Geotechnical Evaluation Report

NOVATO CREEK LEVEE EVALUATION

PREPARED FOR:

Marin County Flood Control and Water Conservation District

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GEI Consultants

Consulting
Engineers and
Scientists

Geotechnical Evaluation Report

Novato Creek Levee Evaluation
Marin County, California

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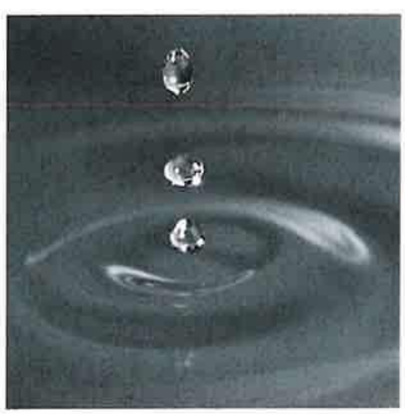

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Acronyms and Abbreviations

cm/sec	centimeters per second
c	undrained cohesion
c′	drained cohesion
District	Marin County Flood Control & Water Conservation District
CPT	cone penetration test
DWR	California Department of Water Resources
EM	Engineer Manual
GEI	GEI Consultants, Inc.
GEO-SLOPE	GEO-SLOPE International, Ltd.
GER	Geotechnical Evaluations Report
HEM	helicopter-borne electromagnetic
HTOL	hydraulic top of levee
i	average exit gradient
i_c	critical gradient
k_h	horizontal hydraulic conductivity
k_v	vertical hydraulic conductivity
LL	liquid limit
N_{60}	standard penetration resistance corrected for hammer system efficiency and sampler dimensions
$N_{1,60}$	normalized standard penetration resistance value
NAD83	North American Datum of 1983
NAVD88	North American Vertical Datum of 1988
NFCP	Novato Flood Control Project
OCR	overconsolidation ratio
pcf	pounds per cubic foot
N_f	uncorrected field standard penetration resistance
PI	plasticity index
psf	pounds per square foot
PTOL	physical top of levee
q_{t1}	normalized tip resistance
SHANSEP	stress history and normalized soil engineering properties
SMART	Sonoma Marin Area Rail Transit
SPT	standard penetration test
SR	State Route
S_u	undrained shear strength
ULDC	Urban Levee Design Criteria
ULE	Urban Levee Evaluations Project
USACE	United States Army Corps of Engineers

WSE	water surface elevation
ϕ	total friction angle
ϕ'	effective friction angle
σ_v'	effective vertical consolidation stress

1.0 Introduction

This Geotechnical Engineering Report (GER) was prepared by GEI Consultants, Inc. (GEI) for Marin County Flood Control & Water Conservation District (District) to present GEI's geotechnical assessment of the Novato Creek levee system.

1.1 Study Area Description

The Novato Creek levee system is located in the City of Novato in Marin County, California (Figure 1-1). The levee segments being evaluated as part of this project includes the following:

- Novato Creek left bank[1], which is approximately 8,500 feet long and extends from the Sonoma Marin Area Rail Transit (SMART) trestle to the northwest and ends at the State Route (SR) 37 bridge to the southeast (Figure 1-2).

- Lynwood Levee is approximately 6,800 feet and extends from the SMART trestle to SR-37 (Figure 1-2). It separates the Lynwood stormwater detention basin on the west side from two wildlife preserve ponds to the east (Duckbill and Heron's Beak Ponds). The detention basin was created primarily for wildlife habitat, but also functions to provide stormwater detention as the Lynwood pump station has three pumps discharging directly to Novato Creek and one pump discharging into the Duckbill Pond before and after high water events.

- Pacheco Pond levee is approximately 3,400 feet long and is located south of SR-37 near Bel Marin Keys (Figure 1-3).

1.2 Project Background, Description, and Purpose

The Novato Creek levees are located within Flood Control Zone No. 1, which was formed in 1955 to address flooding issues in downtown Novato and surrounding areas and encompasses the entire City of Novato as well as a sizeable amount of unincorporated area around the City, making it the District's largest flood control zone. The Zone includes the entire watershed tributary to Novato and Rush Creeks, which includes the project levees (Novato Creek, Lynwood levee, and Pacheco Pond). This area has regularly experienced significant flooding, especially in the areas of Novato where two major creeks converge (Novato and Warner Creeks). In 1984, the residents of Novato voted to fund the Novato Flood Control Project (NFCP). The NFCP was implemented in eight phases that began in 1985 and was completed in 2006. In addition to these improvements, maintenance of lower Warner, Arroyo Avichi, and Novato Creek has required the District to conduct sediment removal operations

[1] Nomenclature for left bank are for views looking downstream.

comprising a range of 25,000-75,000 cubic yards of sediment removal every 4-years to maintain the design-level (50-year) flood protection.

1.3 Evaluation Scope

The objective of this GER is to help the District prioritize, plan, budget for future maintenance and improvement projects to reduce overall flood risk, and to work towards levee certification. The purpose of this report is to present the geotechnical findings of the system-wide study, including existing conditions.

This report addresses geotechnical-related performance issues of select levee reaches related to embankment and foundation seepage and stability only. Civil and/or geotechnical elements needed to achieve Federal Emergency Management Agency (FEMA) accreditation and/or Urban Levee Design Criteria (ULDC) compliance are not addressed in this Study, including:

- Levee penetrations
- Encroachments
- Vegetation
- Animal burrows
- Right-of-way and toe access
- Seismic vulnerability
- Security

Each of these issues would need to be addressed separately as part of an overall FEMA or United States Army Corps of Engineers (USACE) accreditation assessment and ULDC finding either by GEI or by other members of the Project team.

GEI performed subsurface explorations to help fill in data gaps and to aid in the assessment of the levees. The results of the subsurface explorations and lab data are in the geotechnical data report (GEI, 2019).

2.0 Project Levee Information and Geotechnical Design Criteria

2.1 Project Datum and Stationing

The vertical datum used for this Study is the 1988 North American Vertical Datum (NAVD88).

The stationing presented in this report was developed for this project and is intended to be used by the District moving forward. The stationing increases looking upstream for Novato Creek and Lynwood levee. For Pacheco Pond, the stationing increases starting from north to south.

2.2 Water Surface Elevations

The water surface elevations (WSEs) used for analysis in this project are the District provided 50-year peak flow as described in Stetson's H&H report in Section 1 and Table 2a (Stetson, 2019) and the 100-year flow based on the 2017 FEMA Flood Insurance Study (FIS). These will be referred to in this report at the 50-year WSE and 100-year WSE respectively. The WSE data was provided in the hydraulic evaluation report and is summarized in Table 1 (Riverine Flow for the Baseline alternative and FEMA Accredited alternative) of Stetson's Report (Stetson, 2019).

Both the 50-year and 100-year WSE's for Pacheco Pond are approximately 0.5 feet higher than the physical top of levee at the analysis cross section. Therefore, the top of levee was analyzed in place of the 50-year and 100-year WSE's. Overtopping of the Pacheco Pond levee is necessary as the overtopping provides flood protection for the businesses on the west side of Pacheco Pond. Raising the Pacheco Pond levee could therefore reduce the level of flood protection for businesses on the west side of Pacheco Pond (Stetson, 2019).

2.3 Geotechnical Seepage and Slope Stability Design Criteria

This section summarizes the criteria used to evaluate seepage and slope stability geotechnical analyses results for alternatives and design analyses. The criteria for the analyzed water surface elevation based on USACE's design guidelines are consistent with DWR's ULDC Section 7. The design criteria for the different analyses are described below.

2.3.1 Underseepage

Underseepage may occur when a levee is subjected to a differential hydraulic head caused by a river or channel stage that is higher than the ground surface elevation along the landside of the levee. The severity of underseepage depends on several factors, such as the magnitude of the hydraulic head differential; duration of the high-water event; hydraulic conductivity of aquifer layers underlying the levee; thickness, weight, and hydraulic conductivity of any foundation blanket layer; and waterside seepage entrance conditions. The differential hydraulic head leads to seepage flow beneath the levee toward the landside.

When aquifer layers underlie a less pervious top stratum, the seepage in the aquifer layer initially is confined and a blanket condition exists. During high water stages, if the hydraulic pressure in the aquifer layer landward of the levee becomes high enough, the pressure will cause uplift of the blanket. This uplift may lead to rupture at weak spots or low areas, generating a concentration of seepage flow and sand boils. The concentrated seepage may result in channelization of flow across the blanket and underlying aquifer layer, which also may lead to piping. Where seepage flow is concentrated to the extent that turbulent flow conditions exist, the flow may cause erosion of the foundation material, which can undermine the levee. Progressive underseepage piping and boils may lead to levee failure. Underseepage also may negatively affect slope stability by reducing effective stresses in the foundation soils.

Underseepage conditions generally are expressed by an average exit gradient, i. The average exit gradient is calculated using the following equation:

$$i = \frac{\text{total head differential in feet across a blanket layer}}{\text{total thickness in feet of the blanket layer}}$$

The gradient required to cause uplift is called the critical gradient (i_c). The critical gradient is the ratio of the effective unit weight of the blanket layer to the unit weight of water. For a saturated unit weight of 100 pounds per cubic foot (pcf), the critical gradient is 0.6. The ratio of the critical gradient to the average exit gradient is the uplift factor of safety. The following Engineer Manuals (EM) or guidance documents were used to evaluate underseepage and through-seepage for the Novato Creek study area levees:

- EM 1110-2-1913, *Design and Construction of Levees* (USACE, 2000)
- Engineer Technical Letter 1110-2-569, *Design Guidance for Levee Underseepage* (USACE, 2005)
- *Geotechnical Levee Practice* (USACE, 2008a)
- EM 1110-2-1901, *Seepage Analysis and Control for Dams* (USACE, 1993)
- ULDC (DWR, 2012)

According to these publications, the average hydraulic exit gradient must be equal to or less than the following values for the WSE analyzed (50-year and 100-year):

- Landside levee toe: ≤ 0.37 (FS ≥ 1.6)

The average exit gradients summarized above are based on the assumption that the unit weights of the in-situ landside blanket soils (Young Bay Mud) are approximately 100 pcf.

the hydraulic head differential; duration of the high-water event; hydraulic conductivity of aquifer layers underlying the levee; thickness, weight, and hydraulic conductivity of any foundation blanket layer; and waterside seepage entrance conditions. The differential hydraulic head leads to seepage flow beneath the levee toward the landside.

When aquifer layers underlie a less pervious top stratum, the seepage in the aquifer layer initially is confined and a blanket condition exists. During high water stages, if the hydraulic pressure in the aquifer layer landward of the levee becomes high enough, the pressure will cause uplift of the blanket. This uplift may lead to rupture at weak spots or low areas, generating a concentration of seepage flow and sand boils. The concentrated seepage may result in channelization of flow across the blanket and underlying aquifer layer, which also may lead to piping. Where seepage flow is concentrated to the extent that turbulent flow conditions exist, the flow may cause erosion of the foundation material, which can undermine the levee. Progressive underseepage piping and boils may lead to levee failure. Underseepage also may negatively affect slope stability by reducing effective stresses in the foundation soils.

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- ULDC (DWR, 2012)

According to these publications, the average hydraulic exit gradient must be equal to or less than the following values for the WSE analyzed (50-year and 100-year):

- Landside levee toe: ≤ 0.37 (FS ≥ 1.6)

The average exit gradients summarized above are based on the assumption that the unit weights of the in-situ landside blanket soils (Young Bay Mud) are approximately 100 pcf.

2.3.2 Through Seepage

Through seepage may cause removal of materials from levee embankments because of piping through erodible low-plasticity to non-plastic soils. Through seepage also usually is accompanied by a reduced factor of safety against slope stability failure because of high internal water pressures within the landside slope.

The USACE design manuals do not provide specific criteria for through seepage. Accordingly, the levees were evaluated by considering historical performance observations and numerical seepage analyses, based on the location of the phreatic surface break-out on the landside levee slope and the composition of the levee. During the existing conditions analysis phase, levees shown to have a phreatic line emerging on the landside levee slope were evaluated for piping potential and potential for through seepage induced sloughing of the landside slope. Levees with erodible soils that may be prone to piping or through seepage induced sloughing are considered to require remediation

2.3.3 Landside Slope Stability

The requirement for a minimum factor of safety for landside stability in EM 1110-2-1913, *Manual for Levee Design and Construction* (USACE, 2000) is the same as the minimum factor of safety in the ULDC (DWR, 2012). Minimum required factor of safety for the Novato Creek study area is 1.4 at the 100-year WSE.

2.3.4 Rapid Drawdown Slope Stability

Design criteria under rapid drawdown conditions are based on EM 1110-2-1913, *Design and Construction of Levees* (USACE, 2000), which requires a minimum factor of safety between 1.0 and 1.2, depending on the duration of pool levels before drawdown. A minimum factor of safety of 1.0 is required for rapid drawdown analyses where pool levels before drawdown are unlikely to persist for long periods. A minimum factor of safety of 1.2 is required when the pool levels before drawdown are likely to persist for long periods.

Because the water surface elevations in the Novato Creek Study Area are flashy and unlikely to persist for long periods of time, a minimum required factor of safety of 1.0 was adopted for the Novato Creek Study Area.

3.0 Background Information

3.1 Levee Construction, Performance, and Improvement History

3.1.1 Levee Construction

The Novato Creek levees were likely constructed in the late 1800’s by local interests. By the late 1800’s, agriculture was well established and tidal marshlands were diked and drained for farming and grazing. In the early 1900’s the area was modified for flood control and land reclamation purposes.

The District constructed the Lynwood Levee with crown elevations of 14 to 15 feet (NAVD 88); 1 to 2 feet higher than the levee on the Novato Creek right bank levee. The Lynwood Levee is a setback levee to the existing Novato Creek right bank levee and may be the future primary flood protection structure on the right bank of Novato Creek. However, between Station 279+50 and 289+50 the Lynwood Levee is adjacent to Novato Creek, and the only levee on the right bank for this stretch. This levee protects homes, businesses, the SMART commuter rail lines, low lying areas along HWY 101 and two pump stations operated and maintained by the District. Failure of the Lynwood Levee would cause inundation of the Lynwood Basin and also likely cause upstream stormwater flooding and possibly tidal water to flow upstream through the system.

Pacheco Pond was created in 1980 as mitigation for construction of the adjacent Ignacio Industrial Park. The pond is fed by Arroyo de San Jose and Pacheco Creeks and is maintained both as a flood control basin and as wildlife habitat. These two creeks, which serve 18 percent of the Novato watershed drainage area, generate significant discharges to Novato Creek. Inflows from these large and steeply-sloped drainages have a relatively short travel time to Pacheco Pond, but can only flow to Novato Creek during periods of low tide when water levels in Pacheco Pond are higher than tidal elevations in Novato Creek. The tide gates also limit brackish water incursions into this predominantly freshwater pond and preserve Pacheco Pond for stormwater runoff storage capacity. Additionally, the tide gates accommodate creek flow from Pacheco Creek and Arroyo de San Jose that cannot drain against Novato Creek high tides.

3.1.2 Historical Flood Events and Past Performance

Flooding has occurred multiple times over the years, with Novato Creek experiencing the worst impacts. In recent history, the winter storms of 1970, 1973, 1982, 1983, 1986, 1998, 2005, 2006, 2014, 2016/2017, and 2019 caused significant damage. A summary of recent breaches in the Novato Creek Study Area are summarized in Table 3-1.

3.1.3 Levee Improvements

Post construction improvements have been made to the Novato Creek study area and include levee raises and widening, breach repairs, and erosion repairs. Descriptions of these improvements are summarized in Table 3-1.

3.2 Historical Geotechnical Data, Geotechnical Field Investigation, and Laboratory Testing

3.2.1 Historical Geotechnical Data

Historical explorations relevant to the study area levees were compiled from available references and used in the interpretation of subsurface conditions and assessment of the levee condition. Most explorations were located on the crown of the levee, with six explorations performed on the landside toe or field.

The locations of the previous explorations are shown in Figures 1-2 and 1-3 and in the profiles on Plates 10 through 19.

3.2.2 Geotechnical Field Investigation

As part of the site-specific field program, GEI performed field explorations to support a levee evaluation of the Novato Creek study area levees. The field exploration program consisted of the following:

- Preparing a Geotechnical Exploration Work Plan (GEI, 2019) prior to the field explorations.
- Drilling, sampling, and logging 6 exploratory borings along the levee crown;
- Advancing 15 CPT soundings along the levee crown and 4 CPT soundings along the landside levee toe;

The locations of the explorations, sampling intervals, sample types, and target depths were developed based on our review of existing information (GEI, 2019). The explorations were generally located to fill in data gaps where no explorations had previously been performed.

A summary of the borings and CPTs performed for this study is provided in the GDR (GEI, 2019). Exploration plan views showing the locations of the explorations performed for this Study and the locations of explorations performed by others for previous investigations are shown in Plates 1 through 9. Profiles to accompany the plan views are shown in Plates 10 through 19.

Geotechnical laboratory testing was performed on selected samples obtained from the explorations to assist with characterization of the geotechnical engineering properties of the subsurface materials. The testing included dry density, moisture content, Atterberg limits, grain-size distribution tests, consolidation, and triaxial tests.

The data gathered from the field exploration and laboratory testing programs are presented and summarized in the GDR (GEI, 2019).

4.0 Novato Creek Study Area

4.1 Levee Features

4.1.1 Levee Geometry

Levee geometry was evaluated using survey data performed in 2018 (GEI, 2019) for this project as well as data obtained from the District that included historic survey data from 2016. A summary of the geometric characteristics for the Novato Creek levees by segment is provided below.

4.1.1.1 Novato Creek Left Bank

The crown width of the Novato Creek Left Bank levee averages approximately 16 feet (ranging from 13 to 28 feet), with an average height of approximately 13 feet (ranging from 10 to 16 feet). Landside slopes average approximately 1.8H:1V (ranging from 1.5H:1V to 2.7H:1V), and waterside slopes average approximately 2H:1V (ranging from 1.3H:1V to 4.8H:1V).

4.1.1.2 Lynwood

The crown width of the Lynwood levee averages approximately 34 feet (ranging from 19 to 69 feet), with an average levee height of approximately 12 feet (ranging from 8 to 20 feet). Landside slopes average approximately 1.8H:1V (ranging from 1.4H:1V to 5.5H:1V), and waterside slopes average approximately 2H:1V (ranging from 1.5H:1V to 4.1H:1V).

4.1.1.3 Pacheco Pond

The crown width of the Pacheco Pond levee averages approximately 12 feet (ranging from 8 to 14 feet), with an average height of approximately 13 feet (ranging from 12 to 15 feet). Landside slopes average approximately 2.5H:1V (ranging from 1.9H:1V to 3.9H:1V), and waterside slopes average approximately 4H:1V (ranging from 3.2H:1V to 6.1H:1V).

4.1.2 Features

Features presented along the project levees include, but are not limited to pipe penetrations, pumps, and ramps. It was not within the scope of this study to evaluate levee anomalies. For any future designs, anomalies and features should be evaluated because they could affect the embankment and foundation seepage and/or stability.

4.2 Subsurface Conditions

Historical boring and CPT logs compiled from available references and site-specific borings and CPTs performed for this Study were used to evaluate the subsurface conditions along the

Study area levees. In cases of conflicting information between the current and historical data, the more recent information was generally given more weight in interpreting subsurface conditions. Copies of the boring and CPT logs performed for this Study are presented in the GDR. Levee embankment and foundation materials are discussed individually in the following sections.

4.2.1 Embankment Materials

4.2.1.1 Novato Creek Left Bank

Based on available subsurface explorations, embankment soils encountered along the Novato Creek levees generally consisted of clay with interbedded layers of silt.

4.2.1.2 Lynwood

Based on available subsurface explorations, embankment soils encountered along the Lynwood levees generally consisted of fine-grained soils (i.e., silt and clay) and sand (silty sand and clayey sand) layers that have a fines content ranging between 16 and 46 percent.

4.2.1.3 Pacheco Pond

Based on available subsurface explorations, embankment soils encountered along the Pacheco Pond levees generally consisted of elastic silt and fat clay. The silt materials are considered potentially erodible and susceptible to piping and through seepage breakout, while the clay materials are not considered erodible.

4.2.2 Foundation Materials

Based on the regional geology and available geotechnical explorations the levee embankments are predominantly underlain by soft Young Bay Mud deposits ranging from 10 to 70 feet thick. Beneath the Young Bay Mud layer is an Old Bay Mud layer with interbedded layers of clayey and silty sands as well as clayey gravel layers.

4.2.3 Groundwater Conditions

In the explorations performed for this Project, groundwater was encountered at the time of exploration at depths of approximately 0 to 6 feet below the landside levee toe, corresponding to elevations of about -4 to 2 feet (NAVD88), and depths of approximately 4 to 14 feet below the levee crown, corresponding to elevations of about -4 to 6 feet. The estimated depths to groundwater encountered in the explorations performed for this Project can be found in the GDR (GEI, 2019). It should be noted that the depths to groundwater are based on pore pressure dissipation tests performed during the CPT soundings and are not direct measurements. Groundwater is expected to vary seasonally with river levels, changes in seasons, variations in rainfall, human activities, and other factors.

5.0 Geotechnical Evaluations Methodology

5.1 Geotechnical Parameter Characterization

Recommended material properties were developed for each stratigraphic layer for each modeled cross section. Available, site specific geotechnical exploration and testing information was reviewed within the evaluation reach of each cross section including geomorphology, geophysical data, subsurface explorations, and laboratory testing. The material properties were developed considering the guidance outlined in EM 1110-2-1913 (USACE 2000) and the Urban Levee Evaluation (ULE) Guidance Document for Geotechnical Analyses (DWR 2015).

5.1.1 Hydraulic Conductivity

Hydraulic conductivities for seepage analyses were selected for each soil type based on material index properties, laboratory testing, and review of relevant geotechnical references. Hydraulic conductivities were developed for each material type encountered within the levee embankment and foundation soils. Further discussion of the development of hydraulic conductivity values is provided in Appendix A. A summary table of horizontal and vertical hydraulic conductivities for each material type is provided on the cross sections (Appendix C).

5.1.2 Soil Strength Parameters

Soil strength parameters for slope stability analyses were selected for each soil type. Strength parameters vary based on a number of factors such as material type, relative density, current and maximum past pressures, and plasticity. These factors were considered during development of strength parameters as described in Appendix B. Unit weights for each soil strata were selected based on available laboratory test data and typical ranges for each soil type.

In selecting strength parameters, distinction was made between free-draining materials and non-free-draining materials. Free-draining materials are defined as coarse-grained materials with little or no plastic fines such that, when sheared, do not generate excess pore water pressure. Free-draining materials were assumed to remain drained and hence their shear strength was characterized with effective stress drained parameters for all loading conditions. Effective stress parameters were used for steady-state slope stability analyses for all soil types modeled. Fine-grained soils were assumed to drain slowly and not dissipate excess pore pressures. For rapid loading cases (such as rapid drawdown), we assigned undrained strength parameters to fine-grained soils that were not considered free-draining materials.

Strength parameter development for each analysis cross section is discussed in more detail in Appendix B.

5.2 Methodology for Steady-State Seepage Analyses

Seepage analyses were performed using SEEP/W, a two-dimensional finite element modeling computer program, developed by GEO-SLOPE International, Ltd (2018). SEEP/W was used to calculate the steady-state phreatic surface and pore water pressure within the levee and foundation soils at the 50-year and 100-year WSEs. The seepage analyses are discussed in more detail below.

5.2.1 Seepage Parameters

Available, pertinent geotechnical exploration and testing information was reviewed within the evaluation reach of each cross section to develop recommended seepage parameters for the analyses. The parameters were developed considering the guidance outlined in EM 1110-2-1913 (USACE, 2000) and the ULE Guidance Document (DWR 2015). A summary of the hydraulic conductivities used in the analyses are provided on the cross-sections Appendix C.

Hydraulic conductivities were selected for each soil type encountered within the levee embankment and foundation based on material index properties and correlations with grain size distribution and plasticity characteristics.

5.2.2 Model Development

Important elements for consideration in developing seepage models include model cross section development (levee geometry, surface conditions, and soil stratigraphy), seepage parameter selection, and boundary condition selection. Model cross section selection and development for each levee reach is discussed in Section 7.2. Seepage parameters and boundary conditions are discussed below.

5.2.3 Seepage Model Boundary Conditions

Boundary conditions were generally applied to the seepage model as follows:

- “No-flow” boundary condition at the waterside vertical edge and bottom edge of the model;
- Constant head boundary condition equivalent to the flood level being evaluated along the riverside ground surface and riverside levee slope below the analysis water level (either 50-year or 100-year WSEs);
- Constant head boundary condition applied to the landside vertical edge of the model equal to the natural ground surface elevation landward of the levee toe, avoiding anomalous high or low points; and

- Potential seepage face review boundary condition along the levee crown, landside levee slope and landside ground surface to the landward extent of the model.

It should be noted that the landside vertical edge boundary condition represents a conservative condition, because the likely presence of standing water above the landside levee toe during a flood event would help decrease the overall net seepage head differential across the levee.

The extents of the transverse sections for SEEP/W models were selected as follows:

- The SEEP/W models extended landward approximately 1,000 to 2,000 feet to assure that boundary elements did not adversely affect the analyses;
- On the waterside, models were extended to the middle of the main channel or to the middle of the pond, whichever was further away from the levee centerline; and
- The bottoms of the models were extended downward to a minimum depth of 4H below the landside ground surface where H is the height of the levee embankment. This is consistent with the Guidance Document recommendations (DWR, 2015).

5.2.4 Underseepage

Underseepage analyses were performed assuming steady-state seepage conditions developed during the flood condition being analyzed. Underseepage for steady-state conditions is evaluated by calculating the average vertical exit gradient across at the landside levee toe and at potentially critical locations away from the levee toe based on variations in subsurface and surface conditions. The calculated average vertical exit gradient is compared to the maximum allowable gradient for the location under consideration. Alternatively, the potential for underseepage problems can be expressed as a factor of safety against uplift by dividing the critical gradient associated with the blanket soil by the calculated gradient. The critical gradient is determined by taking the total submerged unit weight of the soil and dividing it by the unit weight of water, and it represents the gradient at which uplift of the blanket might be initiated.

Maximum allowable vertical exit gradients and factor of safety criteria used for this Study are consistent with the ULDC and are tabulated below.

Condition	Max. Allowable Exit Gradient(1)*	Minimum FS(1)
Landside Levee Toe	0.37	1.6
Landside Levee Toe with Seepage Berm (min width = 4 x height, max 300 ft.)	0.37	1.6
Seepage Berm Toe (min width = 4 x height, max 300 ft.)	0.6 (Note 2)	1.0
Ditch, Canal or Depression (at the levee toe)	0.37	1.6
Ditch, Canal or Depression (150 ft. from the levee toe)	0.6 (Note 3)	1.0

1. The saturated unit weights of the "in-situ" landside blanket soils must be at or above 100 pcf to use these exit gradient criteria.
2. Instances where the toe exit gradient exceeds 0.6 at the toe of a 300-ft-wide seepage berm are considered unlikely to affect levee performance during a flood event since seepage-related issues, such as sand boils, would occur 300 ft from the landside levee toe. Thus, the seepage berm is truncated at a width of 300 ft.
3. Exit gradient criteria are linearly interpolated from 0.37 at the landside levee toe to 0.6 at a distance of 150 ft from the landside levee toe.

Additionally, if no fine-grained blanket material was present beneath the levee, referred to in this report as a "leaker" condition, a Creep Ratio calculation was performed where sandy soil layers exist in the upper foundation. Creep Ratio is a metric for evaluating the risk of backward erosion of a sandy layer below a hypothetical impermeable roof, which is considered not erodible. Creep Ratios were originally based on observations of piping occurring from foundations supporting masonry dams, but the use of Creep Ratios for evaluation of levees provides an indication of conditions that may lead to piping and backward erosion of the foundation. The calculation compares the seepage flow distance, or the levee base width (W), to the Net Head (h_{cr}).

Specific critical Creep Ratios, or creep factors, have been identified for different soil types, with more erodible soils (i.e. fine sands or silt) requiring a greater base width for a given hydraulic head. Bligh (1927) provides a creep factor based on the grain size and material type of layer and if the base width/net head ratio is less than this value, it would be susceptible to backward erosion and piping (assuming no flow through the overlying structure) (CIRIA, 2013). The use of Creep Ratios for this evaluation provides a relative indication of conditions that may be more vulnerable to "leaker" seepage and/piping.

5.2.5 Through Seepage

In the case where through seepage is determined to exit on the landside levee slope, the effects of levee through seepage need to be evaluated. For the case of levee through seepage, or "face-exiting" seepage, the levee did not meet criteria if there was a phreatic surface breakout of at least 1-foot above the landside levee toe and the embankment was comprised of sand or low plasticity, fine-grained material considered potentially erodible, or if there were reported instances of past through seepage events.

The presence of rodent holes and animal burrows was not explicitly analyzed as part of this study; however, it should be noted that their presence increases the through seepage risk and could potentially lead to piping, erosion, or levee failure especially if the holes are observed on both the waterside and landside slope at the same location.

5.3 Methodology for Steady-State Slope Stability Analyses

Slope stability analyses were performed on the same analysis cross sections evaluated for seepage using SLOPE/W, a slope stability analysis software program developed by GEO-SLOPE International, Ltd (2018). Slope stability was evaluated using the Spencer limit equilibrium method of analysis, which satisfies both moment and force equilibrium by assuming that the resultant interslice forces are of constant slope throughout the sliding mass. Circular slip surfaces were evaluated and defined using the entry-and-exit method. Non-circular surfaces and wedge analyses were not considered for this Study. The slope stability parameters and loading conditions considered for the analyses are discussed below.

5.3.1 Slope Stability Parameters

Slope stability parameters include soil strength and unit weight. These parameters were selected for each soil layer in each analysis cross section. The parameters were developed considering the guidance outlined in EM 1110-2-1913 (USACE, 2000) and the ULE Guidance Document and published reports in similar materials such as *The Properties of San Francisco Bay Mud at Hamilton Air Force Base, California, University of California at Berkeley*, (Bonaparte and Mitchell, 1979).

Strength parameters vary based on a number of factors such as material type, relative density, current and maximum past pressures, and plasticity. A summary of the strength parameters used in the analyses are provided in Appendix B.

Fine-grained soils were assumed to drain slowly and not dissipate excess pore pressures for rapid loading conditions such as rapid drawdown. For the rapid drawdown analyses, undrained strength parameters were assigned to fine-grained soils that were not considered free-draining materials.

5.3.2 Slope Stability Analysis

Critical slip surfaces were identified for each load case. Slip surfaces less than five-feet-deep were not considered to be indicative of failed criteria, since they can be categorized as localized sloughing failures that are a maintenance concern rather than a levee safety issue. These shallow, localized failures are not considered an immediate threat to the levee and can be repaired between flood events. Failure surfaces were limited to circles that would impact the levee crown creating a potential levee safety issue.

For this case, it is assumed that the duration of the flood is sufficient to establish steady-state seepage conditions through the levee embankment, in accordance with USACE guidelines. The phreatic surfaces and pore water pressures from the seepage analyses were used in the stability evaluations. Because steady-state seepage is a long-term condition, drained strengths were assigned to both coarse- and fine-grained soils.

End of Construction for new levees or Earthquake Loading were not included in the scope of this Study. End of Construction is typically only evaluated for levee reaches if major modifications in the levee cross section (crown raises or significant berms) are required to mitigate seepage, stability, or overtopping. Earthquake Loading evaluations are only required for development of an Emergency Action Plan and are not required for evaluating the need for or design of remedial mitigation measures. This is because the Novato Creek levees have been classified as only "Intermittently Loaded" levees per ULDC, which are levees that do not experience a water surface elevation of one foot or higher above the elevation of the levee toe at least once a day for more than 36 days per year on average. Since the water surface against the Novato Creek levees exceeds one foot above the toe of the levees for less than 36 days per year on average, earthquake stability evaluations are not part of the evaluation of existing conditions presented in this report.

5.4 Methodology for Rapid Drawdown Slope Stability Analyses

The waterside rapid drawdown stability condition of a levee slope is evaluated when a drop in water level is relatively quick, so that soils within the slope do not have sufficient time to completely drain. Rapid drawdown analysis results depend on the embankment and foundation materials, drop level, and waterside slope geometry.

Water level drop rates for rapid drawdown analyses were estimated using available hydrographs prepared by Stetson Engineering for the project. The drop rates used for Novato Creek and Lynwood were based on the Mean Lower Low Water elevation of 2.96 feet. Based on an evaluation of this information, a drop of 10.8 feet is considered appropriate for rapid drawdown analysis along the Novato Creek Left Bank levee and a drop of 9.5 feet for Lynwood Levee. Due to the ponding condition along Pacheco Pond Levee, a drop from the Physical top of levee (PTOL) to the waterside toe was used. The drop of 7.3 feet was considered appropriate. Analyses were performed assuming that the drop occurred from the 100-year WSE or PTOL, whichever was lower.

Rapid drawdown analyses were performed in accordance with the three-stage rapid drawdown procedure developed by Duncan, Wright, and Wong (1990). This procedure is recommended for levees in EM 1110-2-1913, *Design and Construction of Levees* (USACE, 2000). The procedure is summarized as follows:

- First-stage computations: First-stage computations estimate effective stresses along the slip surface before drawdown. The phreatic surface used to estimate effective stress before drawdown is used to estimate undrained shear strength in the second-stage computation.
- Second-stage computations: Using the effective stresses calculated during first stage computations, undrained shear strengths are estimated for second-stage computations from shear strength relationships for undrained shear strength and the effective consolidation stress. For SLOPE/W, these computations are done internally in the

program and traditional “R” (consolidated undrained) strength envelopes are input. Factors of safety for undrained conditions are calculated for slip surfaces using the phreatic surface before drawdown.

- Third-stage computations: Third-stage computations are begun by estimating the fully drained shear strengths of the soil (assuming all excess pore water pressures due to drawdown have dissipated).

After computing drained strengths, undrained shear strengths used for second-stage computations are compared to drained strengths for each slice along the slip surface. For any slice in the sliding mass where the drained strength is lower than the undrained strength, the drained strength is assigned to that slice. After the appropriate drained or undrained shear strength has been assigned for each slice (i.e., the lower of the two), the third-stage computations are performed. The factor of safety computed during the third stage represents the factor of safety for rapid drawdown.

Similar to steady-state stability analysis, rapid drawdown analyses also were performed using SLOPE/W software. SLOPE/W software is an acceptable tool to perform three-stage rapid drawdown analyses, in accordance with the Duncan, Wright, and Wong (1990) procedure. A white paper by GEO-SLOPE (Krahn, 2004) indicates that GEO-SLOPE’s rapid drawdown analyses results are similar to the original analyses results published by Duncan, Wright, and Wong.

5.5 Methodology for Settlement Analysis

Settlement analyses of existing conditions were not performed as part of this study. Laboratory testing performed on select soil samples provided estimates for the compressibility characteristics of the soil layers beneath the levees, however the construction documentation of the levee embankments was not available at the time of this study. This data is necessary to determine the amount of and duration of loading on the in-situ bay mud layers. If available, historic records of original construction and subsequent levee raises would make it possible to perform estimated settlement calculations for all known construction (loading) starting with the time the levees were built. These calculated settlements could then be compared to the estimated actual settlement from current survey data. Without this information on the as-built conditions of the embankment and constructed levee improvements a meaningful existing conditions settlement analysis could not be performed.

Due to the presence of thick layers of Young Bay Mud underlying the levee embankment, settlement has been occurring as a result of the levee construction and subsequent levee raises. To facilitate continued operations and maintenance activities as well as future engineering design it is recommended that a settlement monitoring program be implemented to track and document the impact of settlement on the levee embankments.

5.6 Methodology for Erosion Assessment

GEI evaluated whether appreciable erosion of the levee embankment can be expected during the base flood (100-year) as a result of either currents or waves, and whether the anticipated erosion will result in failure of the levee embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability.

For the Novato Creek Levee System, the primary factors addressed in this evaluation included a levee geometry check, expected flow velocities, existing slope protection techniques, and the ability of embankment and shallow foundation materials to withstand expected flow velocity without eroding or scouring.

The levee geometry check compares a standard levee prism to a given levee cross section and matches the top of the prism's landside to the levee's landside intersection with a given water surface elevation (plus freeboard). This elevation is defined by the 100-year WSE plus three feet of freeboard. Per Table 7.1 in the ULDC (DWR, 2012) the Novato Creek LB us 37 and Lynwood levees are considered to be "major stream levees". Therefore, a 20-foot crown width with a wasterside slope of 3 horizontal to 1 vertical ratio (3H:1V) and a landside slope of 2H:1V was considered the standard levee prism. For levees along Pacheco Pond, the standard levee prism used has a 12-foot crown width with a wasterside slope of 3H:1V and a landside slope of 2H:1V consistent with "minor stream levees". At any area on the waterside where the standard levee prism exceeds the existing levee section, the levee's integrity is considered compromised (Figure 5-1). Areas with extensive erosion may be subject to significant risk of erosion failure.

Peak flow and local velocities have been summarized by Stetson and are presented in Appendix D. Riverine erosion typically occurs at locations where levee materials are not able to resist the scouring forces of high-velocity flow. EM 1110-2-1601 recommends a set of maximum mean channel velocities as a guide to design non-scouring flood control channels. Table 5-1 summarizes the maximum mean channel velocity that a given material can withstand before it begins to scour.

6.0 Reach Selection

6.1 Overview

The findings from the review of historic explorations and site-specific explorations performed for this project of existing levee conditions were used to divide the levee alignment into reaches that have reasonably consistent characteristics (e.g., levee geometry, subsurface conditions). After reach selection, analysis locations were selected.

6.2 Study Area Reaches

A total of three reaches were designated for the Novato Creek levees: 1) Novato Creek Left Bank (Reach NCLB), 2) Lynwood Levee (Reach LL), and 3) Pacheco Pond Levee (Reach PP). The station limits of each reach are summarized in Table 6-1.

The information summarized in Table 6-1 at each cross-section location is grouped by reach. A summary of the conditions that form the basis for each reach are also summarized in Table 6-1. A summary of the average existing levee conditions, past performance, improvement history, and analysis WSE is in Table 6-1.

The reaches were selected such that each reach can be adequately represented in terms of geotechnical characterization and analysis by one longitudinal soil profile, one associated transverse cross section, and one set of associated geotechnical analysis input parameters. When selecting reaches, the following factors and characteristics were considered:

- Levee composition, geotechnical properties of levee materials, and levee construction method (where known);
- Levee geometry, including height and slope angles;
- Levee performance history and types of distress; and
- WSEs to be used for assessment (relative degree of loading to be evaluated)

7.0 Existing Conditions Analysis

7.1 Overview and Approach

Numerical seepage and stability analyses were performed to aid in the evaluation of each of the Novato Creek levee reaches using site-specific field and laboratory test data collected and developed as part of this Study.

Analysis cross sections were first developed to represent the conditions within the selected identified reaches. Two-dimensional seepage and stability analyses were then performed along the analysis cross sections in general accordance with USACE EM 1110-2-1913, Design and Construction of Levees (USACE, 2000), ETL 1110-2-569, Design Guidance for Levee Underseepage (USACE, 2005), EM 1110-2-1902, Slope Stability (USACE, 2003) and the DWR Urban Levee Evaluation Guidance Document for Geotechnical Analyses (DWR, 2015; hereafter referred to as "ULE Guidance Document"). Design criteria presented in the ULE Guidance Document were adopted for this Study and are referred to as the Design Criteria. Three dimensional effects were not considered for this Study.

7.2 Analysis Cross Sections

For each reach, cross sections consisting of embankment and foundation materials were developed for seepage and stability analyses. Soil stratigraphy was interpreted for each cross section and the contacts between interpreted soil layers were typically drawn as horizontal lines. Cross sections were typically chosen near field explorations where detailed stratigraphic and material property information was available. Seepage and stability parameters were developed for each stratigraphic layer in the cross section. The development of seepage and slope stability parameters for each layer in each cross section are presented in Appendix A and Appendix B, respectively. The analysis cross sections developed to represent the reach for seepage and stability analyses are discussed in the sections below.

As noted previously, the Novato Creek levees are founded on materials predominantly classified as clay, including softer, young Bay Mud materials. Deeper old bay mud deposits, are commonly interbedded and irregularly stratified, and layers are potentially discontinuous over long distances. It should be noted that geomorphic features such as natural channels (as observed in the vicinity of the Novato Creek Left Bank) will potentially impact the subsurface stratigraphy and soil layer continuity. The stratigraphic interpretations discussed below and presented in the figures referenced in this section are based on our engineering judgment. Because of the inherent variability of these deposits, other interpretations are possible.

7.2.1 Novato Creek

The analysis cross section selected for the Novato Creek Levee is located at STA 300+89. Key embankment characteristics at the cross-section location are:

Height =	12 feet
Width =	16 feet
Waterside slope =	1.6H:1V
Landside slope =	1.6H:1V

The embankment materials are modeled as clay (CL) that is interpreted as not potentially erodible. The foundation is modeled with an approximately 25-foot-thick clay (CH) blanket layer over a more permeable silty sand (SM) material which extends to the maximum depth of the model.

7.2.2 Lynwood

The analysis cross section selected for the Lynwood Levee is located at STA 260+68. Key embankment characteristics at the cross-section location are:

Height =	10 feet
Width =	30 feet
Waterside slope =	1.4H:1V
Landside slope =	1.9H:1V

The embankment materials are modeled as a lean to fat clay (CL/CH) over a silty sand (SM) layer in the lower half of the levee. The lower portion of the levee is interpreted to be potentially erodible. The foundation is modeled with an approximately 5-foot-thick silty sand layer that creates an unconfined seepage path beneath the levee (referred to as a “leaker” condition). Approximately 35 feet of soft clay (CH) is modeled below the shallow SM layer.

7.2.3 Pacheco Pond

The analysis cross section selected for Pacheco Pond Levee is located at STA 33+22. Key embankment characteristics at the cross-section location are:

Height =	13 feet
Width =	10 feet
Waterside slope =	2.6H:1V
Landside slope =	2.2H:1V

The embankment materials are modeled as elastic silt (MH) and fat clay (CH) that is interpreted to be potentially erodible. The foundation is modeled with an approximately 25-foot-thick clay (CH) blanket (soft, younger Bay Mud) overlying and 8-foot-thick silty sand

(SM) material (Old Bay Mud). This silty sand is underlain by a stiff clay (CH) layer (Old Bay Mud).

8.0 Geotechnical Analysis Results

As discussed, the Novato Creek levee system was divided into three reaches. An analysis cross section was selected for each of the reaches and were analyzed to assess the existing conditions. At the completion of existing conditions analysis, the project team documented whether a levee reach met or did not meet design criteria.

8.1 Seepage and Stability

Steady-state seepage analyses, steady-state stability analyses and rapid drawdown stability analyses were performed on the three analysis cross sections developed to represent Novato Creek, Lynwood, and Pacheco Pond (Section 7.2). Each cross section was analyzed for flood levels corresponding to the 50-year and 100-year WSE. The results of the analyses are discussed below and summarized in Table 8-1 by reach. The analysis results are discussed in detail and presented by reach in Appendix C.

For each cross section, the seepage analysis results are illustrated in figures in Appendix C that show the seepage model with soil layering and total head contours for the 100-year WSE. Exit gradients were estimated at the levee toe for each cross section and are annotated on the figures. Likewise, the stability analysis results are illustrated in figures in Appendix C that show the soil layering, amount of water drop (rapid drawdown), analysis search limits, and critical failure surface with corresponding factors of safety.

The analysis sections were evaluated to assess which reaches meet seepage and stability criteria and if remediation is needed. In determining whether reaches are recommended for remediation, variable foundation conditions, levee composition, and past performance were considered. The 50-year and 100-year WSE elevations (Stetson 2019) were typically close together (within 1 ft) and because of this the analysis conclusions discussed below were the same for both analyses.

The results of the analyses, slope stability appears to be the highest risk potential failure mode in the levee reaches evaluated for this Study. However, due to the composition of the levees and shallow foundation, through seepage and shallow under seepage is also a concern in several reaches, and may be exacerbated by burrowing rodents. The stability deficiencies appear to be largely due the soft bay mud (CH) underlying the levee embankments. A summary of the results for each reach is provided below.

Remedial improvements should be considered to address these stability deficiencies. The mitigation of the landside slope stability would likely be able to address the through seepage and shallow under seepage concerns as well with only minor adjustments. The remedial alternatives are detailed in a separate Remedial Alternatives Report (RAR).

8.1.1 Novato Creek Left Bank

The Novato Creek Left Bank levee seepage analyses results indicate that the levee meets criteria for underseepage and through seepage. The phreatic surface does breakout above the levee toe, but the embankment material is predominantly clay (CH) and not considered erodible.

Based on the stability analyses results, we conclude that the Novato Creek Left Bank levee reach does not meet criteria for landside slope stability. Rapid drawdown waterside stability criteria are met for this reach.

8.1.2 Lynwood

The Lynwood levee seepage analyses results indicate that the levee does not meet criteria for underseepage due to a leaker condition and does not meet criteria for through seepage due to the breakout of the phreatic surface in a potentially erodible SM layer at the base of the embankment.

Based on the stability analyses results, we conclude that Lynwood levee reach does not meet criteria for landside slope stability. Rapid drawdown waterside stability criteria are met for this reach.

8.1.3 Pacheco Pond

The Pacheco Pond levee seepage analyses results indicate that the levee meets criteria for underseepage but does not meet criteria for through seepage due to the breakout of the phreatic surface in a potentially erodible elastic silt layer in the embankment.

Based on the stability analyses results, we conclude that Pacheco Pond levee reach does not meet criteria for landside slope stability. The analysis assumed the WSE was at the same elevation as the levee crown because the 50-year and 100-year elevations exceeded the elevation of the crown. As discussed in Section 5.3, the analysis assumes that steady-state seepage conditions develop through the levee embankment, in accordance with USACE guidelines. This is a conservative assumption that resulted in a very low factor of safety (less than 1.0). To develop a steady state condition in these materials would require sustained loading for a very long period of time, which does not reflect the typical levee condition for Pacheco Pond. As a check of the material parameters, a sensitivity analysis was performed assuming a low WSE that would not saturate the embankment (to reflect a more typical condition for Pacheco Pond) and the analysis resulted in a factor a safety above 1.0 but did not approach the minimum factor of safety of 1.4 as outlined in both the *Manual for Levee Design and Construction* (USACE, 2000) and the ULDC (DWR, 2012) for the 100-year WSE. The factor of safety greater than 1.0 for this condition is consistent with the current condition of the levee (not actively failing). Rapid drawdown waterside stability criteria are met for this reach.

8.2 Erosion

8.2.1 Novato Creek

The Novato Creek levee freeboard analysis was performed by Stetson and can be found in Section 5 of the H&H report (Stetson 2019). The analysis indicates that none of the Novato Creek Levee meets the freeboard requirement of 3 feet above the 100-year WSE. As a result of this, it is not possible to place the standard levee prism as described in Section 5.6, and the levee is considered compromised. Comparison with the standard levee prism indicates that the waterside slope is over steepened with slopes steeper than 3H:1V and in many locations, approaching 2H:1V.

Based on the available geotechnical data, the embankment of the Novato Creek levee predominantly consists of clay (CL and CH) with some areas of clayey gravel (GC). Stetson's model indicates that the velocities in the channel average approximately 2 fps with a peak velocity of approximately 3.7 fps (Appendix D). The clay embankment material will likely be resistant to scour at these velocities per the values outlined in Table 5-1.

Based on the analysis described above, the Novato Creek levee is not considered at risk of erosion-driven failure. However, while the boring logs indicate that the embankment is largely clay and resistant to erosion, it is possible that there are portions of the levee (outside of the discrete points sampled by the borings) that contain more sandy material and could potentially be susceptible to erosion. Monitoring of the slopes should be performed as part of the ordinary operations and maintenance to ensure no areas of erosion develop in the future.

8.2.2 Lynwood

Lynwood levee freeboard analysis as summarized in Section 5 of the H&H report (Stetson 2019) indicates that only a small upstream portion (~1000 feet) meet the freeboard requirement of 3 feet above the 100-year WSE from approximately station LL 297+50 to LL 307+50 (excluding a low point at Station LL 304+50). As a result of this, it is not possible to place standard levee prism as described above in the remaining approximately 6000 feet upstream and this portion of the levee is considered compromised. In the downstream portion the standard levee prism indicates that the waterside slope is oversteepened and the toe has begun encroaching on the theoretical standard levee prism. Additionally, the upstream waterside slopes are oversteepened with slopes steeper than 3H:1V, and in many locations, steeper than 2H:1V.

Based on the available geotechnical data, the embankment in the upstream portion of the Lynwood levee (approximately 2500 feet) predominantly consists of clay (CH). The downstream portion of the embankment (approximately 4500 feet) predominantly consists of sand and silt. Most of the historic borings do not provide detail on the grain size of the sand; as a result, a conservative assumption of fine sand/sandy silt was used in this assessment. Stetson's model indicates that the velocities in the upstream 2500 feet of the channel average approximately 2 fps with a peak velocity of approximately 3 fps (Appendix D) The clay embankment material in this area of the channel will likely be resistant to scour at these velocities per the values outlined in Table 5-1. The downstream 4500 feet of the channel has an average velocity of approximately 3 fps with a peak velocity of 3.8 fps (Appendix D). The silty/sandy material identified in the borings are likely susceptible to scour erosion at these velocities.

Based on the analysis described above, the downstream portion of the Lynwood levee (approximately 4500 feet) is at risk of erosion-driven failure. Additionally, it should be noted, that while the boring logs indicate that the upstream portion is largely clay and resistant to erosion, it is possible that there are portions of the upstream levee (outside of the discrete points sampled by the borings) that contain more sandy material and could potentially be susceptible to erosion as well.

8.2.3 Pacheco Pond

The Pacheco Pond levee freeboard analysis (Stetson 2019) indicates that none of the Pacheco Pond levee meets the freeboard requirement of 3 feet above the 100-year WSE. As a result of this, it is not possible to place standard levee prism as described above and therefore the levee is considered compromised.

Based on the available geotechnical data, the embankment of the Pacheco Pond levee predominantly consists of silt and clay (MH and CH). The Pacheco Pond levee is not subject to typical channel flows. The velocities that fill Pacheco Pond are expected to be negligible and will not exceed the thresholds outlined above in Table 5-1.

Based on the analysis described above, the Pacheco Pond levee is not considered at risk of erosion-driven failure. With an embankment of largely silt with some clay the levee could likely begin to scour if it were subjected to flow velocities greater than 3 feet per second. It is also possible that there are portions of the levee (outside of the discrete points sampled by the borings) that contain more sandy material and could potentially be susceptible to erosion. Monitoring of the slopes should be performed as part of the ordinary operations and maintenance to ensure no areas of erosion develop in the future.

9.0 Evaluation of Results

The results of seepage and stability analyses for the existing conditions were evaluated in conjunction with the record of past performance observations to assess which reaches or portions of reaches meet criteria. Analysis results provided at the end of this section were in general agreement with the observed past performance in the levee system.

Future remedial alternative evaluations should be performed for Novato Creek, Lynwood, and Pacheco Pond levee reaches.

Based on review of levee segments where geotechnical criteria are not met, the potential remediations within the Novato Creek levee system are expected to be either a raised levee crown with earthen fill to address freeboard deficiencies, and berms or cutoff walls to address seepage and stability deficiencies. It is our understanding that the Pacheco Pond levee will not be raised due to potential flood impacts this might have on the surrounding areas. The list below provides potential remediation alternatives that should be evaluated for each reach.

9.1 Pacheco Pond

- Alternative 1: Combination seepage and stability berm
- Alternative 2: Shallow cutoff wall and short stability berm

9.2 Lynwood

- Alternative 1: Levee crown raise and seepage berm
- Alternative 2: Levee crown raise and cutoff wall

9.3 Novato Creek

- Alternative 1: Levee crown raise and widening with a shallow cutoff wall
- Alternative 2: Levee crown raise and widening with a toe drain

Each of the alternatives outlined above have been assessed for viability with respect to the highest risk potential failure mode , land use, environmental, and construction constraints. This detailed discussion as well as the analysis results are presented in the Remedial Alternatives Report (RAR).

10.0 Limitations

The levee system evaluations were performed in accordance with the standard of care and skill ordinarily exercised by members of our profession. Standard of care is defined as the ordinary diligence exercised by fellow practitioners in the same or similar area performing the same services under similar circumstances during the same period.

Discussions of subsurface conditions and improvement alternatives summarized in this report are based on the assumption that subsurface soil and groundwater conditions between the subsurface explorations will not appreciably deviate from those disclosed at the locations of the site-specific explorations. Subsurface explorations may not disclose all adverse conditions in a levee and its foundation. No warranty, either express or implied, is made that actual encountered site and subsurface conditions will conform to the conditions described herein.

Subsurface conditions were directly observed only at the boring locations and directly interpreted at locations where CPT soundings were performed. Geomorphic data were utilized during interpretation of foundation conditions, and for interpolation of conditions where no exploration data were available. However, as is always the case with interpretations of subsurface conditions between widely spaced explorations, it is understood that conditions between explorations may differ from those shown in the profiles and described in this report.

A compilation of prior geotechnical borings and other subsurface data developed by others has been utilized in preparing this report. GEI has relied upon the prior geotechnical information in developing subsurface stratigraphic profiles and strength and hydraulic conductivity parameters for geotechnical analyses of the levee. Inaccuracies in some of the geotechnical data developed by others could lead to incomplete or faulty analyses or interpretations of geotechnical conditions and levee behavior during high water events. GEI does not attest to the accuracy, completeness, or reliability of geotechnical borings and other subsurface data by others that are included in this report; an independent validation or verification of data by others has not been performed.

The analyses results do not constitute a final opinion about the condition of a levee reach relative to levee performance and the ability of a levee reach to provide reliable flood protection, because such determinations can be affected by conditions beyond the scope of work. The findings of this report may be refined as design of remedial measures are developed during the design and review process.

Any data presented in this report are time sensitive in that they apply solely to locations and conditions existing at the time of exploration and during preparation of this report. Data should not be applied to any other projects in or near the area of this study, nor should it be applied at a future time without appropriate verification.

This report is for the use and benefit of Marin County and its consultants. Use by any other party is at their own discretion and risk.

11.0 References

California Department of Water Resources (DWR), 2012, Urban Levee Design Criteria, May.

California Department of Water Resources (DWR), 2015, Urban Levee Evaluation Guidance Document for Geotechnical Analyses.

Federal Emergency Management Agency (FEMA), 2017, Flood Insurance Study (FIS).

GEI Consultants, Inc. (GEI), 2019, Geotechnical Data Report Novato Creek Levee Evaluation Project, Marin County, California, October.

GEO-SLOPE International, Ltd., 2018, GeoStudio 2018 Edition, Calgary, Alberta, Canada.

Stetson Engineers, Inc (Stetson), 2019, Hydraulic Evaluation of Novato Creek Levees. September.

U.S. Army Corps of Engineers (USACE), 2000, Design and Construction of Levees, EM 1110-2-1913.

U.S. Army Corps of Engineers (USACE), 2003, Slope Stability, EM 1110-2-1902

U.S. Army Corps of Engineers (USACE), 2005, Design Guidance for Levee Underseepage, ETL 1110-2-569

Tables

Table 3-1. Past Performance Summary

Flood Year	Reach	Approximate Station	Past Performance Description	Mitigation Description
1983	NCLB	Unknown	Novato Creek levees were damaged during the 1983 high water event. No additional data describing was found.	Levee raise and widening was performed in 1983. The levee crown was built to 15 feet wide with a 2H:1V slope on the landside.
2014	NCLB	306+10 to 306+30	A 20-foot wide section was intentionally breached approximately 370 feet downstream of the SMART railroad bridge. The breach was an emergency measure to direct water away for businesses and nearby homes.	The breach was repaired, but in 2016 a 100-foot section was rebuilt to a self-eroding weir to allow water to overflow and relieve the hydraulic pressures upstream.
2014	LL	307+50 to 307+70	A 20-foot wide section blew out at an abandoned 60-inch culvert approximately 230 feet downstream of the SMART railroad bridge. Water overtopped the Novato Creek right bank into Duckbill Pond creating enough pressure to blow-out the culvert.	The abandoned culvert was removed and the levee was repaired in 2015.
December 2014	NCLB and LL	Unknown	A combination of high tides and high intensity rains resulted in levee overtopping, a breach on Lynwood levee, and an intentional breach along Novato Creek left bank.	The overtopping and levee breach along Lynwood levee was repaired in 2015 as part of the Novato Creek Levee Repair Project. The Novato Creek left bank breach was permanently repaired in 2016 as part of the Novato Creek Sediment Removal project.
December 2014	PP	Unknown	Near overtopping	No documented mitigation
2016-2017	LL	Multiple Locations between 242+16 to 310+00	Bank erosion	The District completed temporary repairs right after the storm.
2017	LL	Unknown	Following the Novato Creek Right Bank breach, Lynwood Levee was observed to have sustained erosion leaving near vertical bank cuts in several locations.	Repairs were completed in 2018
February 2019	PP	31+50 to 32+50	A 75 feet to 100 feet wide breach	Emergency repairs were completed on February 17, 2019. The repairs consisted of bringing in 1-ton class rock to close the breach. Other materials included crushed rock and clay soils to stabilize the breach.
February 2019	PP	30+50 to 31+50	A 100 feet wide levee instability and overtopping occurred adjacent to the levee breach. Severe erosion was observed along the levee crest, landside, and waterside slopes.	Emergency repairs were completed on February 28, 2019. The repair along the crest and slopes included drain rock, crushed rock, and clay soils.
February 2019	PP	10+00 to 15+00	Crest and slope erosion was observed during the 2019 storm event.	Emergency repairs were completed on March 13, 2019. The erosion repair along the crest and slope included drain rock, crushed rock, clay soil, aggregate subbase, and geotextile fabric.
February 2019	NCLB	225+36 to 235+00	During the February 2019 storm overtopping occurred along Novato Creek left bank near SR-37.	No documented mitigation
February 2019	LL	242+16 to 247+50	During the February 2019 storm overtopping occurred along Lynwood levee near SR-37.	No documented mitigation

Table 5-1. Maximum Mean Channel Velocity for Material to Resist Erosion

Levee Material	Maximum Mean Channel Velocity (ft/sec)
Fine Sand/Sandy Silt	2
Silt Clay	3.5
Coarse Sand	4
Clay	6
Fine Gravel	6

TABLE 6-1. Summary of Reach and Cross-Section Characteristics
Novato Creek Levee Evaluation Project

<table>
<tr><th>Reach ID</th><th>Station Limits</th><th>Cross-Section Station Location</th><th colspan="2">Reach Details</th><th>Rationale for Reach Selection</th><th>Rationale for Analysis Cross-Section Selection</th><th>Rationale for Inclusion of Explorations in Cross-Section</th></tr>
<tr><td rowspan="11">LL</td><td rowspan="11">LL 242+16 to LL 310+00</td><td rowspan="11">LL 260+68</td><td>Levee Height :</td><td>Min: 5.3 ft.
Max: 15.1 ft.</td><td rowspan="11">• Reach extents were selected based on the similarity of embankment materials, foundation materials, and levee geometry.</td><td rowspan="11">• Section shows representative stratigraphy

• Sections shows representative levee geometry
Crown width: ~32.1 ft.
Landside slope: ~2.1H:1V
Waterside slope: ~1.9H:1V

• Geotechnical data available near section</td><td rowspan="11">• GEI_003B: located at cross-section; crown exploration for stratigraphy

• GEI_011C: located 25.7 ft. up-station from cross-section location; crown exploration projected for stratigraphy

• S-1(2017): located 451.8 ft. down-station from cross-section location; crown exploration projected for stratigraphy

• B22 (2016): located 465.1 ft. down-station from cross-section location; crown exploration projected for stratigraphy

• HA-17 (2016): located 556.8 ft. up-station from cross-section location; crown exploration projected for stratigraphy</td></tr>
<tr><td>Crown Width:</td><td>Typical range: 19.0 to 69.0 ft.
Min: 19.0 ft.
Max: 117.2 ft.</td></tr>
<tr><td>Landside Slope:</td><td>Typical range: 1.4 to 2.2 (H:1V)
Min: 1.4
Max: 5.5</td></tr>
<tr><td>Waterside Slope:</td><td>Typical range: 1.5 to 2.6 (H:1V)
Min: 1.5
Max: 4.1</td></tr>
<tr><td>Riverine, District 50-year, without Sea Level Rise WSE</td><td>12.36 ft.</td></tr>
<tr><td>Riverine, FEMA 100-year, without Sea Level Rise WSE</td><td>12.49 ft.</td></tr>
<tr><td>Past Performance:</td><td>1. <u>Overtopping and levee breach (2014)</u>:
A 20-foot section of the levee was breached approximately 230-ft downstream from SMART Railroad Bridge into Lynwood Basin. The breach resulted in the blow out of an abandoned 60-inch culvert that connected Duckbill Pond and Lynwood Basin.
2. <u>Erosion (2017)</u>:
Several points on the Pond-side of the levee were noted to have sustained erosion leaving vertical banks cut in several locations.</td></tr>
<tr><td>Improvement History:</td><td>1. Levee breach of 2015 was repaired after storm event and abandoned culvert was removed.
2. Erosion repairs from the 2017 event were completed in the summer of 2017 and further repairs were scheduled for 2018.</td></tr>
<tr><td>Embankment Materials:</td><td>Predominantly fine grained material generally composed of Silty Sand (SM), Lean Clay (CL), and Fat Clay (CH)</td></tr>
<tr><td>Foundation Materials:</td><td>Predominantly fine grained material generally composed of Clayey Sand (SC), Silty Sand (SM), Elastic Silt (MH), Lean Clay (CL), and Fat Clay (CH). Transitions to Claystone, Sandstone, and gravelly soils at approximate elevation of -55 feet to -66 feet.</td></tr>
</table>

TABLE 6-1. Summary of Reach and Cross-Section Characteristics
Novato Creek Levee Evaluation Project

Reach ID	Station Limits	Cross-Section Station Location	Reach Details		Rationale for Reach Selection	Rationale for Analysis Cross-Section Selection	Rationale for Inclusion of Explorations in Cross-Section
NCLB	NCLB 225+36 to NCLB 310+00	NC 300+89	Levee Height :	Min: 4.6 ft. Max: 16.1 ft.	• Reach extents were selected based on the similarity of embankment materials, foundation materials, and levee geometry.	• Section shows representative stratigraphy • Sections shows representative levee geometry Crown width: ~15.1 ft. Landside slope: ~1.8H:1V Waterside slope: ~1.7H:1V • Geotechnical data available near section	• GEI_001B: located at cross-section; crown exploration for stratigraphy • GEI_001C: located 2.7 ft. down-station from cross-section location; crown exploration for stratigraphy • B3 (2016): located 20.7 ft. up-station from cross-section location; landside field exploration projected to complete landside stratigraphy • B2 (2016): located 363.8 ft. down-station from cross-section location; landside toe exploration projected to complete landside stratigraphy • B4 (2016): located 381.6 ft. up-station from cross-section location; landside field exploration projected to complete landside stratigraphy • GEI_001C_TOE: located 382.8 ft. down-station from cross-section location; landside toe exploration for stratigraphy • B1 (2016): located 526.2 ft. up-station from cross-section location; crown exploration projected for stratigraphy
			Crown Width:	Typical range: 13.0 to 21.3 ft. Min: 10.2 ft. Max: 27.9 ft.			
			Landside Slope:	Typical range: 1.5 to 2.2 (H:1V) Min: 1.1 Max: 2.7			
			Waterside Slope:	Typical range:1.3 to 2.7 (H:1V) Min: 1.3 Max: 4.8			
			Riverine, District 50-year, without Sea Level Rise WSE	13.56 ft.			
			Riverine, FEMA 100-year, without Sea Level Rise WSE	13.76 ft.			
			Past Performance:	1. Overtopping and intentional levee breach (2014): A 20-foot-wide section of the left bank levee was intentionally breached about 370-ft downstream of SMART Railroad Bridge into Deer Island Basin as an emergency measure to direct water during a storm. 2. Levee breach (2019): Breach occurred just south of SR-37 and is located just outside project limits.			
			Improvement History:	1. Intentional levee breach temporarily repaired after storm event in 2014 and then permanently repaired in 2016. An approximate 100 linear foot section of the levee was rebuilt to be self-eroding to allowing the top two-feet to erode at high flow conditions. 2. No records of levee repairs of levee breach in 2019 have been located.			
			Embankment Materials:	Predominantly fine grained material generally composed of Silt (ML), Lean Clay (CL), and Fat Clay (CH)			
			Foundation Materials:	Predominantly fine grained material generally composed of Clayey Sand (SC), Silty Sand (SM), Elastic Silt (MH), Lean Clay (CL), and Fat Clay (CH). Transitions to Sandstone, Shale and gravelly soils at approximate elevation of -55 feet to -59 feet.			

TABLE 6-1. Summary of Reach and Cross-Section Characteristics
Novato Creek Levee Evaluation Project

Reach ID	Station Limits	Cross-Section Station Location	Reach Details		Rationale for Reach Selection	Rationale for Analysis Cross-Section Selection	Rationale for Inclusion of Explorations in Cross-Section
PP	PP 10+00 to PP 43+90	PP 33+22	Levee Height :	Min: 2.6 ft. Max: 17.5 ft.	• Reach extents were selected based on the similarity of embankment materials, foundation materials, and levee geometry.	• Section shows representative stratigraphy • Sections shows representative levee geometry Crown width: ~13.1 ft. Landside slope: ~2.3H:1V Waterside slope: ~3.8H:1V • Geotechnical data available near section • Section near location of 2019 levee breach	• GEI_005B: located at cross-section; crown exploration for stratigraphy • GEI_015C: located 15.2 ft. up-station from cross-section location; crown exploration for stratigraphy
			Crown Width:	Typical range: 8.2 to 17.1 ft. Min: 8.2 ft. Max: 17.1 ft.			
			Landside Slope:	Typical range: 1.9 to 2.7 (H:1V) Min: 1.9 Max: 3.9			
			Waterside Slope:	Typical range: 3.6 to 4.2 (H:1V) Min: 3.2 Max: 6.6			
			Riverine, District 50-year, without Sea Level Rise WSE	9.80 ft.			
			Riverine, FEMA 100-year, without Sea Level Rise WSE	9.80 ft.			
			Past Performance:	1. <u>Levee breach (2019)</u>: Between approximate Station PP 31+00 and PP 32+00, approximately 75 to 100 feet of levee was blown out in an area just north of the location of GEI_005B during a storm. 2. <u>Levee breach (year unknown)</u>: Breach reported at the southern-most end of levee. Breach extent/location unknown.			
			Improvement History:	1. No records of levee repairs of levee breach in 2019 have been located. 2. Levee breach at southern-most end of levee repaired by District but no records of repair have been located.			
			Embankment Materials:	Predominantly fine grained material generally composed of Elastic Silt (MH) and Fat Clay (CH)			
			Foundation Materials:	Predominantly fine grained material generally composed of Clayey Sand (SC), Silty Sand (SM), Silt (ML), Elastic Silt (MH), Lean Clay (CL), and Fat Clay (CH)			

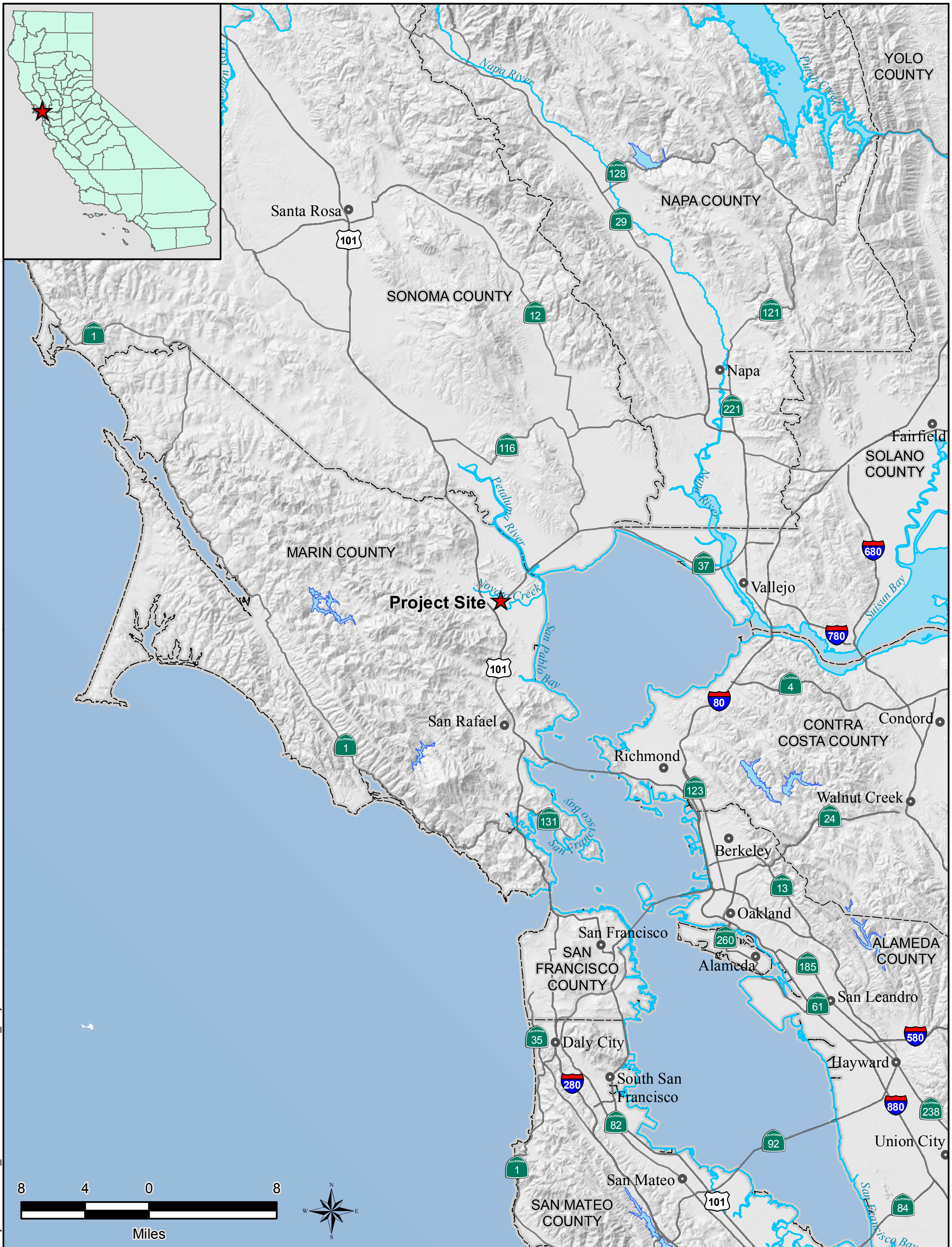
Table 8-1. Existing Conditions Analysis Results Summary

Levee	Station	Model	WSE	WSE	Underseepage	Through Seepage	Slope Stability	Rapid Drawdown
					Gradient	Seepage Breakout above Toe (ft)	FS	FS
Pacheco Pond	PP 33+22	Existing Conditions	9.32	Top of Levee	0.26	9.53	0.5	1.5
Novato Creek LB us 37	NCLB 300+89	Existing Conditions	13.56	50-Year	0.06	2.74	1.2	N/A
Novato Creek LB us 37	NCLB 300+89	Existing Conditions	13.76	100-Year	0.06	2.74	1.2	1.8
Lynwood Levee	LL 260+68	Existing Conditions	12.36	50-Year	N/A	1.00	1.3	N/A
Lynwood Levee	LL 260+68	Existing Conditions	12.49	100-Year	N/A	1.00	1.3	1.3

Red = Does not meet criteria

Criteria for underseepage, through seepage, slope stability, and rapid drawdown can be found in Section 2.3 of this GER.

Figures



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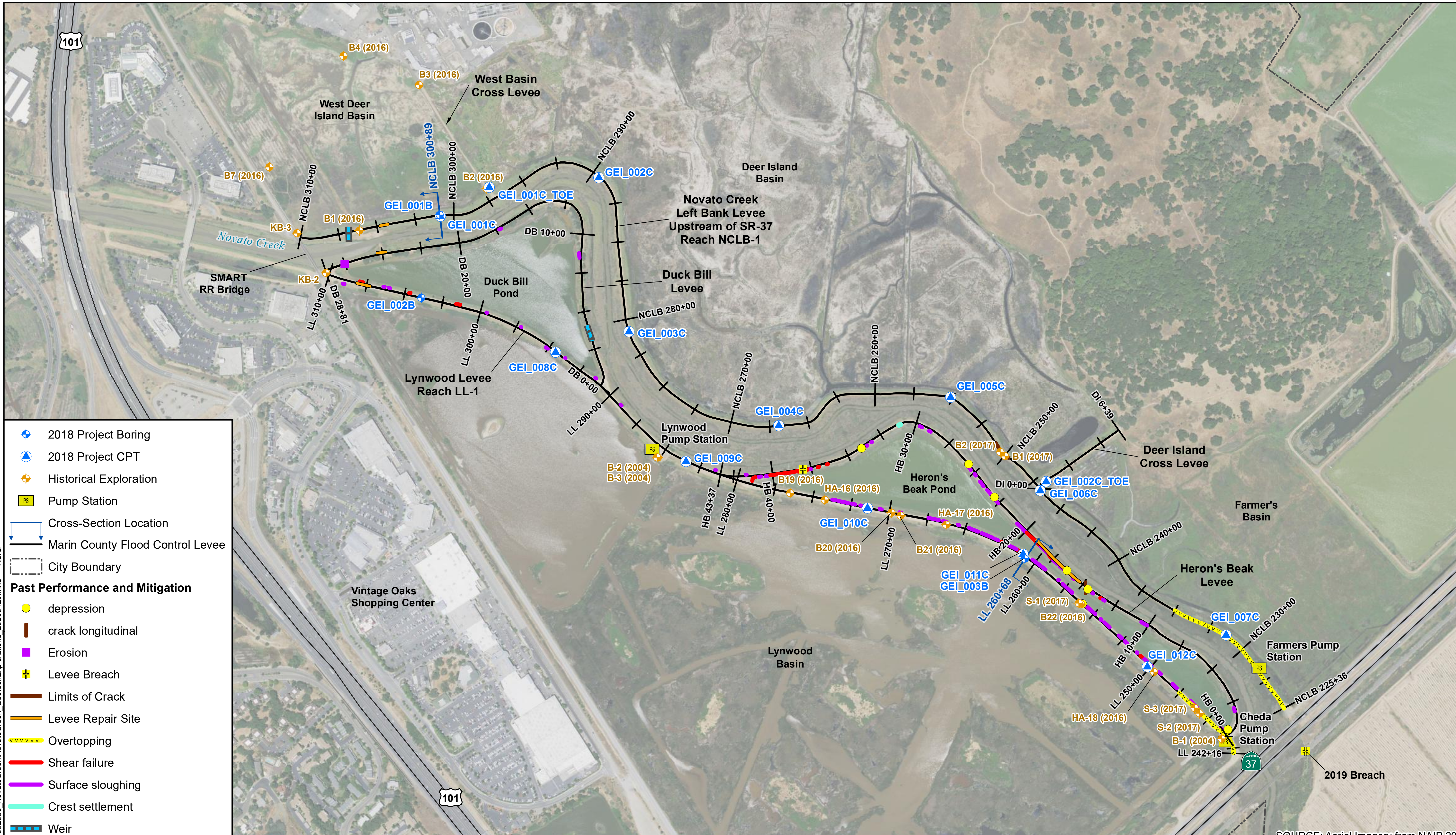
27Nov2019

Novato Creek Levee Evaluation Project Novato, California	GEI Consultants	Project Location
Marin County Flood Control and Water Conservation District		DECEMBER 2019 FIGURE 1-1



0 300 600 1,200
Feet

Novato Creek Levee Evaluation Project
Novato, California

Marin County Flood Control and Water Conservation District

GEI Consultants

GEOTECHNICAL EXPLORATIONS
NOVATO CREEK AND LYNWOOD BASIN LEVEES

JANUARY 2020

FIGURE 1-2

Z:\Projects\1802696_NovatoCreek\NovatoCreek_GeotechExplorations_20200120.mxd RS/SI 20-Jan-2020

0 250 500 1,000

Feet

Novato Creek Levee Evaluation Project
Novato, California

Marin County Flood Control
and Water Conservation District

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GEOTECHNICAL EXPLORATIONS
PACHECO POND LEVEE

JANUARY 2020

FIGURE 1-3

07-Jan-2020 Z:\Projects\1802696_NovatoCreek\PachecoPond_GeotechExplorations_20200107.mxd RS/SI

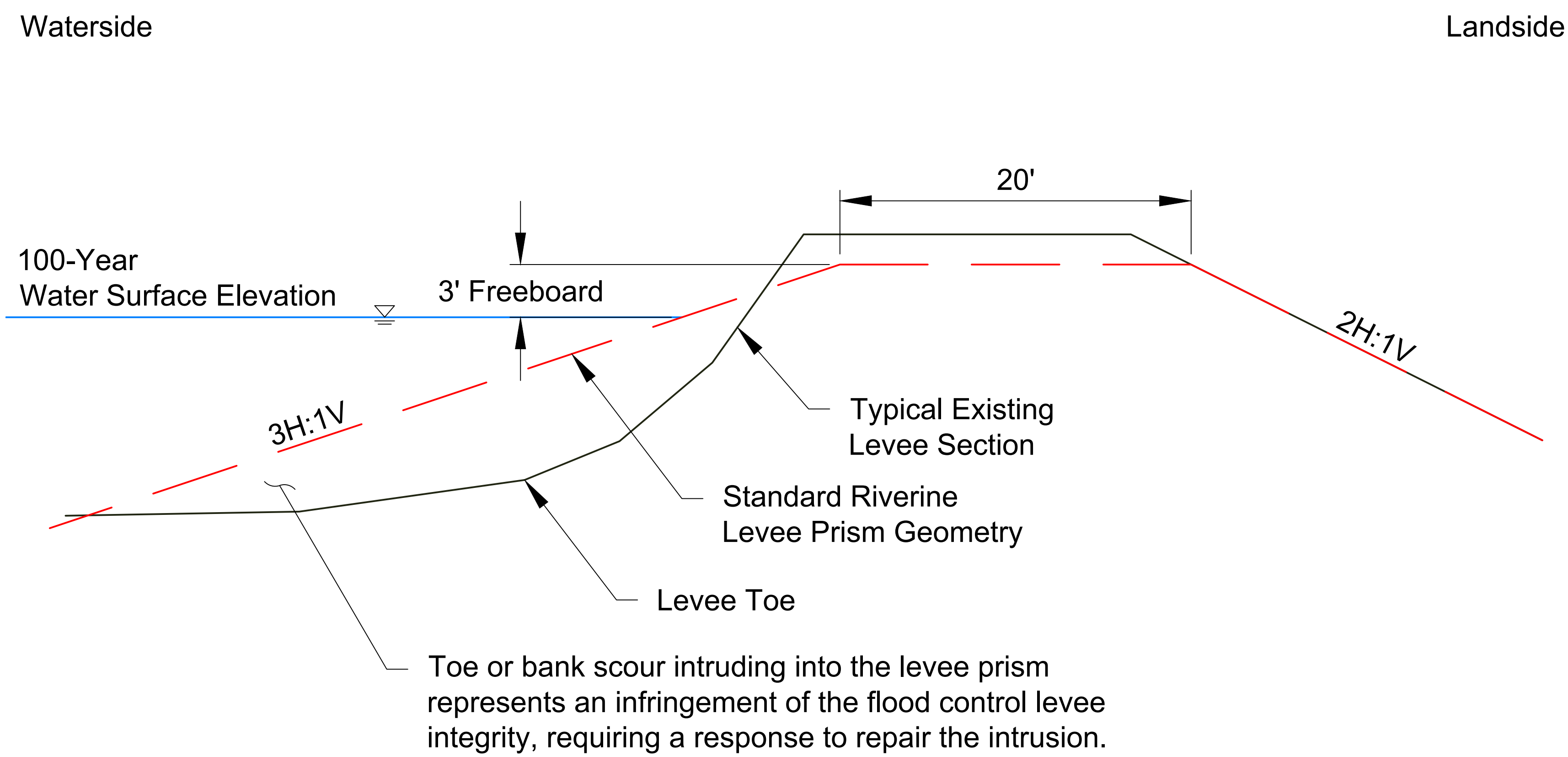

FIGURE 5-1. Standard Levee Prism Geometry within a Riverine Levee Section

Plates

27-Nov-2019 Z:\Projects\1802696_NovatoCreek\NovatoCreek_PlanView_mapbook.mxd RS

0 100 200 400
Feet

Novato Creek Levee Evaluation Project
Novato, California

Marin County Flood Control
and Water Conservation District

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NOVATO CREEK LEFT BANK LEVEE
EXPLORATION AND SITE PLAN
STATION NCLB 225+36 TO 248+00

DECEMBER 2019

PLATE 1

27-Nov-2019 Z:\Projects\1802696_NovatoCreek\NovatoCreek_PlanView_mapbook.mxd RS

0 100 200 400
Feet

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Novato, California

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NOVATO CREEK LEFT BANK LEVEE
EXPLORATION AND SITE PLAN
STATION NCLB 248+00 TO 276+00

DECEMBER 2019

PLATE 2

0 100 200 400
Feet

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Novato, California

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NOVATO CREEK LEFT BANK LEVEE
EXPLORATION AND SITE PLAN
STATION NCLB 276+00 TO 304+00

DECEMBER 2019

PLATE 3

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Feet

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N

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Novato, California

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and Water Conservation District

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NOVATO CREEK LEFT BANK LEVEE
EXPLORATION AND SITE PLAN
STATION NCLB 304+00 TO 310+00

DECEMBER 2019

PLATE 4

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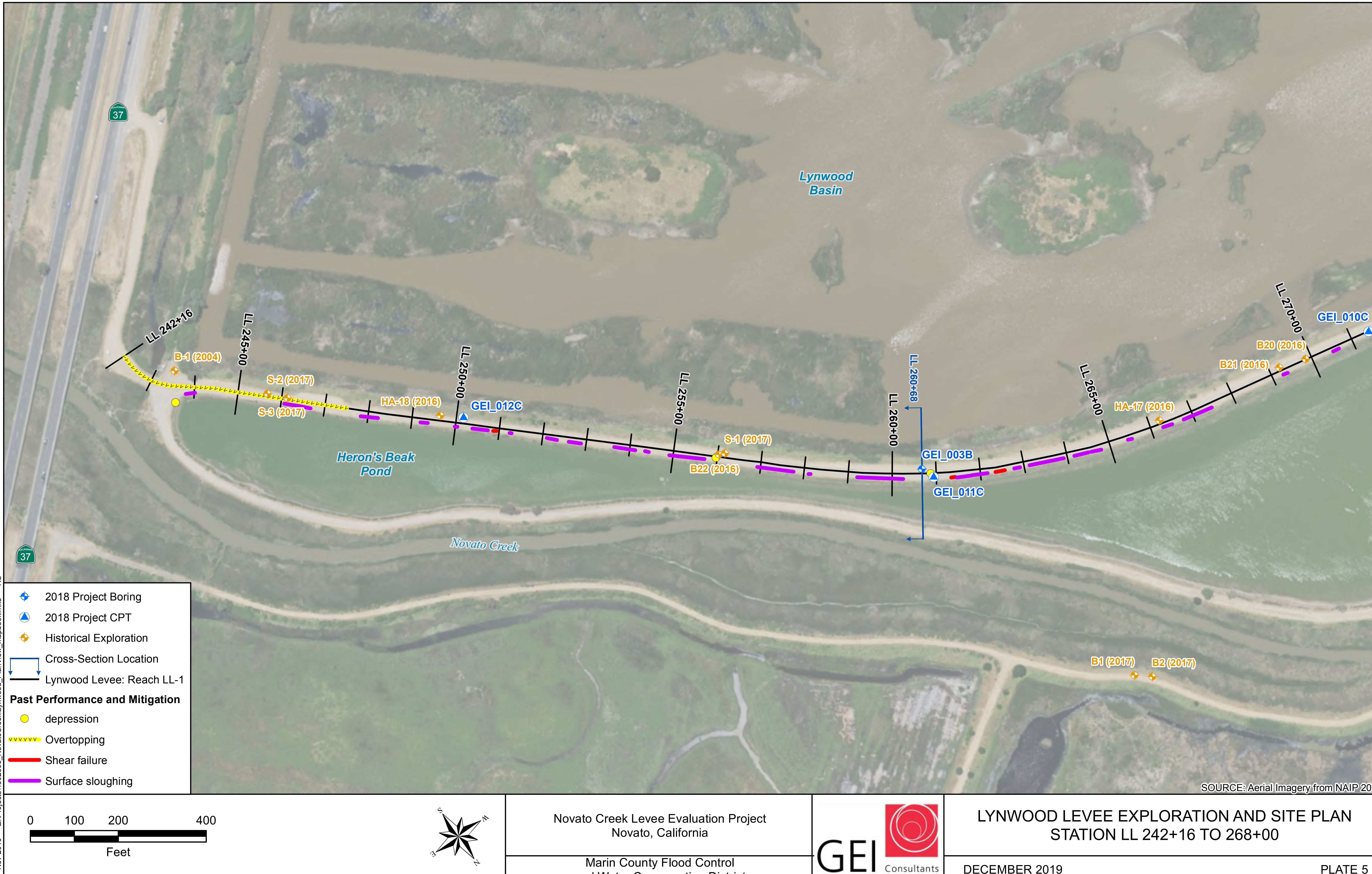

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Novato Creek Levee Evaluation Project
Novato, California

Marin County Flood Control
and Water Conservation District

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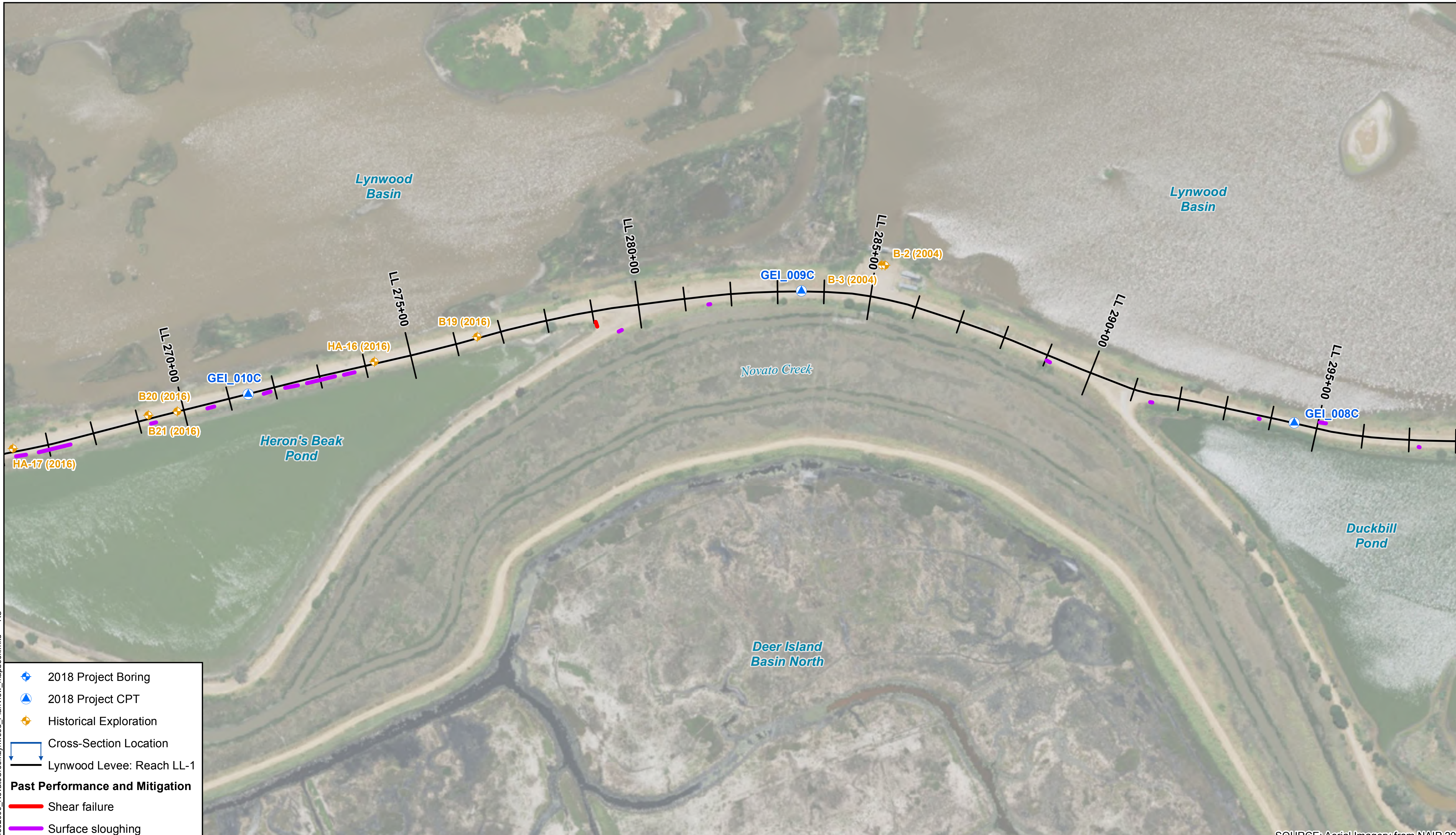
LYNWOOD LEVEE EXPLORATION AND SITE PLAN
STATION LL 242+16 TO 268+00

DECEMBER 2019

PLATE 5



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Novato, California

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LYNWOOD LEVEE EXPLORATION AND SITE PLAN
STATION LL 268+00 TO 296+00

DECEMBER 2019

PLATE 6

0 100 200 400

Feet

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LYNWOOD LEVEE EXPLORATION AND SITE PLAN
STATION LL 296+00 TO 310+00

DECEMBER 2019

PLATE 7

27-Nov-2019 Z:\Projects\1802696_NovatoCreek\Lynwood_PlanView_mapbook.mxd RS

0 100 200 400
Feet

Novato Creek Levee Evaluation Project
Novato, California

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PACHECO POND LEVEE EXPLORATION AND SITE PLAN
STATION PP 10+00 TO 38+00

DECEMBER 2019

PLATE 8

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0 100 200 400
Feet

Novato Creek Levee Evaluation Project
Novato, California

Marin County Flood Control
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PACHECO POND LEVEE EXPLORATION AND SITE PLAN
STATION PP 38+00 TO 43+90

DECEMBER 2019

PLATE 9

27-Nov-2019 Z:\Projects\1802696_NovatoCreek\PachecoPond_PlanView_mapbook.mxd RS

SOIL GROUP SYMBOLS AND NAMES

Graphic	Group Names	Graphic	Group Names	Graphic	Group Names
	Lean Clay (CL)		Lean Clay with Sand (CL)		Lean Clay with Gravel (CL)
	Sandy Lean Clay (CL)		Sandy Lean Clay with Gravel (CL)		Gravelly Lean Clay (CL)
	Gravelly Lean Clay with Sand (CL)		Fat Clay (CH)		Fat Clay with Sand (CH)
	Fat Clay with Gravel (CH)		Sandy Fat Clay (CH)		Sandy Fat Clay with Gravel (CH)
	Gravelly Fat Clay with Sand (CH)		Silt with Sand (ML)		Sandy Silt (ML)
	Elastic Silt (MH)		Sandy Elastic Silt (MH)		Peat (PT)
	Poorly Graded Sand (SP)		Poorly Graded Sand with Gravel (SP)		Poorly Graded Sand with Clay and Gravel (SP-SC)
	Poorly Graded Sand with Silt and Gravel (SP-SM)		Well-Graded Sand with Clay and Gravel (SW-SC)		Clayey Sand (SC)
	Clayey Sand with Gravel (SC)		Silty Clayey Sand (SC-SM)		Silty Sand (SM)
	Silty Sand with Gravel (SM)		Poorly Graded Gravel (GP)		Poorly Graded Gravel with Sand (GP)
	Poorly Graded Gravel with Clay and Sand (GP-GC)		Well-Graded Gravel (GW)		Well-Graded Gravel with Clay and Sand (GW-GC)
	Clayey Gravel (GC)		Clayey Gravel with Sand (GC)		Silty Gravel (GM)
	Silty Gravel with Sand (GM)				

ROCK SYMBOLS AND NAMES

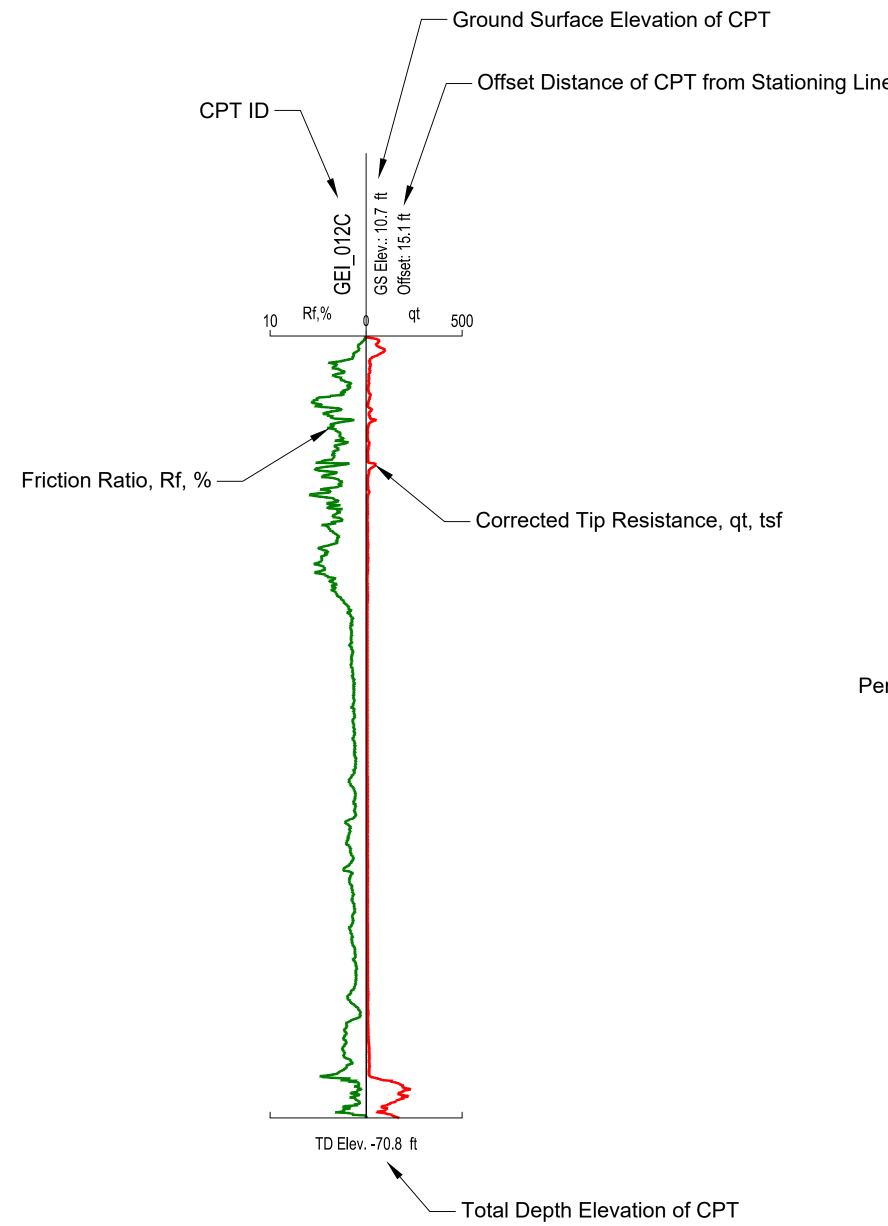
Graphic	Rock Type	Graphic	Rock Type	Graphic	Rock Type
	Claystone		Sandstone		Shale

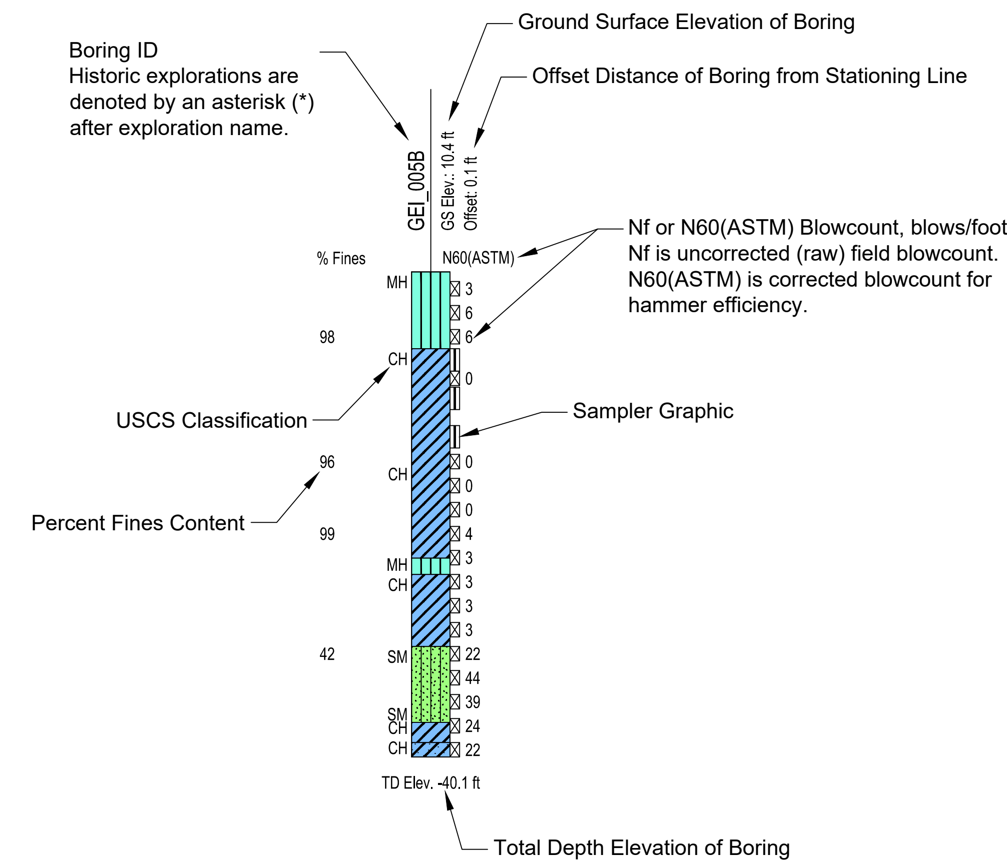
Note:

Soils were classified in the field in general accordance with ASTM D2488-06, Standard Practice for Description and Identification of Soils (Visual Manual Procedure). Where laboratory testing was performed, classifications were modified in general accordance with ASTM D2487-06, Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System).

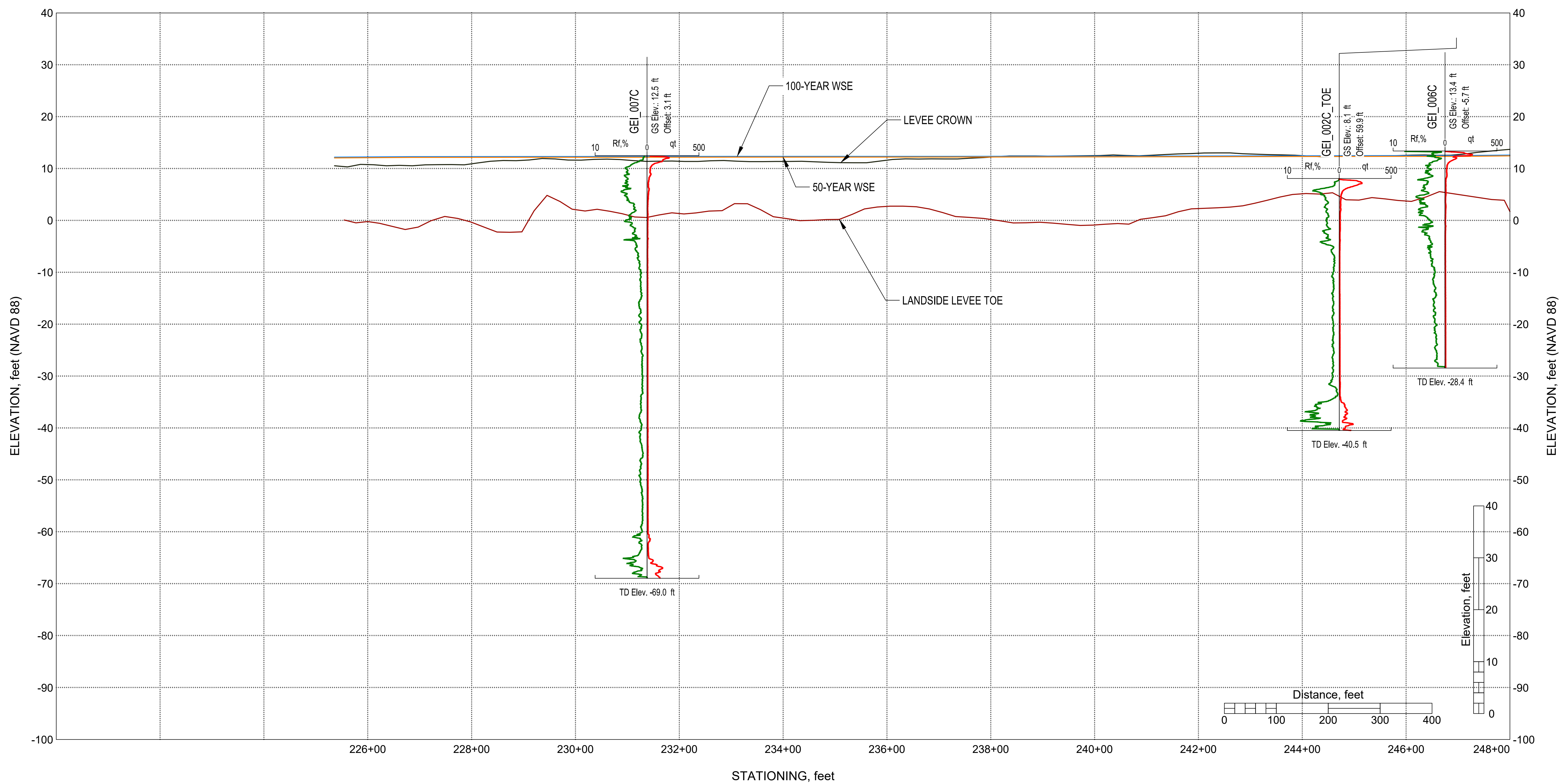
SAMPLER GRAPHIC SYMBOLS

Standard Penetration Test (1.4 in. ID)	Standard California (2.5 in. ID)
Modified California (2.0 in. ID)	Shelby Tube (2.87 in. or 2.37 in. ID)
Rock Core	Bag Sample





Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**PROFILE LEGEND**
Marin County Flood Control and Water Conservation District		December 2019 — Project #1802696 — Plate 10



Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

PROFILE ALONG NOVATO CREEK LEVEE
STATION NCLB 225+36 TO 248+00

December 2019 | Project #1802696 | Plate 11

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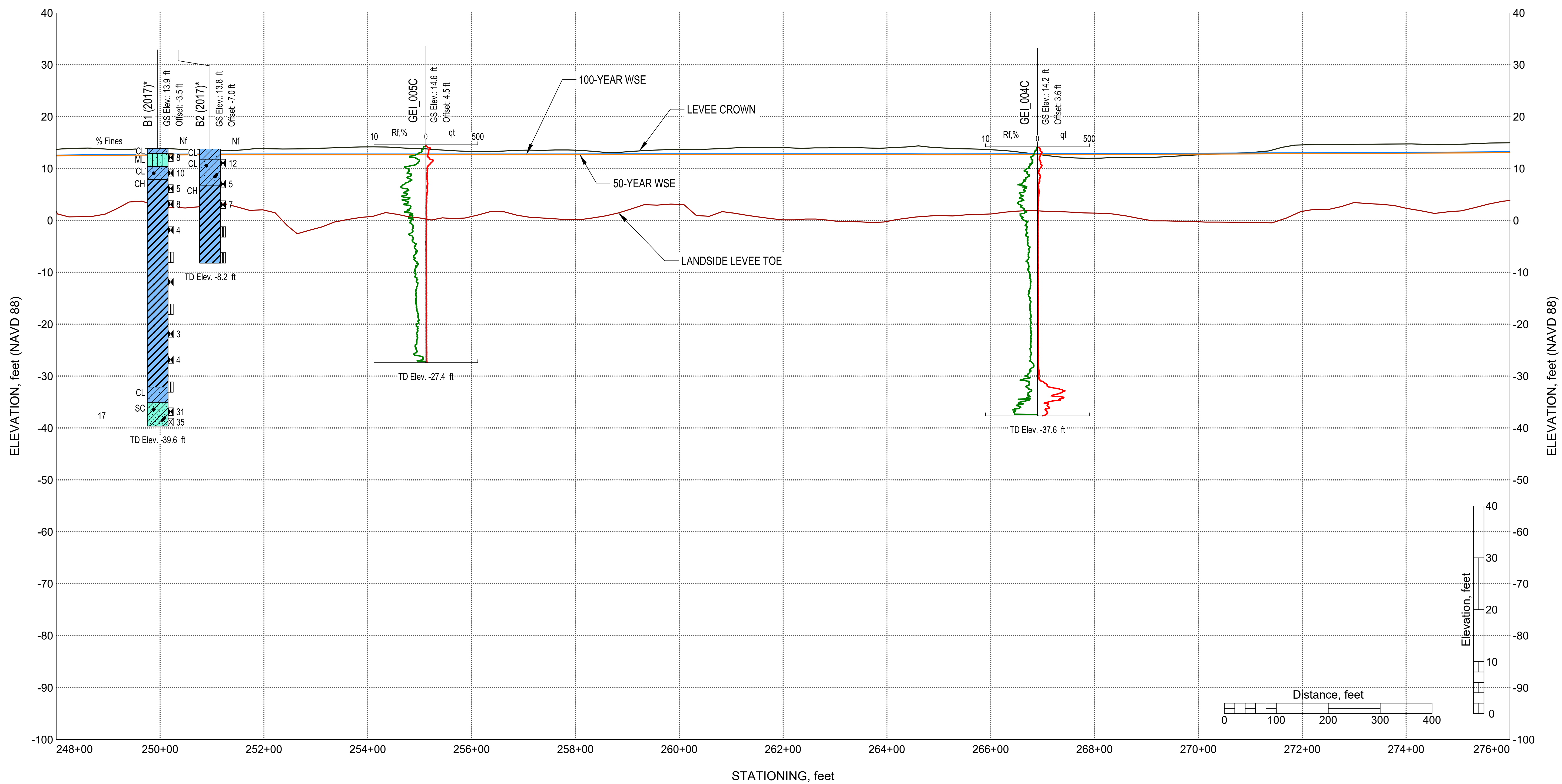

Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

PROFILE ALONG NOVATO CREEK LEVEE
STATION NCLB 248+00 TO 276+00

December 2019 Project #1802696 Plate 12

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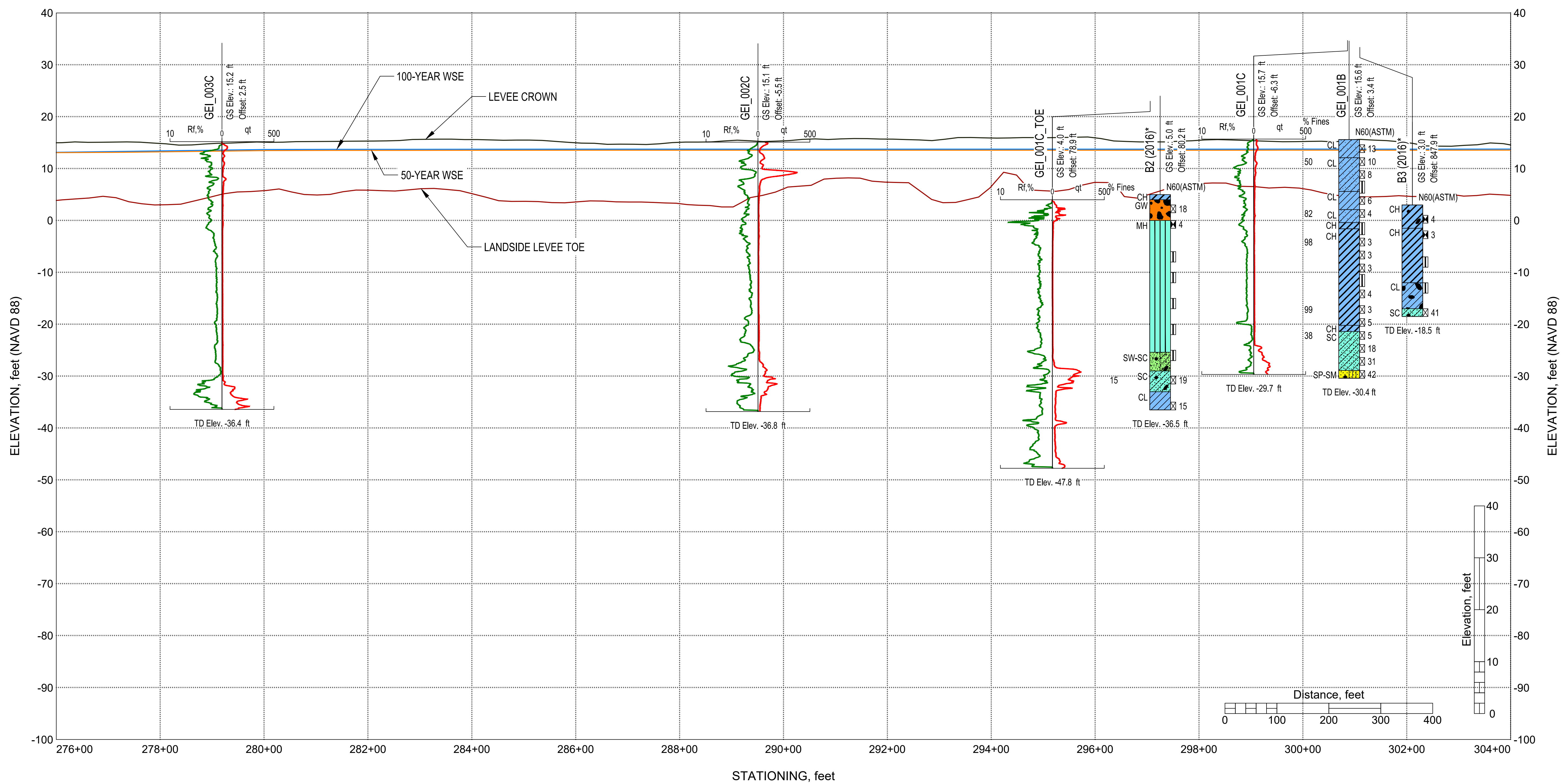


Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

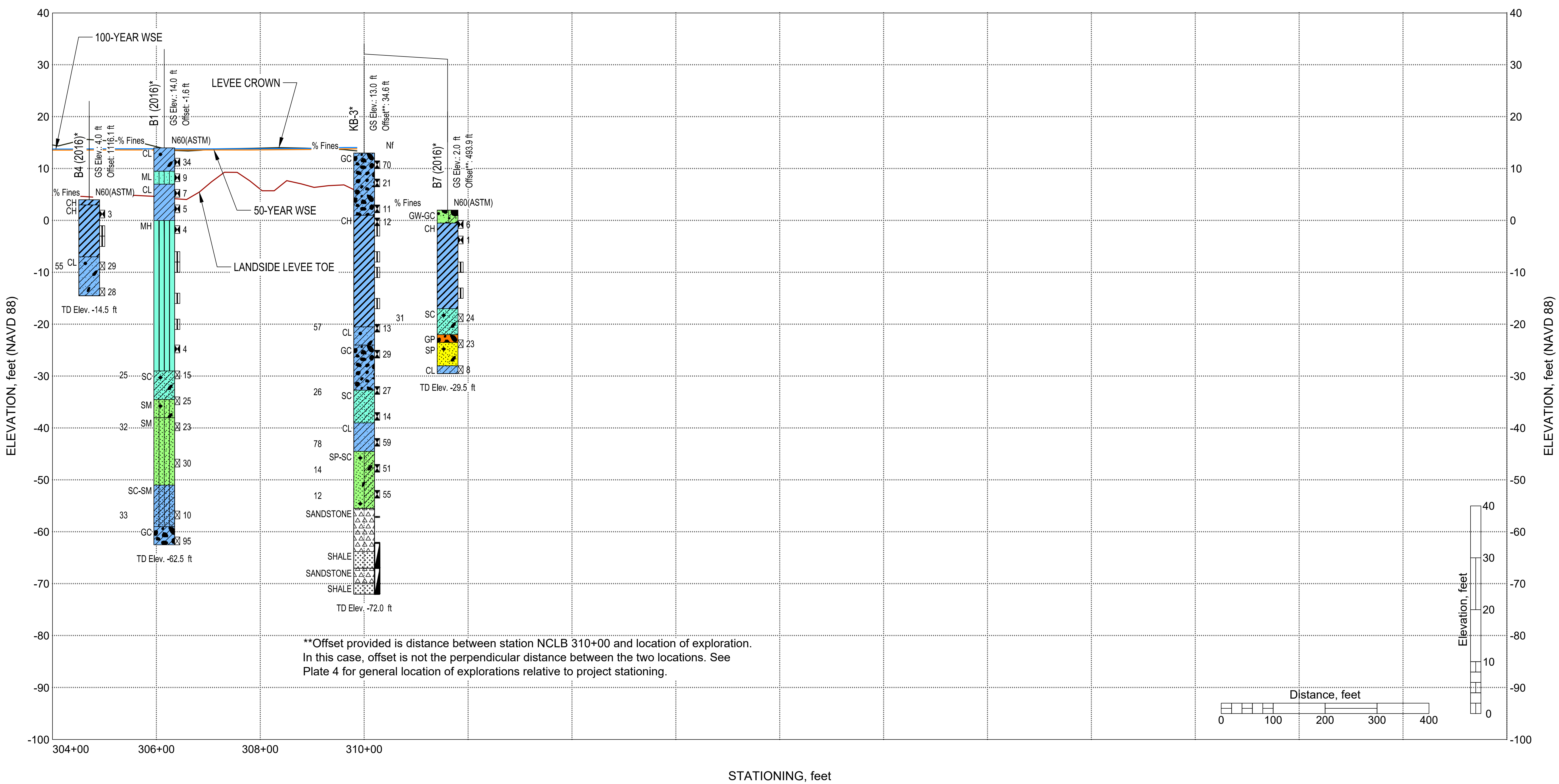
Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

PROFILE ALONG NOVATO CREEK LEVEE
STATION NCLB 276+00 TO 304+00

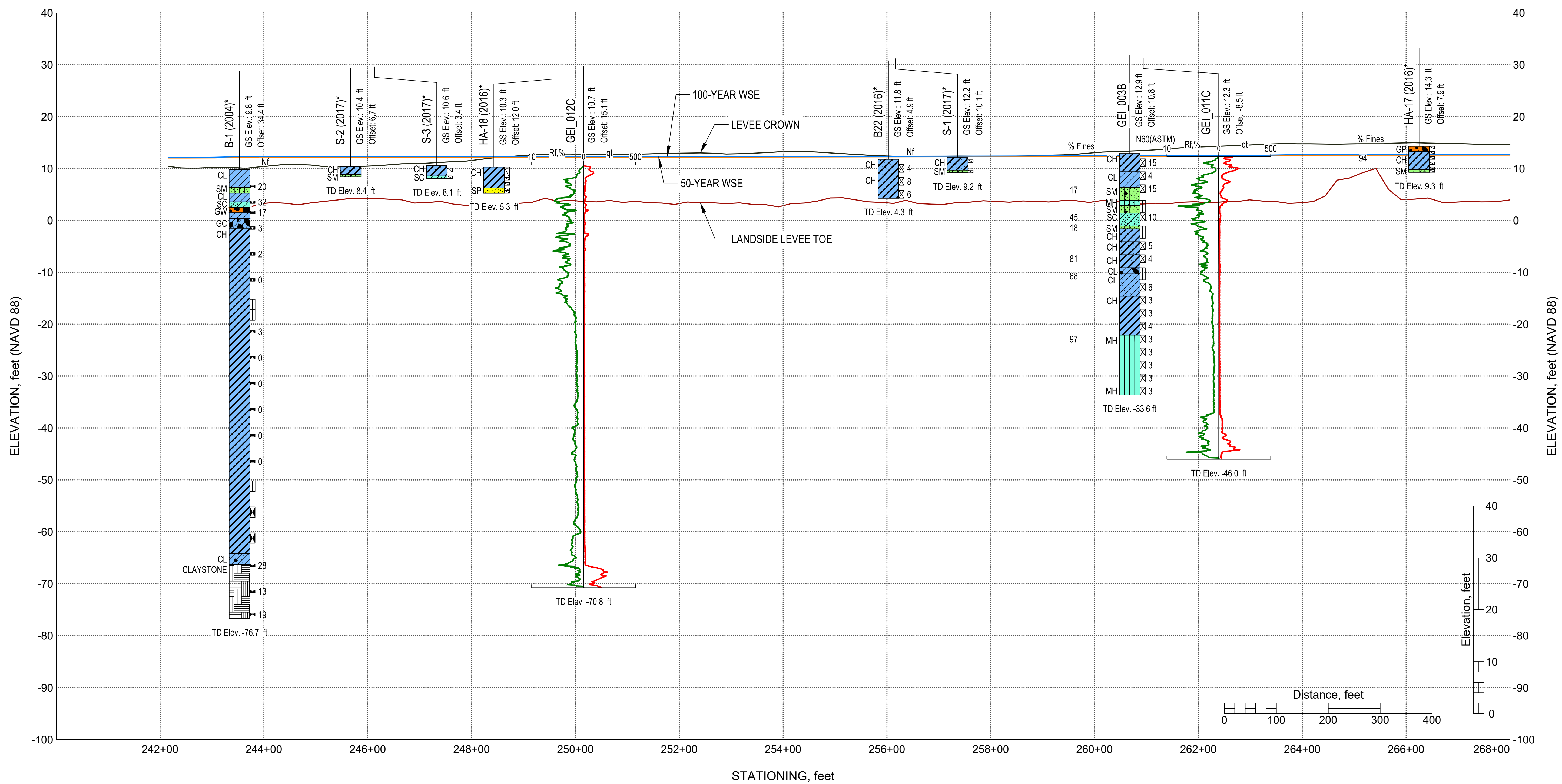
December 2019 | Project #1802696 | Plate 13



Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**PROFILE ALONG NOVATO CREEK LEVEE** STATION NCLB 304+00 TO 310+00
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Plate 14



Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**PROFILE ALONG LYNWOOD LEVEE** STATION 242+16 TO 268+00
Marin County Flood Control and Water Conservation District		December 2019 / Project #1802696 / Plate 15

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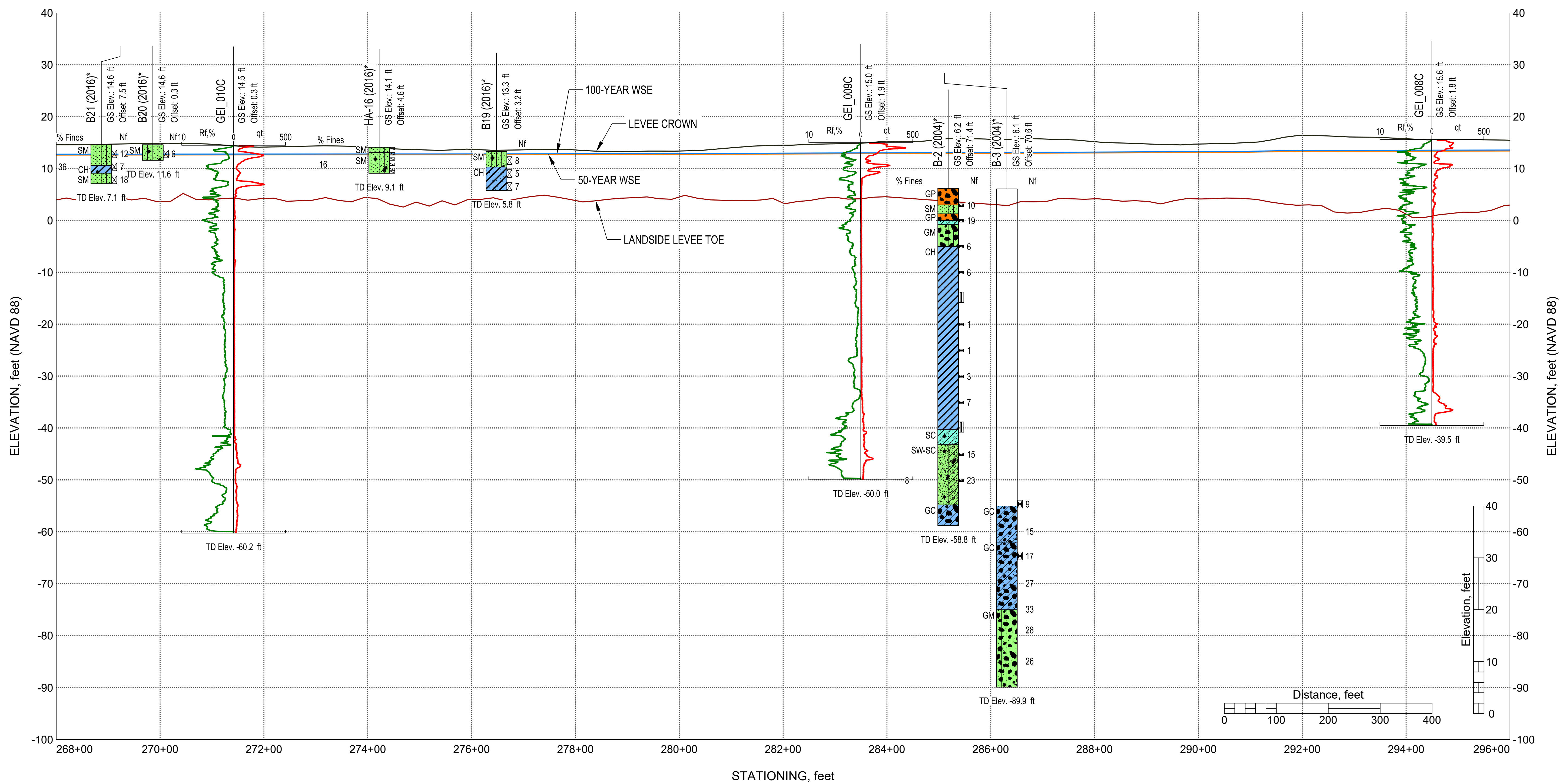
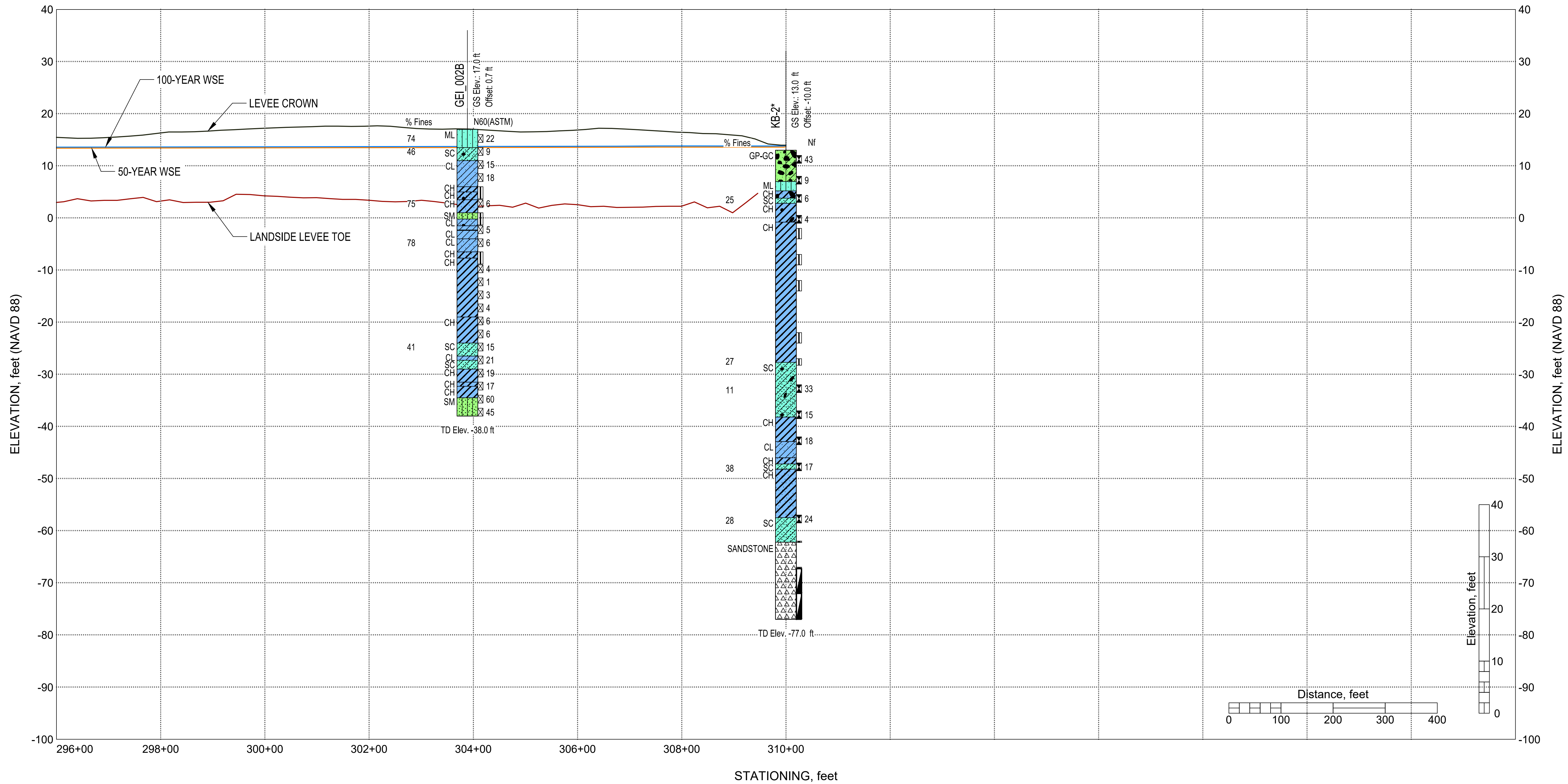


Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**PROFILE ALONG LYNWOOD LEVEE** STATION 268+00 TO 296+00
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Plate 16



Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

PROFILE ALONG LYNWOOD LEVEE
STATION 296+00 TO 310+00

December 2019 | Project #1802696 | Plate 17

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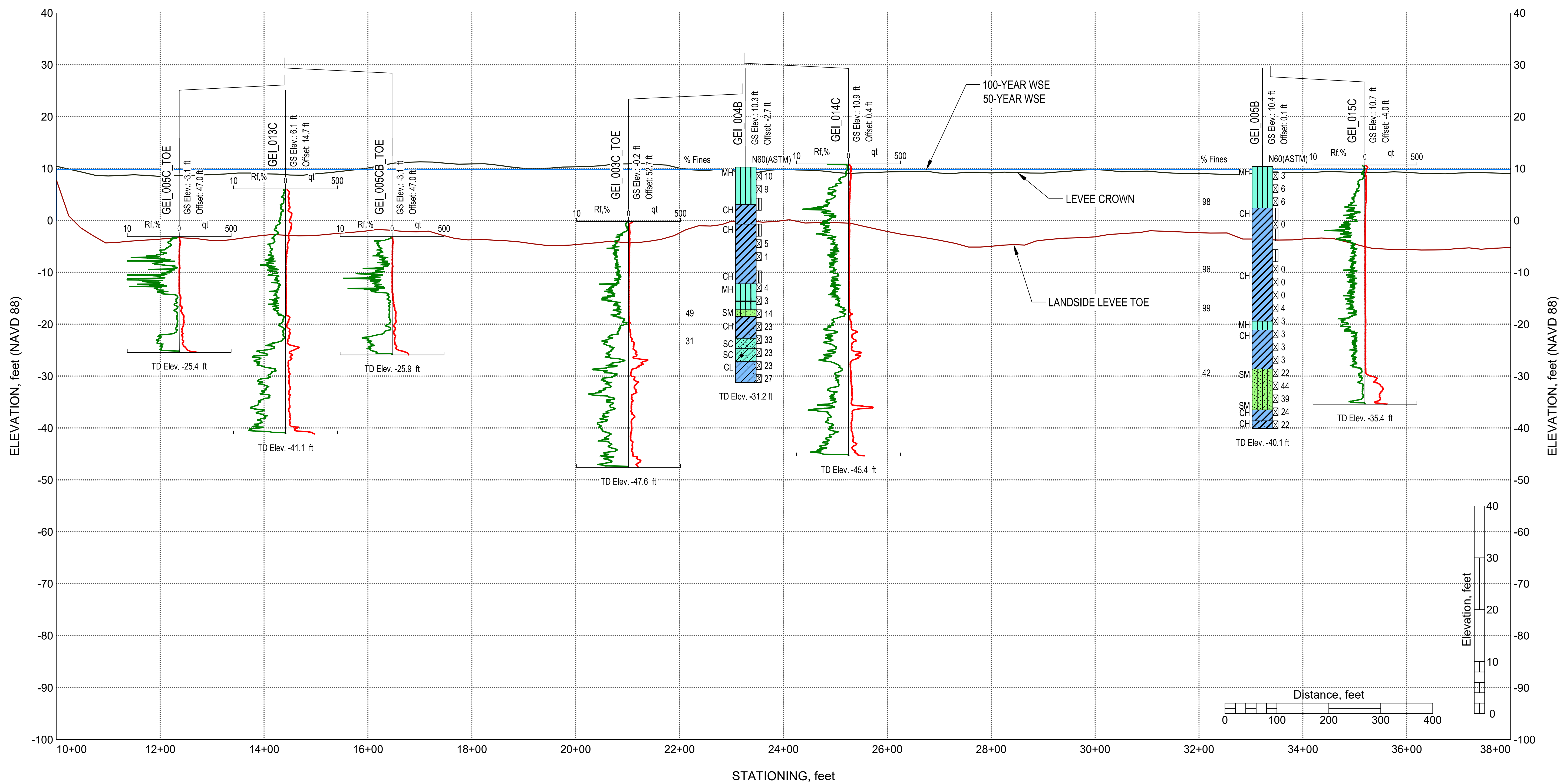

Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

PROFILE ALONG PACHECO POND LEVEE
STATION 10+00 TO 38+00

December 2019	Project #1802696	Plate 18

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Notes:

1. Levee crown and landside toe elevations are approximate. All toe data (including toe line) has been projected to the levee crown stationing alignment. Due to the curvature of the levee this projection causes distances shown to vary from actual spacing of toe features.
2. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations and profiles for these records are based on available information. USCS classifications may be interpreted from the available information.
3. Exploration offset is the perpendicular distance between project stationing and location of exploration. A positive offset indicates exploration is located on the landside of the levee while a negative offset indicates exploration is located on the waterside.

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

PROFILE ALONG PACHECO POND LEVEE
STATION 38+00 TO 43+90

December 2019 | Project #1802696 | Plate 19

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Appendix A
Development of Hydraulic Conductivities for Analysis

Appendix A-Development of Hydraulic Conductivities for Analysis

Purpose

To select initial analytical hydraulic conductivity parameters for the soil layers at analysis cross sections using current and historical field and lab testing data and published references.

References

California Department of Water Resources (DWR) (2014), Guidance Document for Geotechnical Analyses, Urban Levee Evaluations Project, Version 14, prepared by URS Corp.

Demetrious Koutsoftas Geotechnical Consultants, Inc. (DKGC) (2010), In-Situ Permeability Testing Bay Area Experience for California Department of Water Resources.

Demetrious Koutsoftas Geotechnical Consultants, Inc. (DKGC) (2013), Summary of the results of Hydraulic Conductivity Tests and Recommendations for Revision of the Presumptive Hydraulic Conductivity Values for Seepage Analyses.

Terzaghi, K., Peck R., Mesri (1996), Soil Mechanics in Engineering Practice Third Edition, Wiley Interscience Publications.

USACE (1993), Engineer Manual 1110-2-1901 Seepage Analysis and Control for Dams, Department of the Army U.S. Army Corps of Engineers, Washington, DC.

Approach

Hydraulic conductivities for seepage analyses for the Novato Creek Levee Evaluation Project (Project) were developed based on review of laboratory and in-situ test results for soils in the San Francisco Bay Area and review of geotechnical literature on hydraulic conductivities. The data sources and procedures for examining the data are described in detail in the sections below.

Data Sources

Several data sources were reviewed to develop the initial hydraulic conductivities for analysis. The test data sources are described below.

DWR ULE Special Testing Program

A database of laboratory-measured hydraulic conductivities was developed by DKGC (2013) as part of the California Department of Water Resources (DWR) special testing program for the Urban Levee Evaluation (ULE) project. The database consists of 302 hydraulic conductivity tests on intact samples taken from DWR ULE study areas on samples of relatively clean sands (SP, SP-SM, SP-SC), silty sands (SM, SM-SC), sandy silt, clayey silt, and silts (ML, CL-ML), clayey sands (SC), clays (CL, CH), and organic soils. The results of the hydraulic conductivity tests were summarized by DKGC (2013). Soil index test results, estimates of in situ stresses, and soil classifications were provided for each sample. The measured hydraulic conductivities were plotted against fines content (percent passing the No. 200 sieve) in a series of plots for different soil types.

Laboratory-measured hydraulic conductivities were performed with flexible wall permeameters, in accordance with ASTM D5084-00. Constant head tests were performed on samples expected to have hydraulic conductivities greater than 1.0×10^{-3} cm/sec and falling head tests were performed on samples with hydraulic conductivities less than or equal to 1.0×10^{-3} cm/sec.

San Francisco Bay Specific DWR Laboratory and In-Situ Testing Program

Additionally, in-situ and laboratory permeability testing of soils in the San Francisco Bay Area was summarized by DKGC (2010) for DWR and discussed further in the ULE Guidance Document (2014). Relevant in-situ testing was performed on over 20 locations for the Downtown Extension Project and Transbay Transit Center Project on Colma, Marine, and Dune Sands with fines content ranging from 0% to 49%. An additional five falling head tests for horizontal conductivity were performed on Sandy Bay Muds and Bay Muds. Additionally, the ULE Guidance Document (2014) notes that incremental loading (IL) consolidation tests were performed by Dames and Moore in 1989 on samples of San Francisco Bay Mud. Dames and Moore concluded hydraulic conductivity values corresponding with the highest void ratio (representing in-situ conditions for Bay Mud) ranged between 1×10^{-7} and 4×10^{-7} cm/sec.

Selection of Initial Hydraulic Conductivities for Analysis

The initial hydraulic conductivities selected for analysis in the Project were developed by reviewing available test data and established literature (Terzaghi, Peck, Mesri, 1996 and USACE, 1993). The steps for selecting initial hydraulic conductivities are described below.

The initial hydraulic conductivities for analysis, shown in Figure A-1 and Table A-1, were developed by examining the DWR ULE laboratory-measured conductivity plots developed by DKGC (2013) as shown on the attached Figures A-2 to A-6, and comparing (or adjusting when necessary) these assigned values to San Francisco Bay specific laboratory and in-situ testing as shown on Figure A-1. Figures are titled as shown below:

Figure A-1: Assigned Vertical Hydraulic Conductivities versus San Francisco Bay Specific Testing and Established Literature

Figure A-2: Vertical Hydraulic Conductivities – SP and SP-SM (0 to 12% fines)

Figure A-3: Vertical Hydraulic Conductivities –SM (12 to 49% fines)

Figure A-4: Vertical Hydraulic Conductivities – SP-SC, SC, and SC-SM

Figure A-5: Vertical Hydraulic Conductivities – ML, CL-ML, and CL

Figure A-6: Vertical Hydraulic Conductivities – San Francisco Bay Mud

In Figures A-2 through A-5, the laboratory test results are plotted as individual data points. Additional annotations on the plots include the approximate boundaries of the dataset (shown as dashed lines) and trendlines with project-specific assigned hydraulic conductivity values shown as red points indicating initial values for analysis. In Figure A-6, the test results are also plotted as individual data points with additional annotations on the plot illustrating the analytical value chosen for in-situ bay mud.

The DWR ULE test database created by DKGC (2013) were compared to the San Francisco Bay specific test results mentioned above. The range of hydraulic conductivities for the Dune Sand, Marine Sand, and Colma Sand appear in reasonable agreement with the assigned values based on ULE test data. The Bay Mud data is shown to have a lower hydraulic conductivity than the other CL and CH materials presented in Figure A-4. Therefore, the vertical hydraulic conductivity value for intact natural deposits of CL and CH material was adjusted to 1E-7 cm/sec to better match the hydraulic conductivities expected at the Project site. Otherwise, the complete DWR ULE hydraulic conductivity database was considered for developing initial hydraulic conductivities for analyses.

The selected initial hydraulic conductivities and conductivity ratios for analysis are presented in Table A-1. The table is separated into groups based on soil type, fines content, and plasticity of fines (where applicable). Further considerations included the location of the materials (embankment or foundation), quality of material placement (controlled or uncontrolled placement), and the potential for defective fine-grained blankets due to desiccation or penetrations.

Table A-1. Novato Creek Hydraulic Conductivities Summary

Material Type	Soil Type	Soil Description		k_v (cm/sec)	k_v (ft/day)	k_h/k_v (Note 1)	k_h (cm/sec)	k_h (ft/day)
Clay	CL CL-ML CH	Embankment		1.0.E-06	2.83E-03	4	4.0.E-06	1.13E-02
		Natural Deposits - Shallow (< 10 ft) Dessicated or Damaged		2.5.E-06	7.09E-03	4	1.0.E-05	2.83E-02
		Natural Deposits- Intact (> 10 ft)		1.0.E-07	2.83E-04	2	2.0.E-07	5.67E-04
Silt (70-100% fines)	ML, MH	Embankment		5.0.E-06	1.42E-02	4	2.0.E-05	5.67E-02
		Natural Deposits		5.0.E-06	1.42E-02	4	2.0.E-05	5.67E-02
	SM	Natural Deposits / Embankment - Uncontrolled Placement	12-25% fines	5.0.E-04	1.42E+00	4	2.0.E-03	5.67E+00
			25-35% fines	1.5.E-04	4.25E-01	4	6.0.E-04	1.70E+00
			35-49% fines	4.0.E-05	1.13E-01	4	1.6.E-04	4.54E-01
		Embankment - Controlled Placement	12-49% fines	3.0.E-05	8.50E-02	4	1.2.E-04	3.40E-01

Notes: 1) Anisotropy ratios may be adjusted to account for the effects of interbedding or other environmental considerations.

Figure A-1. Assigned Vertical Hydraulic Conductivities versus San Francisco Bay Specific Testing and Established Literature

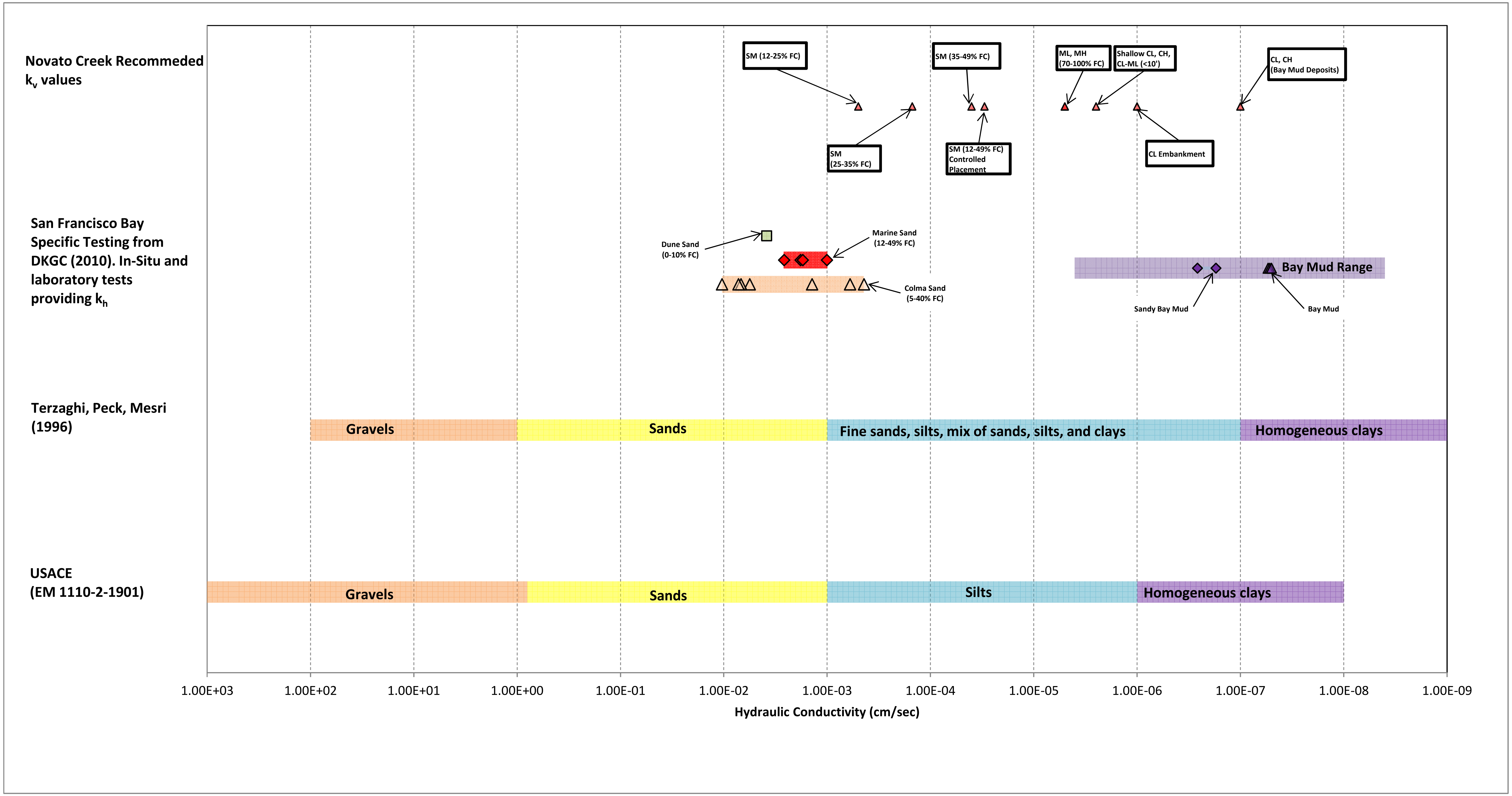
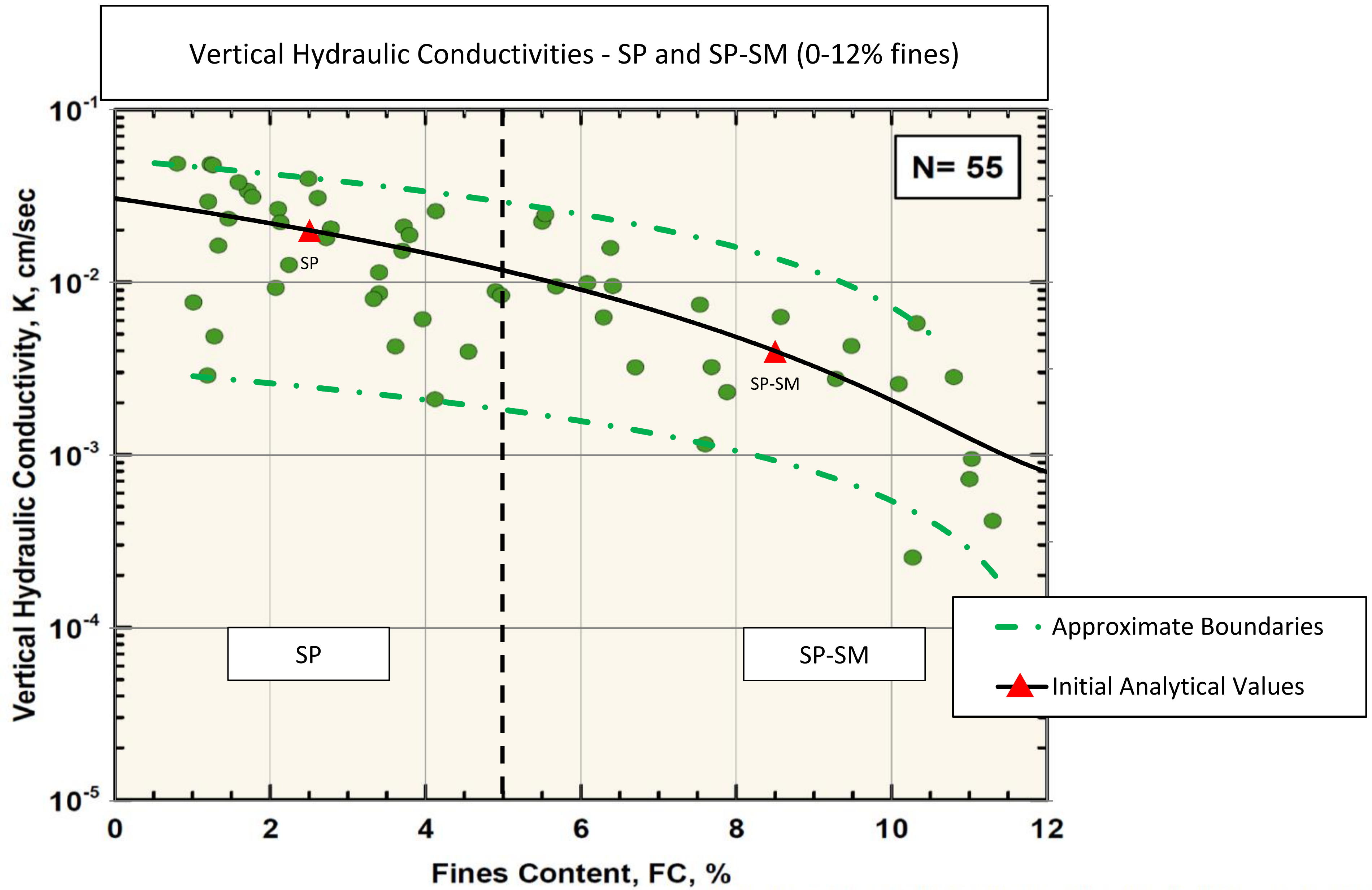


Figure A-2. Vertical Hydraulic Conductivities – SP and SP-SM (0 to 12% fines)

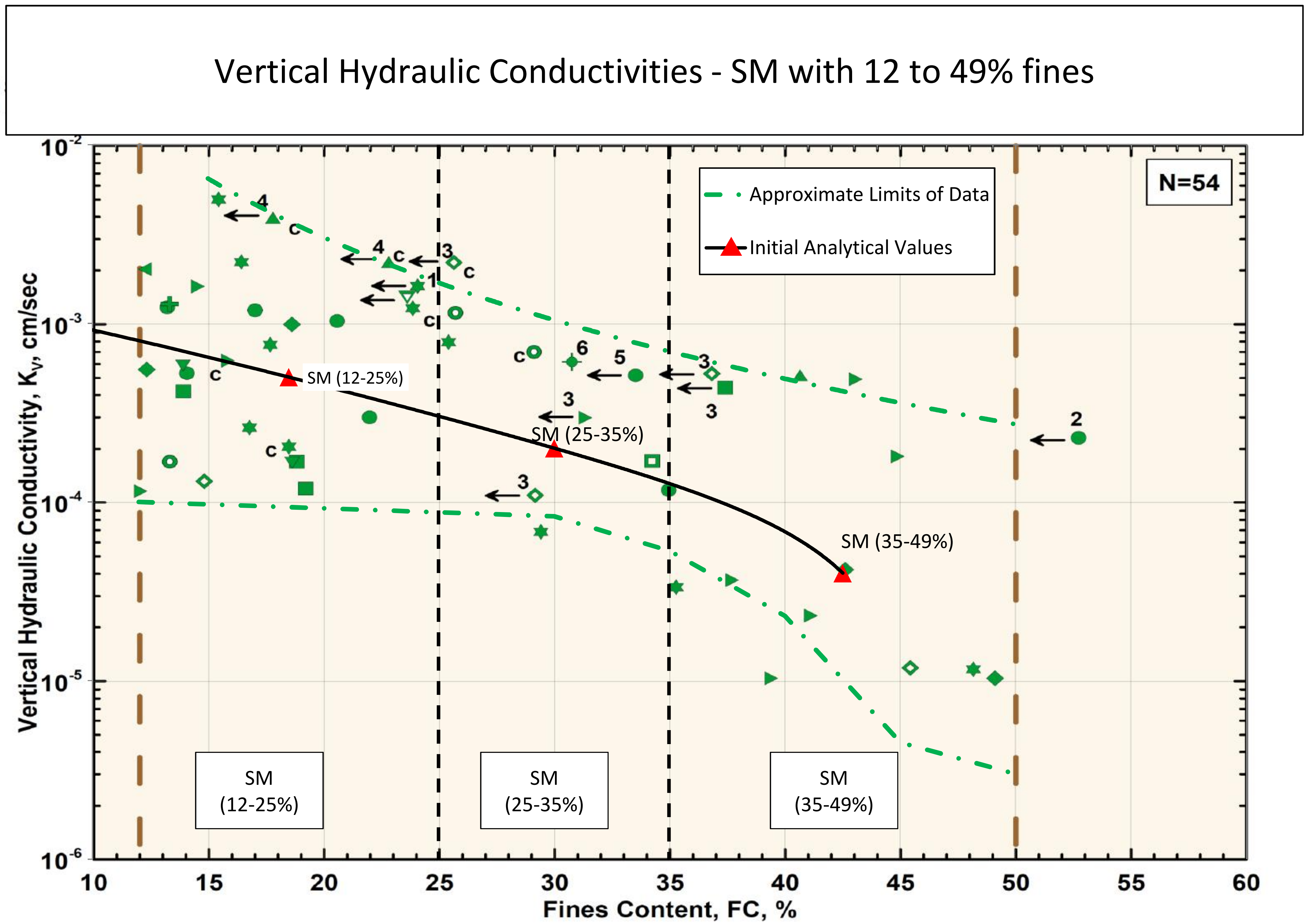


VARIATION OF HYDRAULIC CONDUCTIVITY WITH FINES CONTENT: RESULTS OF TESTS ON HOMOGENEOUS, RELATIVELY CLEAN SANDS (SP, SP-SM): ULE PROGRAM

DWR Levee Evaluation Program
ULE Special Testing Program
DWR Guidance Document
California Department of Water Resources
November 2013 Sacramento, CA

Q:\132217\4 Internal Project Data\4-05 Reports & Narratives\2013-01 Appendix A-2 DWR Guidance Document\Appendix A-2 New Version\Figures\K vs FC Clean Sands.grf

Figure A-3. Vertical Hydraulic Conductivities –SM (12 to 49% fines)



Symbol	Study Area
✕	Walthall Slough
◀	Boggs Tract
▼	Woodland
▽	Davis
◻	Shima Tract
△	SJAFCA (Non-Project Area)
◇	Smith Canal
✚	NEMDC
○	Rough and Ready Island
●	Bear Creek
■	Brookside
▲	Calaveras River
◆	Lincoln Village
✸	RD 17
✛	South Sacramento Streams
▶	Sacramento River
c	Other tests performed using the constant head method

Special conditions of numbered tests:

1. The fines content is affected by the presence of small clay lumps in the test specimen (SP-SM with small clay lumps)
2. Sand with a wedge of cohesive material at the base
3. SM soil with pockets of fine grained soil
4. Sand soil with pockets of fine grained soil
5. Silty sand with pockets of fine grained soil causing higher fines content than the bulk of the sample
6. Lumps of cohesive material causing higher fines content and higher plasticity than the bulk of the specimen

Note:
Fines Content = Percent Passing the No. 200 Sieve

VARIATION OF VERTICAL HYDRAULIC CONDUCTIVITY WITH FINES CONTENT: SILTY SANDS (SM)

DWR Levee Evaluation Program
ULE Special Testing Program
DWR Guidance Document
California Department of Water Resources
November 2013 Sacramento, CA

Figure A-4. Vertical Hydraulic Conductivities – SP-SC, SC, and SC-SM

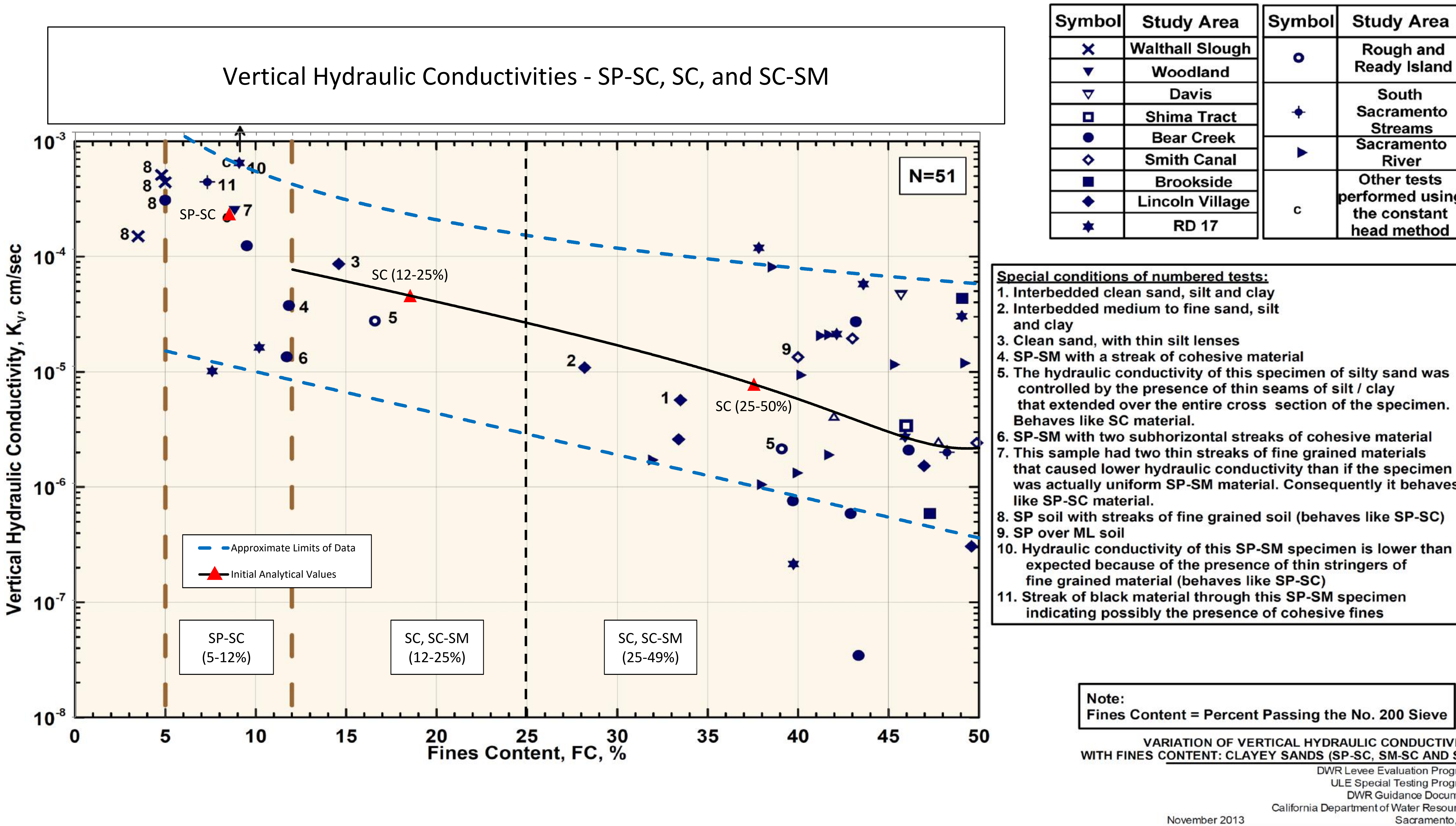


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Figure A-5. Vertical Hydraulic Conductivities – ML, CL-ML, and CL

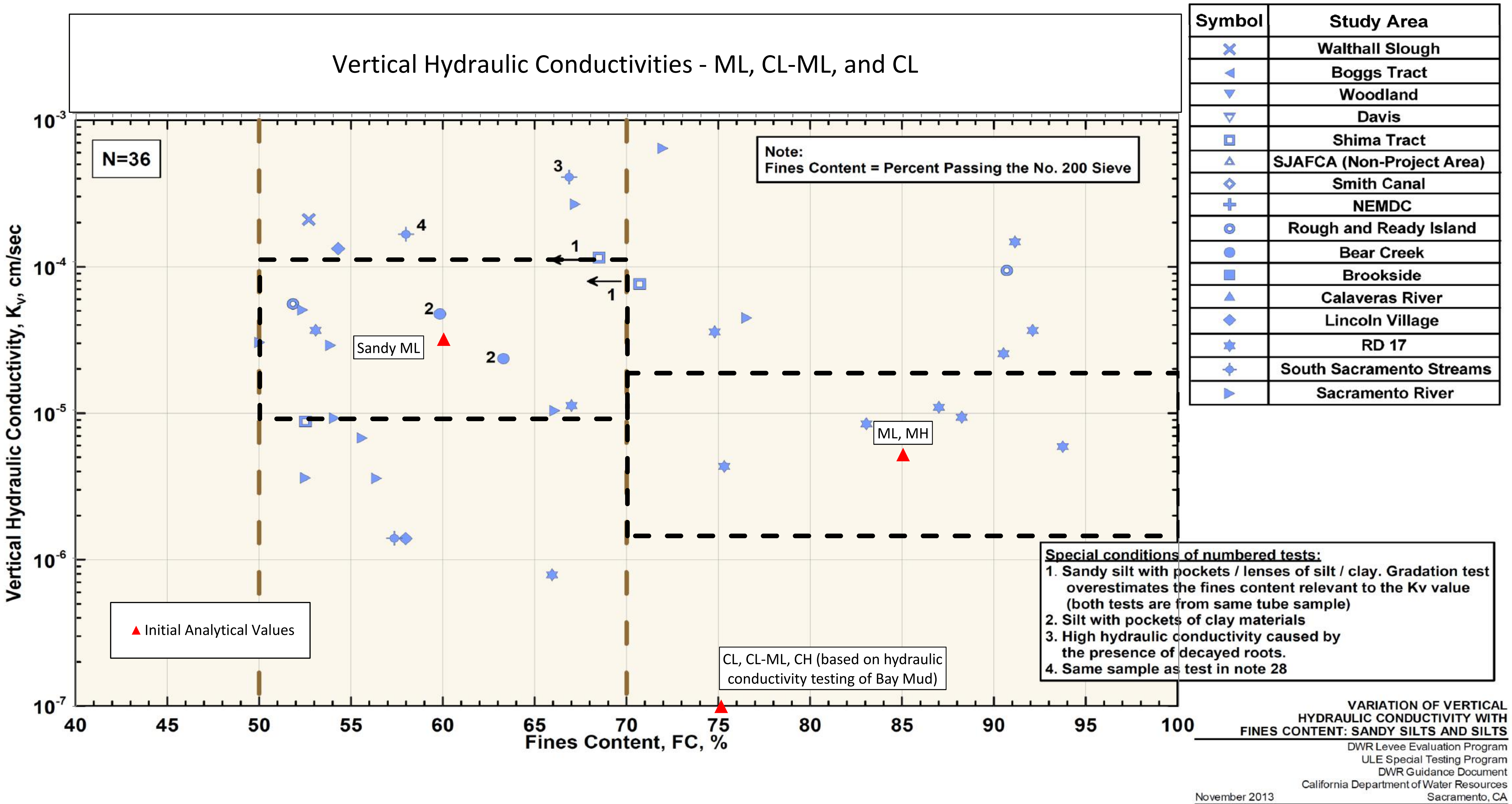

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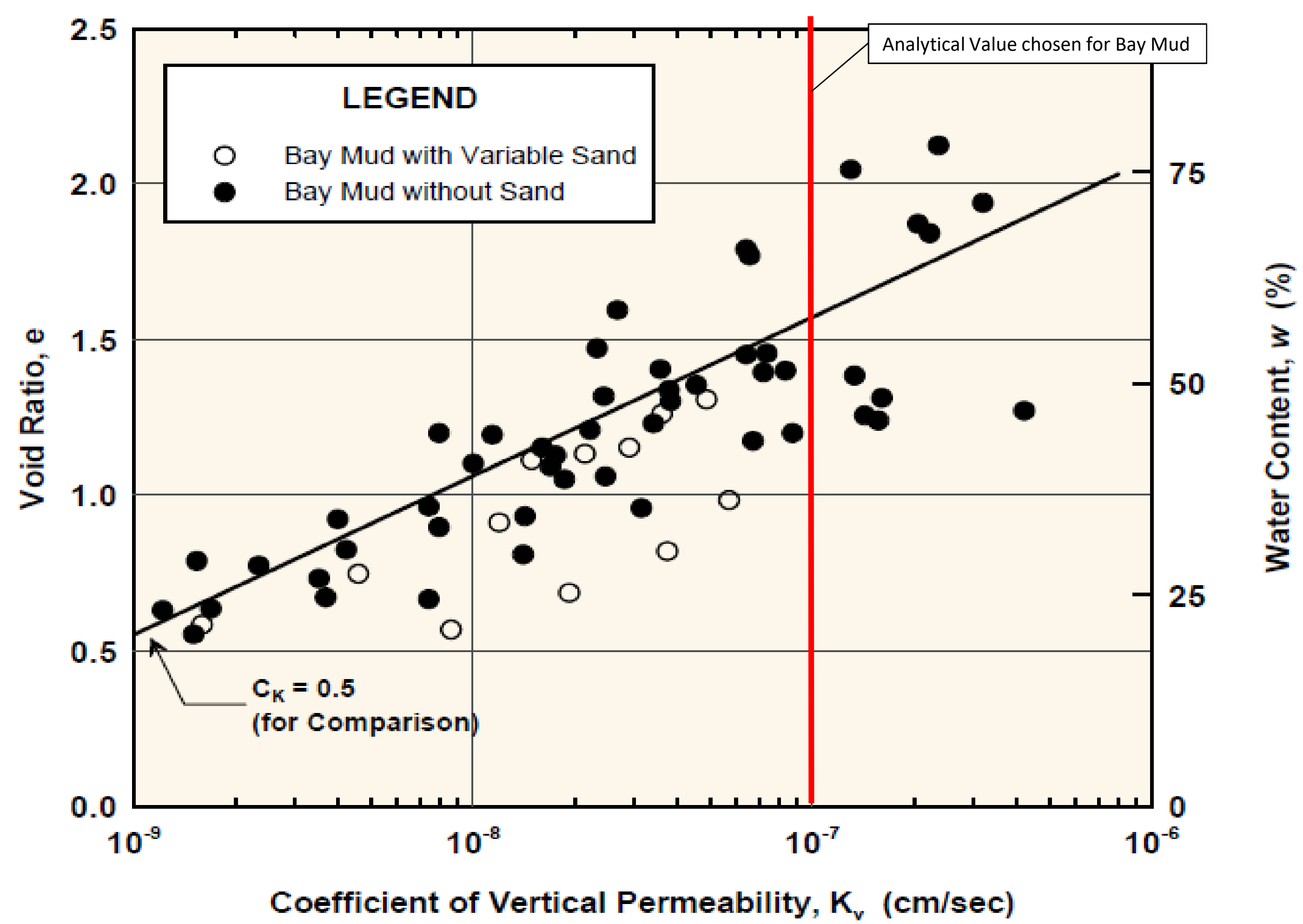
Figure A-6. Vertical Hydraulic Conductivities – San Francisco Bay Mud

Vertical Hydraulic Conductivities - Bay Mud



HYDRAULIC CONDUCTIVITY OF BAY MUD ESTIMATED
FROM INCREMENTAL LOADING CONSOLIDATION TESTS

DWR Levee Evaluation Program
Guidance Document - Appendix A
California Department of Water Resources
August 2011 Sacramento, CA

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Appendix B
Development of Soil Strength Parameters for Analysis

Appendix B-Development of Strength Parameters for Analysis

Purpose

To select shear strength parameters for the soil layers at evaluation cross sections using site-specific and historical field and lab testing data and published references. Selected values are used in our stability analyses.

References

ASTM (2003), Standard Test Method for Unconsolidated Undrained Triaxial Compression Test of Cohesive Soils, ASTM Standard D2850-03a.

ASTM (2004a), Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading, ASTM Standard D2435-04.

ASTM (2004b), Standard Test Method for Consolidated Undrained Triaxial Compression Test of Cohesive Soils, ASTM Standard D4767-04.

Bonaparte and Mitchell (1979), The Properties of San Francisco Bay Mud at Hamilton Air Force Base, California, University of California at Berkeley

California Department of Water Resources (DWR) (2015), Guidance Document for Geotechnical Analyses, Urban Levee Evaluations Project, prepared by URS Corp., April.

Demetrious Koutsoftas Geotechnical Consultants, Inc. (DKGC) (2013), Development of Total Stress Envelopes for Rapid Drawdown Stability Analysis: Comparison of Simplified Procedure of Appendix F with Conventional Approach using Results of Triaxial Tests, Project Memorandum.

DeJong, J.T., Jaeger, R.A., Boulanger, R.W., Randolph, M.F., and Wahl, D.A.J. (2012), Variable Penetration Rate Cone Testing for Characterization of Intermediate Soils, Geotechnical and Geophysical Site Characterization 4, Coutinho and Mayne eds., Taylor & Francis Group, London.

Duncan et al. (1990) Slope Stability during Rapid Drawdown. Proceedings of H. Bolton Seed Memorial Symposium. Vol. 2. Electric Power Research Institute (1990) Manual on Estimating Soil Properties for Foundation Design.

EPRI EL-6800 Final Report. August 1990.

GEI (2015), Geotechnical Data Report, Coyote Creek Levee Evaluation Project.

Federal Highway Administration (FHWA) (2002), NHI Course No. 132031, Subsurface Explorations – Geotechnical Site Characterization.

Hatanaka, M. and Uchida, A. (1996), Empirical correlation between penetration resistance and effective friction of sandy soil, Soils & Foundations, Vol. 36, No. 4, Japanese Geotechnical Society.

Kulhawy, F.H. and Mayne, P.W. (1990), Manual on Estimating Soil Properties for Foundation Design, EL-6800, Electric Power Research Institute.

Lambe, T.W., and Whitman, R.V. (1969), Soil Mechanics, John Wiley & Sons, Inc., New York.

Ladd, C.C. (1991), Stability Evaluation During Staged Construction, Journal of Geotechnical Engineering, Vol. 117, No. 4. Ladd, C.C. and DeGroot, D.J. (2003), Recommended Practice for Soft Ground Site Characterization, Proceedings of the 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering, Boston.

Lunne T., Robertson, P.K., and Powell, J.J.M. (1997). Cone Penetration Testing in Geotechnical Practice. Chapman & Hall, London.

Mayne, P.W. (2007), Cone Penetration Testing, A Synthesis of Highway Practice, NCHRP Synthesis 368, Transportation Research Board, Washington, D.C. Mitchell, J.K. (1993), Fundamentals of Soil Behavior, 2nd Edition, John Wiley & Sons, Inc., New York.

Robertson, P.K. and Cabal, K.L. (2014), Guide to Cone Penetration Testing, 6th Edition, Gregg Drilling & Testing, Inc.

Terzaghi, K., Peck, R.B., and Mesri, G. (1996), Soil Mechanics in Engineering Practice, 3rd Edition, John Wiley & Sons, Inc., New York.

U.S. Naval Facilities Engineering Command (NAVFAC) (1986), Design Manual 7.01, Soil Mechanics.

Wong et al. (1983) Comparisons of Methods of Rapid Drawdown Stability Analysis. Report No. UCB/GT/82-05. University of California at Berkeley

Summary

This write-up describes our general approach for developing shear strength parameters. Details of the selection of shear strength parameters are provided in the attachments that follow this write-up. There is one attachment for each analysis cross section that we evaluated. At the beginning of each attachment, a summary table is provided, showing the selected shear strength parameters for the cross section.

Approach

We selected soil strength parameters based on site-specific subsurface explorations and laboratory testing of samples obtained within the Novato Creek Levee Evaluation Project study area. Additionally, we considered lab testing results from the California Department of Water Resources (DWR) Urban Levee Evaluation (ULE) special testing program performed by Demetrious Koutsoftas Geotechnical Consultants, Inc. (DKGC). When appropriate, we used correlations to field and lab index test data to develop parameters.

We used some historical boring logs, SPT data, and lab testing results to supplement our current explorations to develop strength parameters and stratigraphy, but in cases of conflicting information, we generally applied more weight to more recent information.

Strength parameters were estimated for each individual evaluation cross section. For each section we evaluated the data from subsurface explorations adjacent to the section location. We also considered data from subsurface explorations within the reach represented by the cross section where appropriate.

In some instances where soil layers had limited data within the reach we also used data from additional reaches to estimate strength parameters specific to the entire project, particularly for laterally continuous soil units, such as young bay mud (see Figure B-1). In principle, the use of data from nearby reaches will be limited to those material properties demonstrating a high degree of consistency within the study area or having minimal or no impact on the analysis results.

Mohr-Coulomb Failure Envelope

We performed our stability evaluations using limit-equilibrium analyses with shear strengths defined by a Mohr-Coulomb failure envelope. The Mohr-Coulomb strength envelope is a straight-line simplification of a failure envelope that is defined by a slope angle (ϕ) and an intercept (c) defined by the following equation:

$$\tau = c + \sigma * \tan(\phi)$$

Where τ is the shear strength on the failure plane, c is a cohesion intercept, σ is the normal stress on the failure plane, and ϕ is a friction angle.

For fine-grained soils and non-freely draining coarse-grained soils, we assigned both a drained strength envelope using effective strength (c' and ϕ') parameters, and an undrained strength envelope using total stress (c and ϕ) parameters. Freely draining coarse-grained soils do not retain high pore pressures during a rapid drawdown condition, and thus only drained strengths have been developed. We assigned the drained strength envelope using effective stress (c' and ϕ') parameters. In general, we assumed an effective cohesion of zero ($c' = 0$) for freely draining coarse-grained soils.

Our approach to develop strength parameters is outlined in the following steps:

1) Develop Representative Stratigraphy

At each evaluation cross section, we developed representative subsurface stratigraphy based on our interpretation of the nearby subsurface explorations.

2) Estimate Drained Shear Strength Parameters For Coarse-Grained Soil Layers

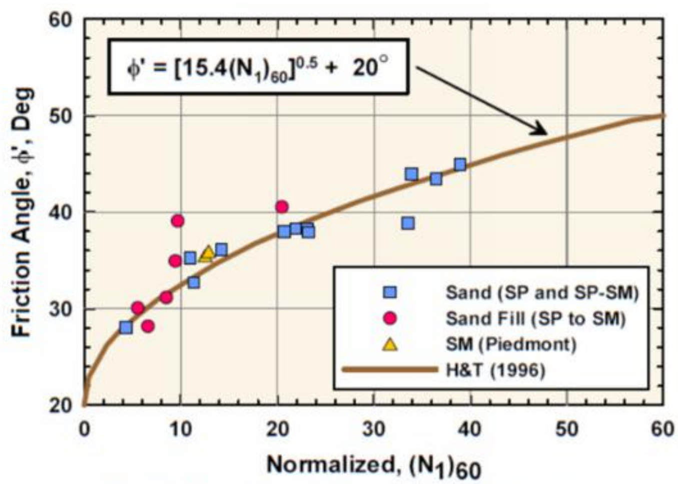
We estimated drained shear strengths of predominantly coarse-grained soils using empirical correlations to SPT N-values and CPT normalized tip resistance.

As recommended in the Urban Levee Geotechnical Evaluations Program, Guidance Document for Geotechnical Analyses, (Guidance Document) (DWR, 2015) we used the following correlations:

<u>SPT N-Value Corrections and Correlations</u>:

Correlation SPT N-value to Friction Angle (ϕ')

For SPT N-Value data, from FHWA (2002), adapted from Hatanaka & Uchida (1996):



Peak Friction Angle of Sands from SPT Resistance
(adapted from Hatanakata & Uchida, 1996; Figure from FHWA NHI, 2002).

As part of a previous project we contacted Prof. Paul Mayne, the author of the NHI publication, to confirm which corrections were incorporated into the correlation. Dr. Mayne indicated that only the N_{60} energy correction and the N_1 overburden correction were included. Other corrections, such as those for rod length, borehole diameter, and sampler type were not included.

N_{60} Energy Correction

Various correlations between strength parameters and SPT N-values are available. The correlations are generally based on N-values corrected for 60% of the theoretical energy delivered by the hammer (N_{60}) and for 1 tsf effective overburden pressure ($N_{1(60)}$).

We corrected the field N-values (N_{field}) to N_{60} values as follows:

$$N_{60} = N_{field} * C_E$$

Where C_E is a correction for the hammer energy ratio (ER), which is calculated as:

$$C_E = \frac{ER}{60}$$

For our current explorations, Cascade Drilling provided recent hammer energy correlations for the drill rig used during the exploration program. For historical borings, we used the hammer energy or the N_{60} noted on the boring logs.

$N_{1(60)}$ Overburden Correction

An overburden correction factor CN is applied to the SPT N-values to account for the dependency of N-values on effective overburden stress. The overburden correction factor is calculated as:

$$C_N = \sqrt{\frac{P_a}{\sigma'_{vo}}}$$

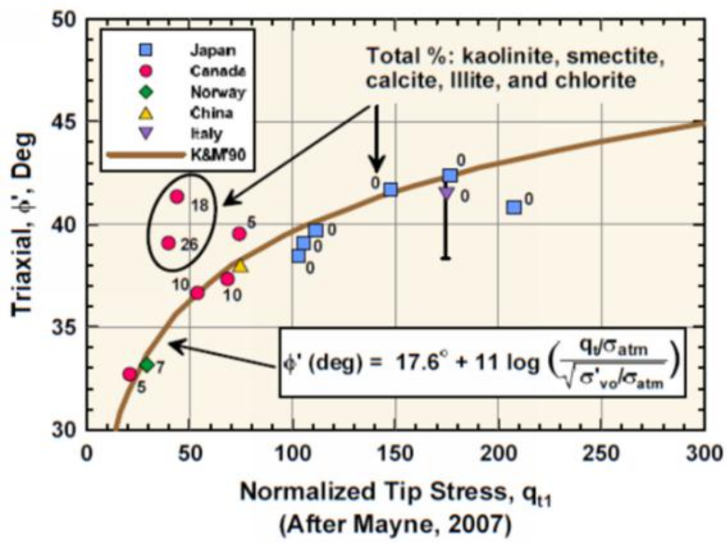
Where P_a is the atmospheric pressure (equal to 2,116 psf) and σ'_{vo} is the in situ effective overburden stress.

To simplify the calculation of the overburden correction factor, we assumed a total unit weight of 100 pcf for young bay mud soils and 120 pcf for all other soils. If noted, we used groundwater depths on the boring logs. Otherwise, we assumed a depth to groundwater based on the groundwater elevations of nearby explorations. For the purpose of estimating strength properties, we assumed other correction for $N_{1(60)}$ including adjustments for rod length and borehole diameter were equal to 1, because these parameters were not used in the development of the correlations to strength parameters.

CPT Correlation:

Correlation CPT Tip Stress to Drained Friction Angle (ϕ')

For CPT normalized tip stress, from Kulhawy & Mayne (1990), published in Mayne (2007):



We selected representative drained shear strengths for each coarse-grained soil layer by estimating the typical drained friction angles estimated with the above SPT and CPT correlations.

3) Estimate Drained Shear Strength Parameters For Fine-Grained Soil Layers

The site-specific strength testing program included isotropically consolidated undrained triaxial compression (CIUC) tests (American Society for Testing and Materials, ASTM, D4767) without pore pressure measurements and incremental load consolidation (ILC) tests (ASTM D2435) on Shelby tube samples taken from the fine-grained soil layers (predominantly young bay mud). Our testing program consisted of 5 CIUC tests and 4 ILC tests on young bay mud. Additional details and test data of the strength testing program were provided in the Geotechnical Data Report (GDR) issued in October of 2019 (GEI, 2019).

As noted above, no site-specific drained strength testing to estimate drained shear strengths of the fine-grained soils was available for this evaluation. Therefore, we estimated drained shear strength parameters (c' and ϕ') of predominantly fine-grained soils using the Guidance Document (DWR, 2015).

A table from the Guidance Document (DWR, 2015) presenting recommended ranges of values for steady-state seepage stability (drained) parameters is provided on the following page. The document is intended as guidance for Urban Levee Evaluations, which are screening-level analyses, and tend towards conservative selection of parameters.

Table 5-4 Summary of Strength Relationships for Non-Free-Draining Soils for Steady-State Conditions

Soil Type [1, 4]	Site-Specific Drained Strength Related Data Available	Site-Specific Strength Related Data Limited or Not Available	
		OCR ≥ 2, or Liquidity Index ≤ 0.6	OCR < 2, or Liquidity Index > 0.6
Group 1 Soils Foundation Layers	c′ + Φ′ as determined from strength tests c′ ≤ 200 psf Φ′ ≤ 35 degrees	$c' = 0.015 \times \sigma'_p \leq 150$ psf, if σ'_p is known c′ ≤ 100 psf, if σ_p is not known Φ′ = 28 to 32 degrees	c′ = 0 Φ′ = 30 to 32 degrees
Group 1 Soils Embankment Layers [3]	c′ + Φ′ as determined from strength tests c′ ≤ 100 psf Φ′ ≤ 32 degrees	$c' = 0.01 \times \sigma'_p \leq 100$ psf, if σ'_p is known c′ = 50 psf, if σ'_p is not known Φ′ = 28 to 32 degrees [2]	c′ = 0 Φ′ =27 to 30 degrees
Group 2 Soils Foundation Layers	c′ + Φ′ as determined from strength tests c′ ≤ 200 psf Φ′ ≤ 32 degrees	$c' = 0.02\ \sigma'_p \leq 100$ psf, if σ'_p is known c′ ≤ 75 psf, if σ'_p is not known Φ′ = 27 to 30 degrees	c′ = 0 Φ′ = 27 to 30 degrees
Group 2 Soils Embankment Layers	c′ + Φ′ as determined from strength tests, but: c′ ≤ 100 psf Φ′ ≤ 32 degrees	$c' = 0.01 \times \sigma'_p \leq 75$ psf, if σ'_p is known c′ ≤ 50 psf, if σ'_p is not known Φ′ = 27 to 30 degrees [2]	c′ = 0 Φ′ = 27 to 30 degrees [2]
Group 3 Soils/Organic Soils Foundation Layers	c′ + Φ′ as determined from strength tests c′ ≤ 100 psf Φ′ ≤ 32 degrees	$c' = 0.02\ \sigma'_p \leq 75$ psf, if σ'_p is known c′ ≤ 50 psf, if σ'_p is not known Φ′ = 27 to 31 degrees[2, 3]	c′ = 0 Φ′ = 27 to 30 degrees[2]
Group 3 Soils/Organic Soils Embankment Layers	c′ + Φ′ as determined from strength tests c′ ≤ 75 psf Φ′ ≤ 32 degrees	$c' = 0.01 \times \sigma'_p \leq 50$ psf, if σ'_p is known c′ ≤ 0 psf, if σ'_p is not known	c′ = 0 Φ′ = 27 to 28 degrees[2, 3]
Group 4 Soils Foundation Layers	c′ + Φ′ as determined from strength tests c′ ≤ 150 psf Φ′ ≤ 36 degrees	$c' = 0.015\ \sigma'_p \leq 75$ psf, if σ'_p is known c′ ≤ 50 psf, if σ'_p is not known Φ′ = 31 to 34 degrees	c′ = 0 Φ′ = 30 to 34 degrees
Group 4 Soils Embankment Soils	c′ + Φ′ as determined from strength tests c′ ≤ 100 psf Φ′ ≤ 34 degrees	$c' = 0.01\ \sigma'_p \leq 50$ psf, if σ'_p is known c′ = 0 psf, if σ'_p is not known Φ′ = 30 to 33 degrees	c′ = 0 Φ′ = 28 to 32 degrees

Legend:
c′ = effective cohesion
Φ′ = effective friction angle
σ'_{vm} = effective vertical maximum past pressure

Notes:
1 See Figure 5-1 for definition of soil groups.
2 ULE analysts must consider construction processes and potential variability that might not have been captured by available tests when selecting parameters for slope stability.
3 For embankment soils, ULE analysts should consider the effects of construction processes and potential variability of the placement conditions when selecting conservative parameters for slope stability analysis.
4 For Group 1 soils used in a remediation effort, see recommendations in Section 5.8.3.2

The table is organized by soil group, which is a convention unique to the guidance document. Where the group of soils are classified as:

- Group 1 Soils: CL and CH with a liquid limit less than 65
- Group 2 Soils: CH with an liquid limit greater than or equal to 65
- Group 3 Soils: Organic soils (OL and OH), excluding peat
- Group 4 Soils: Inorganic silts, ML, and clayey silts CL-ML with a plasticity index lower than 7, referred to as Group 4A; and silts with a liquid limit between 25 and 65, referred to as Group 4B.

The maximum past pressure was used to develop effective cohesion values as described in the table above. However, where the fine-grained soils were shown to be normally to lightly overconsolidated with a high liquidity index (LI), such as bay mud, the effective cohesion was assigned as 0 psf.

Drained friction angles were estimated from the range provided in the table based on the soil's overconsolidation ratio. The lesser value of the drained friction angle range was typically chosen for a conservative analysis.

4) Estimate Total Stress Parameters for Fine-Grained Soils

4a) Estimate SHANSEP Parameters for Fine-Grained Soils

Ladd and DeGroot (2003) suggest that overconsolidation can cause a strength increase that can be modeled by the Stress History and Normalized Strength Engineering Properties (SHANSEP) method. The stress ratio (S_u/σ'_{vo}) of an overconsolidated soil can be predicted by the following equation:

$$S_u/\sigma'_{vo} = S \cdot OCR^m$$

Where S_u is undrained shear strength, σ'_{vo} is the effective overburden stress, S is the strength ratio for normally consolidated soil, OCR is the overconsolidation ratio, and m is a curve-fitting parameter.

The overconsolidation ratio is defined as:

$$OCR = \frac{\sigma'_{vm}}{\sigma'_{vo}}$$

Where σ'_{vm} is the maximum past pressure and σ'_{vo} is the effective overburden stress.

Using the correlation between undrained strength ratio for isotropic consolidation and effective friction angle as shown in the EPRI EL-6800 Final Report (1990), the undrained strength ratio was estimated using the effective friction angle estimated using the Guidance Document (DWR, 2015). For this project an S value of 0.325 (based on a typical effective friction angle of 27° for fine-grained soils) and a typical m value of 0.8 were chosen.

4b) Calibrate Undrained Shear Strength (Su) Estimates from CPT Data to Laboratory Strength Tests

We estimated undrained shear strength of fine-grained soils from the CPT with the following formula (as defined in Lunne et. al. 1997):

$$S_u = \frac{q_t - \sigma_{vo}}{N_{kt}}$$

Where S_u is the undrained shear strength, qt is the cone tip resistance, σ_{vo} is the total vertical stress, and N_{kt} is a constant typically ranging from 10 to 18 (Robertson and Cabal 2014). N_{kt} values between 14 and 16 are often used at sites where limited site-specific data has been obtained (Robertson and Cabal 2014). To estimate undrained shear strengths from CPT data, an N_{kt} value of 11 was selected based on strength data available to GEI from nearby projects that encountered bay mud.

Comparisons of undrained shear strengths estimated from SHANSEP and CPT data are provided in Figures B-2 through B-4.

4c) Maximum Past Pressure (σ'_{vm}) Estimates from CPT Data to Laboratory Consolidation Tests

We estimated maximum past pressures from CPTs with the approach presented by Kulhawy and Mayne (1990):

$$\sigma'_{vm} = k_{OCR}(q_t - \sigma_{vo})$$

Where k_{OCR} is a constant ranging from 0.2 to 0.5 (Robertson and Cabal 2014), q_t is the corrected cone resistance, and σ_{vo} is the vertical total stress. For this evaluation a k_{OCR} value of 0.3 was used based on calibration of estimated maximum past pressures from CPT data with estimated maximum past pressures from Casagrande's Graphical Method of the 4 current ILC test results. As shown in Figures B-2 through B-4 of the attachments a k_{OCR} value of 0.3 generally results in a normally to lightly overconsolidated soil for the young bay mud.

The vertical total stresses were estimated using current CPT data, more specifically Soil Behavior Type. The vertical total stress for all soils (except bay mud) was estimated based on normalized Soil Behavior Type (SBT_{Qtn}) as defined by Robertson (2009) which uses a variable stress ratio exponent for normalization based on the Soil Behavior Type Index, I_c. The soil unit weights assigned based on SBT_{Qtn} are summarized in Table B-1 shown below.

Table B-1. Total Soil Unit Weights based on SBT_{Qtn} (Robertson 2009)

Zone	Total Unit Weight (pcf)	Description
0	118.6	Undefined
1	111.4	Sensitive, Fine Grained
2	79.5	Organic Soils
3	111.4	Clays
4	114.6	Silt Mixtures
5	120.9	Sand Mixtures
6	124.1	Sands
7	127.3	Gravelly Sand to Sand
8	130.5	Stiff Sand to Clayey Sand
9	120.9	Very Stiff Fine Grained

The vertical total stress for bay mud was estimated using a unit weight of 100 pcf which is the average unit weight reported for all current CIUC and ILC tests.

The vertical effective stresses were also estimated and are plotted in Figures B-2 through B-4 to allow for comparison with the maximum past pressures. To estimate the vertical effective stresses, pore pressures were estimated based on the water surface elevations encountered during the current CPT explorations. To simplify calculations, a unit weight of 100 pcf was used for bay mud and 120 pcf was used elsewhere.

4d) Estimate total stress parameters for fine-grained soil layers

For the second stage of the three-stage rapid drawdown analysis, SLOPE/W uses undrained strengths to evaluate the stability factor of safety. Typically, an R-envelope from undrained triaxial tests is used to define undrained strengths. An R-envelope is essentially a line defining undrained shear strength (S_u) as a function of vertical effective stress. The linear fit to develop total stress parameters for undrained strength versus effective stress using the SHANSEP correlation and CPT data, provides the undrained shear strength as a function of vertical effective stress. As described by Wong et al. (1982), the linear fit to undrained shear strength for a given vertical effective stress provides a cohesion value of “a” and a slope parameter “b” as shown in the equation below:

$$Su = a + \sigma'_v * \tan(b)$$

However, SLOPE/W 2018 requires the undrained strength parameters "c" and "ϕ", which define an undrained strength relationship between the shear stress on the failure plane at failure τ_{ff} and the normal stress on the failure plane at failure σ_{ff}. For this envelope the cohesion parameter "c" and slope parameter "ϕ" are used as shown in the equation below:

$$\tau_{ff} = c + \sigma'_{\mathrm{v}} * \tan(\phi)$$

The parameters "c" and "ϕ" can be calculated directly from the parameters "a", "b", and ϕ′ using the following relationships derived from equations presented in Duncan and Wright (2005):

$$\phi = \sin^{-1}\left(\frac{\tan(b)}{\cos(\phi') + \tan(b)}\right)$$

$$c = a\left(\frac{1 - \sin(\phi)}{\cos(\phi)\cos(\phi')}\right)$$

We have verified this procedure with the developers of SLOPE/W via personal communication on a previous project.

The attachments provided include both the "a" and "b" parameters which were developed by fitting to undrained strength versus effective stress, and the parameters "c" and "ϕ" which will be used for analysis as an input in SLOPE/W 2018. For the laterally continuous young bay mud soil unit, nearby undrained strength data estimated by current CPT explorations was plotted versus vertical effective stress to understand the consistency of undrained soil strength (see Figure B-1). This plot provides a project-specific fit of undrained strength versus effective stress for younger bay mud that is applied consistently to every Reach. Two trends can be observed in Figure B-1 within the undrained shear strength profiles with a break point at an approximate effective stress of 1.8 ksf. Both trends have been characterized by "a" and "b" parameters; however, in creating the simplified model, and "a" = 0.21 ksf and "b" = 10° were respectively chosen to characterize young bay mud providing conservative c and ϕ values. Note, that these values capture an approximate trend in undrained strength versus depth for the existing stress state in the soil based on in-situ tests and engineering judgement. Where relevant, sensitivities will be considered during existing condition and remedial condition analysis to understand the impact the soil properties have on results.

Novato Creek Left Bank Levee, Sta. NCLB 300+89

<u>Summary:</u>

Our selected shear strength parameters are summarized in the following table:

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c'	ϕ'	c	ϕ
	(pcf)	(psf)	(deg)	(psf)	(deg)
(1) CL	120	50	28	1200	0
(2) CH	100	0	27	200	10
(3) SM	125	0	38	--	--

Layers 1 and 2 are considered fine-grained soils. Layer 3 is considered coarse-grained soil.

We primarily considered the following borings and CPTs in our evaluation of the cross-section.

Exploration	Station	Offset	Location	Ground Surface Elevation	Drilled Depth
	(NCLB)	(ft)		(ft, NAVD 88)	(ft)
GEI_001B	300+89	3.4	Crown	15.6	46.0
GEI_001C	300+86	-6.3	Crown	15.7	45.4
B3 (2016)	301+10	847.9	Field	3.0	21.5
B2 (2016)	297+25	80.2	Toe	5.0	41.5
B4 (2016)	304+71	1116.1	Field	4.0	18.5
GEI_001C_TOE	297+06	78.9	Toe	4.0	51.8
B1 (2016)	306+15	-1.6	Crown	14.0	76.5

Figure B-2 is a summary of the data from these explorations.

<u>Fine-Grained Soils:</u>

Layer 1 (CL)

Layer 1 is an embankment layer generally consisting of lean clay. This layer is overconsolidated with an OCR greater than 2. Figure B-2 shows corrected SPT N values (N_{60} for samples classified as clay-like soils and $N_{1,60}$ for sand-like soils) from current and historic explorations. Using current and historic boring logs, the typical SPT N_{60} range can be estimated to be between 5 and 18 blows per foot (bpf) with an average of 10 bpf. Per the Guidance Document (DWR, 2015) the recommended range of values for steady-state seepage stability drained friction angle for a Group 1 (CL and CH with liquid limit less than

65) embankment soil is between 28° to 32°. The estimated drained friction angle of 28° was chosen from the recommended range to be conservative.

Per the Guidance Document (DWR, 2015), since no in-situ vane shear tests or consolidation tests were performed on samples from this layer, the maximum past pressure is not known and the drained cohesion is assumed to be equal to 50 psf.

The undrained strength value for the layer was estimated using the SHANSEP approach which depends on strength ratio parameter (S), curve-fitting parameters (m), and OCR. An S value of 0.325 and m value of 0.8 were used for this entire project. The OCR was estimated using data from GEI_001C and GEI_001C_TOE. To estimate OCR, the maximum past pressure was estimated using the method proposed by Kulhawy and Mayne (1990) with a site-specific k_{OCR} value of 0.3. The layer's average OCR value of 8 was assumed in this strength calculation. Undrained shear strengths were also estimated using the cone tip resistance correlation presented by Lunne et. al. (1997). The undrained shear strengths calculated estimated from GEI_001C and GEI_001C_TOE data were assuming an N_{kt} value of 11. Both undrained strengths are shown in Figure B-2. The average undrained strength value of 1,200 psf estimated from SHANSEP for the layer was used for analysis. The total friction angle is assumed to be equal to 0°.

<u>We used drained strength parameters of ϕ' = 28° and c' = 50 psf.</u>
<u>We used total strength parameters of ϕ = 0° and c = 1,200 psf.</u>

Layer 2 (CH)
Layer 2 is a foundation layer generally consisting of high plasticity fat clay (young bay mud). Generally, this layer is lightly overconsolidated with an OCR less than 2. Figures B-2 through B-4 show corrected SPT N values (N_{60} for samples classified as clay-like soils and $N_{1,60}$ for sand-like soils) from current and historic explorations. Using current and historic boring logs, the typical SPT N_{60} range for young bay mud can be estimated to be between 0 and 6 bpf. Figures B-2 through B-4 also show results from Atterberg Limit tests performed on samples from current and historic explorations. From these figures, the results of 16 tests performed on young bay mud soil samples can be observed. The liquid limit range between all 16 results is 42 to 137, with only 4 samples having a liquid limit less than 65. Per the Guidance Document (DWR, 2015), the recommended range of values for steady-state seepage stability drained friction angle for a Group 2 (CH with liquid limit greater or equal to 65) foundation soil is between 27° to 30°. The estimated drained friction angle of 27° was chosen from the recommended range to be conservative. The drained cohesion is assumed to be equal to 0.

The undrained strength values for a and b were determined by fitting the parameters to nearby undrained strength data estimated by current CPT explorations plotted versus vertical effective stress (see Figure B-1). This plot provides a project-specific fit of undrained strength versus effective stress for young bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.21 ksf and 10 degrees respectively.

<u>We used drained strength parameters of ϕ' = 27° and c' = 0 psf.</u>

We used total strength parameters of b = 10° and a = 210 psf, which for analysis purposes convert to: ϕ = 10° and c = 200 psf.

Coarse-Grained Soils:

Layer 3 (SM)

Layer 3 is a foundation layer generally consisting of silty sand, clayey sand, and silty, clayey sand. The SPT $N_{1(60)}$ value in this layer ranges between 4 and 65 bpf with an average of 21 bpf, indicating a medium dense soil. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in the Guidance Document (DWR, 2015) ranges from 29° to 45° with an average of 38°. Based on the correlation to tip resistance provided in the Guidance (DWR, 2015), the friction angle estimated from CPT soundings ranges from 34° to 45° with an average of 40°. Based on data available, a typical friction angle of 38° was chosen for analysis.

We used drained strength parameters of ϕ' = 38° and c' = 0 psf.

Lynwood Levee, Sta. LL 260+68

Summary:

Our selected shear strength parameters are summarized in the following table:

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c'	ϕ'	c	ϕ
	(pcf)	(psf)	(deg)	(psf)	(deg)
(1) CH	120	0	27	500	0
(2) CL	120	50	28	1500	0
(3) SM	115	0	36	--	--
(4) SM	115	0	33	--	--
(5) CH	100	0	27	200	10
(6) SM	115	0	38	--	--

Layers 1, 2, and 5 are considered fine-grained soils. Layer 3, 4, and 6 are considered coarse-grained soil.

We primarily considered the following borings and CPTs in our evaluation of the cross-section.

Exploration	Station	Offset	Location	Ground Surface Elevation	Drilled Depth
	(LL)	(ft)		(ft, NAVD 88)	(ft)
GEI_003B	260+68	10.8	Crown	12.9	46.5
GEI_011C	260+94	-8.5	Crown	12.3	58.3
S-1 (2017)	256+16	10.1	Crown	12.2	3.0
B22 (2016)	256+03	4.9	Crown	11.8	7.5
HA-17 (2016)	266+25	7.9	Crown	14.3	5.0

Figure B-3 is a summary of the data from these explorations.

Fine-Grained Soils:

Layer 1 (CH)

Layer 1 is an embankment layer generally consisting of fat clay. This layer is overconsolidated with an OCR greater than 2. Figure B-3 shows corrected SPT N values (N_{60} for samples classified as clay-like soils and $N_{1,60}$ for sand-like soils) from current and historic explorations. Using current and historic boring logs, the typical SPT N_{60} range can be estimated to be between 4 and 15 bpf with an average of 9 bpf. Figure B-3 also shows results from Atterberg Limit tests performed on samples from current and historic explorations. From this figure, the results of 1 test performed on samples from this layer can be observed. The liquid limit is 66. Per the Guidance Document (DWR, 2015), the recommended range of values for steady-state seepage stability drained friction angle for a Group 2 (CH with liquid limit greater than or equal to 65) embankment soil is between 27° to 30°. The estimated drained friction angle of 27° was chosen from the recommended range to be conservative.

Per the Guidance (DWR, 2015), since no in-situ vane shear tests or consolidation tests were performed on samples from this layer, the maximum past pressure is not known and the drained cohesion is assumed to be less than or equal to 50 psf. We used a drained cohesion of 0 psf.

The undrained strength value for the layer was estimated using the SHANSEP approach which depends on strength ratio parameter (S), curve-fitting parameters (m), and OCR. An S value of 0.325 and m value of 0.8 were used for this entire project. The OCR was estimated using data from GEI_011C. To estimate OCR, the maximum past pressure was estimated using the method proposed by Kulhawy and Mayne (1990) with a site-specific k_{OCR} value of 0.3. The OCR was capped at 15 in order to prevent overestimating the undrained shear strength. The average undrained strength value of 500 psf estimated from SHANSEP for the layer was used for analysis. The total friction angle is assumed to be equal to 0°.

<u>We used drained strength parameters of ϕ' = 27° and c' = 0 psf.</u>
<u>We used total strength parameters of ϕ = 0° and c = 500 psf.</u>

Layer 2 (CL)

Layer 2 is an embankment layer generally consisting of lean clay. This layer is overconsolidated with an OCR greater than 2. Figure B-3 shows corrected SPT N values (N_{60} for samples classified as clay-like soils and $N_{1,60}$ for sand-like soils) from current and historic explorations. Using current and historic boring logs, the typical SPT N_{60} range can be estimated to be between 4 and 18 bpf with an average of 11 bpf. Per the Guidance Document (DWR, 2015), the recommended range of values for steady-state seepage stability drained friction angle for a Group 1 (CL and CH with liquid limit less than 65) embankment soil is between 28° to 32°. The estimated drained friction angle of 28° was chosen from the recommended range to be conservative.

Per the Guidance Document (DWR, 2015), since no in-situ vane shear tests or consolidation tests were performed on samples from this layer, the maximum past pressure is not known and the drained cohesion is assumed to be equal to 50 psf.

The undrained strength value for the layer was estimated using the SHANSEP approach which depends on strength ratio parameter (S), curve-fitting parameters (m), and OCR. An S value of 0.325 and m value of 0.8 were used for this entire project. The OCR was estimated using data from GEI_011C. To estimate OCR, the maximum past pressure was estimated using the method proposed by Kulhawy and Mayne (1990) with a site-specific k_{OCR} value of 0.3. The OCR was capped at 15 in order to prevent overestimating the undrained shear strength. Undrained shear strengths were also estimated using the cone tip resistance correlation presented by Lunne et. al. (1997). The undrained shear strengths calculated estimated from GEI_011C data were assuming an N_{kt} value of 11. Both undrained strengths are shown in Figure B-3. The average undrained strength value of 1,500 psf estimated from SHANSEP for the layer was used for analysis. The total friction angle is assumed to be equal to 0°.

<u>We used drained strength parameters of ϕ' = 28° and c' = 50 psf.</u>
<u>We used total strength parameters of ϕ = 0° and c = 1,500 psf.</u>

Layer 5 (CH)
Layer 5 is a foundation layer generally consisting of high plasticity fat clay (young bay mud). Generally, this layer is lightly overconsolidated with an OCR less than 2. Figures B-2 through B-4 show corrected SPT N values (N_{60} for samples classified as clay-like soils and $N_{1,60}$ for sand-like soils) from current and historic explorations. Using current and historic boring logs, the typical SPT N_{60} range for young bay mud can be estimated to be between 0 and 6 bpf. Figures B-2 through B-4 also show results from Atterberg Limit tests performed on samples from current and historic explorations. From these figures, the results of 16 tests performed on young bay mud soil samples can be observed. The liquid limit range between all 16 results is 42 to 137, with only 4 samples having a liquid limit less than 65. Per the Guidance Document (DWR, 2015), the recommended range of values for steady-state seepage stability drained friction angle for a Group 2 (CH with liquid limit greater or equal to 65) foundation soil is between 27° to 30°. The estimated drained friction angle of 27° was chosen from the recommended range to be conservative. The drained cohesion is assumed to be equal to 0.

The undrained strength values for a and b were determined by fitting the parameters to nearby undrained strength data estimated by current CPT explorations plotted versus vertical effective stress (see Figure B-1). This plot provides a project-specific fit of undrained strength versus effective stress for young bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.21 ksf and 0 degrees respectively.

<u>We used drained strength parameters of ϕ' = 27° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 10° and a = 210 psf, which for analysis purposes convert to: ϕ = 10° and c = 200 psf.</u>

<u>Coarse-Grained Soils:</u>

Layer 3 (SM)
Layer 3 is an embankment layer generally consisting of silty sand. Based on the correlation to tip resistance provided in the Guidance Document (DWR, 2015), the friction angle estimated from CPT soundings ranges from 36° to 40° with an average of 39°. Due to the limited amount of data available, a conservative friction angle of 36° was chosen for analysis to be within the expected range of 32° to 37° for free-draining soils in the embankment as mentioned in the Guidance Document (DWR, 2015).

We used drained strength parameters of $\phi' = 36°$ and $c' = 0$ psf.

Layer 4 (SM)

Layer 4 is a foundation layer generally consisting of silty sand and clayey sand. A single SPT $N_{1(60)}$ value of 13 bpf was recorded in this layer indicating a medium dense soil. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in the Guidance Document (DWR, 2015) is 33°. Due to the limited amount of data available, a conservative friction angle of 33° was chosen for analysis to be within the expected range of 32° to 37° for free-draining soils in the embankment as mentioned in the Guidance Document (DWR, 2015).

We used drained strength parameters of $\phi' = 33°$ and $c' = 0$ psf.

Layer 6 (SM)

Layer 6 is a foundation layer that is believed to be generally consisting of silty sand based on limited CPT data available. No soil samples were recovered within this layer. Based on the correlation to tip resistance provided in the Guidance Document (DWR, 2015), the friction angle estimated from CPT soundings ranges from 37° to 42° with an average of 39°. Based on data available, a typical friction angle of 38° was chosen for analysis.

We used drained strength parameters of $\phi' = 38°$ and $c' = 0$ psf.

Pacheco Pond Levee, Sta. PP 33+22

Summary:

Our selected shear strength parameters are summarized in the following table:

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c'	ϕ'	c	ϕ
	(pcf)	(psf)	(deg)	(psf)	(deg)
(1) MH	120	0	27	1000	0
(2) CH	115	0	27	400	0
(3) CH	100	0	27	200	10
(4) SM	125	0	40	--	--
(5) CH	100	0	27	200	10

Layers 1, 2, 3, and 5 are considered fine-grained soils. Layer 4 is considered coarse-grained soil.

We primarily considered the following borings and CPTs in our evaluation of the cross-section.

Exploration	Station	Offset	Location	Ground Surface Elevation	Drilled Depth
	(PP)	(ft)		(ft, NAVD 88)	(ft)
GEI_005B	33+22	0.1	Crown	10.4	50.5
GEI_015C	33+37	-4.0	Crown	10.7	46.1

Figure B-4 is a summary of the data from these explorations.

Fine-Grained Soils:

Layer 1 (MH)

Layer 1 is an embankment layer generally consisting of elastic silt. This layer is overconsolidated with an OCR greater than 2. Figure B-4 shows corrected SPT N values (N_{60} for samples classified as clay-like soils and $N_{1,60}$ for sand-like soils) from current explorations. Using current boring logs, the typical SPT N_{60} range can be estimated to be between 3 and 6 bpf with an average of 5 bpf. The Guidance Document (DWR, 2015), does not specify a recommended range of values for steady-state seepage stability drained friction angle for a Group 3 (OL and OH) embankment soil. Elastic silt is not explicitly stated to be part of Group 3 based on Section 5.8.1.2 of the Guidance Document (DWR, 2015) but based on Figure 5-1 the same document, it is assumed that MH is part of the same group. However, the Guidance Document (DWR, 2015) does provide a recommended range of 27° to 28° for the drained friction angle of a Group 3 embankment soil that is slightly overconsolidated with an OCR less than 2. Since soils with larger OCR values typically have larger friction angles, the estimated drained friction angle of 27° that was chosen for analysis is conservative.

Per the Guidance Document (DWR, 2015), since no in-situ vane shear tests or consolidation tests were performed on samples from this layer, the maximum past pressure is not known and the drained cohesion is assumed to be equal to 0 psf.

The undrained strength value for the layer was estimated using the SHANSEP approach which depends on strength ratio parameter (S), curve-fitting parameters (m), and OCR. An S value of 0.325 and m value of 0.8 were used for this entire project. The OCR was estimated using data from GEI_015C. To estimate OCR, the maximum past pressure was estimated using the method proposed by Kulhawy and Mayne (1990) with a site-specific k_{OCR} value of 0.3. Undrained shear strengths were also estimated using the cone tip resistance correlation presented by Lunne et. al. (1997). The undrained shear strengths calculated estimated from GEI_015C data were assuming an N_{kt} value of 11. Both undrained strengths are shown in Figure B-4. The average undrained strength value of 1,050 psf estimated from SHANSEP for the layer was used for analysis. The total friction angle is assumed to be equal to 0°.

<u>We used drained strength parameters of ϕ’ = 27° and c’ = 0 psf.</u>
<u>We used total strength parameters of ϕ = 0° and c = 1,050 psf.</u>

Layer 2 (CH)
Layer 2 is an embankment layer generally consisting of fat clay. This layer is generally lightly overconsolidated with an OCR less than 2. Figure B-4 shows corrected SPT N values (N_{60} for samples classified as clay-like soils and $N_{1,60}$ for sand-like soils) from current explorations. Using current boring logs, the typical SPT N_{60} value can be estimated to be 0 bpf. Figure B-4 also shows results from Atterberg Limit tests performed on samples from current explorations. From this figure, the results of 2 tests performed on samples from this layer can be observed. The liquid limit range is 70 to 89. Per the Guidance Document (DWR, 2015), the recommended range of values for steady-state seepage stability drained friction angle for a Group 2 (CH with liquid limit greater than or equal to 65) embankment soil is between 27° to 30°. The estimated drained friction angle of 27° was chosen from the recommended range to be conservative.

Per the Guidance Document (DWR, 2015), since no in-situ vane shear tests or consolidation tests were performed on samples from this layer, the maximum past pressure is not known and the drained cohesion is assumed to be equal to 0 psf.

The undrained strength value for the layer was estimated using the SHANSEP approach which depends on strength ratio parameter (S), curve-fitting parameters (m), and OCR. An S value of 0.325 and m value of 0.8 were used for this entire project. The OCR was estimated using data from GEI_015C. To estimate OCR, the maximum past pressure was estimated using the method proposed by Kulhawy and Mayne (1990) with a site-specific k_{OCR} value of

0.3. Undrained shear strengths were also estimated using the cone tip resistance correlation presented by Lunne et. al. (1997). The undrained shear strengths calculated estimated from GEI_015C data were assuming an N_{kt} value of 11. Both undrained strengths are shown in Figure B-4. The average undrained strength value of 400 psf estimated from SHANSEP for the layer was used for analysis. The total friction angle is assumed to be equal to 0°.

<u>We used drained strength parameters of ϕ' = 27° and c' = 0 psf.</u>
<u>We used total strength parameters of ϕ = 0° and c = 400 psf.</u>

Layer 3 (CH)
Layer 3 is a foundation layer generally consisting of high plasticity fat clay (young bay mud). Generally, this layer is lightly overconsolidated with an OCR less than 2. Figures B-2 through B-4 show corrected SPT N values (N_{60} for samples classified as clay-like soils and $N_{1,60}$ for sand-like soils) from current and historic explorations. Using current and historic boring logs, the typical SPT N_{60} range for young bay mud can be estimated to be between 0 and 6 bpf. Figures B-2 through B-4 also show results from Atterberg Limit tests performed on samples from current and historic explorations. From these figures, the results of 16 tests performed on young bay mud soil samples can be observed. The liquid limit range between all 16 results is 42 to 137, with only 4 samples having a liquid limit less than 65. Per the Guidance Document (DWR, 2015), the recommended range of values for steady-state seepage stability drained friction angle for a Group 2 (CH with liquid limit greater or equal to 65) foundation soil is between 27° to 30°. The estimated drained friction angle of 27° was chosen from the recommended range to be conservative. The drained cohesion is assumed to be equal to 0.

The undrained strength values for a and b were determined by fitting the parameters to nearby undrained strength data estimated by current CPT explorations plotted versus vertical effective stress (see Figure B-1). This plot provides a project-specific fit of undrained strength versus effective stress for young bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.21 ksf and 0 degrees respectively.

<u>We used drained strength parameters of ϕ' = 27° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 10° and a = 210 psf, which for analysis purposes convert to: ϕ = 10° and c = 200 psf.</u>

Layer 5 (CH)
Layer 5 is a foundation layer generally consisting of high plasticity fat clay. Due to the limited amount of information available for this soil layer, engineering judgment was used to model this layer with parameters similar to Layer 3 (CH). Due to the depth of this layer, the strength parameters chosen for our stability models are expected to have no impact on results.

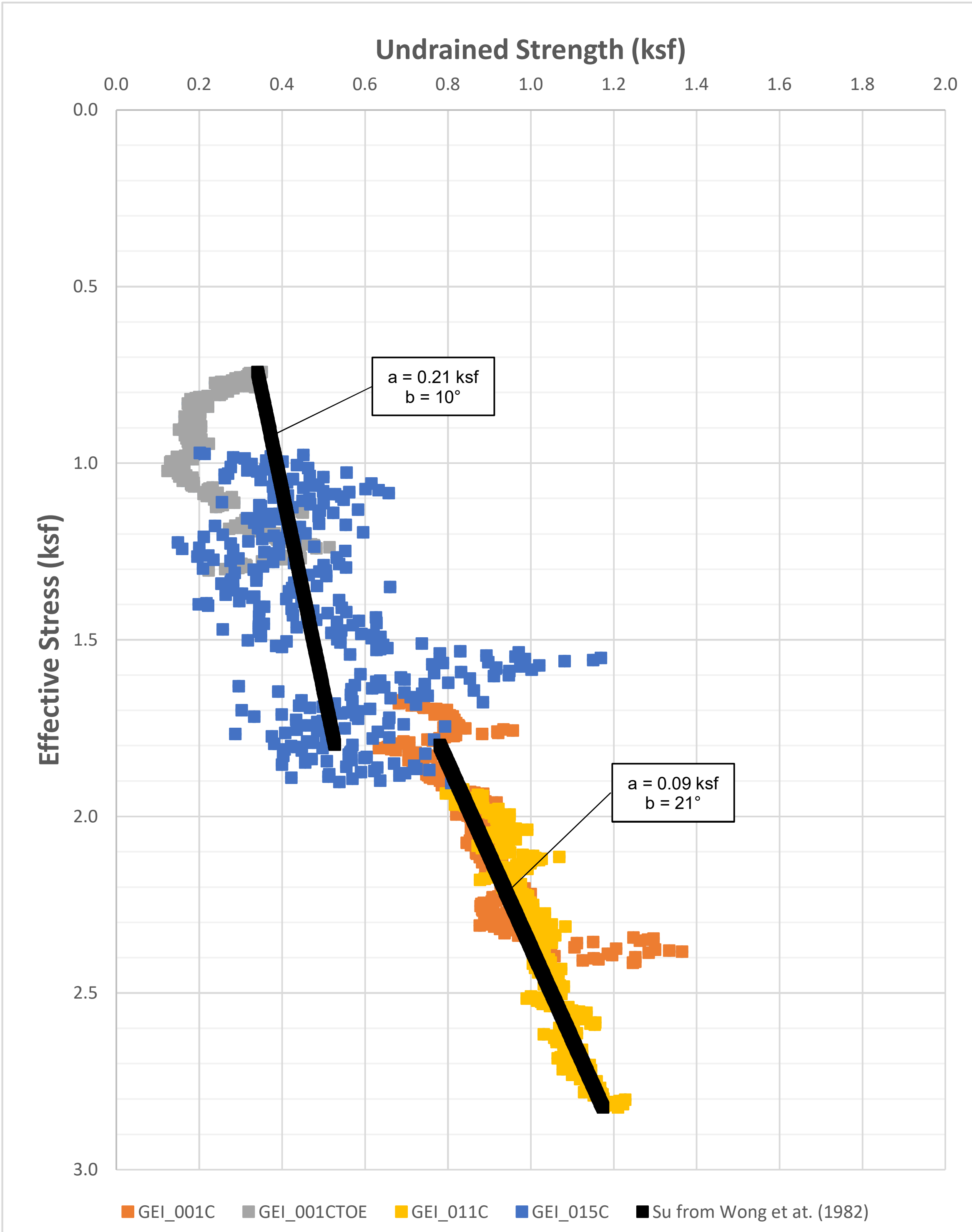
<u>We used drained strength parameters of ϕ' = 27° and c' = 0 psf.</u>
<u>We used total strength parameters of ϕ = 10° and c = 200 psf.</u>

Coarse-Grained Soils:

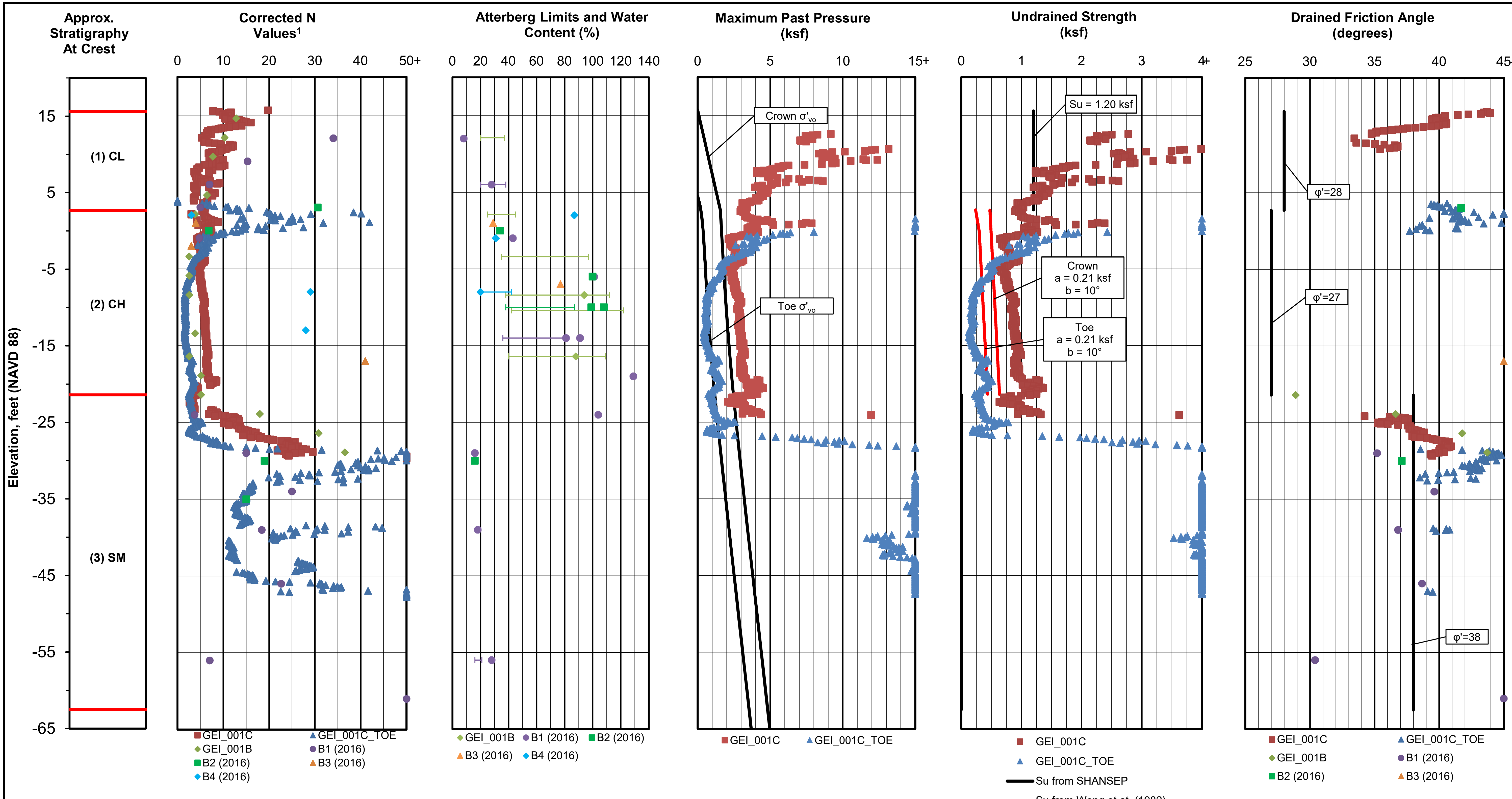
Layer 4 (SM)

Layer 4 is a foundation layer generally consisting of silty sand. The SPT $N_{1(60)}$ value in this layer ranges between 22 to 43 bpf with an average of 31 bpf, indicating a medium dense to dense soil. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in Guidance Document (DWR, 2015) ranges from 39° to 46° with an average of 43°. Based on the correlation to tip resistance provided in Guidance Document (DWR, 2015), the friction angle estimated from CPT soundings ranges from 32° to 42° with an average of 40°. Based on data available, a typical friction angle of 40° was chosen for analysis.

We used drained strength parameters of ϕ' = 40° and c' = 0 psf.



Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	YOUNG BAY MUD UNDRAINED STRENGTH VS. EFFECTIVE STRESS
Marin County Flood Control and Water Conservation District		November 2019 Figure B-1



Strength Parameters Used for Analysis:

	Layer	1	2	3
Drained Parameters	c' (ksf)	0.05	0	0
	φ' (deg)	28	27	38
Undrained Parameters	c (ksf)	1.20	0.20	--
	φ (deg)	0	10	--

Legend for Atterberg Limits

PL w LL Atterberg Limits from Soil Sample

● As Received Water Content

Notes:

1. For clay-like soils, use N_{60}. For sand-like soils, use $N_{1,60}$.
2. PL=Plastic Limit, w=As Received Water Content,LL=Liquid Limit.

Novato Creek Levee Evaluation Project
Novato, CA

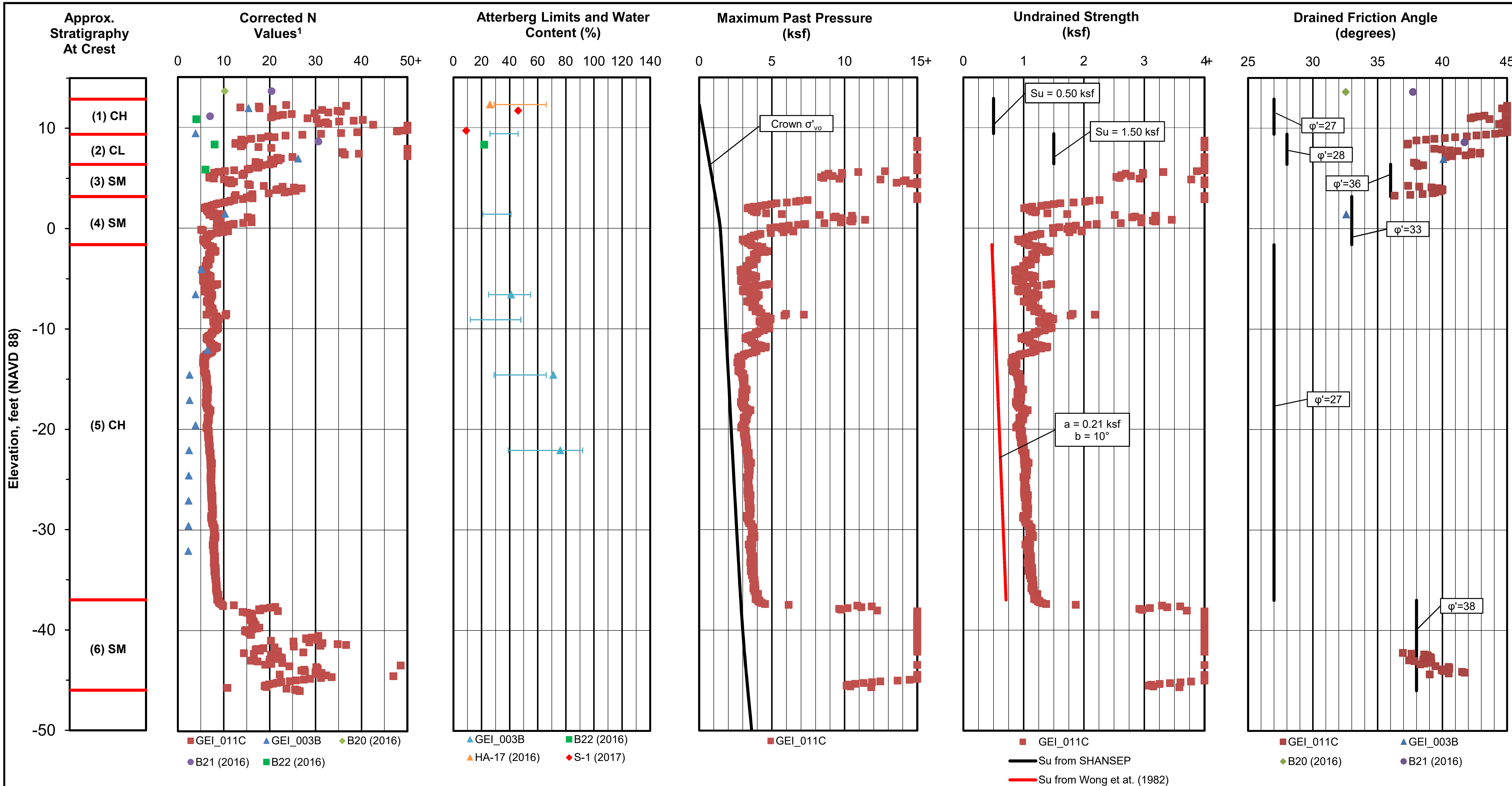
Marin County Flood Control and Water Conservation District

GEI Consultants

SOIL PROPERTIES VS. ELEVATION
NOVATO CREEK LEFT BANK LEVEE
STA NCLB 300+89

November 2019

Figure B-2



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5	6
Drained Parameters	c' (ksf)	0	0.05	0	0	0	0
	ϕ' (deg)	27	28	36	33	27	38
Undrained Parameters	c (ksf)	0.50	1.50	--	--	0.20	--
	ϕ (deg)	0	0	--	--	10	--

Legend for Atterberg Limits

PL w LL — Atterberg Limits from Soil Sample

● As Received Water Content

Notes:

1. For clay-like soils, use N_{60}. For sand-like soils, use $N_{1,60}$.
2. PL=Plastic Limit, w=As Received Water Content, LL=Liquid Limit.

Novato Creek Levee Evaluation Project
Novato, CA

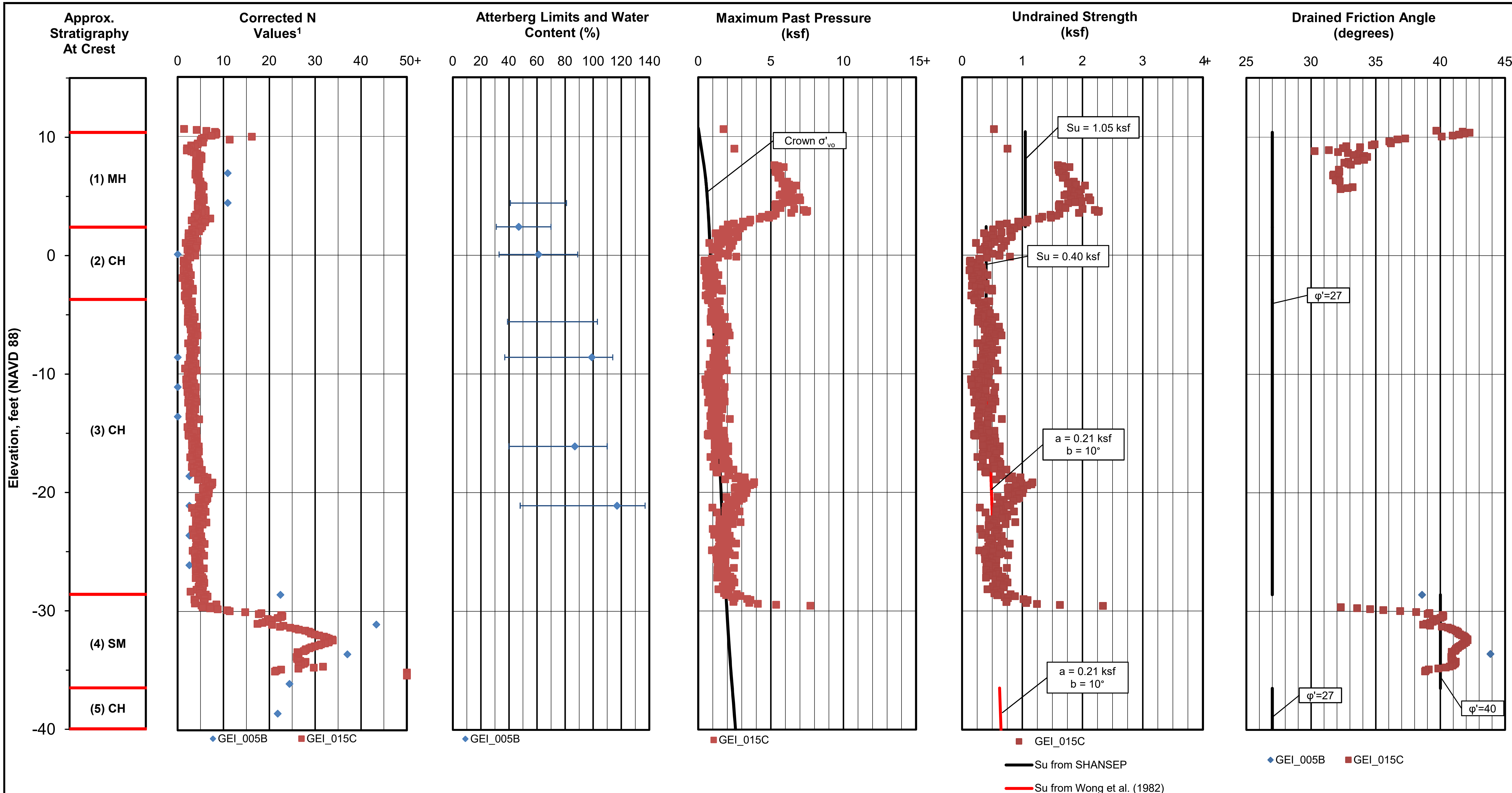
Marin County Flood Control and Water Conservation District

GEI Consultants

SOIL PROPERTIES VS. ELEVATION
LYNWOOD LEVEE
STA LL 260+68

November 2019

Figure B-3



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5
Drained Parameters	c' (ksf)	0	0	0	0	0
	ϕ' (deg)	27	27	27	40	27
Undrained Parameters	c (ksf)	1.05	0.40	0.20	--	0.20
	ϕ (deg)	0	0	10	--	10

Legend for Atterberg Limits

PL w LL — Atterberg Limits from Soil Sample

● As Received Water Content

Notes:

1. For clay-like soils, use N_{60}. For sand-like soils, use $N_{1,60}$.
2. PL=Plastic Limit, w=As Received Water Content,LL=Liquid Limit.

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

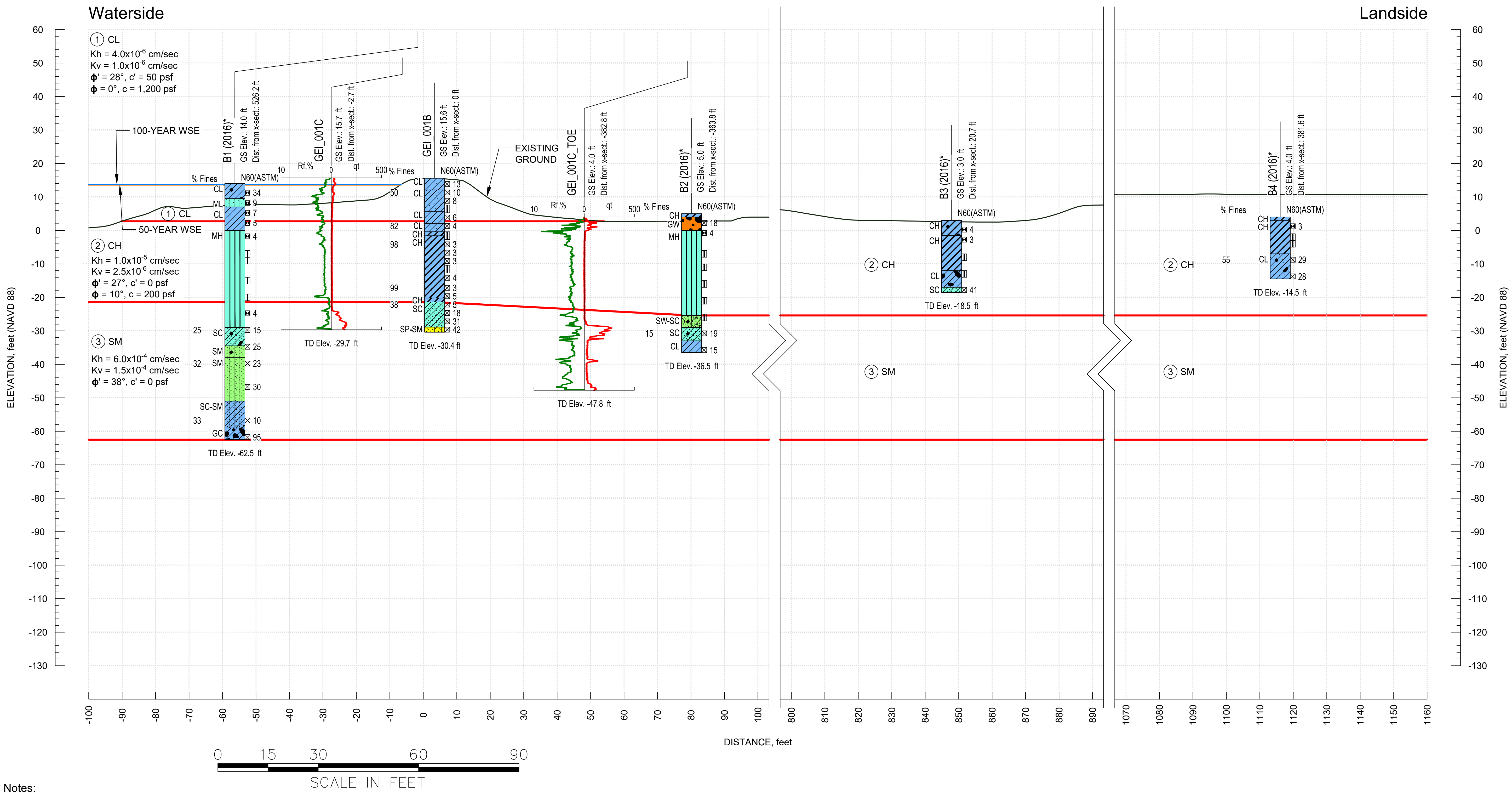
GEI Consultants

SOIL PROPERTIES VS. ELEVATION
PACHECO POND LEVEE
STA PP 33+22

November 2019 Figure B-4

Appendix C

Results of Analysis of Selected Reaches – Existing Conditions



Notes:

1. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations for these records are based on available information. USCS classifications may be interpreted from the available information.
2. Distance from cross-section (Dist. from x-sect.) is the difference in stationing between the exploration and location of cross-section. A positive value indicates the exploration is located up-station of the cross-section while a negative value indicates the exploration is located down-station.
3. The location of the explorations as shown on the cross-section is based on the perpendicular distance between the exploration and the levee stationing alignment.
4. The cross-section is oriented to look up-stream.

Novato Creek Levee Evaluation Project
Novato, CA

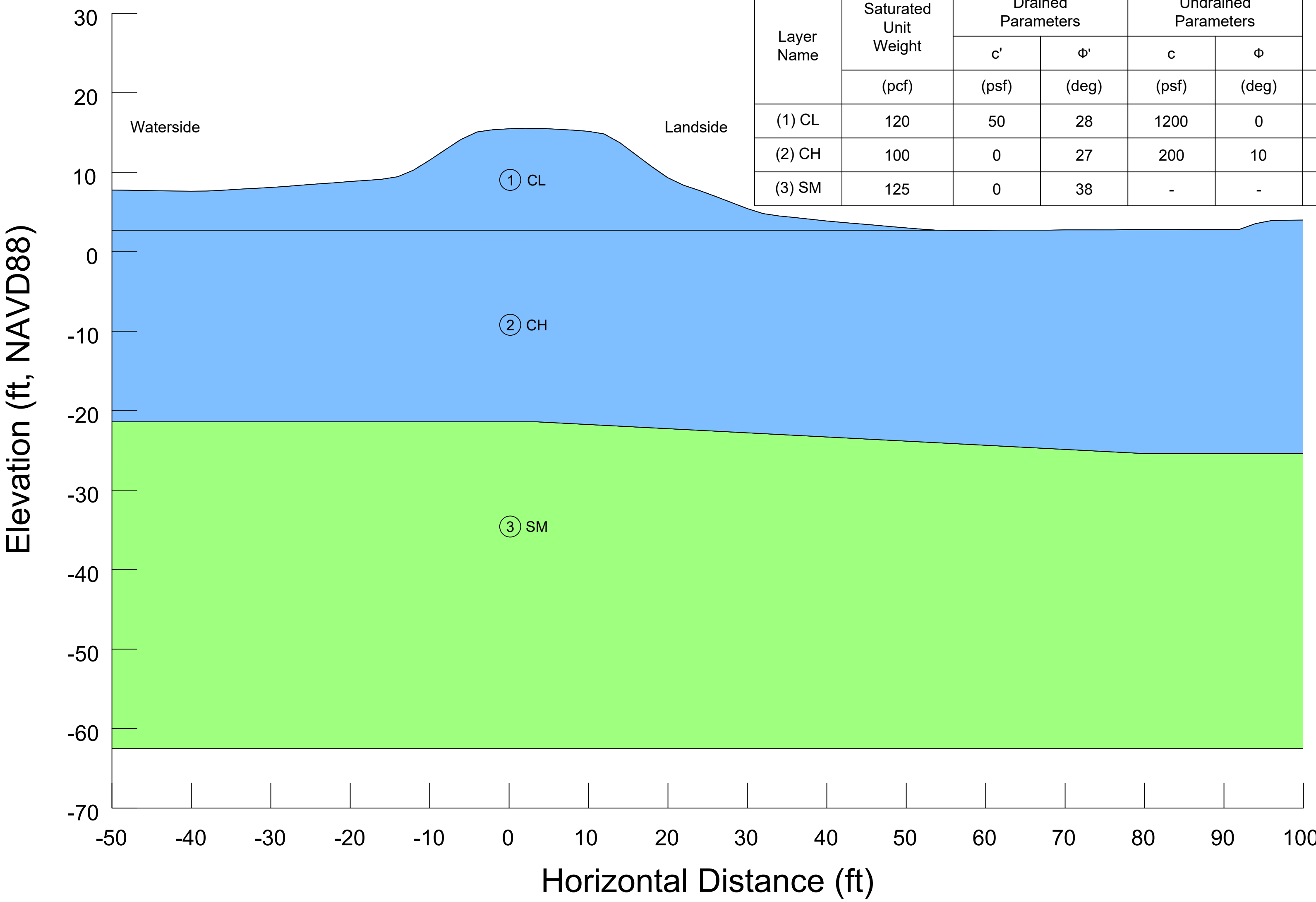
Marin County Flood Control and Water Conservation District

GEI Consultants

ANALYSIS CROSS-SECTION
NOVATO CREEK LEFT BANK LEVEE
STATION NCLB 300+89

December 2019 | Project #1802696 | Figure C-1

DRAWING: J:\Marin County FCD\Projects\1802696_Novato Creek LLAP\Task 3 - Existing Conditions Analysis\Cross Sections\Novato Creek_Cross Sections.dwg



Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters		K_v	K_h
		c'	Φ'	c	Φ		
	(pcf)	(psf)	(deg)	(psf)	(deg)	(cm/sec)	(cm/sec)
(1) CL	120	50	28	1200	0	1.0×10^{-6}	4.0×10^{-6}
(2) CH	100	0	27	200	10	2.5×10^{-6}	1.0×10^{-5}
(3) SM	125	0	38	-	-	1.5×10^{-4}	6.0×10^{-4}

Notes:

1. Model extends 735 feet to the landside and 100 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
BASE MODEL
NOVATO CREEK LEFT BANK LEVEE
STATION NCLB 300+89

December 2019 | Project #1802696 | Figure C-2

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Novato Creek Levee Evaluation Project
Seepage and Slope Stability Analysis Results

Levee: **Novato Creek Left Bank**
Station: **NCLB 300+89**

Analysis Run: **Exisiting Conditions**

Analyzed by: M. Weil
Date: 11/26/2019

Gradient is calculated at: Landside Toe (landside toe, low point at x ft from toe)

Water Surface: 50-Year

Water Surface Elevation 13.56 feet

Exit Gradient (i) Calculation	x-Coordinate	y-Coordinate	Total Head	i
Landside Toe	54.00 feet	2.70 feet	2.70 feet	0.06
Bottom of Blanket	54.00 feet	-24.03 feet	4.22 feet	

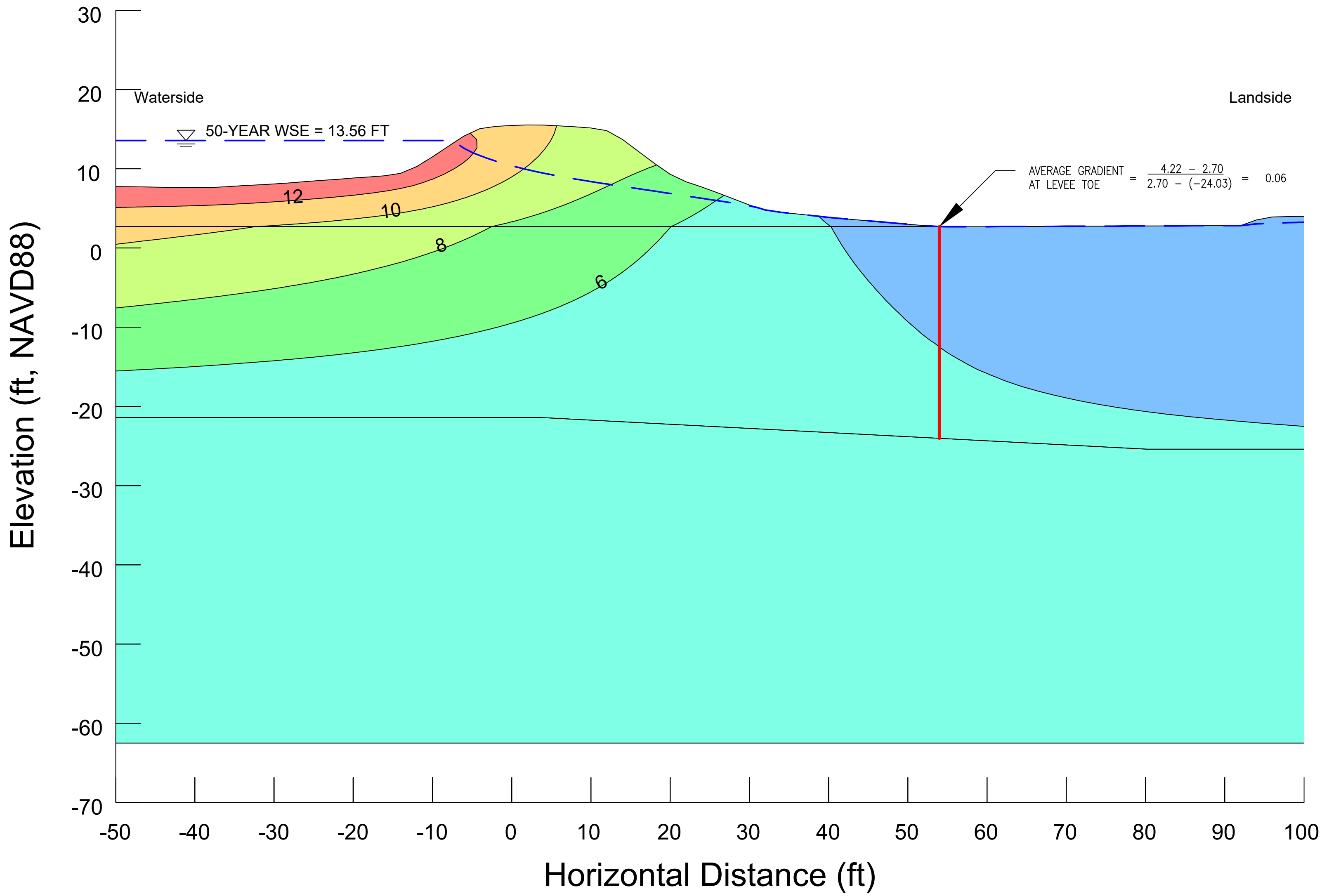
Seepage Breakout above Toe, ft 2.7 ft
Steady State Landside Slope Stability, FS 1.2 (circular)

Water Surface: 100-Year

Water Surface Elevation 13.76 feet

Exit Gradient (i) Calculation	x-Coordinate	y-Coordinate	Total Head	i
Landside Toe	54.00 feet	2.70 feet	2.70 feet	0.06
Bottom of Blanket	54.00 feet	-24.03 feet	4.25 feet	

Seepage Breakout above Toe, ft 2.7 ft
Steady State Landside Slope Stability, FS 1.2 (circular)
Rapid Draw Down Slope Stability, FS 1.8 (circular)



Notes:

1. Model extends 735 feet to the landside and 100 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
50-YEAR WSE - SEEPAGE RESULTS
NOVATO CREEK LEFT BANK LEVEE
STATION NCLB 300+89

December 2019 | Project #1802696 | Figure C-3

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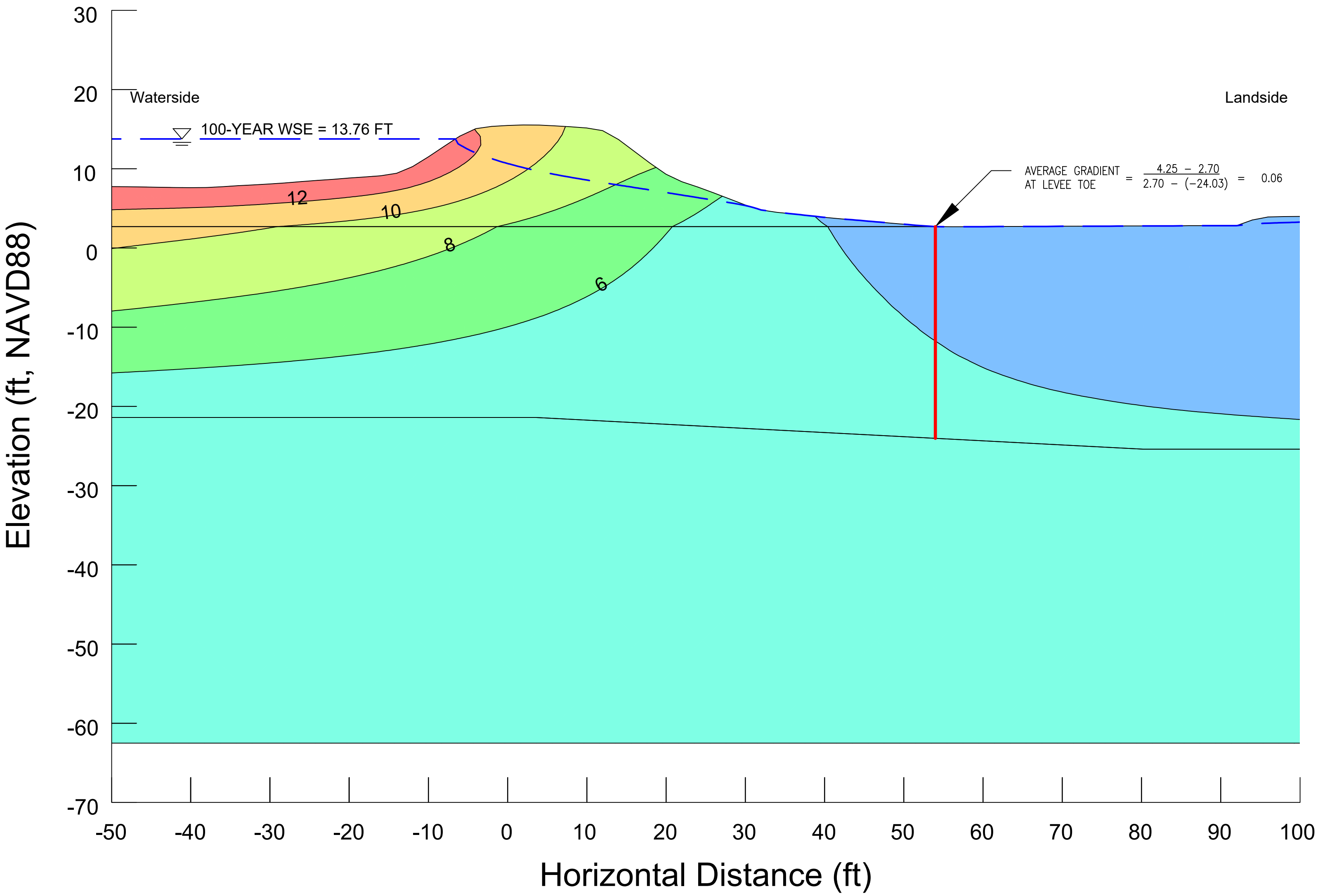

Notes:

1. Model extends 735 feet to the landside and 100 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**EXISTING CONDITIONS** **100-YEAR WSE - SEEPAGE RESULTS** NOVATO CREEK LEFT BANK LEVEE STATION NCLB 300+89
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Figure C-4

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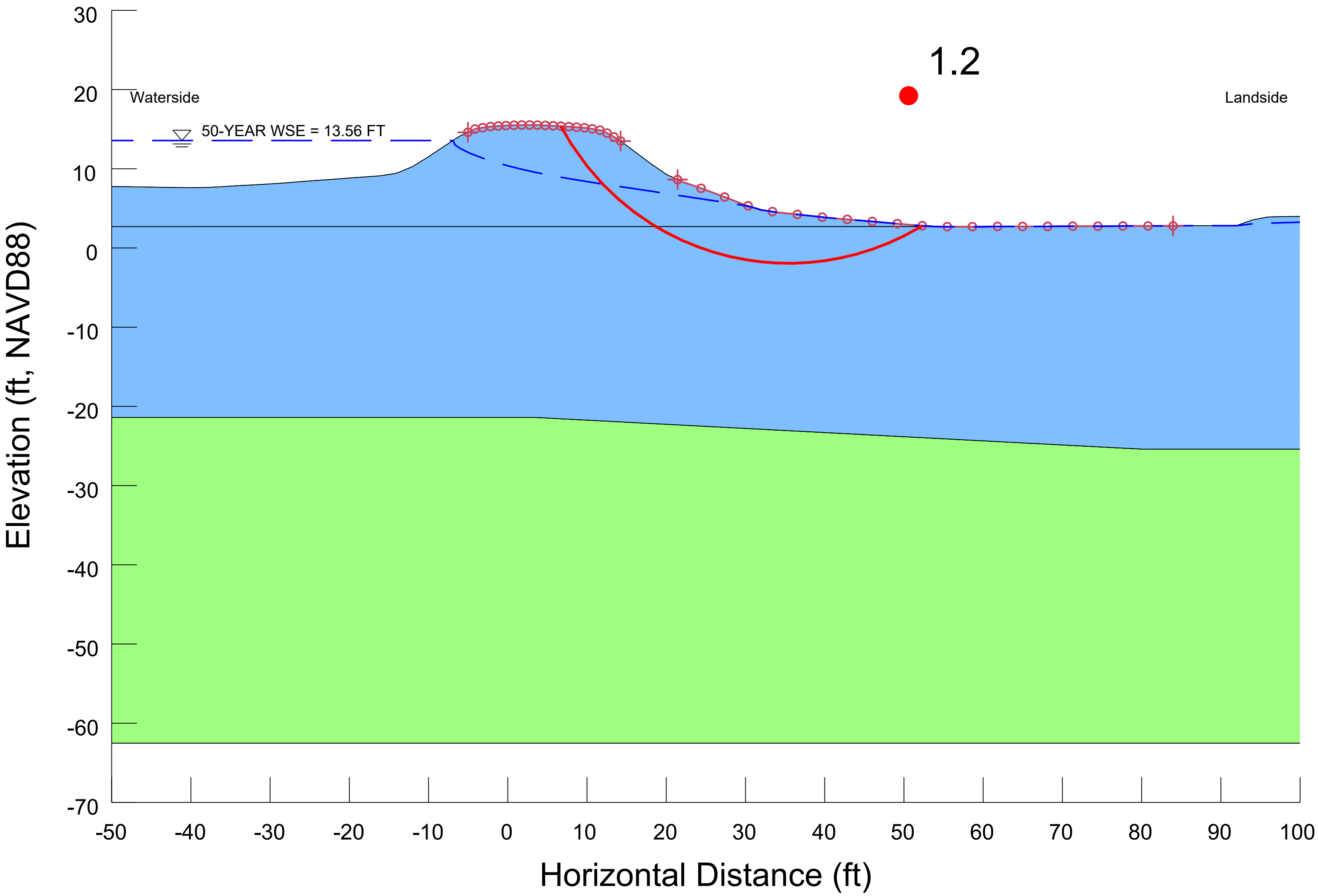


Notes:

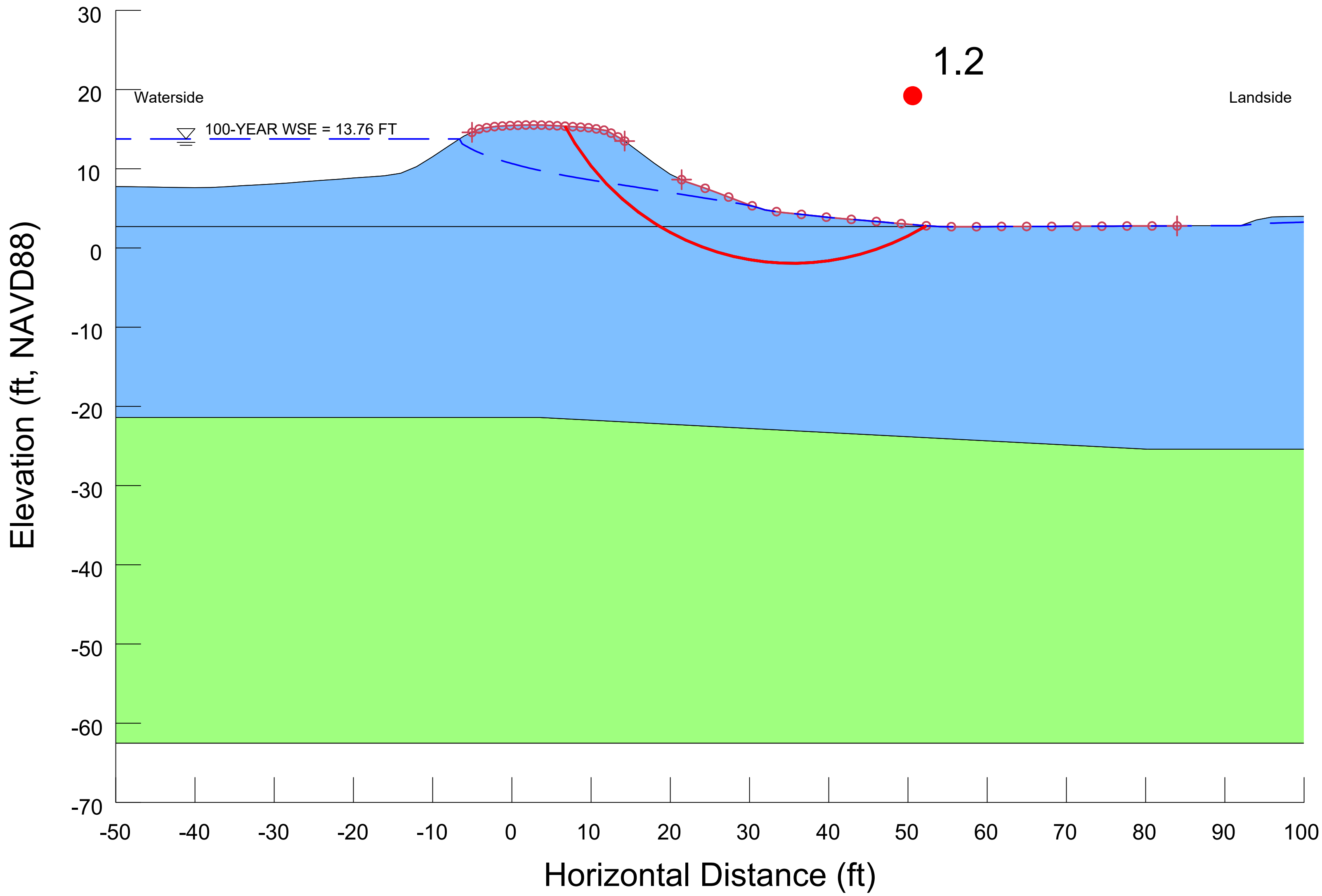
1. Model extends 735 feet to the landside and 100 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**EXISTING CONDITIONS** **50-YEAR WSE - STABILITY RESULTS** NOVATO CREEK LEFT BANK LEVEE STATION NCLB 300+89
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Figure C-5

DRAWING: J:\Marin County FCD\Projects\1802696_Novato Creek LLAP\Task 3 - Existing Conditions Analysis\GER\Figures\CADD\NC_300+89.dwg



Notes:

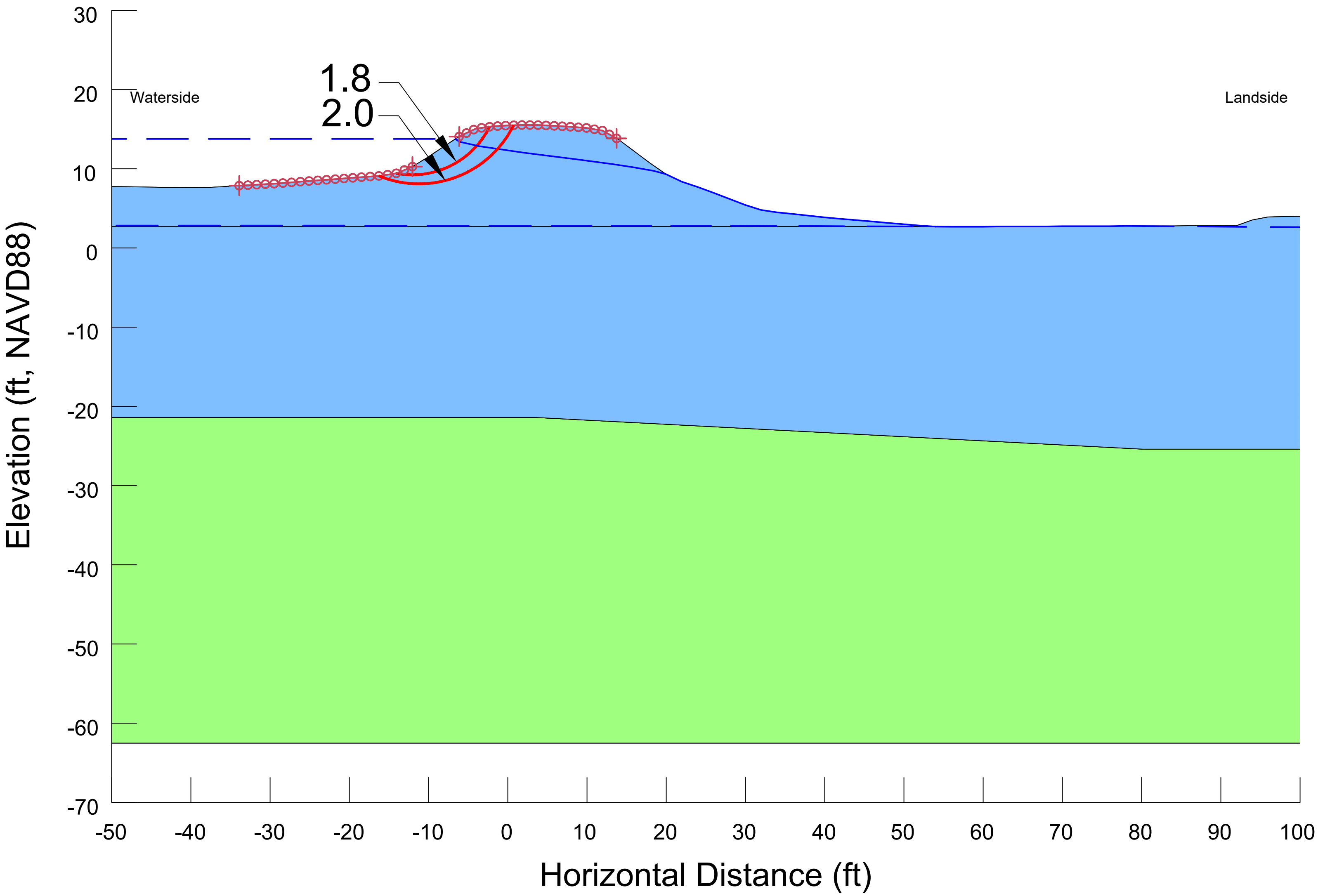
1. Model extends 735 feet to the landside and 100 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**EXISTING CONDITIONS** **100-YEAR WSE - STABILITY RESULTS** NOVATO CREEK LEFT BANK LEVEE STATION NCLB 300+89
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Figure C-6

DRAWING: J:\Marin County FCD\Projects\1802696_Novato Creek LLAP\Task 3 - Existing Conditions Analysis\GER\Figures\CADD\NC_300+89.dwg



Notes:

1. Model extends 735 feet to the landside and 100 feet to the waterside of the alignment.

0 10 20 40
SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
100-YEAR WSE - RAPID DRAWDOWN RESULTS
NOVATO CREEK LEFT BANK LEVEE
STATION NCLB 300+89

December 2019 | Project #1802696 | Figure C-7

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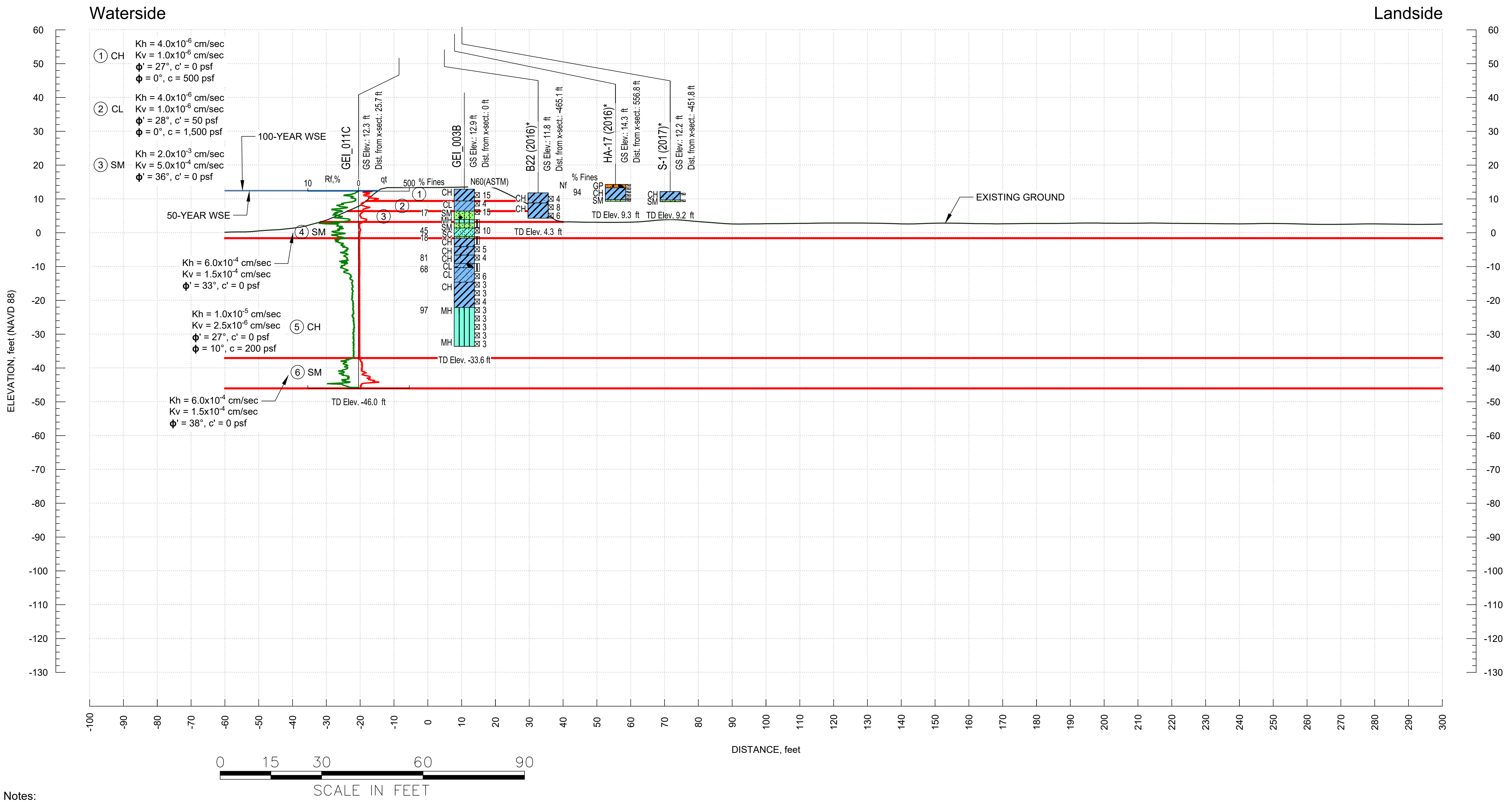

Notes:

1. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations for these records are based on available information. USCS classifications may be interpreted from the available information.
2. Distance from cross-section (Dist. from x-sect.) is the difference in stationing between the exploration and location of cross-section. A positive value indicates the exploration is located up-station of the cross-section while a negative value indicates the exploration is located down-station.
3. The location of the explorations as shown on the cross-section is based on the perpendicular distance between the exploration and the levee stationing alignment.
4. The cross-section is oriented to look down-stream.

Novato Creek Levee Evaluation Project Novato, CA	**ANALYSIS CROSS-SECTION** LYNWOOD LEVEE STATION LL 260+68
Marin County Flood Control and Water Conservation District	December 2019 Project #1802696 Figure C-8

GEI Consultants

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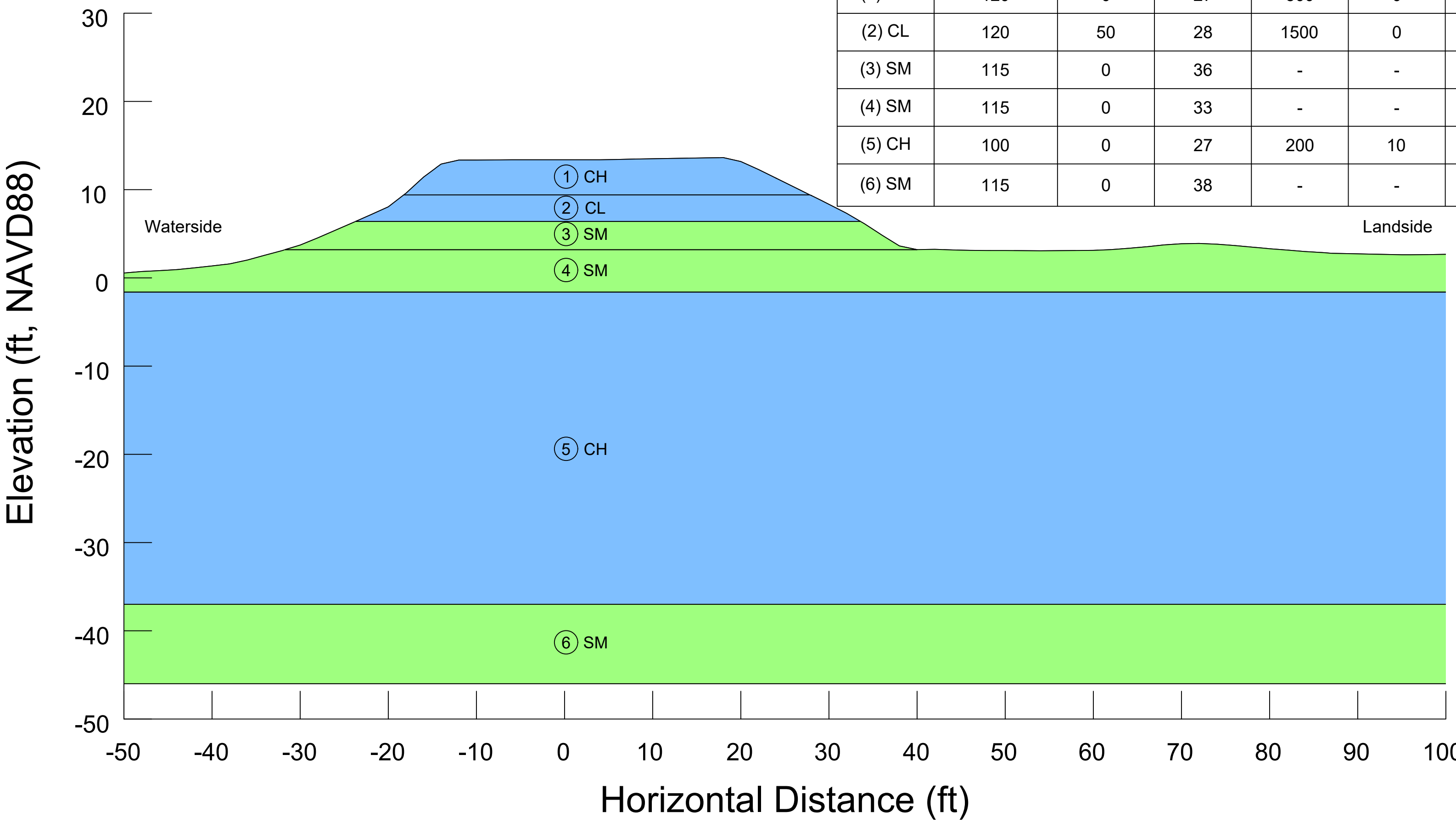
Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters		K_v	K_h
		c'	Φ'	c	Φ		
	(pcf)	(psf)	(deg)	(psf)	(deg)	(cm/sec)	(cm/sec)
(1) CH	120	0	27	500	0	1.0×10^{-6}	4.0×10^{-6}
(2) CL	120	50	28	1500	0	1.0×10^{-6}	4.0×10^{-6}
(3) SM	115	0	36	-	-	5.0×10^{-4}	2.0×10^{-3}
(4) SM	115	0	33	-	-	1.5×10^{-4}	6.0×10^{-4}
(5) CH	100	0	27	200	10	2.5×10^{-6}	1.0×10^{-5}
(6) SM	115	0	38	-	-	1.5×10^{-4}	6.0×10^{-4}



Notes:

1. Model extends 2,000 feet to the landside and 60 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
BASE MODEL
LYNWOOD LEVEE
STATION LL 260+68

December 2019 | Project #1802696 | Figure C-9

DRAWING: J:\Marin County FCD\Projects\1802696_Novato Creek LLAP\Task 3 - Existing Conditions Analysis\GER\Figures\CADD\LL_260+68.dwg

Novato Creek Levee Evaluation Project
Seepage and Slope Stability Analysis Results

Levee: **Lynwood Levee**
Station: **LL 260+68**

Analysis Run: **Exisiting Conditions**

Analyzed by: M. Weil
Date: 11/26/2019

Gradient is calculated at: Landside Toe (landside toe, low point at x ft from toe)

Water Surface: 50-Year

Water Surface Elevation 12.36 feet

Exit Gradient (i) Calculation	x-Coordinate	y-Coordinate	Total Head	i
Landside Toe	40.00 feet	3.20 feet	3.20 feet	**N/A**
Bottom of Blanket	N/A feet	N/A feet	N/A feet	

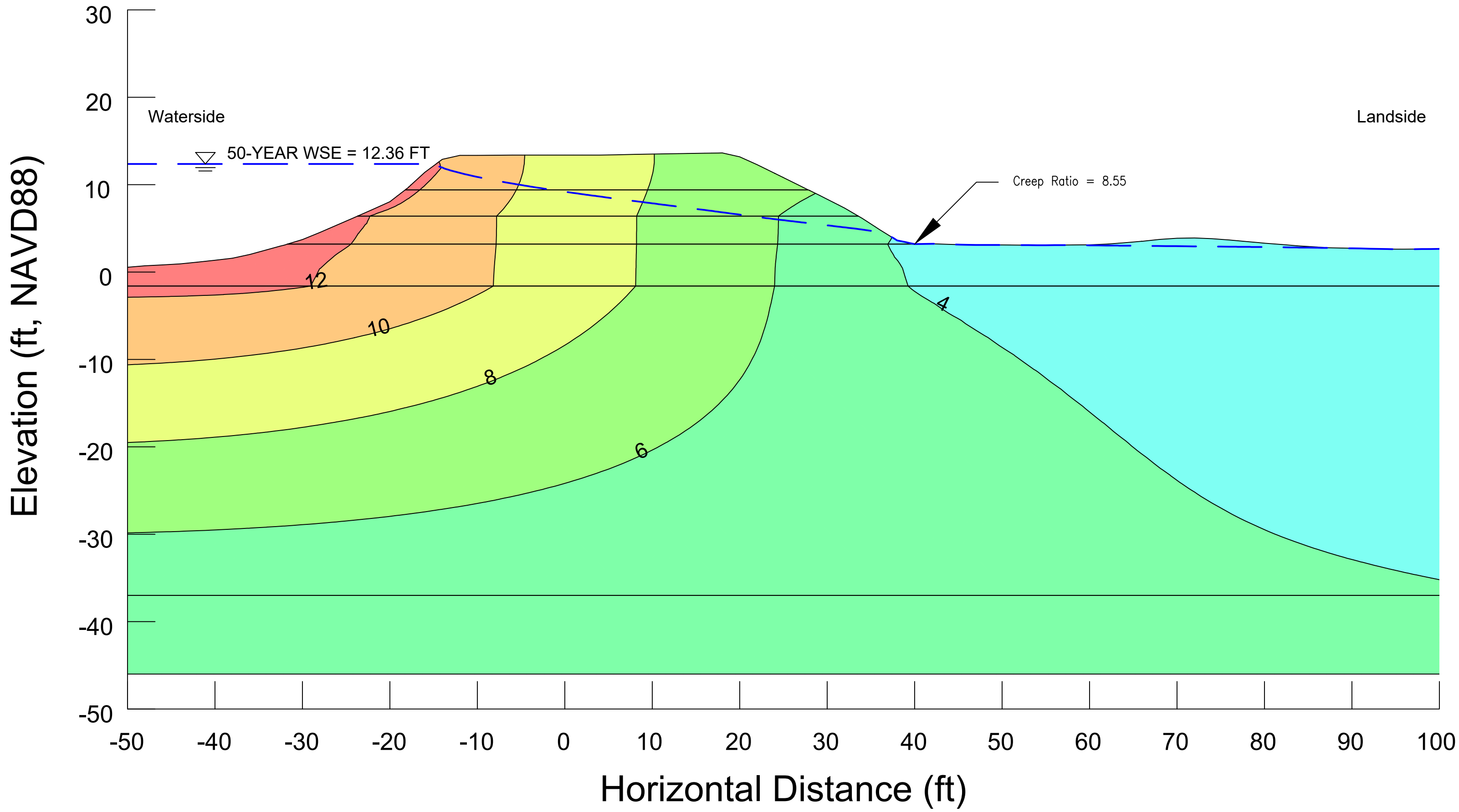
Seepage Breakout above Toe, ft 1.0 ft
Steady State Landside Slope Stability, FS 1.3 (circular)

Water Surface: 100-Year

Water Surface Elevation 12.49 feet

Exit Gradient (i) Calculation	x-Coordinate	y-Coordinate	Total Head	i
Landside Toe	40.00 feet	3.20 feet	3.20 feet	**N/A**
Bottom of Blanket	N/A feet	N/A feet	N/A feet	

Seepage Breakout above Toe, ft 1.0 ft
Steady State Landside Slope Stability, FS 1.3 (circular)
Rapid Draw Down Slope Stability, FS 1.3 (circular)



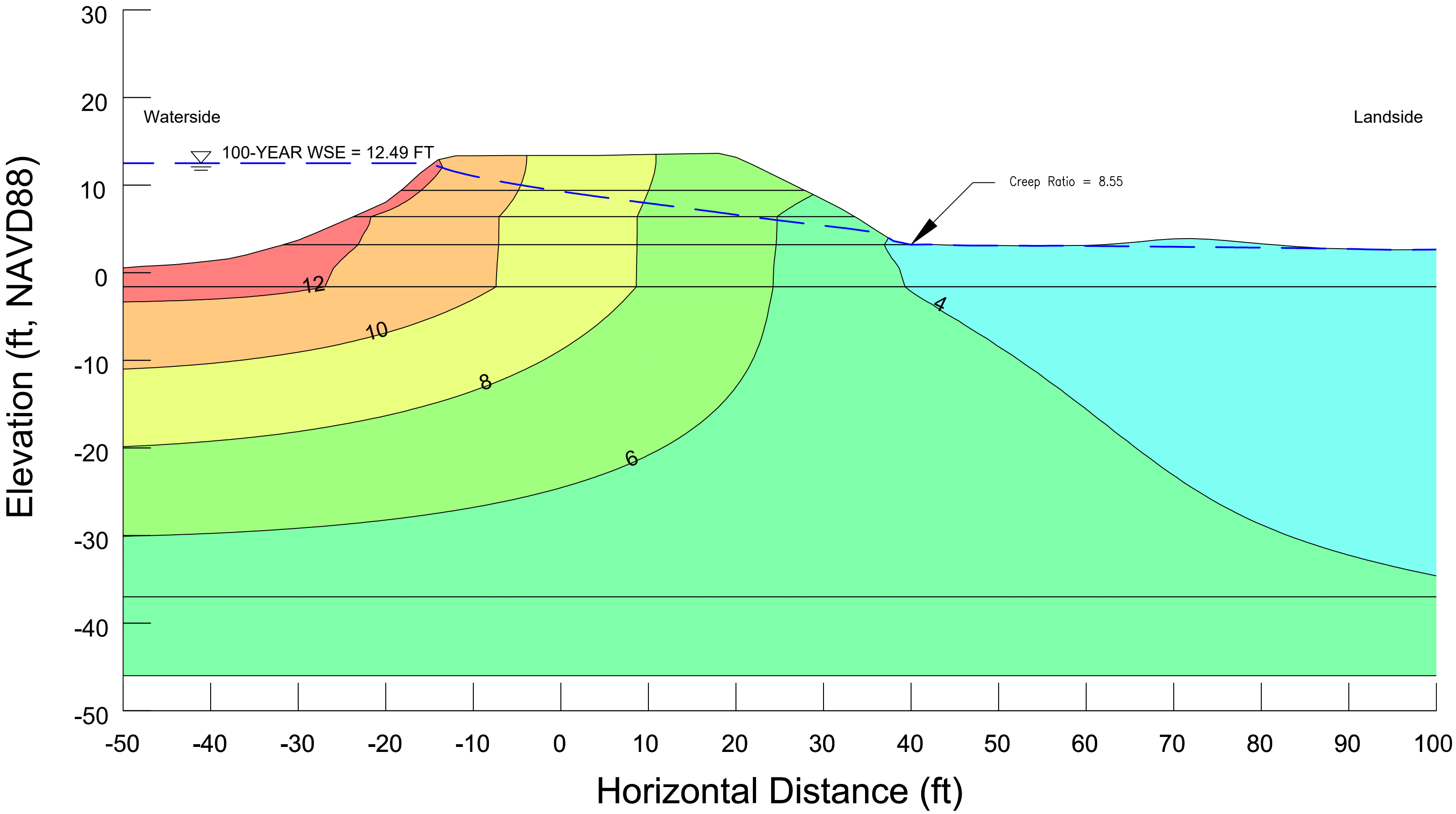
Notes:

1. Model extends 2,000 feet to the landside and 60 feet to the waterside of the alignment.

0 10 20 40
SCALE IN FEET

DRAWING: J:\Marin County FCD\Projects\1802696_Novato Creek LLAP\Task 3 - Existing Conditions Analysis\GER\Figures\CADD\LL_260+68.dwg

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**EXISTING CONDITIONS** **50-YEAR WSE - SEEPAGE RESULTS** LYNWOOD LEVEE STATION LL 260+68
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Figure C-10



Notes:

1. Model extends 2,000 feet to the landside and 60 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
100-YEAR WSE - SEEPAGE RESULTS
LYNWOOD LEVEE
STATION LL 260+68

December 2019 | Project #1802696 | Figure C-11

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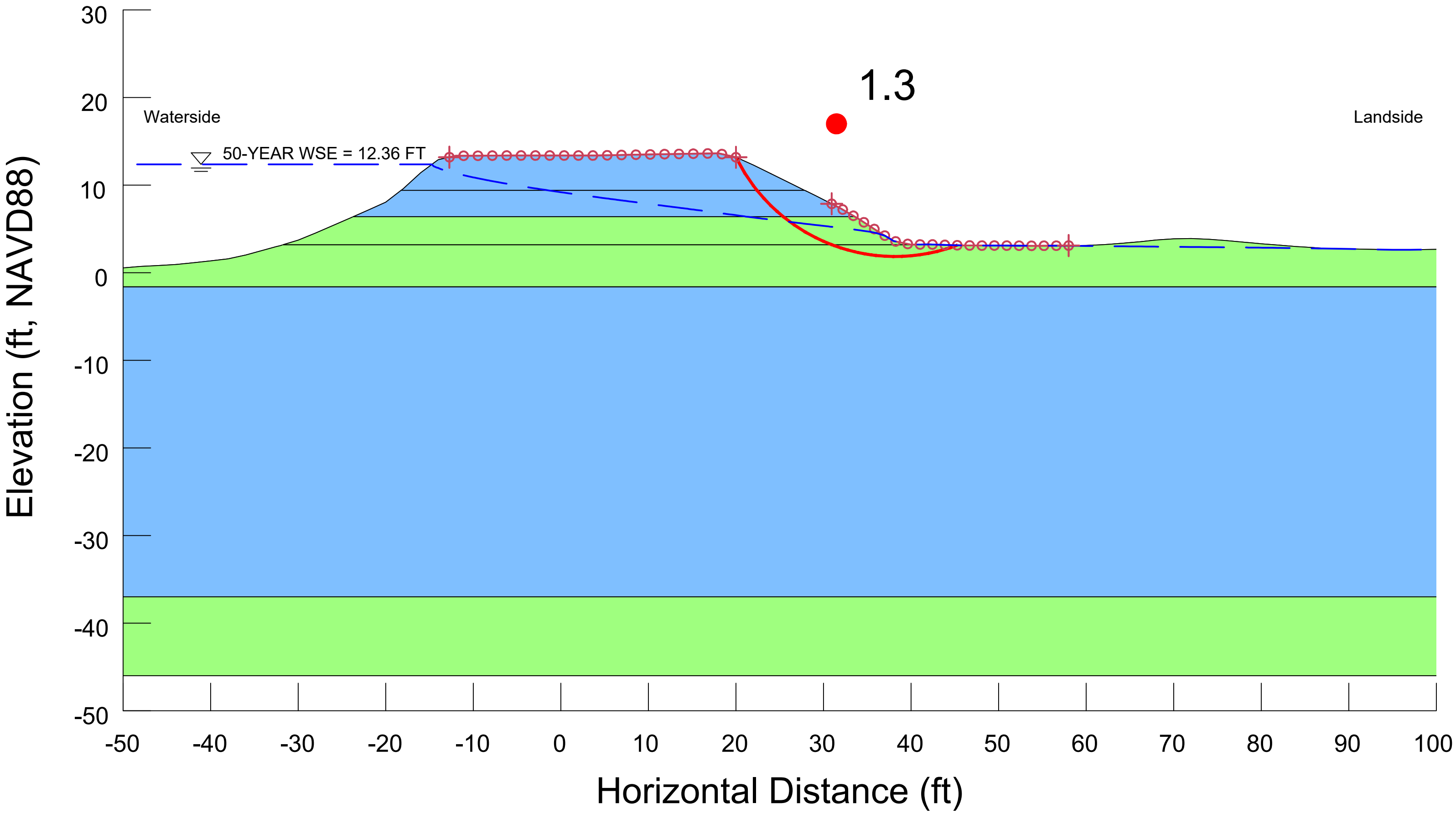

Notes:

1. Model extends 2,000 feet to the landside and 60 feet to the waterside of the alignment.

0 10 20 40
SCALE IN FEET

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**EXISTING CONDITIONS** **50-YEAR WSE - STABILITY RESULTS** LYNWOOD LEVEE STATION LL 260+68
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Figure C-12

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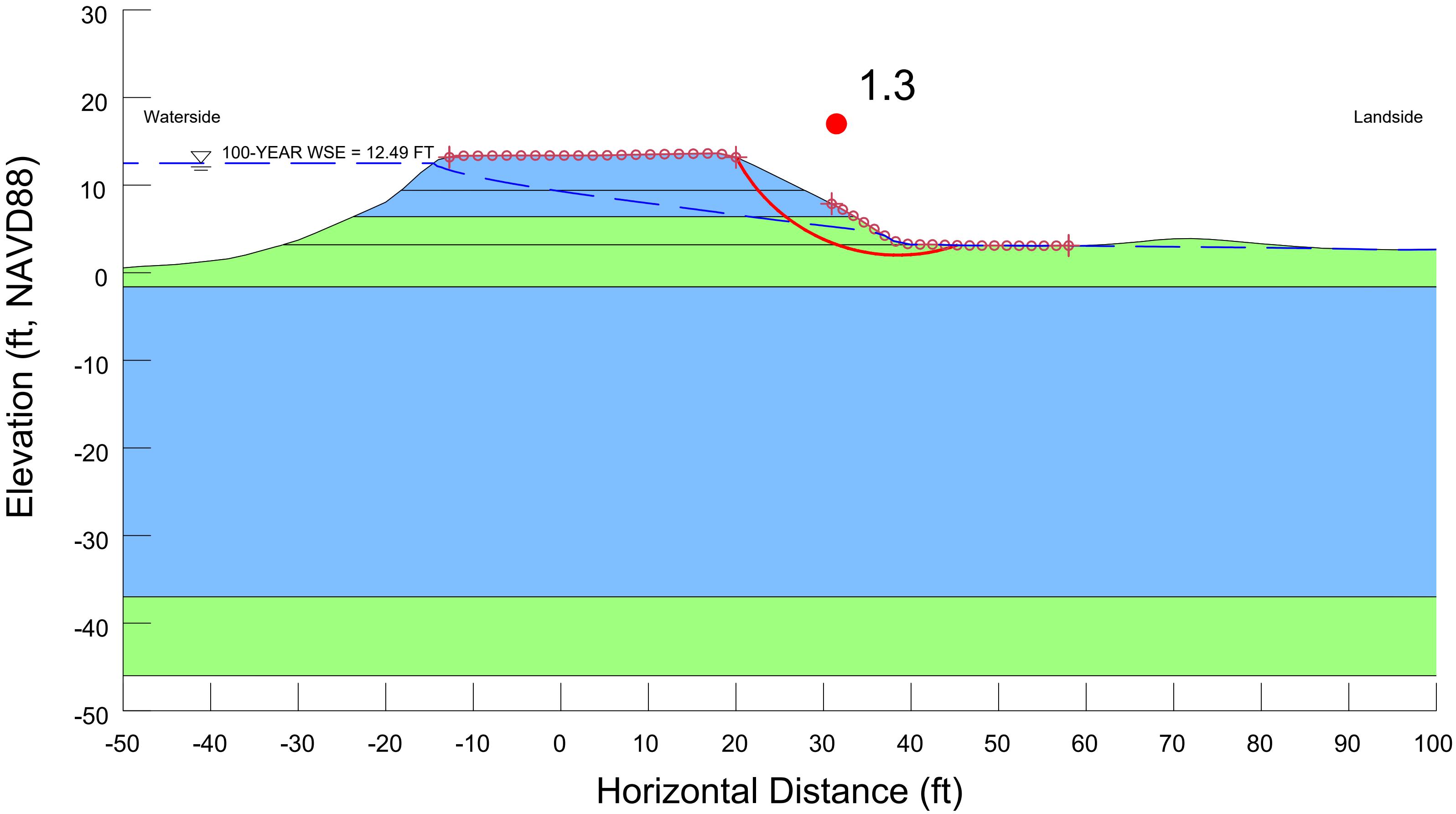

Notes:

1. Model extends 2,000 feet to the landside and 60 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
100-YEAR WSE - STABILITY RESULTS
LYNWOOD LEVEE
STATION LL 260+68

December 2019 | Project #1802696 | Figure C-13

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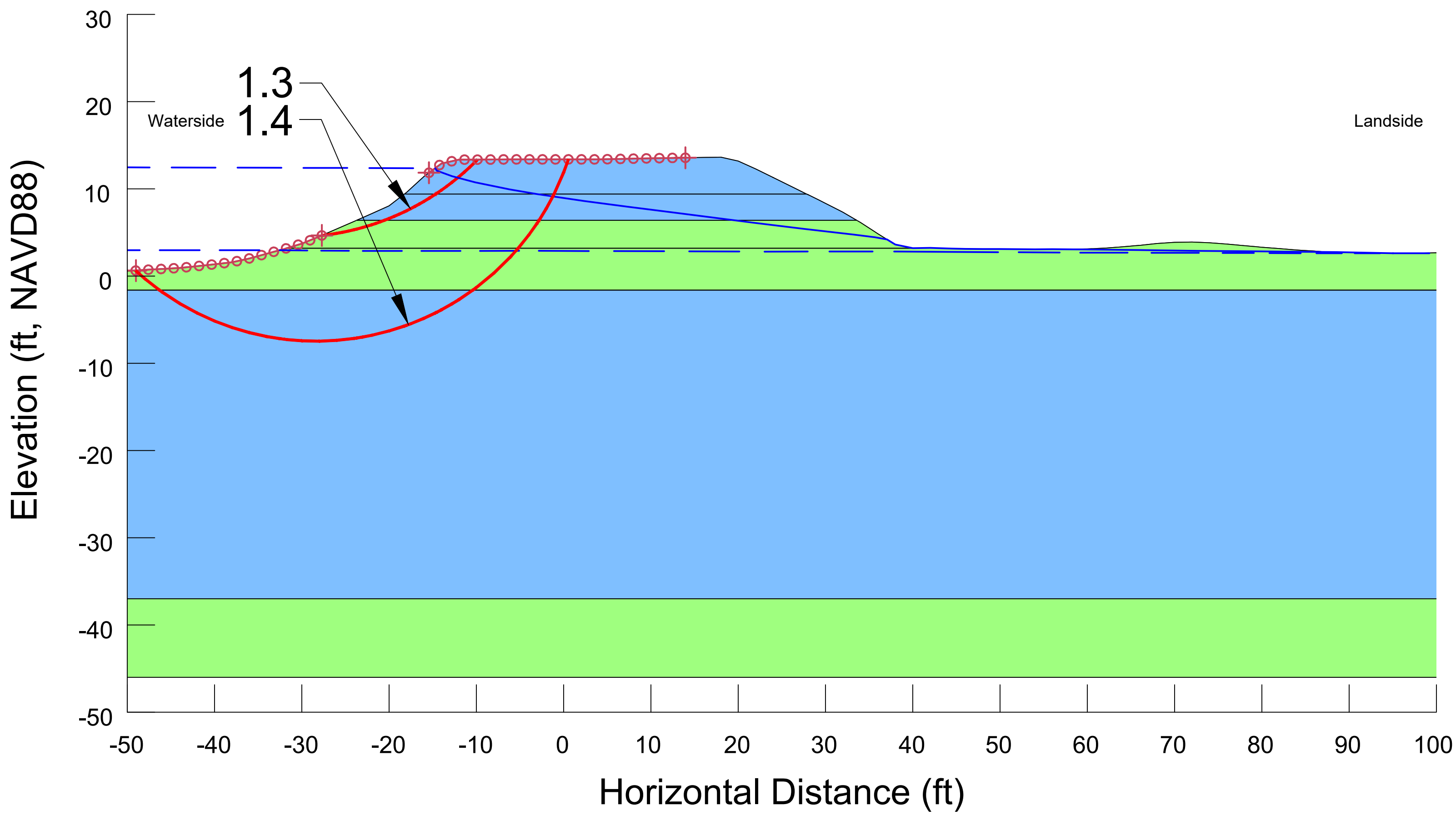

Notes:

1. Model extends 2,000 feet to the landside and 60 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
100-YEAR WSE - RAPID DRAWDOWN RESULTS
LYNWOOD LEVEE
STATION LL 260+68

December 2019 | Project #1802696 | Figure C-14

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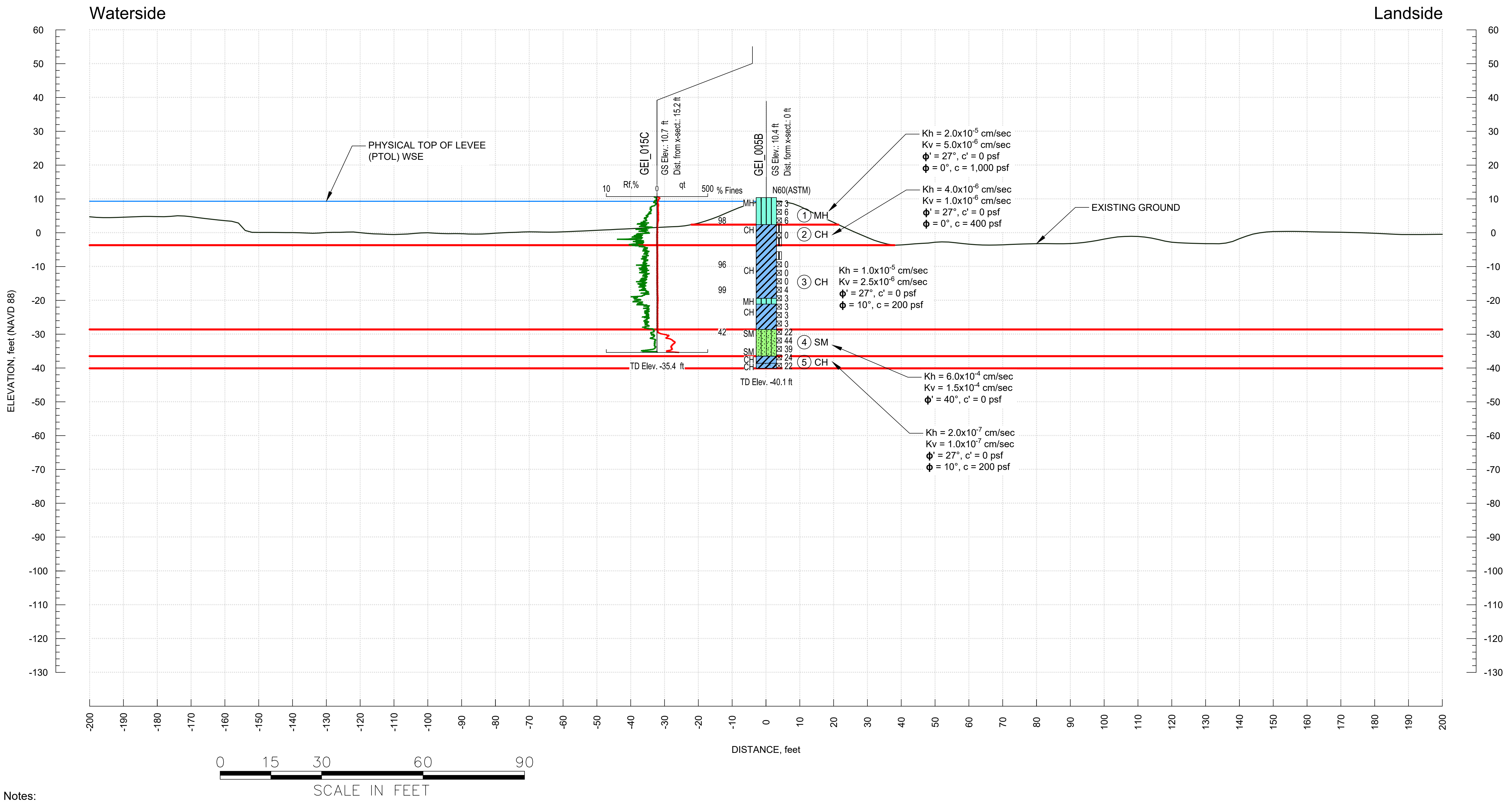


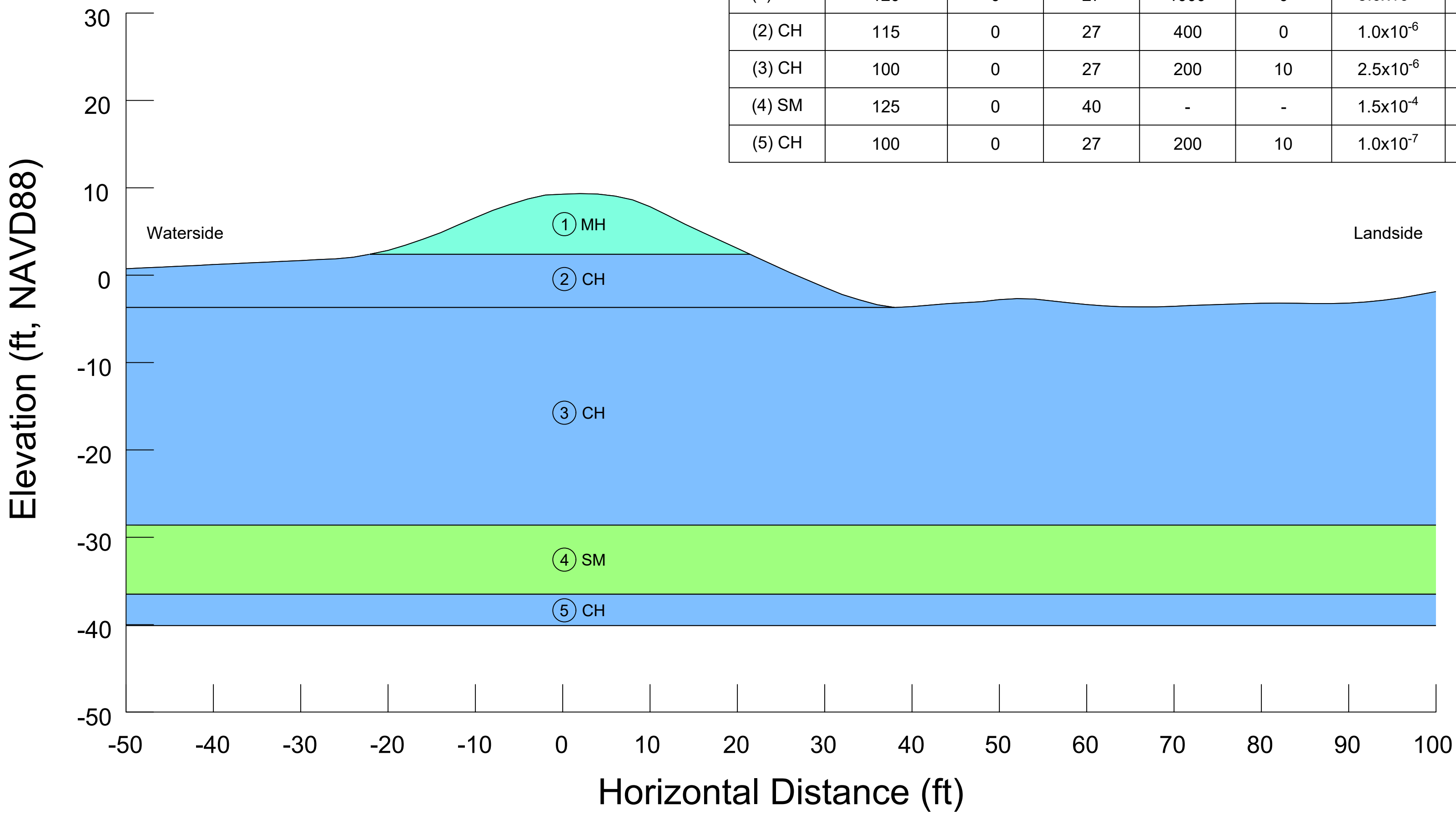
Notes:

1. Historic explorations are denoted by an asterisk (*) after the exploration name. Locations for these records are based on available information. USCS classifications may be interpreted from the available information.
2. Distance from cross-section (Dist. from x-sect.) is the difference in stationing between the exploration and location of cross-section. A positive value indicates the exploration is located up-station of the cross-section while a negative value indicates the exploration is located down-station.
3. The location of the explorations as shown on the cross-section is based on the perpendicular distance between the exploration and the levee stationing alignment.
4. The cross-section is oriented to look down-stream.

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**ANALYSIS CROSS-SECTION** PACHECO POND LEVEE STATION PP 33+22
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Figure C-15

DRAWING: J:\Marin County FCD\Projects\1802696_Novato Creek LLAP\Task 3 - Existing Conditions Analysis\Cross Sections\Novato Creek_Cross Sections.dwg

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters		K_v	K_h
		c'	Φ'	c	Φ		
	(pcf)	(psf)	(deg)	(psf)	(deg)	(cm/sec)	(cm/sec)
(1) MH	120	0	27	1000	0	5.0×10^{-6}	2.0×10^{-5}
(2) CH	115	0	27	400	0	1.0×10^{-6}	4.0×10^{-6}
(3) CH	100	0	27	200	10	2.5×10^{-6}	1.0×10^{-5}
(4) SM	125	0	40	-	-	1.5×10^{-4}	6.0×10^{-4}
(5) CH	100	0	27	200	10	1.0×10^{-7}	2.0×10^{-7}



Notes:

1. Model extends 1,775 feet to the landside and 500 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
BASE MODEL
PACHECO POND LEVEE
STATION PP 33+22

December 2019 | Project #1802696 | Figure C-16

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Novato Creek Levee Evaluation Project
Seepage and Slope Stability Analysis Results

Levee:	**Pachecho Pond**
Station:	**PP 33+22**

Analysis Run: **Exisiting Conditions**

Analyzed by: M. Weil
Date: 11/26/2019

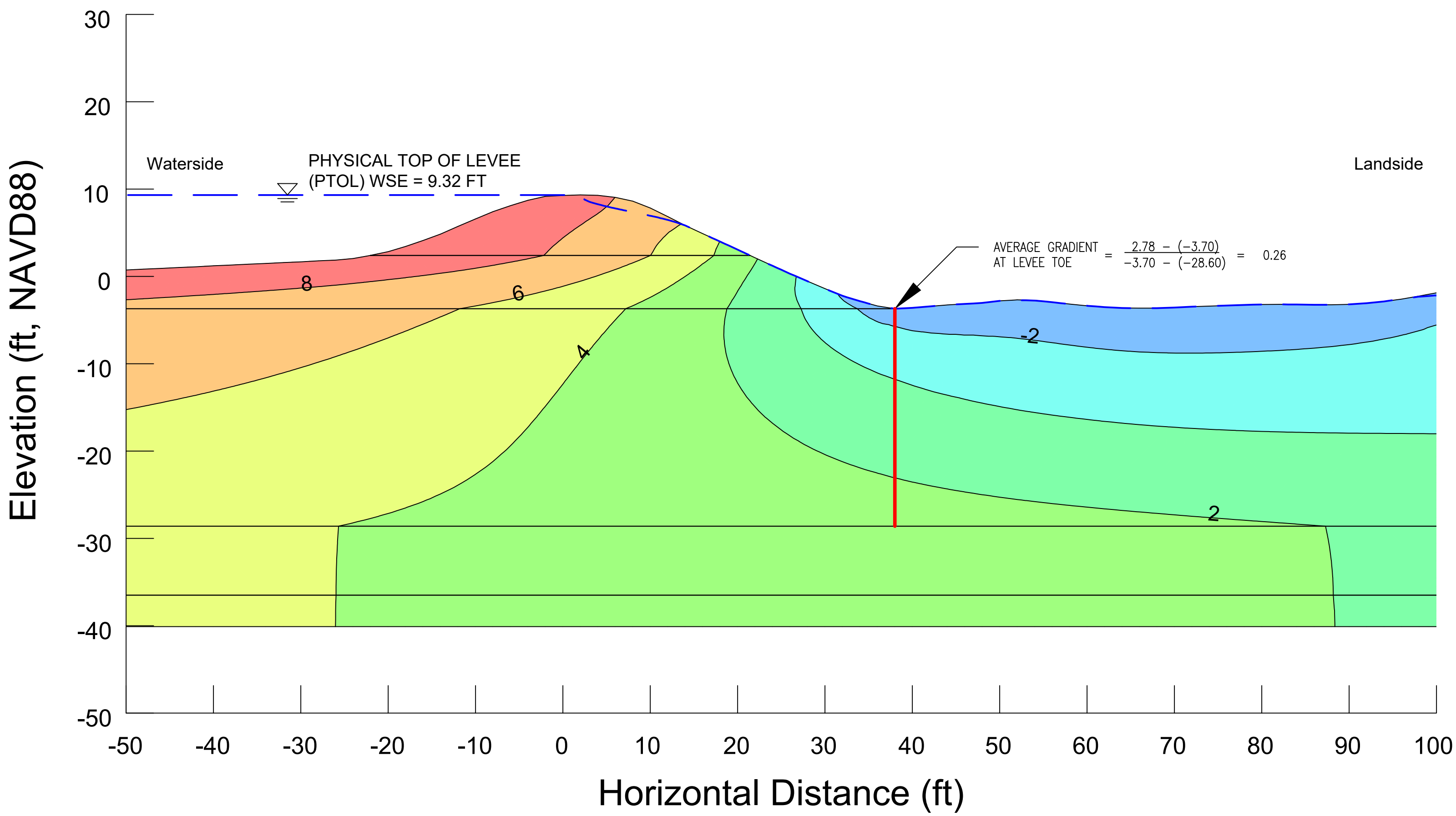
Gradient is calculated at: Landside Toe (landside toe, low point at x ft from toe)

Water Surface: Physical Top of Levee

Water Surface Elevation 9.3 feet

Exit Gradient (i) Calculation	x-Coordinate	y-Coordinate	Total Head	i
Landside Toe	38.00 feet	-3.70 feet	-3.70 feet	**0.26**
Bottom of Blanket	38.00 feet	-28.60 feet	2.78 feet	

Seepage Breakout above Toe, ft	9.5	ft
Steady State Landside Slope Stability, FS	0.5	(circular)
Rapid Draw Down Slope Stability, FS	1.5	(circular)



Notes:

1. Model extends 1,775 feet to the landside and 500 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project Novato, CA	GEI Consultants	**EXISTING CONDITIONS** **PTOL WSE - SEEPAGE RESULTS** PACHECO POND LEVEE STATION PP 33+22
Marin County Flood Control and Water Conservation District		December 2019 Project #1802696 Figure C-17

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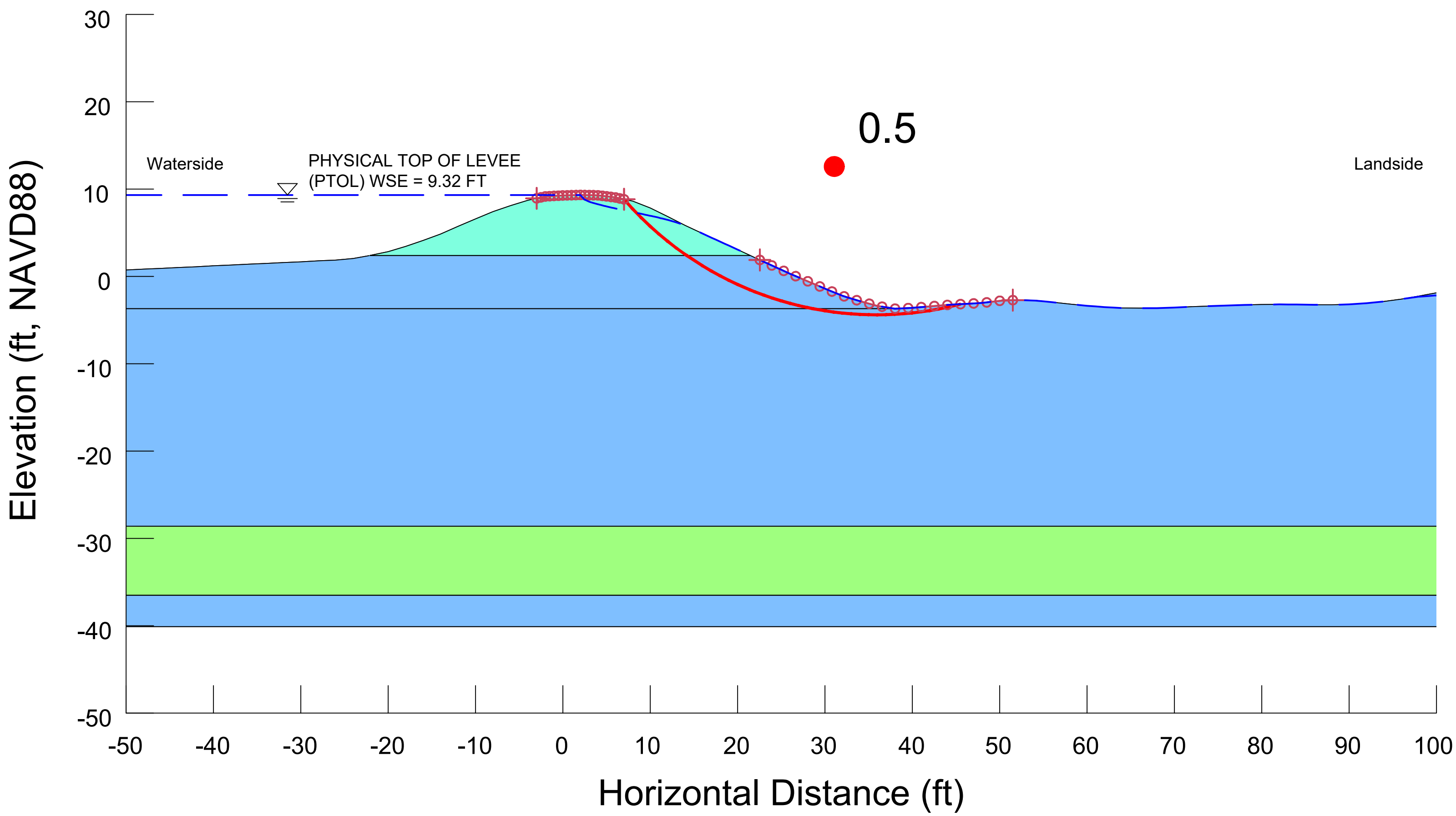

Notes:

1. Model extends 1,775 feet to the landside and 500 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
PTOL WSE - STABILITY RESULTS
PACHECO POND LEVEE
STATION PP 33+22

December 2019 | Project #1802696 | Figure C-18

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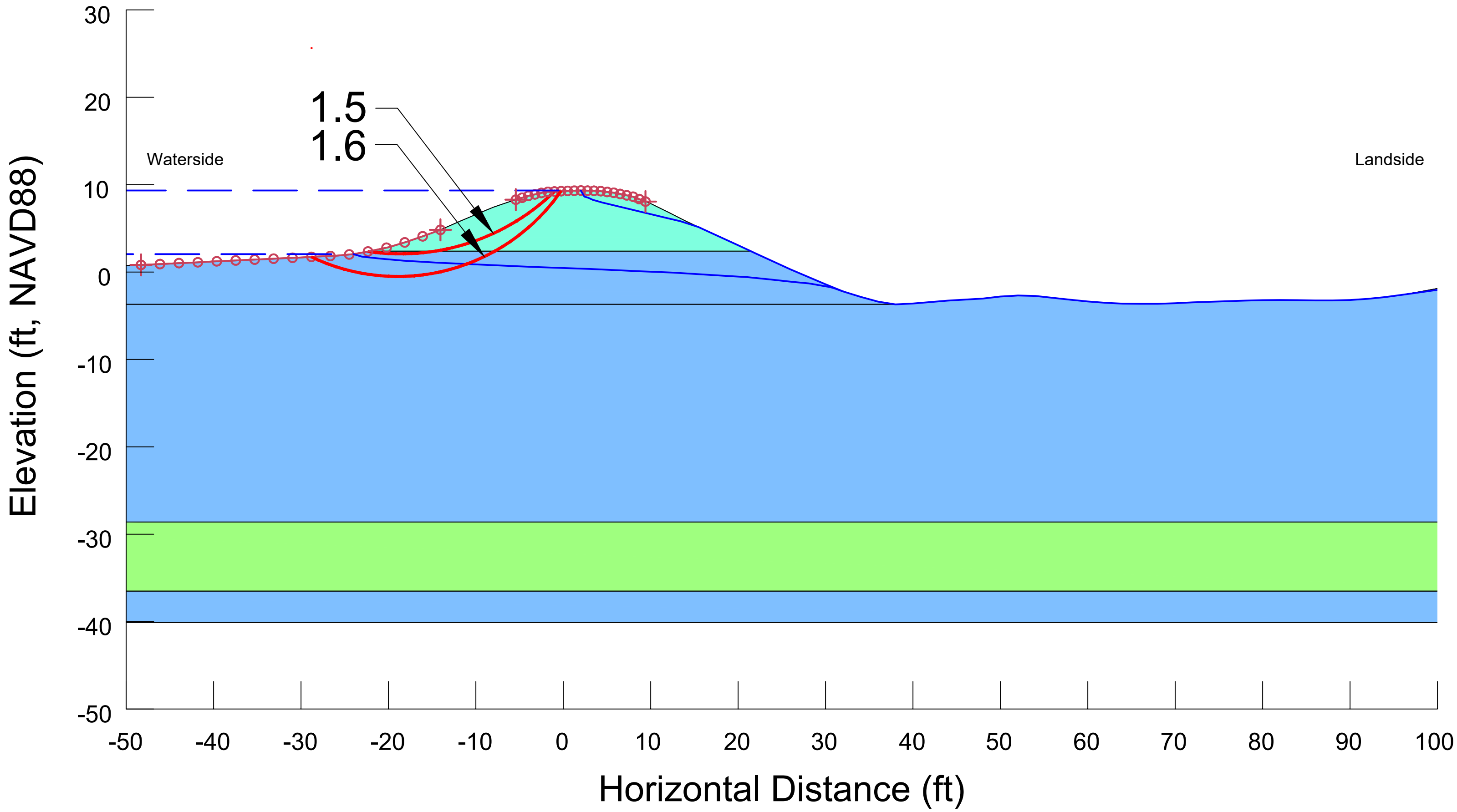

Notes:

1. Model extends 1,775 feet to the landside and 500 feet to the waterside of the alignment.

0 10 20 40

SCALE IN FEET

Novato Creek Levee Evaluation Project
Novato, CA

Marin County Flood Control and Water Conservation District

GEI Consultants

EXISTING CONDITIONS
PTOL WSE - RAPID DRAWDOWN RESULTS
PACHECO POND LEVEE
STATION PP 33+22

December 2019 | Project #1802696 | Figure C-19

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Appendix D
Flow Velocities

Novato Creek Left Bank Reach Velocities			Riverine, 50-Year WSE			Riverine, FEMA 100-Year		
			W.S. Elev	Vel Chnl	Shear Chan	W.S. Elev	Vel Chnl	Shear Chan
River Station	Feature	Project Stationing	(ft NAVD88)	(ft/s)	(lb/sq ft)	(ft NAVD88)	(ft/s)	(lb/sq ft)
20458.89	SR-37	225+36	12.04	2.22	0.09	12.15	2.31	0.09
20615		225+36	12.07	2.52	0.13	12.19	2.61	0.14
20771.29		226+83	12.17	1.52	0.04	12.29	1.59	0.04
21334.73		233+12	12.22	1.31	0.03	12.34	1.37	0.03
21770.51		237+19	12.23	1.87	0.05	12.35	1.95	0.06
22244.39		241+75	12.29	2.07	0.08	12.42	2.15	0.08
22628.08		245+66	12.37	1.69	0.05	12.5	1.76	0.05
22802.8		247+13	12.37	2.84	0.15	12.51	2.96	0.16
22977.6		249+29	12.59	1.6	0.05	12.74	1.69	0.05
23152.3		250+52	12.64	1.29	0.03	12.8	1.35	0.03
23327.14		252+35	12.66	1.22	0.03	12.81	1.27	0.03
23415.8		253+40	12.66	1.25	0.03	12.82	1.31	0.03
23504.6		254+31	12.66	1.31	0.03	12.82	1.37	0.03
23593.3		256+16	12.66	1.41	0.04	12.82	1.48	0.04
23682.1		257+03	12.66	1.74	0.05	12.82	1.82	0.06
23770.87		257+95	12.66	1.84	0.06	12.82	1.93	0.07
23847.4		258+79	12.69	1.56	0.04	12.84	1.63	0.05
23924.1		259+50	12.69	1.52	0.04	12.85	1.6	0.05
24000.7		260+04	12.69	1.45	0.04	12.85	1.52	0.04
24077.3		260+61	12.69	1.39	0.03	12.85	1.46	0.04
24153.94		261+25	12.69	1.47	0.04	12.85	1.54	0.04
24251.7		264+46	12.69	1.69	0.05	12.84	1.77	0.06
24349.5		264+77	12.67	2.1	0.08	12.83	2.19	0.09
24447.36		265+12	12.67	2.34	0.1	12.83	2.43	0.11
24755.4	Heron's Beak Pond	267+68	12.71	2.53	0.12	12.87	2.62	0.12
24881.6		268+56	12.73	2.63	0.12	12.89	2.72	0.13
25007.96		269+51	12.78	2.61	0.12	12.94	2.7	0.13
25327.52		272+09	12.88	2.83	0.15	13.04	2.93	0.15
25720.1		275+21	13.03	2.98	0.16	13.2	3.09	0.17
25986.19		277+28	13.15	3.03	0.17	13.32	3.14	0.18
26167.2		278+80	13.33	2.35	0.1	13.52	2.45	0.11
26348.34		280+04	13.48	1.22	0.03	13.68	1.28	0.03
26481.3		281+40	13.51	0.91	0.01	13.71	0.96	0.02
26614.4		282+70	13.52	0.83	0.01	13.72	0.87	0.01
26747.47		284+02	13.53	0.76	0.01	13.73	0.8	0.01
26969.82		286+20	13.54	0.95	0.02	13.74	1.01	0.02
27066.9		287+20	13.54	0.97	0.02	13.74	1.03	0.02
27164.1		288+53	13.55	0.83	0.01	13.75	0.88	0.01
27261.2		289+83	13.55	0.87	0.01	13.75	0.92	0.02
27358.42		291+05	13.55	0.95	0.02	13.75	1.02	0.02
27452.9		292+18	13.55	1.04	0.02	13.75	1.11	0.02
27547.4		293+61	13.55	1.26	0.03	13.75	1.34	0.03
27641.98		294+67	13.55	1.53	0.04	13.74	1.63	0.05

Novato Creek Left Bank Reach Velocities

River Station	Feature	Project Stationing	Riverine, 50-Year WSE W.S. Elev (ft NAVD88)	Riverine, 50-Year WSE Vel Chnl (ft/s)	Riverine, 50-Year WSE Shear Chan (lb/sq ft)	Riverine, FEMA 100-Year W.S. Elev (ft NAVD88)	Riverine, FEMA 100-Year Vel Chnl (ft/s)	Riverine, FEMA 100-Year Shear Chan (lb/sq ft)
27726.8		295+49	13.55	1.62	0.05	13.75	1.72	0.06
27811.7		296+30	13.55	1.75	0.07	13.75	1.85	0.07
27896.6		297+12	13.55	1.88	0.08	13.75	1.99	0.09
27981.56		297+98	13.55	2.02	0.07	13.75	2.13	0.08
28093.5		298+97	13.54	2.3	0.1	13.74	2.42	0.11
28205.44		299+75	13.56	2.34	0.1	13.76	2.45	0.11
28427.8		302+12	13.55	2.93	0.16	13.75	3.07	0.17
28650.23		304+32	13.57	3.55	0.24	13.77	3.68	0.25
28737.2		305+22	13.6	3.39	0.2	13.81	3.52	0.21
28824.3		306+09	13.6	3.38	0.18	13.81	3.51	0.19
28911.3		306+95	13.6	3.37	0.16	13.8	3.51	0.17
28998.43		307+82	13.6	3.33	0.14	13.81	3.47	0.16
29139.02		309+08	13.69	2.8	0.1	13.9	2.93	0.1
29217.26	SMART Rail Bridge	309+62	13.81	2.35	0.07	14.07	2.45	0.07

Note: Project stationing was added to Stetson's table. Data has not been modified.

Lynwood Reach Velocities			Riverine, 50-Year WSE			Riverine, FEMA 100-Year WSE		
			W.S. Elev	Vel Chnl	Shear Chan	W.S. Elev	Vel Chnl	Shear Chan
River Stationing	Features	Project Stationing	(ft NAVD88)	(ft/s)	(lb/sq ft)	(ft NAVD88)	(ft/s)	(lb/sq ft)
20458.89	SR-37	242+51	12.04	2.22	0.09	12.15	2.31	0.09
20615		243+05	12.07	2.52	0.13	12.19	2.61	0.14
20771.29		243+49	12.17	1.52	0.04	12.29	1.59	0.04
21334.73		248+99	12.22	1.31	0.03	12.34	1.37	0.03
21770.51		253+26	12.23	1.87	0.05	12.35	1.95	0.06
22244.39		257+89	12.29	2.07	0.08	12.42	2.15	0.08
22628.08		261+31	12.37	1.69	0.05	12.5	1.76	0.05
22802.8		262+75	12.37	2.84	0.15	12.51	2.96	0.16
22977.6		264+27	12.59	1.6	0.05	12.74	1.69	0.05
24349.5		274+55	12.67	2.1	0.08	12.83	2.19	0.09
24447.36		275+37	12.67	2.34	0.1	12.83	2.43	0.11
24755.4	Heron's Beak Pond	278+31	12.71	2.53	0.12	12.87	2.62	0.12
24881.6		279+63	12.73	2.63	0.12	12.89	2.72	0.13
25007.96		280+84	12.78	2.61	0.12	12.94	2.7	0.13
25327.52		284+13	12.88	2.83	0.15	13.04	2.93	0.15
25720.1		288+23	13.03	2.98	0.16	13.2	3.09	0.17
25986.19		290+83	13.15	3.03	0.17	13.32	3.14	0.18
26167.2		291+95	13.33	2.35	0.1	13.52	2.45	0.11
28650.23		306+75	13.57	3.55	0.24	13.77	3.68	0.25
28737.2		307+54	13.6	3.39	0.2	13.81	3.52	0.21
28824.3		308+33	13.6	3.38	0.18	13.81	3.51	0.19
28911.3		309+16	13.6	3.37	0.16	13.8	3.51	0.17
28998.43		309+96	13.6	3.33	0.14	13.81	3.47	0.16
29139.02		310+00	13.69	2.8	0.1	13.9	2.93	0.1
29217.26	SMART Rail Bridge	310+00	13.81	2.35	0.07	14.07	2.45	0.07

Note: Project stationing was added to Stetson's table. Data has not been modified.