GEI Consultants

Geotechnical
Environmental
Water Resources
Ecological

Coyote Creek Levee Evaluation Project

Existing Conditions Levee Evaluation and Recommendations for Improvements

Coyote Creek, Marin County, CA

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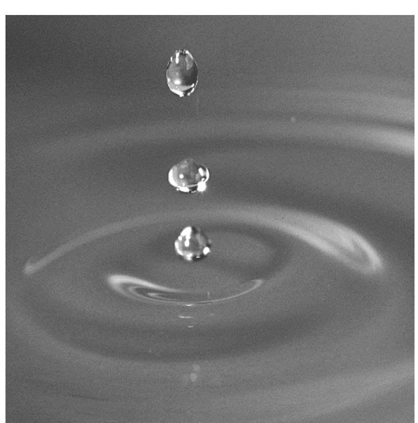
January 2016
Project 1404570



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1. Background and Purpose

GEI Consultants Inc. (GEI) along with project sub-consultant HDR, Inc. (HDR) is assisting the Marin County Flood Control and Water Conservation District (District) in an evaluation of the Coyote Creek Local Flood Protection Project (Project) located in the unincorporated community of Tamalpias Valley. The overall goal of the Project is to provide a comprehensive assessment of the current condition of the levee system and develop initial recommendations for both short- and long-term improvements for future analysis. Improvement alternatives which provide options for maintaining and increasing the level of protection provided by the Project will be developed and assessed with consideration of both their feasibility and cost-effectiveness.

GEI has completed an existing conditions evaluation of the Coyote Creek levee system in order to provide a review of the current levee condition and recommendations on the feasibility of raising or modifying the levee. The work conducted under this existing conditions levee evaluation will be the basis for assessment of remedial alternatives to address any levee segments not meeting design criteria. Existing conditions analyses were performed on 4 representative levee cross sections in the Coyote Creek levee system where existing levee structures are present.

This memorandum includes results for a series of geotechnical analyses including seepage evaluation, slope stability analysis under steady-state seepage conditions, slope stability analysis under rapid drawdown conditions, slope stability analysis under seismic loading (pseudo-static), consolidation settlement, liquefaction potential, liquefaction-induced settlement, and lateral spreading.

2. Project Datum, Stationing, and Base Topographic Map

The vertical datum used for the existing conditions analyses of the Coyote Creek levee system is the 1988 North American Vertical Datum (NAVD88). The levee stationing system has been developed by GEI for the Coyote Creek levee system as shown on Figures 1 through 6. USACE project stationing is also shown on Figures 1 through 6 for reference.

The topographic data utilized for existing conditions analyses, including the elevation contours presented on Figures 1 through 6 and the elevation profiles presented on Figures 7 through 14, was provided by the District. Two separate data sets were provided, which included 1) County of Marin digital topographic-bathymetric surface model, Revision 2013.12.18 (County of Marin, 2013), and 2) topographic and bathymetric data derived from field survey of the Coyote Creek channel and levees, performed by Meridian Surveying Engineering (Meridian, 2013). A brief description of these datasets is provided in the Geotechnical Data Report (GDR) (GEI, 2015).

It should be noted that inconsistencies in elevation data between the two datasets listed above were observed, likely due in part to limitations inherent to data collection methods and survey extents. In areas where data from both survey sources overlap, the data sets were compared, with the data from the source which appeared more accurate based on knowledge of the area used for evaluation. The elevation contours shown on Figures 1-6 were developed using County of Marin, 2013 data, because of the larger spatial extent of the dataset. The elevation profiles shown on Figures 7-14 and 15-22 for each GEI reach alignment and cross section were developed using data from both sources (County of Marin, 2013 and Meridian, 2013), as well as original USACE channel as-built elevations (USACE, 1964), as appropriate. A summary of the topographic data used within each GEI reach alignment is provided in the GDR (GEI, 2015). In addition, the dataset used for each cross section is indicated in Figures 15 – 22.

3. Design Water Surface Elevations

The design water surface elevations used in the existing conditions analyses were determined through HEC-RAS modeling along the Coyote Creek levee system for six different scenarios. Each scenario consisted of the existing conditions geometry (based on surveying performed by Meridian Surveying Engineering, Inc. dated March 2013) with different combinations of riverine flow and tidal downstream boundary condition assumptions as described in the Draft Existing Conditions Hydraulic Analysis and Results for Coyote Creek and Nyhan Creek in Marin County memorandum prepared by HDR, dated February 18, 2015. The HEC-RAS modeling assumes all flow is contained in the channel. These scenarios include:

- Baseline – upstream and downstream boundary conditions used in the design of the Corps project in the 1960s (20-Year Event 1960s Corps design riverine flow and 1960s tidal Mean Higher High Water (MHHW) elevations at Richardson Bay).

- Updated – District revised upstream and downstream boundary conditions that reflect present day conditions equivalent to the design of the Corps project in the 1960s (25-Year Event District riverine flow plus 15-percent and present day tidal MHHW elevation at Richardson Bay).

- Enhanced A (District 50-Yr Event) – District revised upstream and downstream boundary conditions (50-Year Event District riverine flow plus 15-percent and present day tidal MHHW elevation at Richardson Bay).

- Enhanced B (District 100-Yr Event) – District revised upstream and downstream boundary conditions (100-Year Event District riverine flow plus 15-percent and present day tidal MHHW elevation at Richardson Bay).

- FEMA Accredited – FEMA upstream and downstream boundary conditions (100-Year Event FEMA riverine flow and present day tidal MHHW elevation at Richardson Bay).

- FEMA Accredited with Sea Level Rise (SLR) – FEMA upstream and downstream boundary conditions accounting for SLR (100-Year Event FEMA riverine flow and estimated year 2050 tidal MHHW elevation at Richardson Bay).

These scenarios result in a range of overtopping conditions for the study area due to riverine and tidal conditions. Much of the overtopping under riverine conditions occurs in areas of no levee, in areas of extremely low or subsided levees, or in areas of the concrete channel.

GEI assessed existing conditions for the Baseline (20-year event), FEMA Accredited, and FEMA Accredited with SLR scenarios to capture the full range of water surface conditions. This provides a general understanding of existing conditions, as well as provides a means to calibrate models to past performance. The remaining three WSEs (Updated, Enhanced A, and Enhanced B) are intermediate to the analyzed WSEs. Therefore existing conditions analysis results for these scenarios were estimated with linear interpolation based on results from the Baseline, FEMA, and FEMA with SLR WSE analyses.

Remediated conditions analysis will consider all six of the water surface conditions described above. In addition, the remediated conditions analysis may also include one modification to the existing conditions HEC-RAS model if a setback levee is considered as a remedial alternative.

GEI also assessed seismic deformation of the levee system at mean sea level (MSL) conditions. MSL was assessed instead of mean higher high water (MHHW) to capture a typical water surface condition that, for the case of seismic deformation on the waterside slope, is more critical. MSL is considered more critical than MHHW due to the decrease in the stabilizing effect of the higher water level. The MSL was estimated to be approximately 3.2 feet within the Coyote Creek levee system. This was determined using information provided by the National Geodetic Survey for station HT0702, located in the San Francisco Bay.

4. Subsurface Conditions

The interpretation of the subsurface conditions provided below is based on the recent subsurface explorations performed by GEI and review of data from previous subsurface explorations (between 1964 and 2009). Additional explorations were needed, since previous borings were generally shallow and did not provide sufficient geotechnical information for the embankment and foundation necessary to perform a levee evaluation. Current GEI explorations were performed to provide deeper stratigraphy and material properties of the embankment and foundation. Boring logs, cone penetration test (CPT) logs, and laboratory test reports from GEI project explorations and the previous explorations and studies are provided in the GDR.

The locations of the GEI explorations and selected previous explorations are shown in the levee plan views on Figures 1 through 6. Subsurface profiles, including our interpretation of subsurface conditions along the levee alignment, are provided on Figures 7 through 14.

Levee and foundation conditions are based on approximately 107 explorations positioned mostly on the levee crest, landside toe, and landward areas, with a few explorations completed on the waterside of the levee. In general, explorations are at a spacing of less than 500 feet along the levee, with many areas of more densely spaced locations. The total number of explorations along the 2.7-mile study area averages to about 40 explorations per mile. The density of explorations exceeds the federal guidelines described in the USACE Sacramento District's Geotechnical Levee Practice Standard Operation Procedures, which suggest a frequency of approximately 20 explorations per mile. Although the density of explorations is higher than federal guidelines, GEI performed additional explorations for this project because many historic borings were shallow with limited relevant laboratory testing.

It should be noted that along Coyote Creek and Nyhan Creek, the placement of explorations waterside of the levee is generally impractical due to lack of waterside berm and access limitations. Coyote Creek and Nyhan Creek are in very close proximity to the levee; consequently, the need for waterside characterization is greatly reduced since conditions can be assumed based on information collected from beneath and immediately landward of the levee.

Subsurface conditions were observed only at the boring and CPT locations. No geophysical surveys were considered during interpretation of foundation conditions. The soils encountered in the subsurface explorations generally include varying amounts of embankment and surficial fill underlain by Younger Bay Mud (YBM) deposits consisting of soft clay. Stiff material below the YBM likely consisting of Older Bay Mud (OBM) or alluvial and colluvial deposits overlies bedrock of the Franciscan Formation.

The levee embankment soils encountered in the subsurface explorations were variable, but consisted of predominantly clayey sands, sandy clays, clayey gravels, and gravelly sands, but also included clays and poorly graded gravels with sands. The fines content in the embankment material ranged between 9% to almost 55% fines by weight with significant portions of the embankment having more than 30% fines.

Groundwater is expected to vary seasonally according to current and historic explorations. Current explorations encountered the groundwater table at depths between approximately 2.5 and 9.0 ft (elevation between -0.1 and 6.5 ft NAVD88). A 2008 report by Kleinfelder titled "Review of Water Level Survey Data" provided a review of groundwater level data and tidal data for the period between July 3rd and July 30th, 2007 along the south (right) bank of Coyote Creek. The report shows that the tide ranged between approximately 3.2 feet and 6.7 feet (NAVD88) and did not appear to have a significant correlation with groundwater level except when tidal level exceeded approximately 5.9 feet (NAVD88).

5. Selected Reaches and Analysis Cross Sections

5.1 Reach Selection

The goal of reach selection is to divide the levee alignment into a minimum number of analysis reaches that have reasonably consistent characteristics, assumptions and geotechnical analyses objectives. The premise of reach selection is that each reach can be adequately represented in terms of geotechnical characterization and analysis by one longitudinal soil profile and associated transverse cross section, and one set of associated geotechnical analysis input parameters. To reflect potentially critical conditions in any given reach, profile and cross section details may be characterized somewhat differently for seepage and for slope stability analyses. Based on the process described below, the total levee length was divided into 12 reaches (Reach 1 through Reach 12).

The levee was divided into analysis reaches based on observed conditions that likely have a bearing on levee performance. These conditions included the following:

- Interpreted stratigraphy and material properties (inferred from boring logs, laboratory testing results, and geomorphic assessment)
- Existing improvements and structures, such as seepage cutoff walls, floodwalls, or addition of embankment fill.
- Cross sectional levee geometry (height, side slopes)
- Historic performance, including areas of past seepage, erosion or slumping

A detailed review and interpretation of available information was performed to characterize subsurface conditions along the levee alignment. The reviewed information as documented in the GDR included:

- Geotechnical boreholes and CPT explorations
- Soil-index and engineering-property lab testing
- Past studies, levee plans, and available as-builts of remediations
- Geomorphology
- Records of past levee performance observations

Reach selection and characterization depended on existing conditions and available information. First, interpreted stratigraphy, geology, geomorphology, and soil properties were used to define the basic differences between potential reaches. Such differences may include transitions in depositional features, which may lead to variations in stratigraphy and/or soil properties. After the initial characterization was performed, levee improvements, geometry, and performance information were considered. The reach characteristics are described to present the general ranges of conditions within a particular reach; however, some conditions may not be present along the entire reach. Reaches are summarized in Table 1 and the extents are shown on plan and profile in Figures 1 through 14.

5.2 Analysis Cross Sections

Once reach selection was complete, cross sections were developed for geotechnical analyses. Within a particular reach, the location for seepage and stability analyses were determined based on several considerations (soil stratigraphy, past performance, slope geometry, etc.) most relevant to these analyses. Examples of controlling factors for seepage analysis include locations with pervious embankment materials or shallow sand layers in the foundation. Potential controlling factors for stability analysis include locations with soft clays or silts and/or loose sands in the foundation. The cross sections were developed based on engineering judgement to capture vulnerable, but representative conditions within each reach. Note, the analysis discussed in this memorandum focuses on representative conditions within a reach and may not represent an anomalous “worse-case condition” location. During design of improvements, site specific changes may be needed to address anomalous conditions.

Cross sections consisted of levee and foundation materials. Soil stratigraphy was interpreted for each cross section. Cross sections were typically chosen near field explorations where detailed stratigraphic and material property information was available.

A large portion of the Coyote Creek study area levees are shorter than two feet above adjacent landside grades, with the maximum landside levee height of about six feet, and, as such, do not warrant extensive geotechnical analysis of existing conditions. In addition, due to proximity to high ground or currently abandoned extents of levee, only 8 of the 12 reaches either required or will require further analysis and development of cross sections.

For selection of critical cross-sections for existing conditions analyses, we focused on the areas where steady-state seepage and slope stability assessments are appropriate and will provide value (landside levee heights greater than about five feet and oversteepened landside and/or waterside slopes). This includes areas of taller levees, such as the Coyote Creek Middle Reach. We recommended four cross sections to perform existing conditions analysis. These proposed cross sections are located at:

1. Reach 3, Station 28+00 CC-L: Left bank of Coyote Creek Middle Reach with existing flood wall (Reach 3)

2. Reach 4, Station 34+00 CC-L: Left bank of Coyote Creek Middle Reach without a flood wall

3. Reach 7, Station 35+00 CC-R: Right bank of Coyote Creek Middle Reach without a flood wall

4. Reach 9, Station 1+00 NC-L: Left bank of lower Nyhan Creek

We added additional critical cross-sections in the areas of non-existent or low levees where creek flows are not contained and overtopping occurs based on hydraulic analysis and/or observed overland flooding during high tides or extreme events. As the study progresses into analysis of remedial alternatives, these additional cross-sections will be used to evaluate new or taller flood protection structures under the various hydraulic scenarios. These additional cross sections are located at:

1. Reach 5, Station 5+00 CC-R: Settled embankment on the Lower Reach right bank of Coyote Creek

2. Reach 8, Station 4+00 CC-C: Concrete channel in the Upper Reach of Coyote Creek

3. Reach 10, Station 7+00 NC-L: Upper un-leveed portion of Nyhan Creek left bank

4. Reach 12, Station 9+00 BM-L: Bothin Marsh High Ground

Table 1 summarizes the analysis cross sections developed for future analysis as well as rationale for developing cross sections at these locations. Figures 15 through 22 depict the eight cross sections described above, including relevant explorations and interpreted stratigraphy.

6. Material Properties

Recommended material properties were developed for each stratigraphic layer for each modeled cross section. Available, pertinent 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 (URS 2013).

6.1 Seepage Parameters

Hydraulic conductivities for seepage analyses were selected for each soil type based on material index properties, laboratory and in-situ testing by DWR, and review of relevant geotechnical references. Hydraulic conductivities were developed for each material type encountered within the levee embankment and foundation soils as shown on Figures 15 through 22. A summary table of horizontal and vertical hydraulic conductivities for each material type is provided below.

Material Type	Soil Type	Soil Description		k_v (cm/sec)	k_v (ft/day)	k_h/k_v	k_h (cm/sec)	k_h (ft/day)
Cutoff Walls		Soil-bentonite		1.0.E-06	2.83E-03	1	1.0.E-06	2.83E-03
		Soil-Cement-Bentonite		1.0.E-06	2.83E-03	1	1.0.E-06	2.83E-03
		Cement-Bentonite		1.0.E-06	2.83E-03	1	1.0.E-06	2.83E-03
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) Desiccated 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
Sandy Silt (50-70% fines)	ML	Embankment		1.5.E-05	4.25E-02	4	6.0.E-05	1.70E-01
		Natural Deposits		1.5.E-05	4.25E-02	4	6.0.E-05	1.70E-01
Sand	SP, SW	< 5% fines		2.0.E-02	5.67E+01	1	2.0.E-02	5.67E+01
	SP-SM, SW-SM	5-12% fines		4.0.E-03	1.13E+01	2	8.0.E-03	2.27E+01
	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
Sand	SM	Natural Deposits / Embankment - Uncontrolled Placement	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
	SP-SC, SW-SC	5-12% fines		2.0.E-04	5.67E-01	4	8.0.E-04	2.27E+00
	SC	Natural Deposits / Embankment - Uncontrolled Placement	12-25% fines	4.0.E-05	1.13E-01	4	1.6.E-04	4.54E-01
			25-49% fines	7.0.E-06	1.98E-02	4	2.8.E-05	7.94E-02
		Embankment - Controlled Placement	12-49% fines	4.0.E-06	1.13E-02	4	1.6.E-05	4.54E-02
Gravel	GP, GW	< 5% fines		1.0.E-01	2.83E+02	1	1.0.E-01	2.83E+02
	GP-GM, GW-GM	5-12% fines		1.0.E-02	2.83E+01	2	2.0.E-02	5.67E+01
	GM	12-25% fines		1.0.E-03	2.83E+00	4	4.0.E-03	1.13E+01
	GM	25-35% fines		3.0.E-04	8.50E-01	4	1.2.E-03	3.40E+00
	GM	35-49% fines		1.0.E-04	2.83E-01	4	4.0.E-04	1.13E+00
	GP-GC, GW-GC	5-12% fines		3.0.E-03	8.50E+00	4	1.2.E-02	3.40E+01
	GC	12-25% fines		5.0.E-04	1.42E+00	4	2.0.E-03	5.67E+00
	GC	25-49% fines		7.0.E-05	1.98E-01	4	2.8.E-04	7.94E-01

Further discussion of the development of hydraulic conductivity values is provided in Appendix A.

6.2 Slope Stability 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. Rapid loading conditions will also be considered during remedial analyses for after-construction conditions.

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

7. Model Development and Approach

Seepage and stability analyses were performed at the selected cross sections in general accordance with USACE EM 1110-2-1913, Design and Construction of Levees (USACE, 2000), EM 1110-2-1902, Slope Stability (USACE, 2003), ETL 1110-2-569, Design Guidance for Levee Underseepage (USACE, 2005), and EM 1110-2-1901, Seepage Analysis of Dams (USACE, 1993). As previously discussed, seepage and stability analyses have initially been performed for the Baseline, FEMA Accredited, and the FEMA Accredited with SLR scenario water levels.

For some reaches, the computed results of geotechnical analyses at the locations of the analyzed sections meet geotechnical criteria for seepage and stability, but portions of the same reach may not meet criteria due to conditions that differ from the conditions at the analysis cross section. An example of such a condition is within Reach 3, where a portion of the reach contains a clay barrier wall and therefore meets through seepage criteria, but another portion of the reach does not contain a clay barrier wall and does not meet through seepage criteria at water levels for the FEMA Accredited improvement alternative. For this condition, reaches are divided into sub-reaches to differentiate portions of a reach that meet and do not meet criteria.

7.1 Freeboard Analyses

The existing levee crown profiles were compared to composite water surface profiles for the Baseline, FEMA, and FEMA with Sea Level Rise scenarios, which were developed using the greater of the hydraulic grade lines for the subject evaluation scenarios and the high tide conditions as described in the *Draft Existing Conditions Hydraulic Analysis and Results for Coyote Creek and Nyhan Creek in Marin County* memorandum prepared by HDR, dated April 10, 2015. For the Baseline Scenario, the freeboard criteria was adopted from the USACE 1959 Detailed Report on Coyote Creek which required the levee crest elevation to be at least 1 foot above the design water surface elevation corresponding to a 20-year event, and at least 0.5 feet above the highest estimated tide (8.7 feet NAVD88 in 1959). For the FEMA Accredited and FEMA Accredited with SLR Scenarios, the levee met the freeboard requirement if the levee crest elevation was at least 3 feet above the associated composite water surface profile, with the principal difference between the profiles being the tidal condition. The FEMA Accredited Scenario assumes a Stillwater elevation of 9.7 feet NAVD88, corresponding to a 1% annual exceedance probability event. The FEMA Accredited with SLR assumes a Stillwater elevation of 12.7 feet NAVD, based on a projected 3 feet of sea-level rise by 2050. For sections consisting of incised concrete channels (Reach 8), criteria is met if the water surface elevation is contained within the channel.

7.2 Seepage Analyses

Seepage analyses were performed using SEEP/W, a two-dimensional finite element modeling computer program, developed by GEO-SLOPE International, Ltd. The steady-state phreatic surfaces and pore water pressures within the levee and foundation soils were generated using SEEP/W for the Baseline Scenario, FEMA Accredited Scenario, and the FEMA Accredited Scenario with SLR water levels.

The analyses were based on the assumption that steady-state seepage conditions could develop during the design flood. Underseepage for steady state conditions is typically evaluated by estimating the average exit gradient across an impervious blanket overlying a pervious aquifer at the landside levee toe and at potentially critical locations away from the levee toe based on variations in subsurface and surface conditions. The estimated gradient is compared to the maximum allowable gradients. Alternatively, the estimated gradient can be expressed as a factor of safety against uplift by dividing the critical gradient by the estimated gradient. The critical gradient is the gradient where the buoyant unit weight of the soil is equal to the uplift pressure. For conditions where no discrete impervious blanket layer overlays a pervious aquifer, which is common for the Coyote Creek levee system (i.e. levee is founded on semi-pervious to pervious sandy clay or clayey sand overlying impervious bay mud deposits), the average vertical exit gradient was calculated at the landside levee toe across the upper 1-foot of the foundation material. In addition to the average vertical exit gradient calculation discussed above, the horizontal flow through the surficial pervious foundation layer with no overlying blanket was considered, since seepage and piping can result under this “no blanket” condition. The existence of a pervious foundation unit at the levee-foundation contact with hydraulic head potential significantly above the landside toe elevation is generally considered unacceptable in this evaluation unless the pervious unit is fully penetrated by an existing cutoff wall. The volume of seepage discharging from the surficial unit was also considered when assessing the “no blanket” seepage condition.

Maximum allowable vertical exit gradients are tabulated below:

Condition	Max. Allowable Exit Gradient[1,2]	Minimum FS Value
Landside Levee Toe	0.5	1.6
Depression – 150 feet from the levee toe	0.8	1.0

[1]The saturated unit weights of the “in-situ” landside blanket soils must be at or above 112 pounds per cubic foot in order to use these exit gradient criteria. If soils weigh less than 112 pcf, the minimum Factor of Safety (FS) criteria shown shall be followed.

[2] The criteria for an intermediate point up to 150 feet from levee toe will be linearly interpolated from 0.5 to 0.8 to address thinning blankets and/or topographic low points.

7.2.1 Seepage Model Boundary Conditions

Important elements for consideration in developing seepage models for various flood loadings 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 above, and seepage parameter selection is discussed in detail in Appendix A. Boundary conditions were generally applied to the model as follows:

- No-flow boundary condition along the bottom and riverside vertical edge of the model.
- Constant head boundary condition equivalent to the flood level being evaluated along the riverside horizontal ground surface and riverside slope below the flood elevation.
- Constant head boundary condition applied to the landside vertical edge of the model equal to the lowest ground surface elevation landward of the levee toe.

Potential seepage face review boundary condition along the landside slope and landside ground surface to the landward extent of the model. The head at each node with this boundary condition is evaluated within the program after each analysis iteration. At the first iteration, the node is given a no-flow boundary condition, and the head is computed. If a positive pore pressure is computed at this node, SEEP/W converts the boundary condition to a constant head boundary with the total head equal to the node's corresponding elevation and re-computes the solution. If the solution indicates that water is flowing into the system, the node is reassigned a no-flow boundary. The iterations continue until all seepage face review nodes have either zero or negative pore pressure and water is not flowing into the nodes.

7.3 Stability Analyses

Stability analyses were performed on the same analysis cross sections evaluated for seepage. Model cross section development (levee geometry, surface conditions, and soil stratigraphy) is discussed above and stability shear strength parameter selection is discussed in detail in Appendix B.

Stability analyses were performed using SLOPE/W, a slope stability analysis software program developed by GEO-SLOPE International, Ltd. Stability was evaluated using the Spencer analysis method, which satisfies both moment and force equilibrium. Both circular and non-circular slip surfaces were evaluated. Slip surfaces were defined using the entry-and-exit method.

The following load cases are typically considered for stability evaluation:

I. End of Construction

II. Rapid Drawdown

III. Steady-State

IV. Earthquake

Required factors of safety are:

Case	Description	Water Surface Elevation	Minimum FS
I	End of Construction*	Mean Sea Level	1.3
II	Rapid Drawdown	Baseline, FEMA Accredited, FEMA Accredited with SLR	1.2
III	Steady-State	Baseline, FEMA Accredited, FEMA Accredited with SLR	1.4
IV	Earthquake	Mean Sea Level	Not Applicable**

*For new levees and major levee additions only (not applicable for this existing conditions analysis)
**Earthquake loading and stability evaluation are not used for design, but rather for evaluation of seismic displacement

Evaluations for the existing levee conditions were limited to Case II, Case III, and Case IV only. For the existing conditions, Case I does not apply. Case I will be evaluated in remedial analyses for levee reaches if major modifications in the levee cross section (crest raises or significant berms) are required to mitigate seepage or stability.

In general, critical slip surfaces were identified for each load case that would likely compromise overall embankment stability. Slip surfaces less than three feet deep were ignored, 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. Slip surface entry limits were generally assigned from hinge to hinge of the levee crest (i.e. slip surfaces must enter above the levee hinge). Slip surface exit limits were assigned to a portion of the levee slope at the toe and continued landward (or waterward depending on direction of failure) beyond the critical failure surface. If the critical failure surface was controlled by the slip surface exit limit, the exit limit would be extended until the critical failure surface was no longer controlled by the exit limit.

7.3.1 Case II - Rapid Drawdown

For the rapid drawdown case, it is commonly assumed the levee has been saturated for a sufficient length of time under the design flood to develop steady-state seepage conditions, and then the flood recedes quickly. It is also assumed that excess pore pressures would not develop in coarse-grained soils because the coarse-drained soils are relatively free draining. The fine-grained soils were assumed to not be free draining and would generate excess pore pressures.

The Improved Method for Rapid Drawdown was used as outlined in Appendix G of EM 1110-2-1902 (USACE, 2003) to evaluate the rapid drawdown case. The method involves up to three separate stability calculations for each trial slip surface. The first calculation computes the effective stresses before drawdown. The second calculation is performed using undrained shear strengths corresponding to the effective consolidation stresses calculated in the first stage. SLOPE/W estimates undrained strengths based on anisotropic consolidation stresses from the Stage 1 computation by interpolating between the undrained strength envelope (K_c=1) and the drained strength envelope ($K_c = K_f$). This is consistent with guidance in Appendix G of EM 1110-2-1902. If the drained shear strength (using post-drawdown pore water pressures) is less than the undrained shear strength for any slice, a third calculation is performed using the lower strength for each slice. The factor of safety reported is based on the lower of either the effective or undrained shear strengths for each slice along the slip surface.

The method of evaluating rapid-drawdown stability assumes that the river level drops instantaneously from the design flood level to the bottom of the channel, resulting in instantaneous excess pore pressure development in the embankment and foundation soils that is directly proportional to the assumed river level drop. In reality, the flood recedes gradually, and some pore pressure dissipation occurs as the river level drops. As a result, the rapid drawdown analysis is generally considered to be inherently conservative.

7.3.2 Case III - Steady-State Seepage Stability

For this case, it is assumed the duration of the flood is sufficient to establish steady-state seepage conditions through the levee embankment, in accordance with USACE guidelines. This is a conservative assumption for some fine-grained soils given the short duration of flood events. The phreatic surfaces and pore water pressures from our seepage analyses were used in the stability evaluations. Because steady-state seepage is a long-term condition, we assigned drained strengths to all soils, both coarse-grained and fine-grained. Steady-state seepage stability was considered on both the landside and waterside slope of the levee.

7.3.3 Case IV - Earthquake Stability

For this case, we used an approach similar what is described in Section 6.0 of the ULE Guidance Document for Geotechnical Analysis (Guidance Document) (DWR, 2015). This approach considers liquefaction triggering and seismic displacement.

Liquefaction triggering analyses evaluate whether the levee or underlying foundation materials could be susceptible to liquefaction during a flood event. However, for the existing conditions cross sections, all soil layers in Reaches 3, 4, 7, and 9 are screened out of liquefaction triggering analyses due to the criteria listed below:

1) Soils classified as CH or MH can be screened out.

2) Soils classified as CL or ML with plasticity index (PI) greater than or equal to 10, liquid limit (LL) greater than or equal to 35, or water content less than or equal to 80% of the LL.

3) Clayey sand with greater than 20 percent fines.

4) Soils with $(N_1)_{60cs}$ greater than or equal to 23, or $q_{c,1mod}$ greater than or equal to 100 tsf.

To evaluate the seismic displacement of the slopes under long term (drained) and short term (undrained) conditions, we performed pseudo-static analyses and simplified deformation analyses. The seismic deformation analysis is based on the principles of the Newmark deformation analysis (e.g., Makdisi and Seed, 1978). The method used to evaluate the seismic displacement is meant to be a screening-level prediction of seismic deformations. Appendix C provides additional information used to determine seismic displacement. This includes seismic displacement calculations, a probabilistic seismic hazard deaggregation figure from the interactive USGS website used to determine PGA, and referenced figures from the Guidance Document.

The key steps for the seismic displacement evaluation are as follows:

1) Determining the earthquake induced accelerations acting on the slide mass (K_{max}) (see Figure C-1) using the site specific peak ground acceleration (PGA) shown in Figure C-2. The Coyote Creek levee system is located outside the extent of Figure C-2; therefore the earthquake-induced accelerations acting on the slide mass were estimated at the site using the interactive USGS website available at http://geohazards.usgs.gov/designmaps/us/application.php (see Figure C-3). The PGA (peak ground acceleration) for the site is 0.23g (g = acceleration due to gravity). This is associated with a return period of about 94 years. The accelerations are based on a site classification for "Soil Type C" (average shear wave velocities in the upper 30 meters (V_{s30}) of soil ranging between 360 m/s and 760 m/s). The parameter V_{s30} was estimated with the USGS V_{s30} map server online at to be approximately 400 m/s at the site. According to Figure C-1, K_{max} ranges from 0.18g to 0.25g based on a PGA of 0.23g and assuming either "Deep-Medium" or "Deep-Stiff" response of the site, respectively. As discussed in the ULE Guidance Document, a "Deep-Stiff" response is expected for sites where the difference in elevations between the waterside and landside levee toes is greater than about one levee height. Alternatively, a "Deep-Medium" response is expected for sites where the elevations of the waterside and landside levee toes are not equal, but the difference is less than about one levee height.

2) Identify and estimate potential sliding masses and associated seismic yield coefficients (K_y). K_y represents the minimum horizontal acceleration required to produce a factor of safety equal to 1.0. We computed the horizontal yield acceleration (K_y) that results in a factor of safety of 1 using SLOPE/W. Residual strength due to cyclic softening was used for the younger bay mud material. The undrained strength parameters of the younger bay mud layers were assigned to be 80 percent of the proposed undrained shear strengths in Appendix B. All other strengths were assumed equal to the strengths proposed in

Appendix B. The modelled soil strengths were analyzed in SLOPE/W as the minimum of the drained or undrained strengths.

3) Estimate the permanent deformation of a slide mass using K_y and K_{max} as input parameters in Figure C-4. This method assumes that permanent deformation initiates when the earthquake-induced accelerations acting on the slide mass exceed K_y on the slip surface.

7.4 Settlement

Primary and secondary settlements were estimated for the four existing conditions analysis cross sections. Primary settlement refers to the consolidation settlement that occurs in saturated fine-grained soils after construction (loading), and secondary settlement refers to the long term settlement. Secondary settlement, for soils with organics present, is considered to occur directly after loading and occur at a logarithmic rate until additional loading is applied, at which point secondary settlement starts again (Feng, 2013). Estimated settlements were calculated for all known construction (loading) starting with the time the levees were built in 1964. The calculated settlements were compared to the estimated actual settlement. The actual settlement was estimated by determining the difference between the foundation elevation in 1964 (based on elevations provided in USACE As-Constructed Coyote Creek Channel Improvement Plans) and the current foundation elevation. In addition, future settlement based on current levee construction was estimated.

The amount of primary settlement is dependent on the thickness of the layer, the initial void ratio (e_o), the change in stress, the compression ratio (CR), and the recompression ratio (RR). CR and RR are dependent on the initial void ratio (e_o) and the compression index (Cc) or the recompression index (Cr). The field consolidation curve was corrected using Schmertmann's procedure to obtain a corrected Cc value. Consolidation settlement calculations were performed on the bay mud layers using Terzaghi's One-Dimensional Consolidation Theory (1968). Osterberg's stress distribution under a continuous embankment was used to compute the stress increase with depth (Osterberg, 1957). The water level was conservatively assumed to be at the ground surface.

Primary and secondary settlements were assumed to occur in the younger bay mud and the older bay mud layers. Soil parameters used in the analysis were obtained through evaluation of the laboratory test results and also compared to well-established parameters in literature (Bonaparte and Mitchell, 1979). Maximum past pressures estimated from the consolidation test data and obtained from correlations with CPT data indicate the bay mud layers are generally normally to slightly overconsolidated. The soil layers were assumed to be normally consolidated in the settlement calculations. The CR and RR values were calculated based on the average of the seven consolidation tests performed within the Coyote Creek system. The CR and RR value were estimated to be 0.33 and 0.04 respectively. The soil layers were assumed to act "double drained" (drainage occurs at the top and bottom of the layer) based on the assumption that sand seams encountered in several explorations serve as drainage boundaries. The parameter $C_{\alpha\varepsilon}$ was assumed to be equal to 0.004 for the bay mud. This is in agreement with literature from Bonaparte and Mitchell (1979) for an initial void ratio equal to 2.6, which is in the range of measured void ratios of the bay mud laboratory tests.

8. Analysis Results

As discussed, the Coyote Creek levee system was subdivided into 12 reaches. A total of eight representative cross sections were developed, of which four were analyzed for existing conditions. At the completion of existing conditions analysis, the project team documented whether a levee reach met or did not meet design criteria. To validate findings, the analysis results were checked against available records, including past performance data. When there were discrepancies in a given reach between analysis results and past performance, the project team performed additional analyses using reasonable variations in water levels, model geometry, and/or soil properties, as appropriate, to understand the apparent differences and to calibrate models.

Steady-state seepage analyses and steady-state, rapid drawdown, and seismic stability analyses (stability Cases II, III, and IV) were performed for each of the existing conditions levee analysis cross sections discussed above. Each cross section was analyzed for flood levels corresponding to the Baseline, FEMA Accredited, and FEMA Accredited with Sea Level Rise loading scenarios. The analysis results are presented in Figures 23 through 86. For each cross section, the seepage analysis results are illustrated by figures that show the seepage model with soil layering and parameters, and total head plots for the selected loading scenarios. The exit gradient was estimated at the levee toe. Likewise, for each cross section the stability analysis results are illustrated by figures that show the soil layering and parameters, and the trial failure surfaces (circular or non-circular, depending on which is more critical) with corresponding factors of safety for the various loading scenarios. This also includes figures illustrating the soil layering, soil parameters, and K_y values for seismic deformation analysis. The results of the seepage and stability analyses are summarized in Table 2 and illustrated in Figures 87 through 90.

The results of seepage and stability analyses for the existing conditions were evaluated to assess which reaches, or portions of reaches, meet the FEMA accreditation requirements. The results of the assessment are provided at the end of this section.

Based on the seepage analyses results (underseepage and through seepage) we conclude that only Reach 7 and the portion of Reach 3 that has an existing clay barrier (0.24 miles total) met seepage criteria for the Baseline scenario. For the FEMA Accredited and FEMA Accredited with SLR scenarios, only the portion of Reach 3 with the existing clay barrier (0.08 miles) met seepage criteria. The stretches of levee not meeting criteria for seepage were generally due to seepage break-out occurring on the landside slope of an embankment consisting of erodible materials. Seepage analysis results from the additional water surface scenarios can be interpolated from the existing conditions analyses as shown on Figures 87 through 90.

Based on the stability analyses results we conclude that Reach 3 (0.18 miles) met stability criteria for both steady-state seepage and rapid drawdown and does not require remediation for stability. Reaches 4, 7, and 9 do not meet criteria for steady-state landside slope stability under the FEMA with SLR scenario. Additionally, Reach 9 does not meet criteria for steady-state waterside slope stability under the Baseline scenario and does not meet criteria for waterside rapid drawdown under all flood levels considered. This is due primarily to an over steepened waterside slope. Slope stability analysis results from the additional water surface scenarios can be interpolated from the existing conditions analyses as shown on Figures 87 through 90.

Settlement was considered at the existing conditions analysis cross sections. Figures 91 through 94 illustrate the estimated primary and secondary settlement over time. The estimated actual settlement is shown for reference. Note that according to the USACE EM 1110-1-1904 “Settlement Analysis” the accuracy of settlement predictions are reasonable within 50% of actual settlements for many soil types. The results provided in this memorandum are within the 50% accuracy range. Several factors such as discrepancies between actual construction and as-built drawings, unavailable information on maintenance fills, non-uniform soil parameters, and expected deviations in laboratory and field testing all contribute to the expected inaccuracies. Future settlement, assuming no additional loading, is estimated to range between 2 to 3 inches and is due primarily to secondary settlement.

Freeboard was also considered along the levees. For the Baseline Scenario, freeboard was considered to meet criteria if the levee crest elevation was at least 1 foot above the design water surface elevation corresponding to a 20-year event, and at least 0.5 feet above the highest estimated tide (8.7 feet NAVD88 in 1959). For the FEMA Accredited and FEMA Accredited with SLR Scenarios, freeboard was considered to meet criteria if the levee crest elevation was at least 3 feet above the respective composite water surface profiles. Reaches 1, 6, and 11 were not considered for freeboard. Reach 1 is breached, and Reaches 6 and 11 are high ground. Reach 8 (incised concrete channel) met criteria if the water surface elevation and estimated high tide was contained within the channel. Table 3 and Figures 95 through 101 summarize the levee segments not meeting freeboard criteria by reach. Portions of every reach, except the high ground reaches, contained areas that did not meet freeboard requirements for at least one of the evaluation scenarios. Of the reaches considered for freeboard (Reaches 2-5, 7-10, and 12), which totaled 1.97 miles in length, 0.85 miles did not meet freeboard requirements for Baseline Scenario, 1.54 miles did not meet requirements for FEMA Accredited Scenario, and 1.64 miles did not meet requirements for FEMA with SLR Scenario.

Based on the results of the seismic deformation analysis, Reaches 3 and 4 are expected to have little to no seismic deformation. Reach 7 may experience a small amount of seismic deformation (about 7 inches along the slope or 5 inches vertical). Reach 9 is likely to experience the largest seismic deformation (up to 3 feet along the slope or 2 feet vertical). This is primarily due to the steep waterside slope and shallower bay mud deposits.

Freeboard, Seepage and Stability Existing Condition Result Summary Table

	Freeboard[7]			Seepage									Stability								
				Underseepage[1]			Through Seepage			Landside Steady-State			Waterside Steady-State			Waterside Rapid Drawdown					
Reach	Baseline	FEMA	FEMA w/ SLR	Baseline	FEMA	FEMA w/ SLR	Baseline	FEMA	FEMA w/ SLR	Baseline	FEMA	FEMA w/ SLR	Baseline	FEMA	FEMA w/ SLR	Baseline	FEMA	FEMA w/ SLR			
1	DNM	DNM	DNM	Breached																	
2	See Reach 3			Reach conditions will be adequately represented by Reach 12																	
3a[2]	DNM	DNM	DNM	Meet	Meet	Meet	DNM	DNM	DNM	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet			
3b[3]	DNM	DNM	DNM	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet	Meet			
4[4]	DNM	DNM	DNM	Meet	Meet	Meet	DNM	DNM	DNM	Meet	Meet	DNM	Meet	Meet	Meet	Meet	Meet	Meet			
5	DNM	DNM	DNM	No existing levee embankment																	
6	High ground (improvements not necessary)																				
7[5]	DNM	DNM	DNM	Meet	Meet	Meet	Meet	DNM	DNM	Meet	Meet	DNM	Meet	Meet	Meet	Meet	Meet	Meet			
8	DNM	DNM	DNM	No existing levee embankment																	
9[6]	DNM	DNM	DNM	Meet	Meet	Meet	DNM	DNM	DNM	Meet	Meet	DNM	DNM	Meet	Meet	DNM	DNM	DNM			
10	DNM	DNM	DNM	No existing levee embankment																	
11	High ground (improvements not necessary)																				
12	DNM	DNM	DNM	No existing levee embankment (to be analyzed during remedial alternative analysis)																	

Note: DNM = Does Not Meet

[1] Allowable gradient of 0.5 for underseepage criteria per USACE EM-1110-2-1913 Levee Design Manual.

[2] Both FEMA Accredited and FEMA Accredited w/ SLR WSEs were taken at top of levee elevation for Reach 3.

[3] Refers to portion of reach containing clay barrier.

[4] FEMA Accredited w/ SLR WSE was taken at top of levee elevation for Reach 4.

[5] FEMA Accredited w/ SLR WSE was taken at top of levee elevation for Reach 7.

[6] FEMA Accredited w/ SLR WSE was taken at top of levee elevation for Reach 9.

[7] Freeboard is derived from the Hydraulic Analysis Technical Memorandum.

9. 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 the FEMA geotechnical criteria. Analysis results provided at the end of this section were in general agreement with the observed past performance in the levee system. Through seepage has been observed in the past, as well as waterside sloughing.

Future remedial alternative evaluations will be performed for Reaches 2, 3, 4, 5, 7, 8, 9, 10 and 12. Reaches 6 and 11 will not be considered because these reaches represent high ground that will not be susceptible to freeboard, seepage, and stability concerns. Reach 1 will not be considered because this reach has been breached and is not expected to be recovered. Reach 2 is adequately represented by Reach 12, and will be considered in conjunction with remedial alternative evaluation performed on Reach 12.

Based on review of flood capacity and levee segments where geotechnical criteria are not met, the potential remediations within the Coyote Creek levee system are expected to be either a raised levee with earthen fill (where space is available) or raised levee with floodwalls and/or other engineered structures. The list below provides potential remediation options for each reach.

- Reach 1 – This portion of levee is breached and no remedial options will be considered.
- Reach 2 – Earthen fill or floodwall will be evaluated for this portion of levee.
- Reach 3 – Earthen fill or floodwall will be evaluated for this portion of levee.
- Reach 4 – Earthen fill or floodwall will be evaluated for this portion of levee.
- Reach 5 – Earthen fill or floodwall will be evaluated for this portion of levee.
- Reach 6 – This portion of levee is high ground and no remedial options will be considered in Reach 6. A setback levee will be considered near confluence of Nyhan Creek and Coyote Creek to increase channel capacity and create space for Reach 9 waterside repairs.
- Reach 7 – Earthen fill or floodwall will be evaluated for this portion of levee.
- Reach 8 – Floodwall will be considered for this portion of system (concrete-lined channel) where overtopping occurs.
- Reach 9 – Earthen fill or floodwall will be evaluated for this portion of levee.

- Reach 10 – Earthen fill or floodwall will be evaluated for this portion of levee.

- Reach 11 – This portion of levee is high ground and no remedial options will be considered in Reach 11. A setback levee will be considered near confluence of Nyhan Creek and Coyote Creek to increase channel capacity and create space for Reach 9 waterside repairs.

- Reach 12 – Earthen fill or floodwall will be evaluated for this portion of levee.

The reaches to be assessed for remedial alternatives will be assessed for viability with respect to land use, environmental, and construction constraints to be discussed in the Alternatives Evaluation Memorandum.

10. References

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Tables

TABLE 1. SUMMARY OF REACH AND CROSS-SECTION CHARACTERISTICS
Coyote 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="9">1</td><td rowspan="9">0+00 CC-L to 12+80 CC-L</td><td rowspan="9">N/A</td><td>Levee Height :</td><td rowspan="3">According to USACE Record Drawings, a 2 to 3-foot tall levee with 18-foot wide crest was constructed in 1965. Landside of the constructed levee (Bothin Marsh) was designated a disposal area and backfilled with uncompacted fill to the same levee height resulting in a final levee height of 0'. Based on current topography the levee and disposal area has settled between 1 and 3' since levee construction.</td><td rowspan="9">• Extents defined as portion of Lower Coyote Creek with little to no embankment near Bothin Marsh.

• Reach has thick younger bay mud deposits overlying older bay mud and bedrock.

• Settlement noted for the entire reach.</td><td rowspan="9">No reach-specific cross-section developed. This reach is currently breached at multiple locations. Improvement alternatives would likely present prohibitive challenges.</td><td rowspan="9">N/A</td></tr>
<tr><td>Crown Width:</td></tr>
<tr><td>Landside Slope:</td></tr>
<tr><td>Waterside Slope:</td><td>typ. 2H:1V (ranges from 2H to 3H:1V)</td></tr>
<tr><td>Past Performance:</td><td>Settlement</td></tr>
<tr><td>Improvement History:</td><td>None available for review.</td></tr>
<tr><td>Embankment Materials:</td><td>As described above, no indication of the embankment constructed in 1965 remains because of landside backfill and settlement. Subsurface explorations indicate approximately 5 feet of fill material consisting of clayey sand (SC) or clayey gravel (GC).</td></tr>
<tr><td>Foundation Materials:</td><td>Typically 5-foot thick settled fill material consisting of sandy clay (CL) or clayey gravel (GC) underlain by 80-foot thick layer of younger bay mud (CL-CH). Younger bay mud overlays approximately 15-foot thick older bay mud/alluvium on bedrock.</td></tr>
</table>

TABLE 1. SUMMARY OF REACH AND CROSS-SECTION CHARACTERISTICS
Coyote 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
2	12+80 CC-L to 20+00 CC-L	N/A	Levee Height : Crown Width: Landside Slope:	Portion of reach contains floodwall built in 1977 with negligible levee height adjacent to high ground. Upstream portion of Reach contains 1-2' tall levee embankment adjacent to landside high ground. Minor amounts of fill were placed in 1965 and again in 1977. Existing topography indicates settlement occurred within this reach.	• Portion of Coyote Creek with little to no landside levee height. • Portion of Coyote Creek with adjacent high ground on landside. • Reach has thick younger bay mud deposits overlying older bay mud and bedrock.	No reach-specific cross-section developed. This reach is adjacent to high ground on the landside with no landside levee height. Reach conditions will be adequately represented by Reach 12.	N/A
			Waterside Slope:	typ. 2H:1V (ranges from 2H to 3H:1V)			
			Past Performance:	Settlement			
			Improvement History:	Floodwall in 1977, fill added in 1977, Pump station in 1985			
			Embankment Materials:	As described above, minor amounts of fill were placed in in 1965 and 1977 and appears to have experienced settlement. Subsurface explorations indicate approximately 5 feet of fill material consisting of clayey gravel (GC).			
			Foundation Materials:	Typically 5-foot thick settled fill material layer of clayey gravel (GC) overlaying 5-foot thick sandy clay (CL) underlain by 40 to 50-foot thick layer of younger bay mud (CL-CH). Younger bay mud thins upstream. Younger bay mud overlays 15-foot thick older bay mud/alluvium on bedrock.			

TABLE 1. SUMMARY OF REACH AND CROSS-SECTION CHARACTERISTICS
Coyote 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
3	20+00 CC-L to 29+70 CC-L	28+00 CC-L	Levee Height :	typ. 2.5 ft (ranges from 1 to 3.5 ft)	• Downstream reach extent at transition from landside high ground to measureable levee height. • Upstream reach extent at the end of the clay barrier wall. • Reach has generally thinner deposits of younger and older bay mud than adjacent Reaches 2 and 4.	<u>28+00 CC-L</u> • Tall and steep waterside slope • Comparatively taller and steeper landside slope • Narrow crown width • Constrained on landside by residential neighborhood. • Section will be analyzed with and without barrier wall to represent conditions within entire Reach.	• KB-3 (on section for embankment and shallow foundation stratigraphy) • ALB 1993 B-3 (projected for embankment and deeper foundation stratigraphy) • 2F-14 (projected for embankment and shallow foundation stratigraphy) • ALB 1993 B-2 (projected for embankment and deeper foundation stratigraphy) • KB-2 (projected for embankment and shallow foundation stratigraphy) • GEI B-9 (projected for embankment and deeper foundation stratigraphy) • GEI CPT-3 (projected for landward stratigraphy)
			Crown Width:	typ. 11 ft (ranges from 9 to 12 ft)			
			Landside Slope:	typ. 4H:1V (ranges from 2H to 4H:1V, anomalous 15H:1V at downstream portion of reach)			
			Waterside Slope:	typ. 3H:1V (ranges from 2.7H to 5H:1V)			
			Past Performance:	Historical seepage			
			Improvement History:	Floodwall in 1977, Pump station in 1983, Clay Barrier in 2003			
			Embankment Materials:	Embankment material is concrete block floodwall constructed on top of earthen levee. Concrete floodwall consists of two parallel concrete masonry unit (CMU) walls connected with tie-rods. Fill material within the walls are generally poorly graded gravel (GP), poorly graded gravel with silt (GP-GM), poorly graded sand with silt (SP-SM), or poorly graded sand (SP). The floodwall and fill material range from 1 to 3.5 feet tall. Embankment material on downstream portion of reach consists of clayey gravel (GC).			
			Foundation Materials:	Typically 5-foot thick clayey sand (SC) and sandy clay (CL) layer underlain by 20-foot thick younger bay mud (CL-CH). Younger bay mud thickness ranges between 15 and 35 feet thick. Younger bay mud overlays 5 to 15-foot thick older bay mud/alluvium on bedrock.			
4	29+70 CC-L to 36+91 CC-L	34+00 CC-L	Levee Height :	typ. 3 to 4 ft (ranges from 2 to 5 ft)	• Downstream reach extent at end of clay barrier wall. • Upstream reach extent at the transition to the concrete channel section of Coyote Creek. • Reach has generally thicker deposits of younger and older bay mud than adjacent Reaches 3 and 8. • Portion of reach has steepened waterside slopes due to added embankment fill.	<u>34+00 CC-L</u> • Tall and steep waterside and landside slopes • Narrow levee footprint • Contains a sandy lens within the embankment which could contribute to through seepage • Constrained on landside by residential neighborhood	• ALB 1993 B-4 (projected for embankment and deeper foundation stratigraphy) • GEI B-8 (projected for embankment and shallow foundation stratigraphy) • GEI CPT-3 (projected for landward stratigraphy) • GEI GP-2 (projected for landward stratigraphy)
			Crown Width:	typ. 12 ft (ranges from 6 to 18 ft)			
			Landside Slope:	typ. 3H:1V (ranges from 2.0H to 4H:1V, anomalous 12H:1V at upstream portion of reach)			
			Waterside Slope:	typ. 2H:1V (ranges from 1.7H to 4H:1V)			
			Past Performance:	Steepened slopes and erosion on waterside			
			Improvement History:	Added fill in 1977			
			Embankment Materials:	Embankment material is generally clayey gravel (GC), clayey sand (SC), and sandy clay (CL)			
			Foundation Materials:	Typically 5-foot thick clayey sand (SC) and sandy clay (CL) layer underlain by 30-foot thick younger bay mud (CL-CH). Bay mud deposits thin upstream. Younger bay mud ranges between 3 to 30 feet thick overlying 5 to 25-foot thick older bay mud/alluvium on bedrock.			

TABLE 1. SUMMARY OF REACH AND CROSS-SECTION CHARACTERISTICS
Coyote 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
5	0+00 CC-R to 15+50 CC-R	5+00 CC-R	Levee Height : Crown Width: Landside Slope:	According to USACE Record Drawings, a 2 to 7-foot tall levee with 18-foot wide crest was constructed in 1965 between 0+00 CC-R and approximately 10+00 CC-R. Based on current topography the levee has settled to the point that there is negligible levee height.	• Extents defined as portion of Lower Coyote Creek with little to no embankment adjacent to lower ground at marsh and hotel. • Reach has thick younger bay mud deposits overlying older bay mud and bedrock. • Settlement noted for the entire reach.	5+00 CC-R • Thick bay mud deposit with no existing levee. • Constrained on landside by existing hotel structure.	• GEI B-5 (projected for landward stratigraphy) • 2F-19 (projected for deeper foundation stratigraphy) • B-4 (2009) (projected for shallow foundation stratigraphy)
			Waterside Slope:	typ. 3H:1V (ranges from 2H to 10H:1V)			
			Past Performance:	Settlement			
			Improvement History:	Tennessee Valley Pathway in 2013.			
			Embankment Materials:	As described above, no indication of the embankment constructed in 1965 remains because of settlement. Subsurface explorations indicate approximately 5 to 10-foot settled fill material consisting of clayey sand (SC) or clayey gravel (GC).			
			Foundation Materials:	Typically 5 to 10-foot thick settled fill material consisting of sandy clay (CL) or clayey sand (SC) underlain by 80-foot thick layer of younger bay mud (CL-CH). Younger bay mud thins upstream and towards southeastern hillside (ranges between 20-80' thick). Younger bay mud overlays 5 to 15-foot thick older bay mud/alluvium on bedrock.			
6	15+50 CC-R to 29+10 CC-R	N/A	Levee Height : Crown Width: Landside Slope:	Reach is adjacent to high ground and does not include a levee embankment.	• Extents defined as portion of Coyote Creek with little to no embankment adjacent to higher ground at roadway and hillside. • Reach has thinner deposits of younger and older bay mud than adjacent Reaches 5 and 7.	No reach-specific cross-section developed. This reach is adjacent to high ground on the landside with no landside levee height. It is unlikely that improvement alternatives would be needed along this Reach.	N/A
			Waterside Slope:	typ. 4H:1V (ranges from 2.0H to 7.0H:1V)			
			Past Performance:	None available for review.			
			Improvement History:	Tennessee Valley Pathway in 2013.			
			Embankment Materials:	As described above, the reach is adjacent to high ground and does not contain levee embankment material.			
			Foundation Materials:	Typically 5-foot thick layer of sandy clay (CL) fill material, with localized deposits of colluvium extending from the hill slopes, underlain by 15-foot thick layer of younger bay mud (CL-CH). Younger bay mud thins upstream (ranges between 5-20' thick). Younger bay mud overlays 5 to 15-foot thick older bay mud/alluvium on bedrock.			

TABLE 1. SUMMARY OF REACH AND CROSS-SECTION CHARACTERISTICS
Coyote 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
7	29+10 CC-R to 37+29 CC-R	35+00 CC-R	Levee Height :	typ. 3 to 4 ft (ranges from 2 to 5 ft)	• Downstream reach extent at transition from landside high ground to measureable levee height. • Upstream reach extent at the transition to the concrete channel section of Coyote Creek. • Reach has generally thicker deposits of younger and older bay mud than adjacent Reaches 6 and 8. • Portion of reach has steepened waterside slopes due to added embankment fill.	35+00 CC-R • Steep waterside and landside slope. • Narrow levee footprint. • Constrained on landside by residential neighborhood.	• GEI B-2 (on section for embankment and deeper stratigraphy) • KB-3 (2006a) (projected for embankment and shallow foundation stratigraphy) • KB-2 (2006a) (projected for embankment and shallow foundation stratigraphy) • KB-1 (2006a) (projected for shallow landward stratigraphy)
			Crown Width:	typ. 12 ft (ranges from 6 to 18 ft)			
			Landside Slope:	typ. 3H:1V (ranges from 2.0H to 4H:1V, anomalous 10H:1V at upstream portion of reach)			
			Waterside Slope:	typ. 2H:1V (ranges from 1.7H to 6H:1V)			
			Past Performance:	Steepened slopes on waterside.			
			Improvement History:	Added fill in 1977, 2006 and 2007.			
			Embankment Materials:	Embankment material is generally clayey gravel (GC) and clayey sand (SC).			
			Foundation Materials:	Typically 5-foot thick clayey sand (SC) and sandy clay (CL) layer underlain by 15 to 55-foot thick younger bay mud (CL-CH). Younger bay mud overlays 1 to 5-foot thick older bay mud/alluvium on bedrock. Younger and older bay mud thinnest at the downstream limit of reach near adjacent hillside.			
8	0+00 CC-C to 29+38 CC-C and 0+00 CC-C2 to 3+00 CC-C2	4+00 CC-C	Levee Height :	This reach consists of concrete channels with little to no embankment and vertical channel walls.	• Extents defined as portion of Coyote Creek with little to no embankment and concrete channel. • Reach has thinner deposits of younger bay mud and older bay mud than downstream Reaches 4 and 7.	4+00 CC-C • Thicker bay mud deposit. • Low landside adjacent to levee alignment. • Inflection in top of wall elevations results in area of overtopping based on hydraulic analysis.	• GEI B-1 (on section for landward stratigraphy) • 2F-3 (1964) (projected for shallow foundation stratigraphy)
			Crown Width:				
			Landside Slope:				
			Waterside Slope:				
			Past Performance:	None available for review.			
			Improvement History:	None available for review.			
			Embankment Materials:	As described above, there is little to no embankment within this reach. However, material behind the channel wall is approximately 1-foot thick filter material with subdrains and 3 to 4 feet of structural backfill sloped at 0.5H:1V. Native material behind the channel wall is generally sandy silt (ML) and silty sand (SM) with gravelly layers. The native material transitions to clayey sand (SC), sandy clay (CL), and clay (CL) at the downstream portion of the reach.			
			Foundation Materials:	Below the concrete channel is at least 1-foot thick filter material underlain by native soils. Native soils typically consist of 10-foot thick older bay mud (CH) and sandy clay (CL) underlain by bedrock. Layer thins near the northern hillsides (ranges between 5 to 15 feet thick).			

TABLE 1. SUMMARY OF REACH AND CROSS-SECTION CHARACTERISTICS
Coyote 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="8">9</td><td rowspan="8">0+00 NC-L to 4+75 NC-L</td><td rowspan="8">1+00 NC-L</td><td>Levee Height :</td><td>typ. 3.5 ft (ranges from 3 to 4 ft)</td><td rowspan="8">• Downstream reach extent at confluence of Nyhan Creek and Coyote Creek.

• Upstream reach extent at transition from measureable levee height to negligible embankment.

• Reach has generally thicker deposits of younger and older bay mud than adjacent upstream Reach 10.

• Portion of reach has steepened landside and waterside slopes on the embankment.</td><td rowspan="8"><u>1+00 NC-L</u>
• Tall and steep landside and waterside slope.

• Thicker bay mud deposit.

• Sandy lens present in embankment.

• Sandy lens present 20 feet below levee crest.

• Constrained on landside by residential neighborhood.</td><td rowspan="8">• GEI B-10 (projected for embankment and deeper foundation stratigraphy)
• 2F-1 (1964) (projected for shallow foundation stratigraphy)</td></tr>
<tr><td>Crown Width:</td><td>typ. 7.5 ft (ranges from 4 to 13 ft)</td></tr>
<tr><td>Landside Slope:</td><td>typ. 3H:1V (ranges from 1.7H to 3H:1V, anomalous 12H:1V at upstream portion of reach)</td></tr>
<tr><td>Waterside Slope:</td><td>typ. 2H:1V (ranges from 1H to 5H:1V)</td></tr>
<tr><td>Past Performance:</td><td>Steepened slopes on waterside and landside.</td></tr>
<tr><td>Improvement History:</td><td>Pump station in 1978, Added fill in 1977, 2006 and 2007, Erosion repair (pre 2012).</td></tr>
<tr><td>Embankment Materials:</td><td>Embankment material is generally clayey sand (SC) and clay (CL) with portions of clayey gravel (GC).</td></tr>
<tr><td>Foundation Materials:</td><td>Typically 3 to 5-foot thick layer of clayey sand (SC) underlain by 40-foot thick layer of younger bay mud (CL-CH) with a sandy lens present at approximately 20-foot depth. Younger bay mud thins upstream (ranges between 25-50' thick). Younger bay mud overlays 20-foot thick older bay mud/alluvium on bedrock.</td></tr>
<tr><td rowspan="8">10</td><td rowspan="8">4+75 NC-L to 10+50 NC-L</td><td rowspan="8">7+00 NC-L</td><td>Levee Height :</td><td rowspan="3">No levee embankment has been constructed within this reach.</td><td rowspan="8">• Extents defined as portion of Nyhan Creek with little to no embankment adjacent to lower ground.

• Downstream reach extent at transition from measureable levee height to negligible embankment.

• Upstream reach extent at limit of project.

• Reach has thinner deposits of younger and older bay mud than downstream adjacent Reach 9.</td><td rowspan="8"><u>7+00 NC-L</u>
• Sandy lens present 15 feet below levee alignment.

• No existing levee.

• Low point adjacent to levee alignment on landside.</td><td rowspan="8">• GEI B-7 (projected for foundation stratigraphy)
• GEI CPT-9 (projected for foundation stratigraphy)
• KB-4 (2007) (projected for shallow landward stratigraphy)</td></tr>
<tr><td>Crown Width:</td></tr>
<tr><td>Landside Slope:</td></tr>
<tr><td>Waterside Slope:</td><td>typ. 3H:1V (ranges from 1.5H to 10H:1V)</td></tr>
<tr><td>Past Performance:</td><td>None available for review.</td></tr>
<tr><td>Improvement History:</td><td>None available for review.</td></tr>
<tr><td>Embankment Materials:</td><td>As described above, no levee embankment has been constructed within this reach.</td></tr>
<tr><td>Foundation Materials:</td><td>Typically 5-foot thick layer of clayey gravel (GC) underlain by 15 to 20-foot thick layer of younger bay mud (CL-CH) with a sandy lens present at approximately 15-foot depth. Younger bay mud overlays 20-foot thick older bay mud/alluvium on bedrock. Older bay mud thins upstream (ranges between 5-20' thick).</td></tr>
</table>

TABLE 1. SUMMARY OF REACH AND CROSS-SECTION CHARACTERISTICS
Coyote 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="8">11</td><td rowspan="8">0+00 NC-R to 9+70 NC-R</td><td rowspan="8">N/A</td><td>Levee Height :</td><td rowspan="3">Reach is adjacent to high ground and does not include a levee embankment.</td><td rowspan="8">• Extents defined as portion of Nyhan Creek with little to no embankment adjacent to higher ground.

• Downstream reach extent at confluence of Nyhan Creek and Coyote Creek.

• Upstream reach extent at limit of project.

• Reach has thinner deposits of younger bay mud than adjacent downstream Reach 6.</td><td rowspan="8">No reach-specific cross-section developed. This reach is adjacent to high ground on the landside with no landside levee height. It is unlikely that improvement alternatives would be needed along this Reach.</td><td rowspan="8">N/A</td></tr>
<tr><td>Crown Width:</td></tr>
<tr><td>Landside Slope:</td></tr>
<tr><td>Waterside Slope:</td><td>typ. 3H:1V (ranges from 1.5H to 10H:1V)</td></tr>
<tr><td>Past Performance:</td><td>None available for review.</td></tr>
<tr><td>Improvement History:</td><td>None available for review.</td></tr>
<tr><td>Embankment Materials:</td><td>As described above, the reach is adjacent to high ground and does not contain levee embankment material.</td></tr>
<tr><td>Foundation Materials:</td><td>Typically 5-foot thick layer of gravelly clay (CL) underlain by 15-foot thick layer of younger bay mud (CL-CH) on bedrock. No older bay mud is present .</td></tr>
<tr><td rowspan="8">12</td><td rowspan="8">0+00 BM-L to 13+24 BM-L</td><td rowspan="8">9+00 BM-L</td><td>Levee Height :</td><td rowspan="3">Reach has historically been subject to fill placement and dumping of rubble resulting in high ground and a negligible levee height.</td><td rowspan="8">• Extents defined as portion of Bothin Marsh with high ground.

• Reach has thick younger bay mud deposits overlying older bay mud and bedrock.</td><td rowspan="8"><u>9+00 BM-L</u>
• Lower elevations on landside.

• Steeper waterside slope.</td><td rowspan="8">• GEI CPT-8 (projected for foundation stratigraphy)
• GEI B-6 (projected for foundation stratigraphy)</td></tr>
<tr><td>Crown Width:</td></tr>
<tr><td>Landside Slope:</td></tr>
<tr><td>Waterside Slope:</td><td>typ. 6H:1V (ranges from 4.3H to 14H:1V)</td></tr>
<tr><td>Past Performance:</td><td>None available for review.</td></tr>
<tr><td>Improvement History:</td><td>None available for review.</td></tr>
<tr><td>Embankment Materials:</td><td>As described above, reach is high ground with negligible levee height. However, subsurface explorations indicate fill material consists of 5-foot thick layer of clayey gravel (GC).</td></tr>
<tr><td>Foundation Materials:</td><td>Typically 5-foot thick fill layer of clayey gravel (GC) underlain by 15-foot thick layer of interbedded layers of clayey sand (SC), clay (CH), and clayey gravel (GC). Interbedded layers are underlain by 30 to 55-foot thick layer of younger bay mud (CL-CH). Younger bay mud overlays 15-foot thick older bay mud/alluvium on bedrock. No older bay mud is present on the right bank.</td></tr>
</table>

TABLE 2. SUMMARY OF EXISTING CONDITION SEEPAGE AND STABILITY RESULTS
Coyote Creek Levee Evaluation Project

Reach	Reach Length (feet)	Analysis Section Station	Through Seepage							Stability													
			Through Seepage Breakout Height above Landside Toe (ft)			Flow Through Embankment (gpd/ft)			Conclusions	Landside Stability Factors of Safety			Waterside Stability Factors of Safety			Waterside Rapid Drawdown Factors of Safety (Drop to Channel Bottom)			Seismic Displacement				
			Baseline	FEMA Accredited	FEMA Accredited w/ SLR	Baseline	FEMA Accredited	FEMA Accredited w/ SLR		Baseline	FEMA Accredited	FEMA Accredited w/ SLR	Baseline	FEMA Accredited	FEMA Accredited w/ SLR	Baseline	FEMA Accredited	FEMA Accredited w/ SLR	k_y (g)	k_{max} (g)	k_y/k_{max}	Newmark Displacement (ft)	Estimated Vertical Displacement (ft)
3[1]	971	28+00 CC-L	2.4	2.7	2.7	0.1	0.1	0.1	• Portion of Reach consists of concrete floodwall and cutoff wall. • The other portion of Reach consists of earthen embankment with erodible SP and SP-SM. • **The portion of Reach without the cutoff wall does not meet criteria at the Baseline, FEMA Accredited, and FEMA Accredited w/ SLR water surface elevations** due to a high phreatic surface breakout and erodible material.	1.81	1.80	1.80	2.59	2.64	2.64	1.66	1.67	1.67	0.19	0.25	0.77	0.0	0.0
4[2]	721	34+00 CC-L	0.7	0.9	2.0	88.8	125.5	253.5	• A portion of the Reach consists of earthen embankment with erodible SP. • The **Reach does not meet criteria at the Baseline, FEMA Accredited, and FEMA Accredited w/ SLR water surface elevations** due to a high phreatic surface breakout and erodible material.	1.68	1.55	1.25	2.32	2.44	2.40	1.89	1.88	1.88	0.25	0.25	1.00	0.0	0.0
5	1550	5+00 CC-R	Does Not Meet Freeboard							Does Not Meet Freeboard													
7[2]	819	35+00 CC-R	0.4	0.6	1.7	0.2	0.2	0.4	• A portion of the Reach consists of earthen embankment with erodible SM, ML/SM, and ML. • The entire **Reach meets criteria at the Baseline water surface elevation** due to a low phreatic surface breakout. • The Reach **does not meet criteria at the FEMA Accredited and FEMA Accredited w/ SLR water surface elevations** due to a high phreatic surface breakout and erodible material.	1.76	1.63	1.36	1.93	2.12	2.26	1.45	1.45	1.40	0.08	0.25	0.33	0.6	0.4
8	3238	4+00 CC-C	Does Not Meet Freeboard							Does Not Meet Freeboard													
9[2]	475	1+00 NC-L	0.8	1.4	2.5	1.0	1.9	3.4	• A portion of the Reach consists of earthen embankment with erodible SW. • The Reach **does not meet criteria at the Baseline, FEMA Accredited, and FEMA Accredited w/ SLR water surface elevations** due to a high phreatic surface breakout and erodible material.	1.78	1.61	1.37	1.38	1.48	1.51	1.16	1.15	1.16	0.03	0.18	0.16	2.5	1.8
10	575	7+00 NC-L	Does Not Meet Freeboard							Does Not Meet Freeboard													
12	1324	9+00 BM-L	Does Not Meet Freeboard							Does Not Meet Freeboard													

[1] Both FEMA Accredited and FEMA Accredited w/ SLR WSE's are taken at top of levee elevation for cases in Reach 3 because WSE exceeds top of levee elevation.
[2] FEMA Accredited w/ SLR WSE was taken at the top of levee elevation for cases in Reaches 4, 7, and 9 because WSE exceeds top of levee elevation.

TABLE 3. SUMMARY OF LEVEE SEGMENTS NOT MEETING CRITERIA FOR FREEBOARD
Coyote Creek Levee Evaluation Project

Reach ID	Alignment	Reach Sta. Start	Reach Sta. End	Reach Length (ft)	Levee Segments not Meeting Criteria for Freeboard		
					Baseline[1]	FEMA[2]	FEMA w/SLR[3]
1	CC-L	0+00	12+80	1,280	Length: 1,280 ft (100% of Reach) • Sta. 0+00 to 12+80		
2	CC-L	12+80	20+00	720	Length: 206 ft (29% of Reach) • Sta. 12+80 to 14+86	Length: 708 ft (98% of Reach) • Sta. 12+80 to 17+36 • Sta. 17+48 to 20+00	Length: 720 ft (100% of Reach) • Sta. 12+80 to 20+00
3	CC-L	20+00	29+70	970	Length: 615 ft (63% of Reach) • Sta. 21+49 to 22+26 • Sta. 22+57 to 22+62 • Sta. 23+77 to 24+32 • Sta. 24+60 to 24+61 • Sta. 24+93 to 29+70	Length: 970 ft (100% of Reach) • Sta. 20+00 to 29+70	Length: 970 ft (100% of Reach) • Sta. 20+00 to 29+70
4	CC-L	29+70	36+91	721	Length: 322 ft (45% of Reach) • Sta. 29+70 to 32+83 • Sta. 33+14 to 33+23	Length: 721 ft (100% of Reach) • Sta. 29+70 to 36+91	Length: 721 ft (100% of Reach) • Sta. 29+70 to 36+91
5	CC-R	0+00	15+50	1,550	Length: 1,528 ft (99% of Reach) • Sta. 0+00 to 15+28	Length: 1,550 ft (100% of Reach) • Sta. 0+00 to 15+50	Length: 1,550 ft (100% of Reach) • Sta. 0+00 to 15+50
6	CC-R	15+50	29+10	1,360	Length: 0 ft (0% of Reach)		
7[4]	CC-R	29+10	37+29	819	Length: 179 ft (22% of Reach) • Sta. 31+95 to 32+91 • Sta. 34+25 to 34+31 • Sta. 36+52 to 37+29	Length: 629 ft (77% of Reach) • Sta. 31+00 to 37+29	Length: 629 ft (77% of Reach) • Sta. 31+00 to 37+29
8[5]	CC-C1	0+00	29+38	3,238	Length: 733 ft (23% of Reach) • Sta. 3+06 to 10+39	Length: 1,210 ft (37% of Reach) • Sta. 0+00 to 12+10	Length: 1,673 ft (52% of Reach) • Sta. 0+00 to 16+73
	CC-C2	0+00	3+00		Length: 0 ft (0% of Reach)		
9	NC-L	0+00	4+75	475	Length: 83 ft (17% of Reach) • Sta. 3+92 to 4+75	Length: 475 ft (100% of Reach) • Sta. 0+00 to 4+75	Length: 475 ft (100% of Reach) • Sta. 0+00 to 4+75
10	NC-L	4+75	10+50	575	Length: 575 ft (100% of Reach) • Sta. 4+75 to 10+50	Length: 575 ft (100% of Reach) • Sta. 4+75 to 10+50	Length: 575 ft (100% of Reach) • Sta. 4+75 to 10+50
11	NC-R	0+00	9+70	970	Length: 0 ft (0% of Reach)		
12	BM-L	0+00	13+24	1,324	Length: 253 ft (19% of Reach) • Sta. 0+00 to 0+22 • Sta. 1+40 to 1+45 • Sta. 3+56 to 3+72 • Sta. 7+99 to 8+33 • Sta. 9+20 to 10+96	Length: 1,277 ft (96% of Reach) • Sta. 0+00 to 0+84 • Sta. 1+02 to 4+00 • Sta. 4+29 to 13+24	Length: 1,324 ft (100% of Reach) • Sta. 0+00 to 13+24

[1]Baseline freeboard requirement calculated as the maximum of the Water Surface Elevation + 1 ft and High Tide (8.7 ft + 0.5 ft).

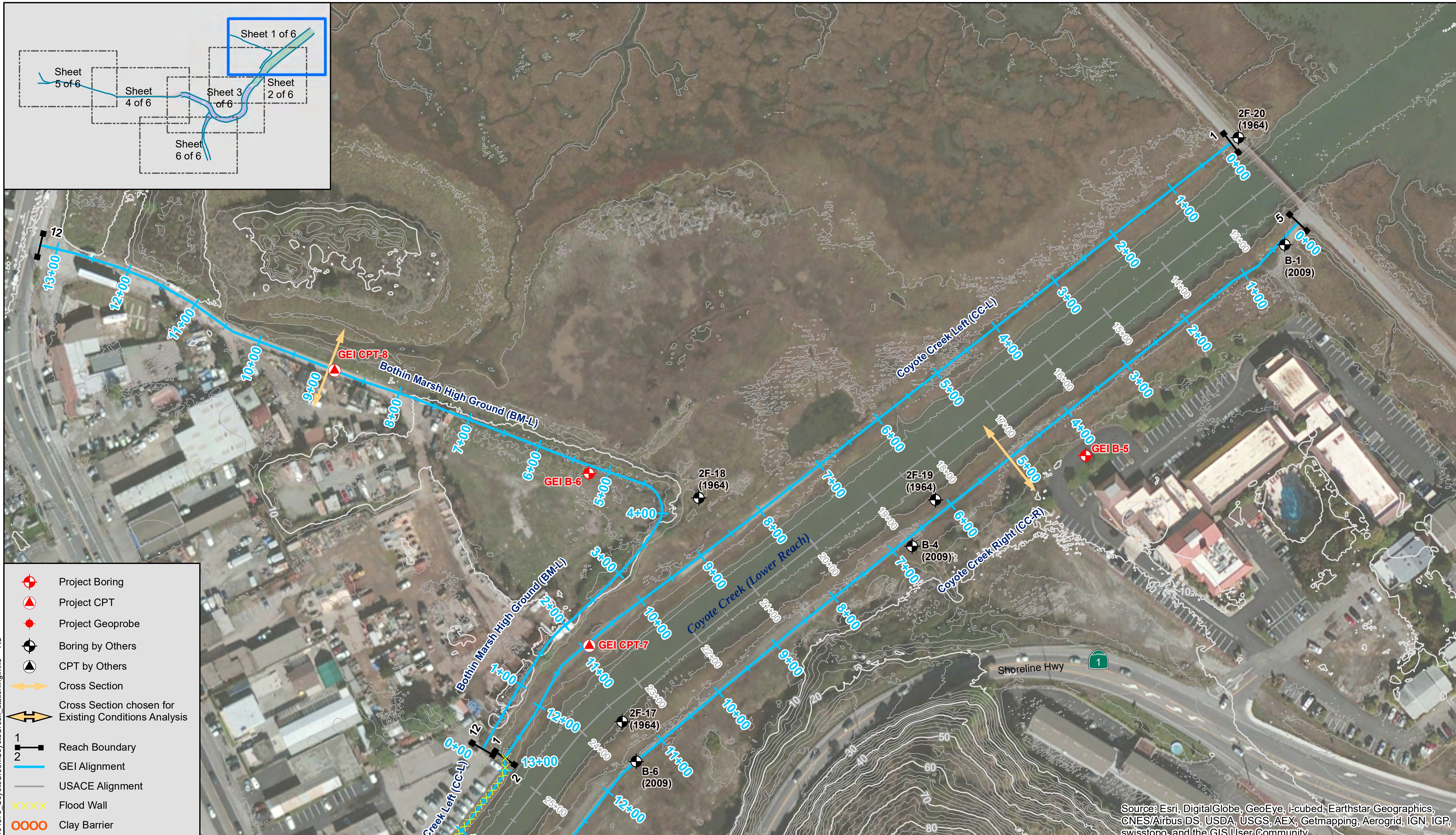
[2]FEMA freeboard requirement calculated as the maximum of the Water Surface Elevation + 3 ft and High Tide (9.7 ft + 3 ft).

[3]FEMA w/SLR freeboard requirement calculated as the maximum of the Water Surface Elevation + 3 ft and High Tide (12.7 ft + 3 ft).

[4]The length of Reach 7 includes the portion of the CC-R alignment spanning the confluence of Coyote Creek and Nyhan Creek (approximately 190 ft).

[5]The freeboard requirement is calculated as the maximum of the Water Surface Elevation and High Tide.

Figures



200 100 0 200

Feet

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

SITE CHARACTERIZATION PLAN

MAY 2015

FIGURE 1

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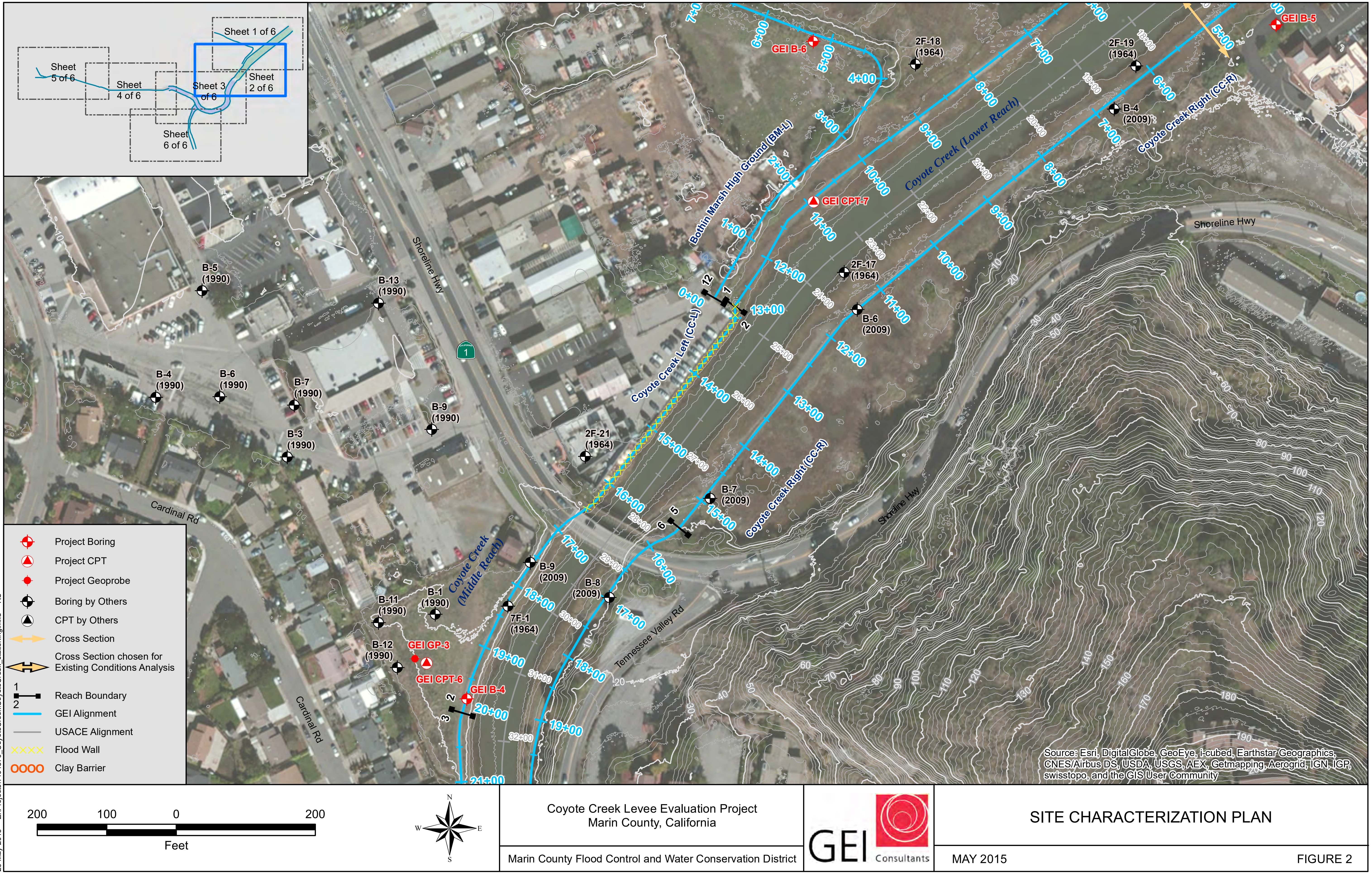

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

SITE CHARACTERIZATION PLAN

MAY 2015

FIGURE 2

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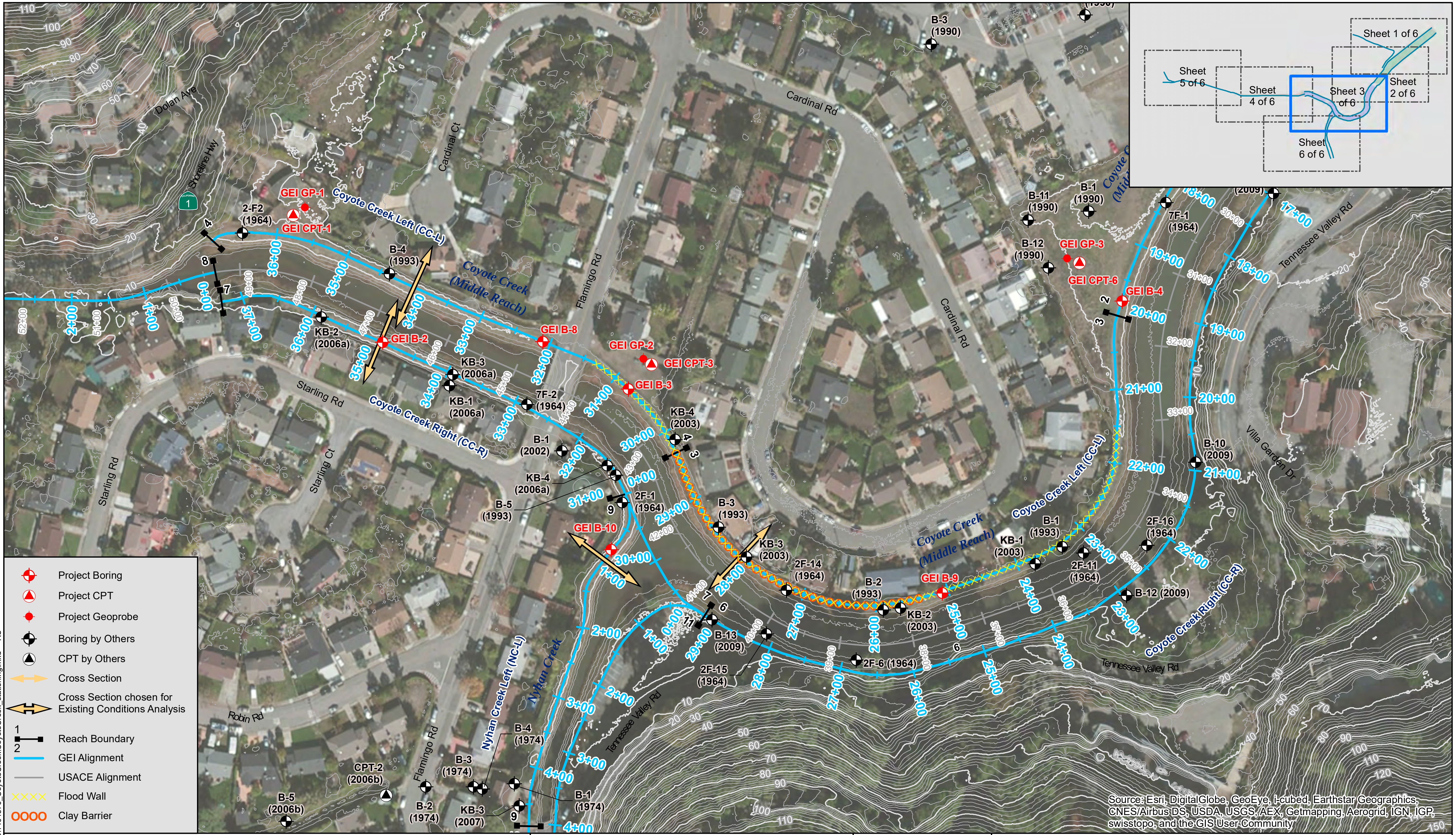


200 100 0 200

Feet

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

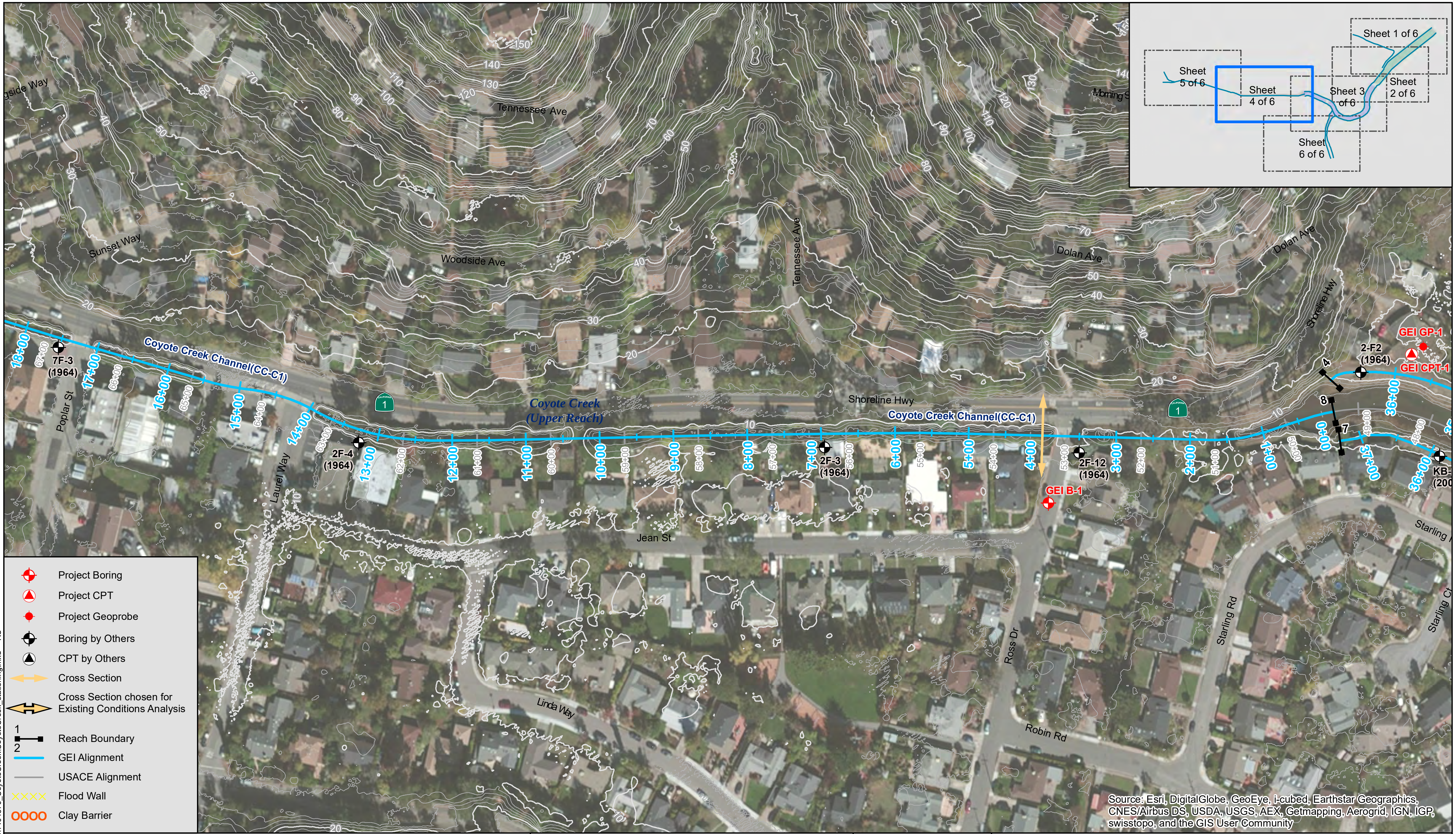
GEI Consultants

SITE CHARACTERIZATION PLAN

MAY 2015

FIGURE 3

26-May-2015 Z:\Projects\1404570_CoyoteCreek\CoyoteCreek_stationing.mxd RS



200 100 0 200

Feet

N E S W

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

SITE CHARACTERIZATION PLAN

MAY 2015

FIGURE 4

26-May-2015 Z:\Projects\1404570_CoyoteCreek\CoyoteCreek_stationing.mxd RS

200 100 0 200

Feet

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

SITE CHARACTERIZATION PLAN

MAY 2015

FIGURE 5

200 100 0 200

Feet

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

SITE CHARACTERIZATION PLAN

MAY 2015

FIGURE 6

26-May-2015 Z:\Projects\1404570_CoyoteCreek\CoyoteCreek_stationing.mxd RS

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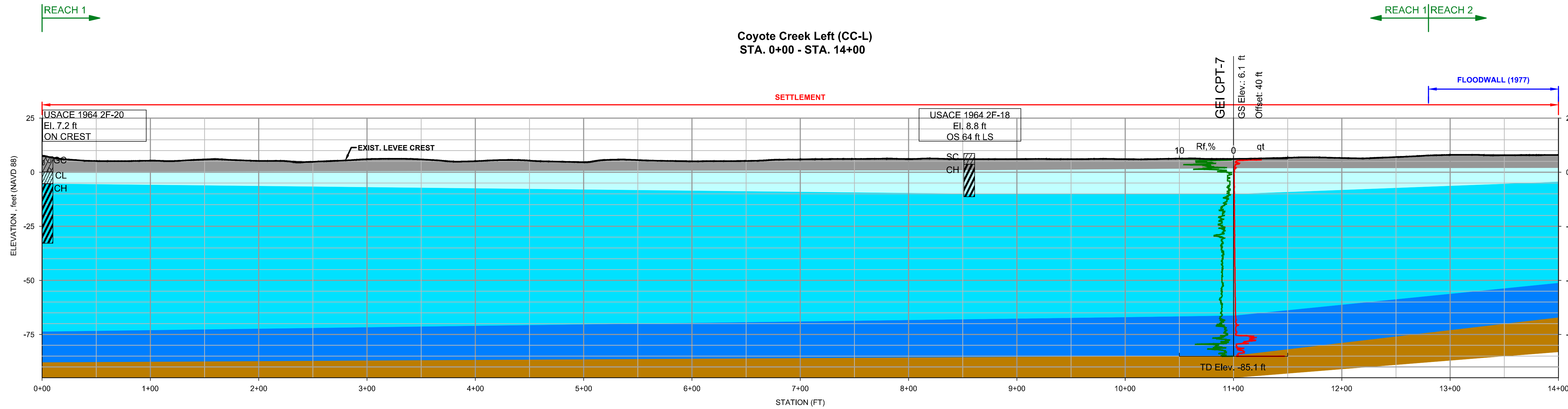
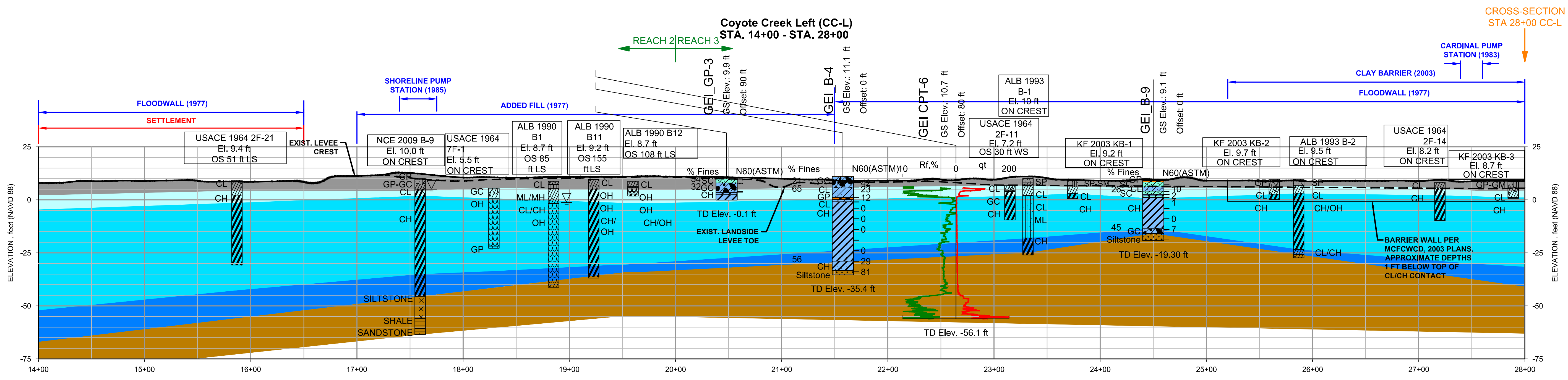




50
25
0
VERT. SCALE (FEET)

0 50 100 200
HORIZ. SCALE (FEET)

Vertical Exaggeration = 2x

LEGEND

- FILL MATERIAL
- CL/SC MATERIAL
- YOUNGER BAY MUD
- OLDER BAY MUD / ALLUVIUM
- BEDROCK
- SAND LENSE

NOTES

ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE

TD = TOTAL DEPTH
OS = OFFSET
LS = LANDSIDE
WS = WATERSIDE
GS = GROUND SURFACE
El. = ELEVATION

EXISTING LEVEE CREST AND TOE PROFILES BASED ON 2013 MERIDIAN CHANNEL SURVEY, USGS 2011 GEOTIFF, AND USACE RECORD DRAWING (1965)

Coyote Creek Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

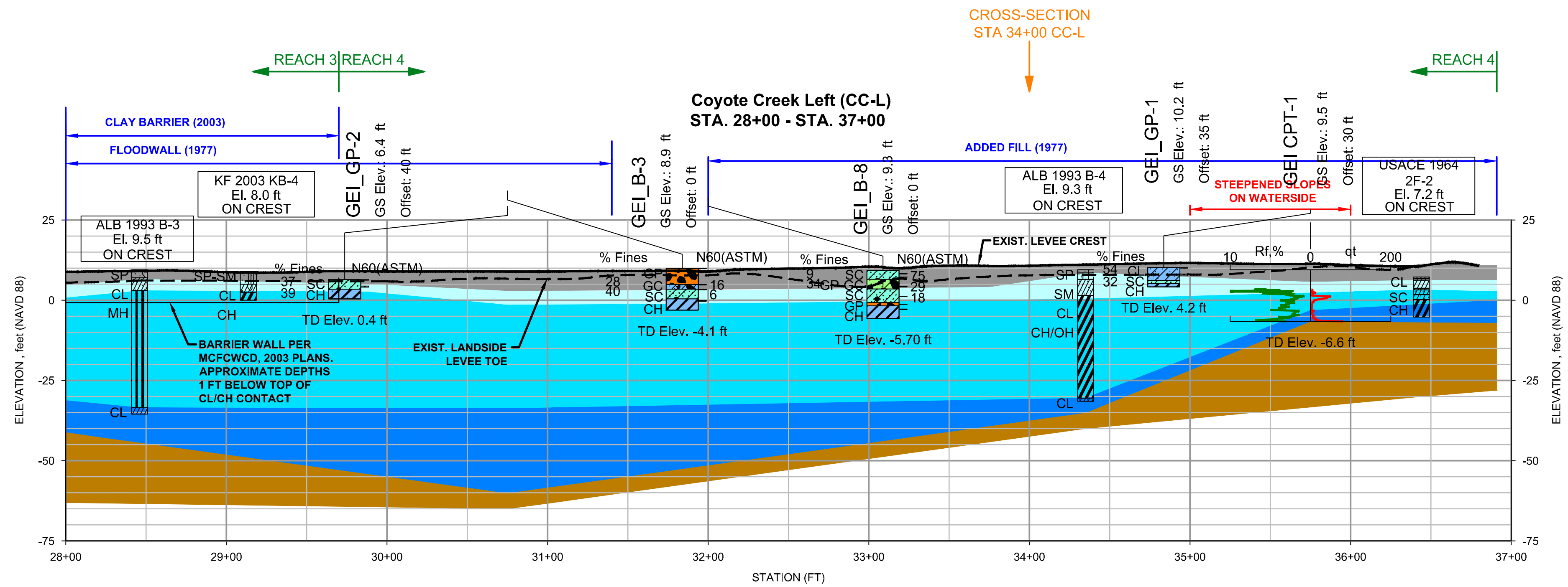
GEI Consultants

Project 140457-0

COYOTE CREEK LEFT PROFILES
(SHEET 1 OF 2)

April 2015

Figure 7



J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Profile Views\ ProfileViews_GEI Explorations_150526.dwg May 2015

VERT. SCALE (FEET): 0, 25, 50

HORIZ. SCALE (FEET): 0, 50, 100, 200

Vertical Exaggeration = 2x

LEGEND

- FILL MATERIAL
- CL/SC MATERIAL
- YOUNGER BAY MUD
- OLDER BAY MUD / ALLUVIUM
- BEDROCK
- SAND LENSE

NOTES

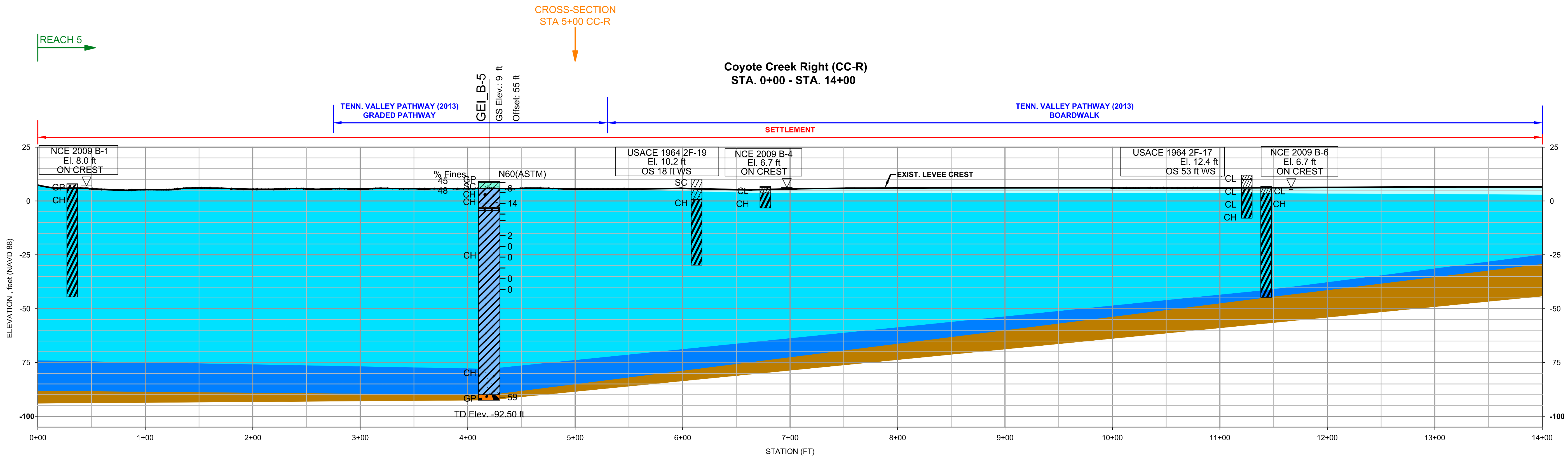
- ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
- TD = TOTAL DEPTH
- OS = OFFSET
- LS = LANDSIDE
- WS = WATERSIDE
- GS = GROUND SURFACE
- El. = ELEVATION

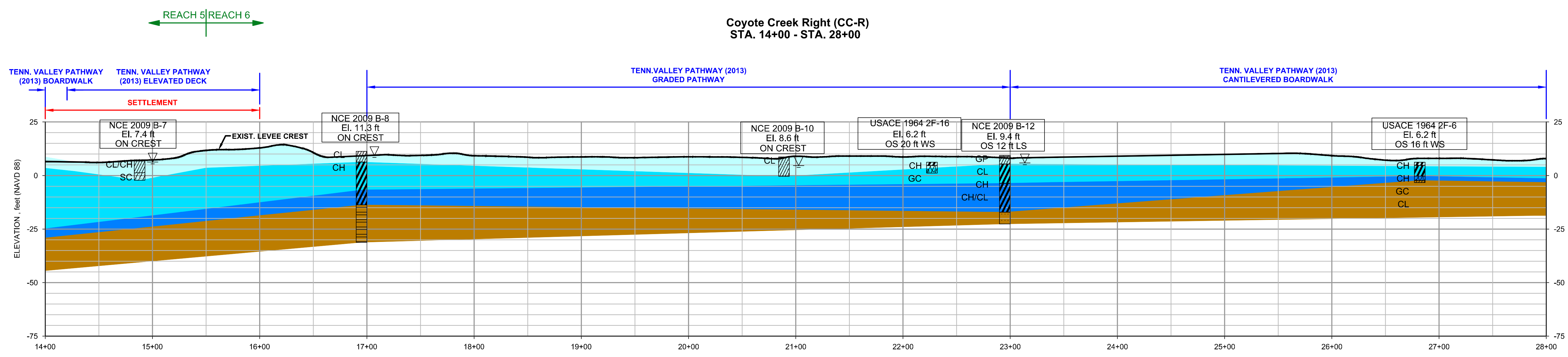
EXISTING LEVEE CREST AND TOE PROFILES BASED ON 2013 MERIDIAN CHANNEL SURVEY, USGS 2011 GEOTIFF, AND USACE RECORD DRAWING (1965)

*MINOR AMOUNTS OF FILL ADDED NEAR FLAMINGO ROAD BRIDGE CC-R AND CC-L (2013)

Coyote Creek Evaluation Project Marin County, California	GEI Consultants	**COYOTE CREEK LEFT PROFILES** **(SHEET 2 OF 2)**
Marin County Flood Control and Water Conservation District	Project 140457-0	April 2015 **Figure 8**

J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Profile Views\ Profile Views_GEI Explorations_150526.dwg May 2015





VERT. SCALE (FEET) 0 25 50

0 50 100 200

HORIZ. SCALE (FEET)

Vertical Exaggeration = 2x

LEGEND

- FILL MATERIAL
- CL/SC MATERIAL
- YOUNGER BAY MUD
- OLDER BAY MUD / ALLUVIUM
- BEDROCK
- SAND LENSE

NOTES

ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE

TD = TOTAL DEPTH
OS = OFFSET
LS = LANDSIDE
WS = WATERSIDE
GS = GROUND SURFACE
El. = ELEVATION

EXISTING LEVEE CREST AND TOE PROFILES BASED ON 2013 MERIDIAN CHANNEL SURVEY, USGS 2011 GEOTIFF, AND USACE RECORD DRAWING (1965)

Coyote Creek Evaluation Project
Marin County, California

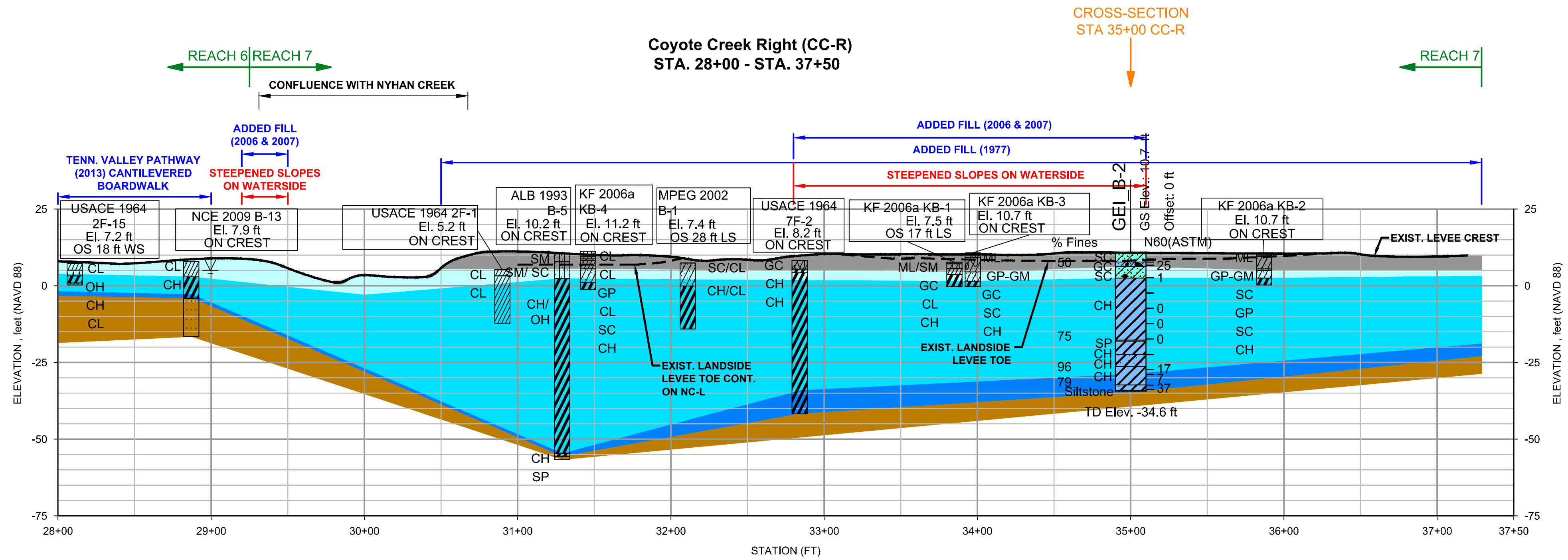
Marin County Flood Control and Water Conservation District

GEI Consultants

COYOTE CREEK RIGHT PROFILES
(SHEET 1 OF 2)

Project 140457-0 | April 2015 | **Figure 9**

J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Profile Views\ ProfileViews_GEI Explorations_150526.dwg May 2015



LEGEND

- FILL MATERIAL
- CL/SC MATERIAL
- YOUNGER BAY MUD
- OLDER BAY MUD / ALLUVIUM
- BEDROCK
- SAND LENSE

NOTES

ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE

TD = TOTAL DEPTH
OS = OFFSET
LS = LANDSIDE
WS = WATERSIDE
GS = GROUND SURFACE
El. = ELEVATION

EXISTING LEVEE CREST AND TOE PROFILES BASED ON 2013 MERIDIAN CHANNEL SURVEY, USGS 2011 GEOTIFF, AND USACE RECORD DRAWING (1965)

*MINOR AMOUNTS OF FILL ADDED NEAR FLAMINGO ROAD BRIDGE CC-R AND CC-L (2013)

VERT. SCALE (FEET): 0 25 50

HORIZ. SCALE (FEET): 0 50 100 200

Vertical Exaggeration = 2x

Coyote Creek Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

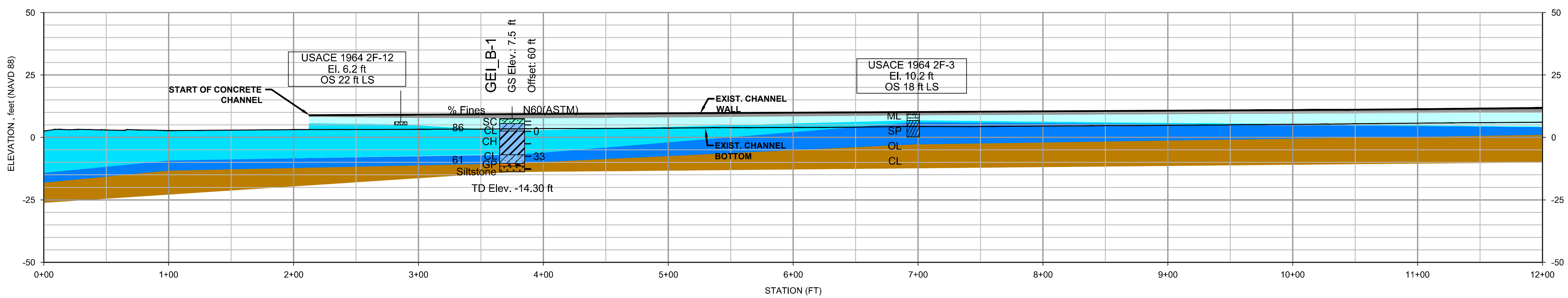
COYOTE CREEK RIGHT PROFILES
(SHEET 2 OF 2)

Project 140457-0 | April 2015 | **Figure 10**

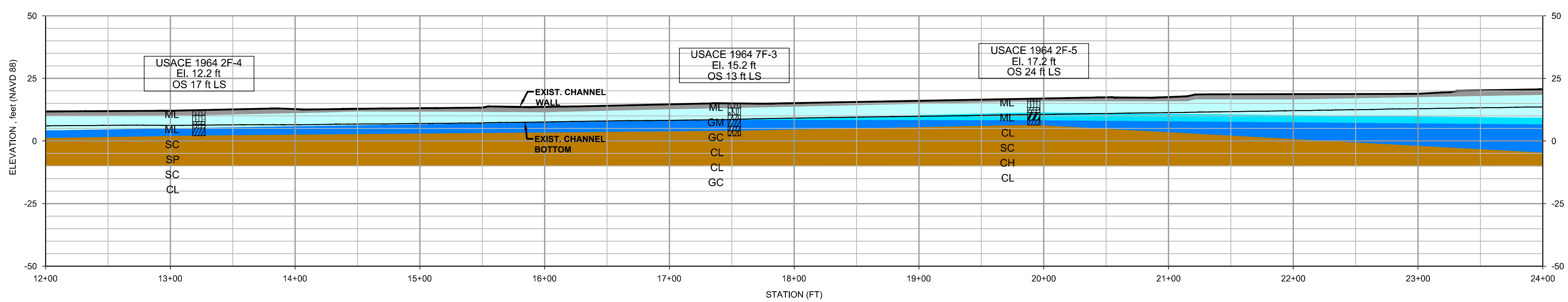
REACH 8

CROSS-SECTION
STA 4+00 CC-C

Coyote Creek Channel (CC-C)
STA. 0+00 - STA. 12+00



Coyote Creek Channel (CC-C)
STA. 12+00 - STA. 24+00



50
25
0
VERT. SCALE (FEET)

0 50 100 200
HORIZ. SCALE (FEET)

Vertical Exaggeration = 2x

LEGEND

- FILL MATERIAL
- CL/SC MATERIAL
- YOUNGER BAY MUD
- OLDER BAY MUD / ALLUVIUM
- BEDROCK
- SAND LENSE

NOTES

ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE

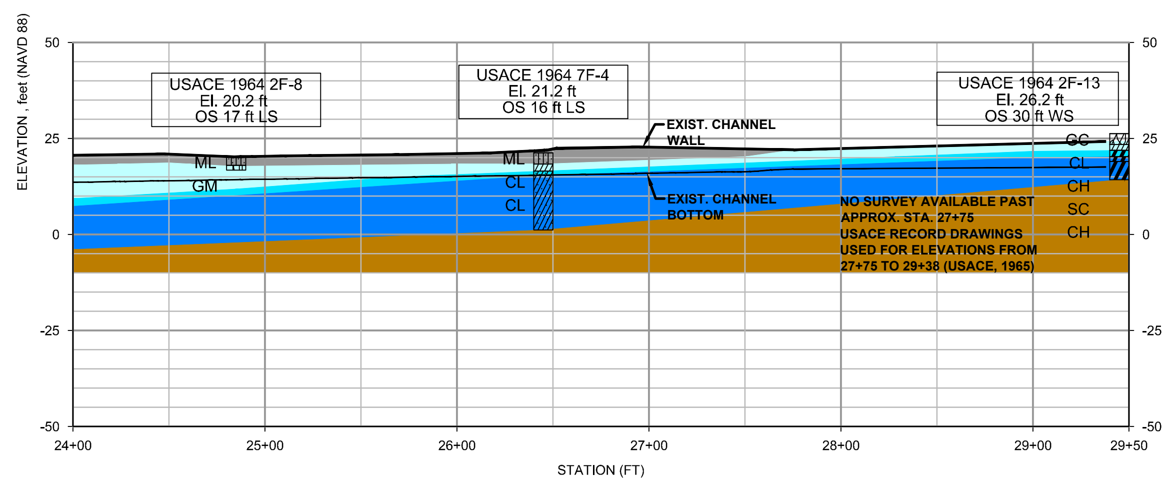
TD = TOTAL DEPTH
OS = OFFSET
LS = LANDSIDE
WS = WATERSIDE
GS = GROUND SURFACE
El. = ELEVATION

EXISTING LEVEE CREST AND TOE PROFILES BASED ON 2013 MERIDIAN CHANNEL SURVEY, USGS 2011 GEOTIFF, AND USACE RECORD DRAWING (1965)

Coyote Creek Evaluation Project Marin County, California	GEI Consultants	**COYOTE CREEK CHANNEL PROFILES** **(SHEET 1 OF 2)**
Marin County Flood Control and Water Conservation District	Project 140457-0	April 2015 **Figure 11**

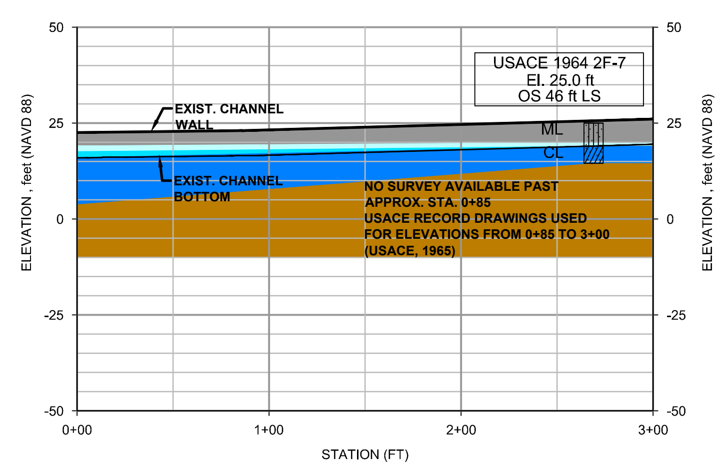
J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Profile Views\ ProfileViews_GEI Explorations_150526.dwg May 2015

Coyote Creek Channel (CC-C)
STA. 24+00 - STA. 29+50



REACH 8

Coyote Creek Channel (CC-C2)
STA. 0+00 - STA. 3+00



50
25
0
VERT. SCALE (FEET)

0 50 100 200
HORIZ. SCALE (FEET)
Vertical Exaggeration = 2x

LEGEND

- FILL MATERIAL
- CL/SC MATERIAL
- YOUNGER BAY MUD
- OLDER BAY MUD / ALLUVIUM
- BEDROCK
- SAND LENSE

NOTES

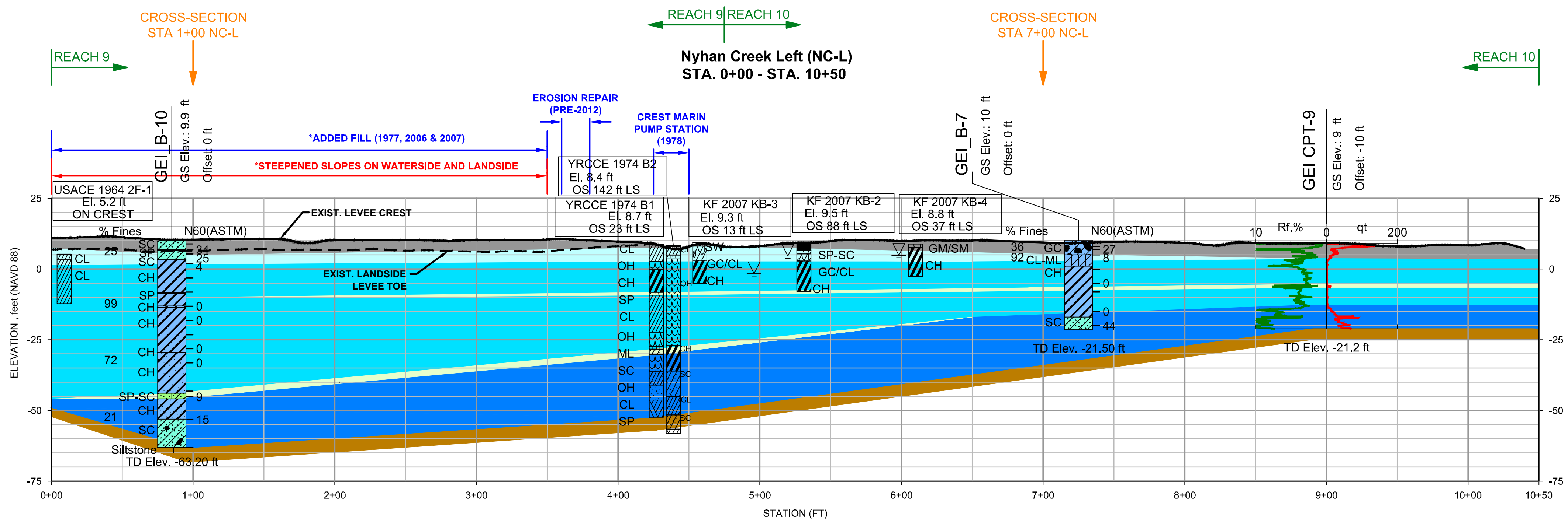
ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE

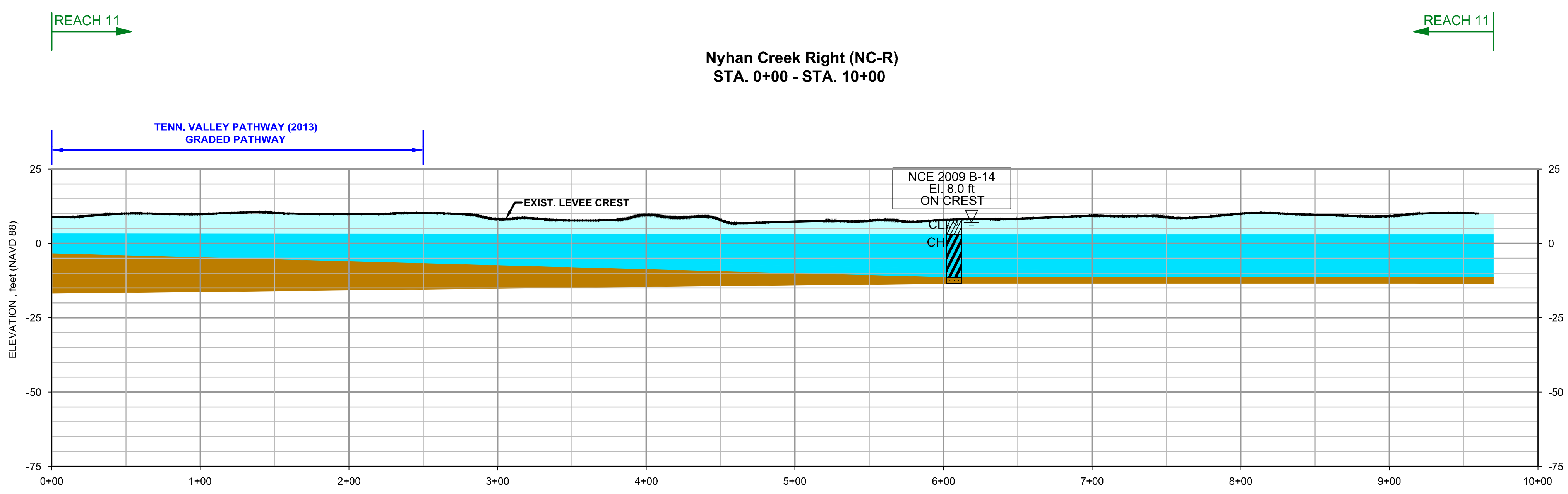
TD = TOTAL DEPTH
OS = OFFSET
LS = LANDSIDE
WS = WATERSIDE
GS = GROUND SURFACE
El. = ELEVATION

EXISTING LEVEE CREST AND TOE PROFILES BASED ON 2013 MERIDIAN CHANNEL SURVEY, USGS 2011 GEOTIFF, AND USACE RECORD DRAWING (1965)

Coyote Creek Evaluation Project Marin County, California	GEI Consultants	**COYOTE CREEK CHANNEL PROFILES** **(SHEET 2 OF 2)**
Marin County Flood Control and Water Conservation District	Project 140457-0	April 2015 **Figure 12**

J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Profile Views\ ProfileViews_GEI Explorations_150526.dwg May 2015





VERT. SCALE (FEET): 0, 25, 50

HORIZ. SCALE (FEET): 0, 50, 100, 200

Vertical Exaggeration = 2x

LEGEND

- FILL MATERIAL
- CL/SC MATERIAL
- YOUNGER BAY MUD
- OLDER BAY MUD / ALLUVIUM
- BEDROCK
- SAND LENSE

NOTES

- ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
- TD = TOTAL DEPTH
- OS = OFFSET
- LS = LANDSIDE
- WS = WATERSIDE
- GS = GROUND SURFACE
- El. = ELEVATION

EXISTING LEVEE CREST AND TOE PROFILES BASED ON 2013 MERIDIAN CHANNEL SURVEY, USGS 2011 GEOTIFF, AND USACE RECORD DRAWING (1965)

*APPROXIMATE LENGTH OF ADDED FILL AND STEEPENED SLOPES

Coyote Creek Evaluation Project
Marin County, California

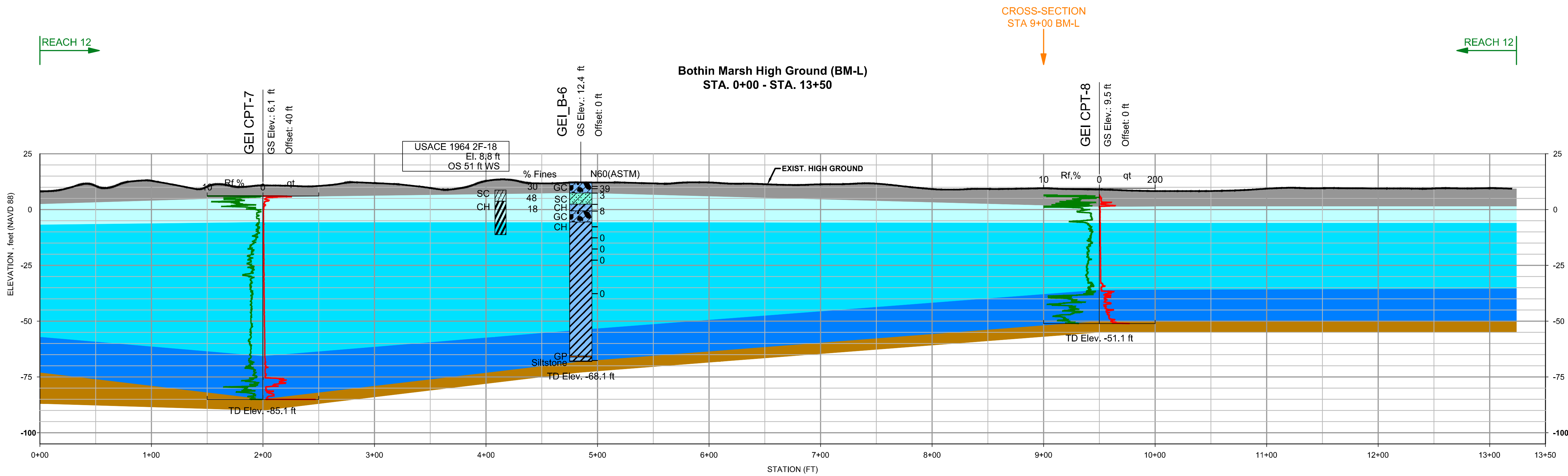
Marin County Flood Control and Water Conservation District

GEI Consultants

NYHAN CREEK LEFT AND NYHAN CREEK RIGHT PROFILES
(SHEET 1 OF 1)

Project 140457-0 | April 2015 | **Figure 13**

J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Profile Views\ ProfileViews_GEI Explorations_150526.dwg May 2015



50
25
0
VERT. SCALE (FEET)

0 50 100 200
HORIZ. SCALE (FEET)

Vertical Exaggeration = 2x

LEGEND

- FILL MATERIAL
- CL/SC MATERIAL
- YOUNGER BAY MUD
- OLDER BAY MUD / ALLUVIUM
- BEDROCK
- SAND LENSE

NOTES

ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE

TD = TOTAL DEPTH
OS = OFFSET
LS = LANDSIDE
WS = WATERSIDE
GS = GROUND SURFACE
El. = ELEVATION

EXISTING LEVEE CREST AND TOE PROFILES BASED ON 2013 MERIDIAN CHANNEL SURVEY, USGS 2011 GEOTIFF, AND USACE RECORD DRAWING (1965)

Coyote Creek Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

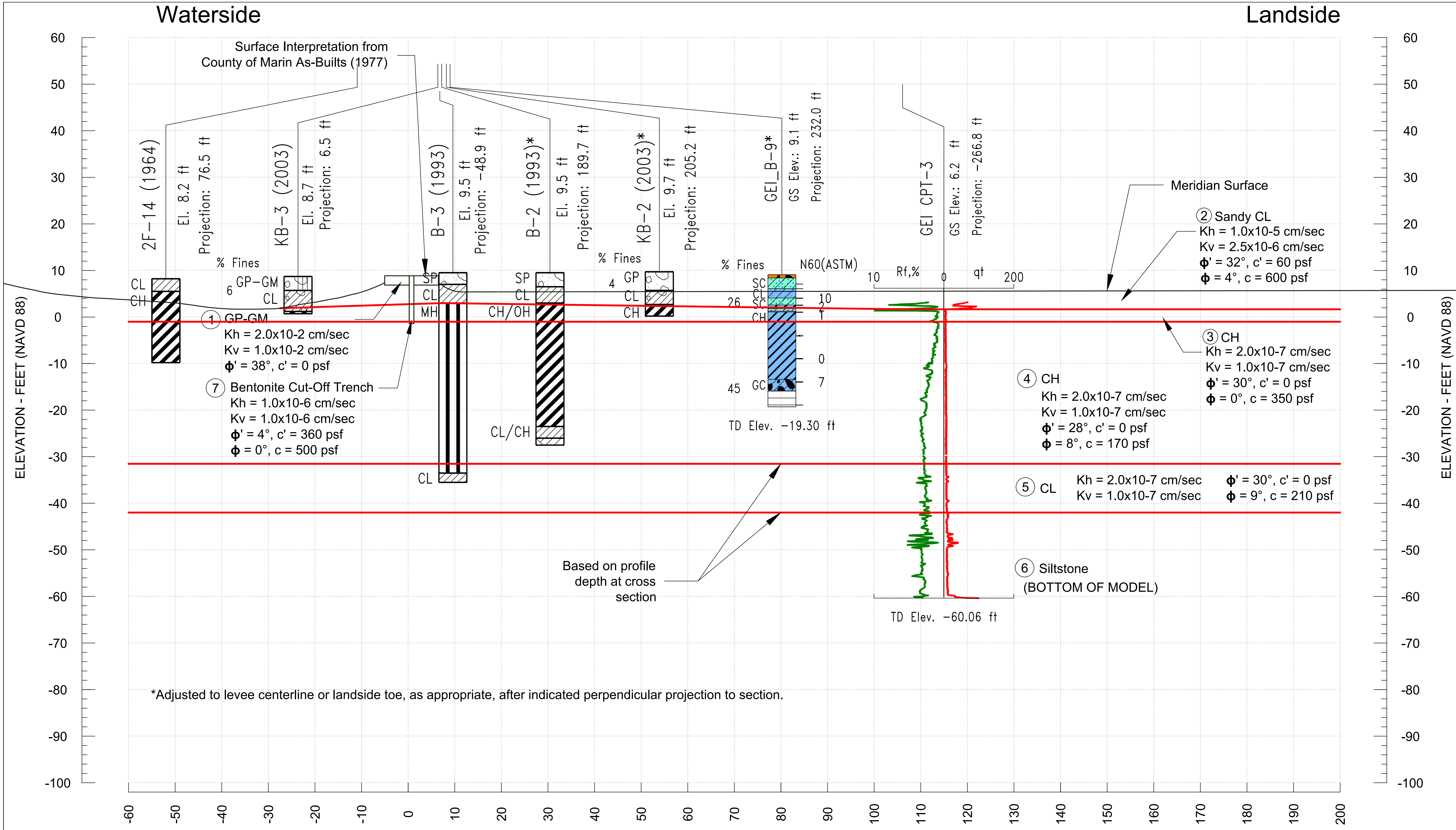
GEI Consultants

Project 140457-0

BOTHIN MARSH HIGH GROUND PROFILES
(SHEET 1 OF 1)

April 2015

Figure 14



0 10 20 40 60
SCALE IN FEET

LEGEND
INTERPRETED STRATIGRAPHY

NOTES
ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
TD = TOTAL DEPTH
GS = GROUND SURFACE
El. = ELEVATION

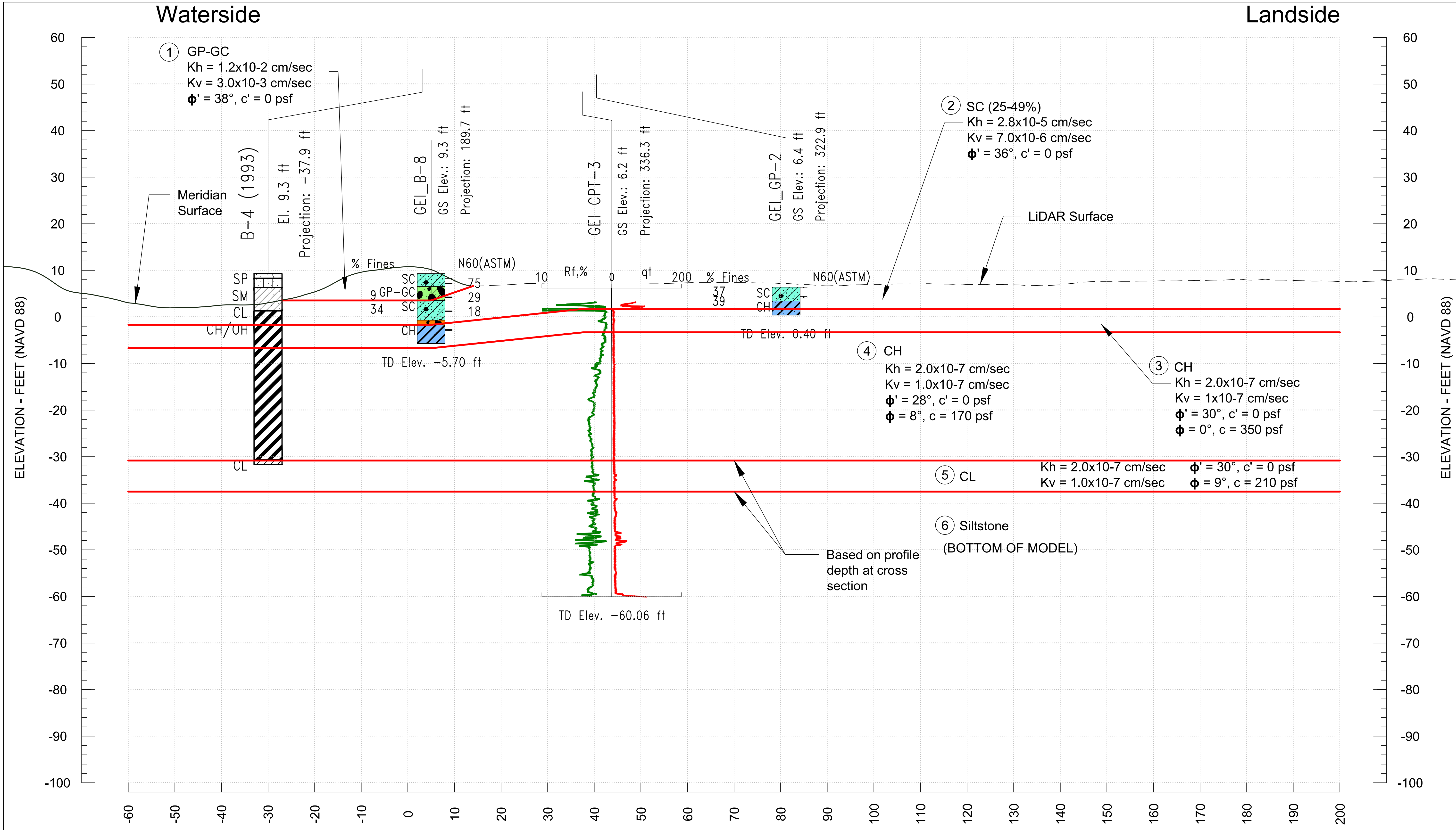
Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

COYOTE CREEK LEFT
REACH 3 STATION 28+00 CC-L

Project 140457-0
August 2015
Figure 15



0 10 20 40 60
SCALE IN FEET

LEGEND
INTERPRETED STRATIGRAPHY

NOTES
ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
TD = TOTAL DEPTH
GS = GROUND SURFACE
El. = ELEVATION

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

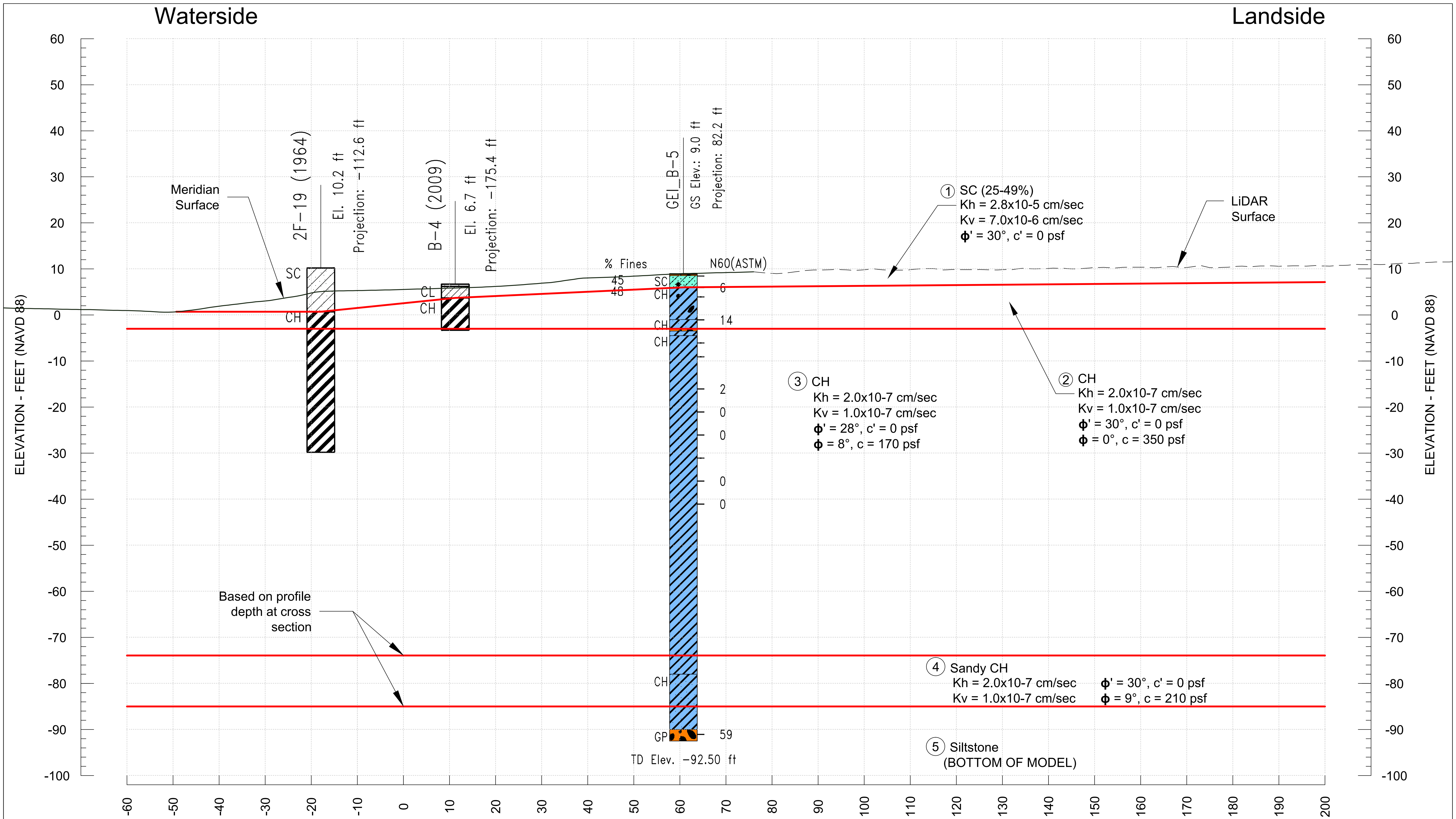
GEI Consultants

Project 140457-0

COYOTE CREEK LEFT REACH 4 STATION 34+00 CC-L

August 2015

Figure 16



SCALE IN FEET (0, 10, 20, 40, 60)

LEGEND

INTERPRETED STRATIGRAPHY

NOTES

ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE

TD = TOTAL DEPTH

GS = GROUND SURFACE

El. = ELEVATION

Coyote Creek Levee Evaluation Project
Marin County, California

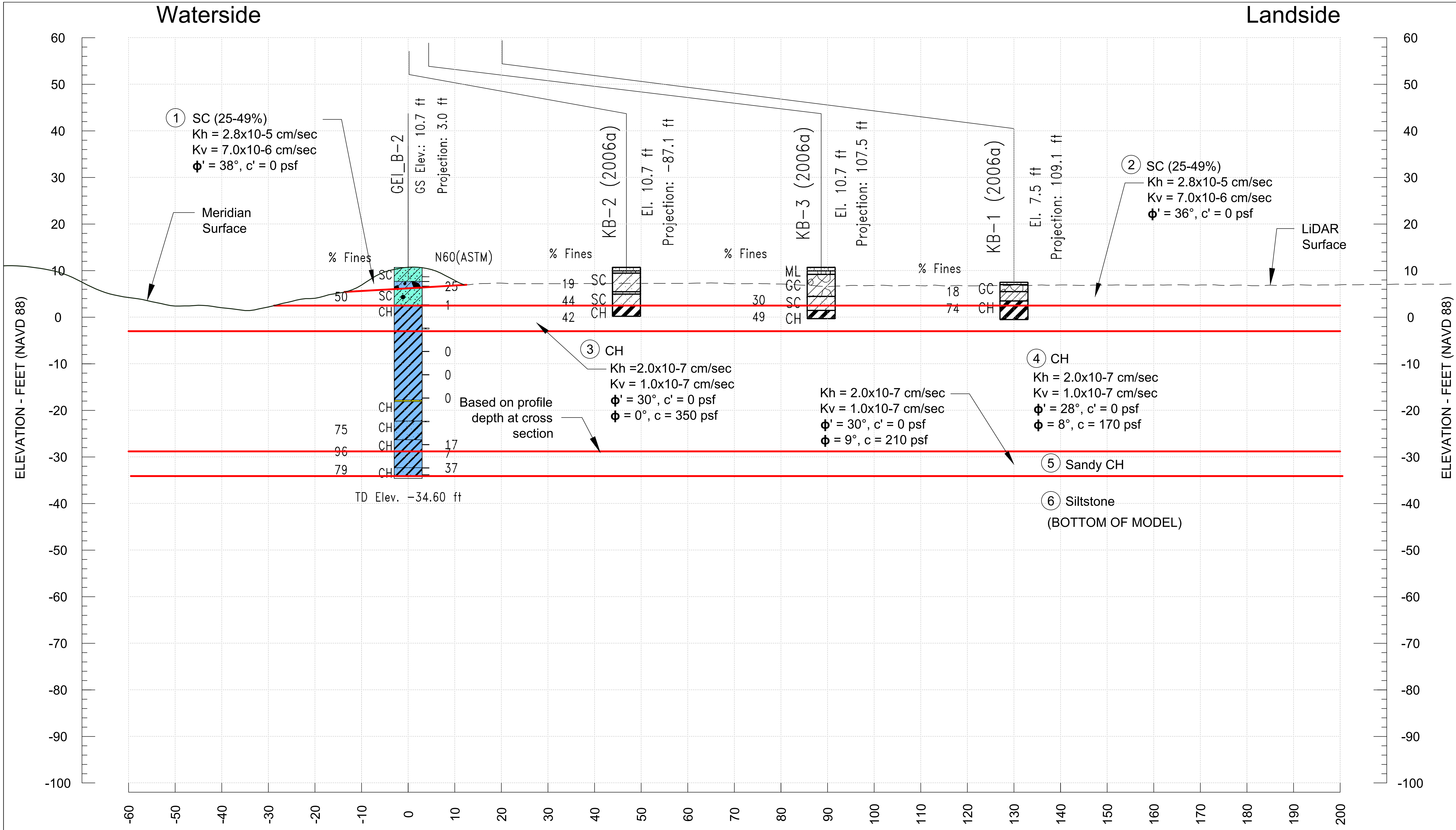
Marin County Flood Control and Water Conservation District

GEI Consultants

COYOTE CREEK RIGHT REACH 5 STATION 5+00 CC-R

Project 140457-0 | August 2015 | **Figure 17**

J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Cross Sections\ Coyote Creek_Analysis Cross Sections 7-17-15.dwg Jul 2015



J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Cross Sections\ Coyote Creek_Analysis Cross Sections 7-17-15.dwg Jul 2015

0 10 20 40 60
SCALE IN FEET

LEGEND
INTERPRETED STRATIGRAPHY

NOTES
ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
TD = TOTAL DEPTH
GS = GROUND SURFACE
El. = ELEVATION

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

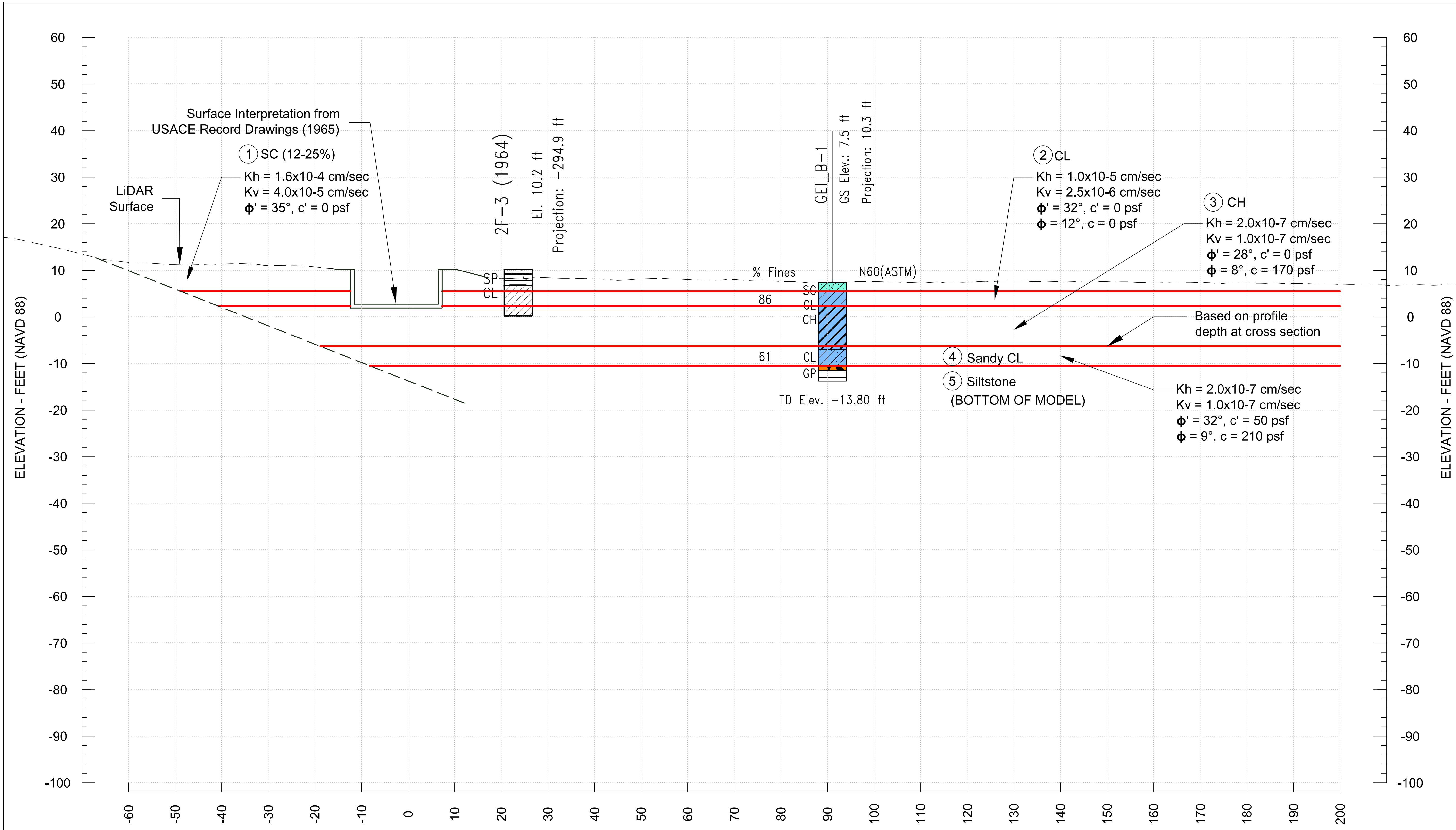
GEI Consultants

Project 140457-0

COYOTE CREEK RIGHT REACH 7 STATION 35+00 CC-R

August 2015

Figure 18



0 10 20 40 60
SCALE IN FEET

LEGEND
INTERPRETED STRATIGRAPHY

NOTES
ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
TD = TOTAL DEPTH
GS = GROUND SURFACE
El. = ELEVATION

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and
Water Conservation District

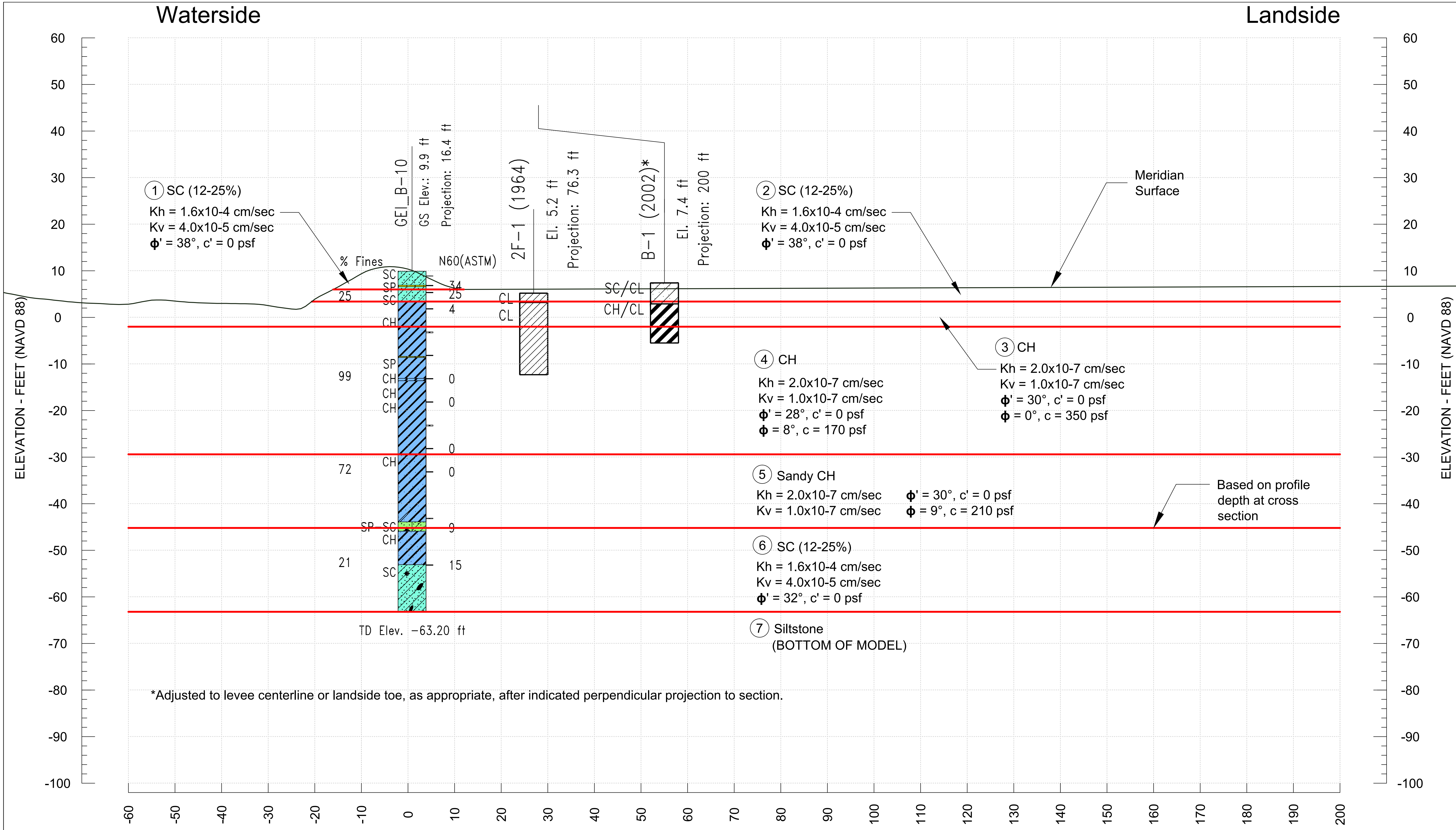
GEI Consultants

Project 140457-0

COYOTE CREEK CHANNEL
REACH 8 STATION 4+00 CC-C

August 2015

Figure 19



0 10 20 40 60
SCALE IN FEET

LEGEND
INTERPRETED STRATIGRAPHY

NOTES
ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
TD = TOTAL DEPTH
GS = GROUND SURFACE
El. = ELEVATION

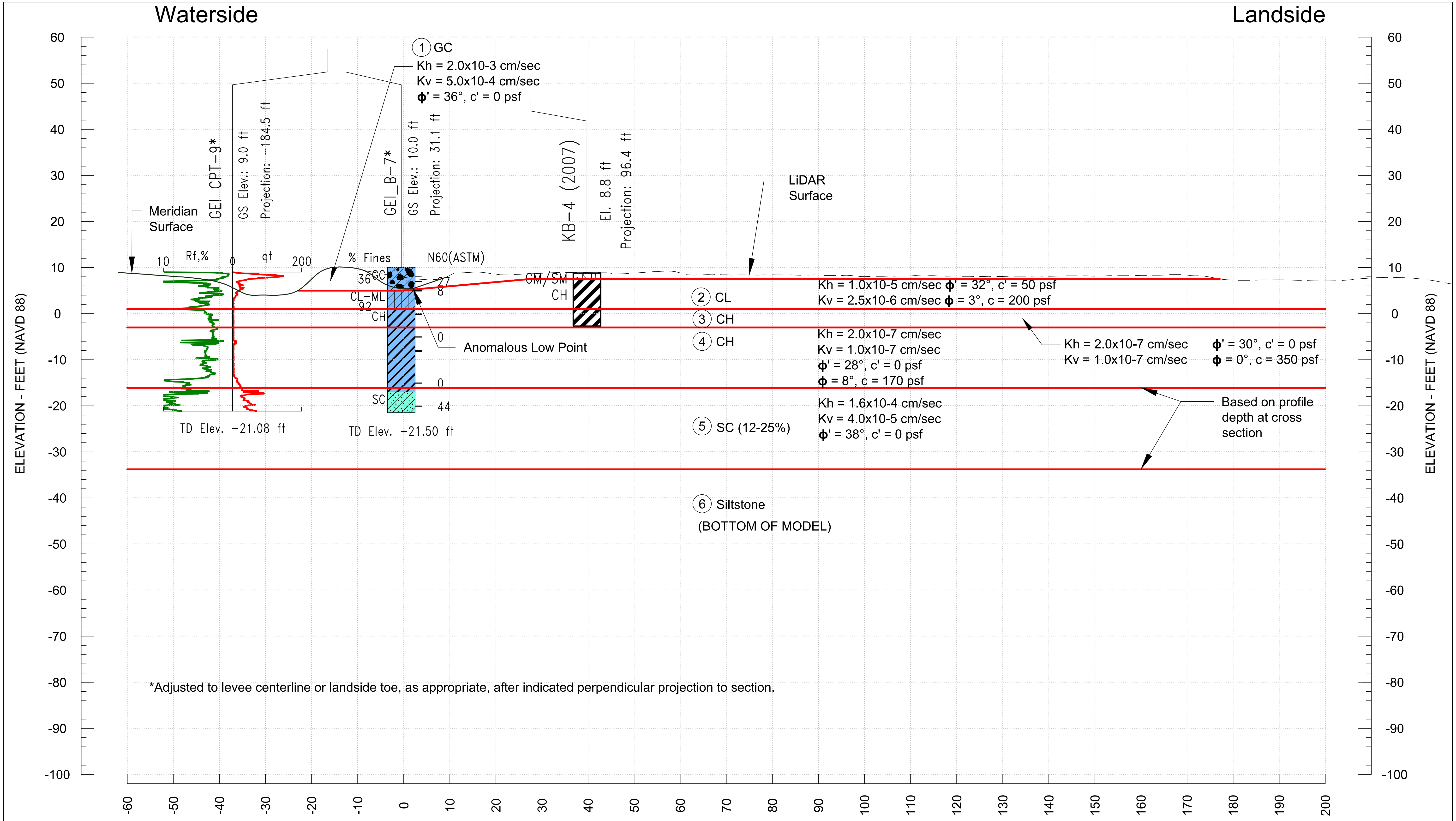
Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

NYHAN CREEK LEFT REACH 9 STATION 1+00 NC-L

Project 140457-0 | August 2015 | **Figure 20**



0 10 20 40 60
SCALE IN FEET

LEGEND
INTERPRETED STRATIGRAPHY

NOTES
ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
TD = TOTAL DEPTH
GS = GROUND SURFACE
El. = ELEVATION

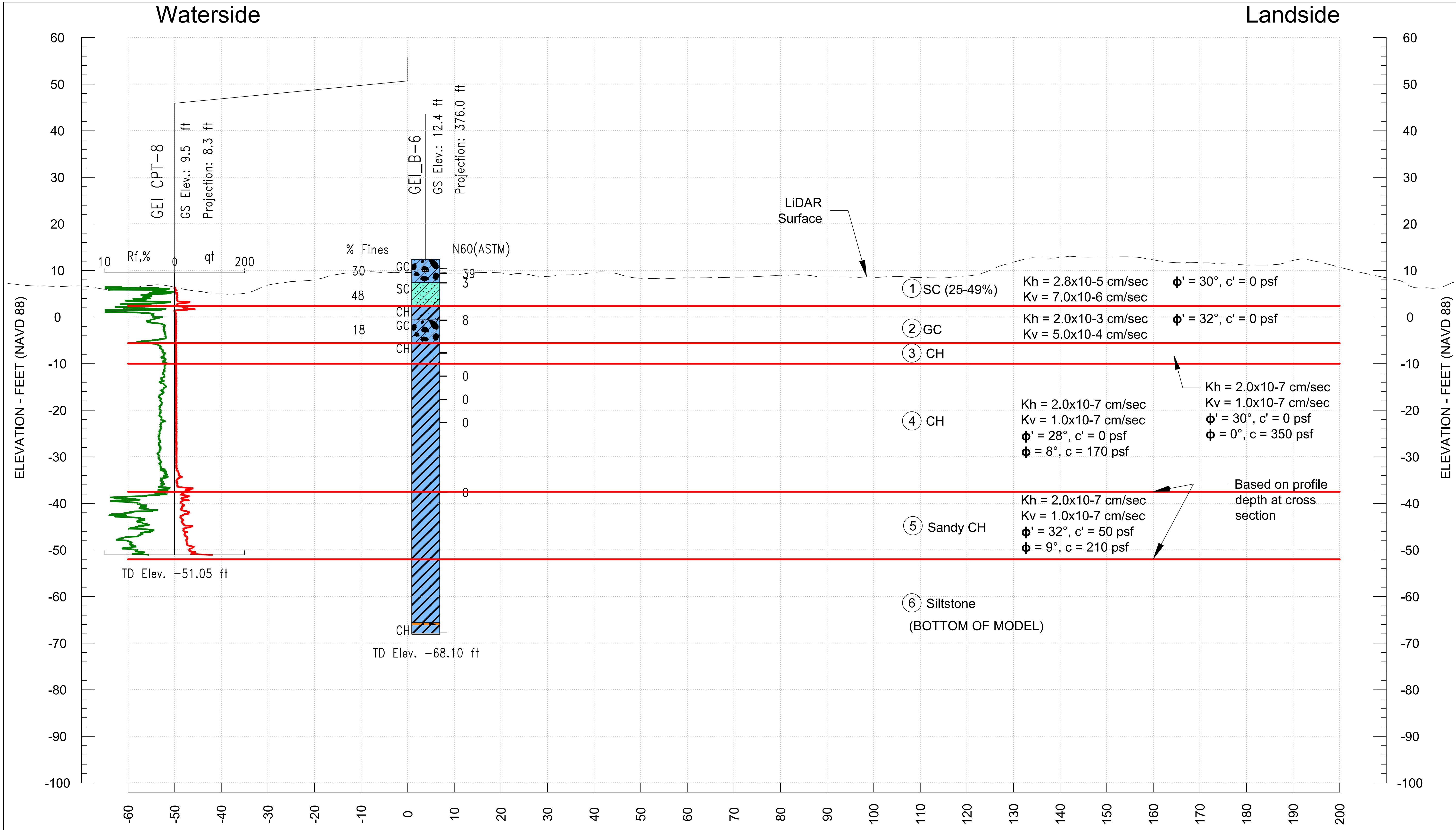
Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

NYHAN CREEK LEFT
REACH 10 STATION 7+00 NC-L

Project 140457-0 | August 2015 | **Figure 21**



J:\Marin County FCD\Projects\1404570_Coyote Creek\6.0 Planning\Task 4 - Analysis\Cross Sections\Coyote Creek_Analysis Cross Sections 7-17-15.dwg Jul 2015

0 10 20 40 60
SCALE IN FEET

LEGEND
INTERPRETED STRATIGRAPHY

NOTES
ACTUAL EXPLORATION LOCATION INDICATED BY OFFSET LINE
TD = TOTAL DEPTH
GS = GROUND SURFACE
El. = ELEVATION

Coyote Creek Levee Evaluation Project
Marin County, California

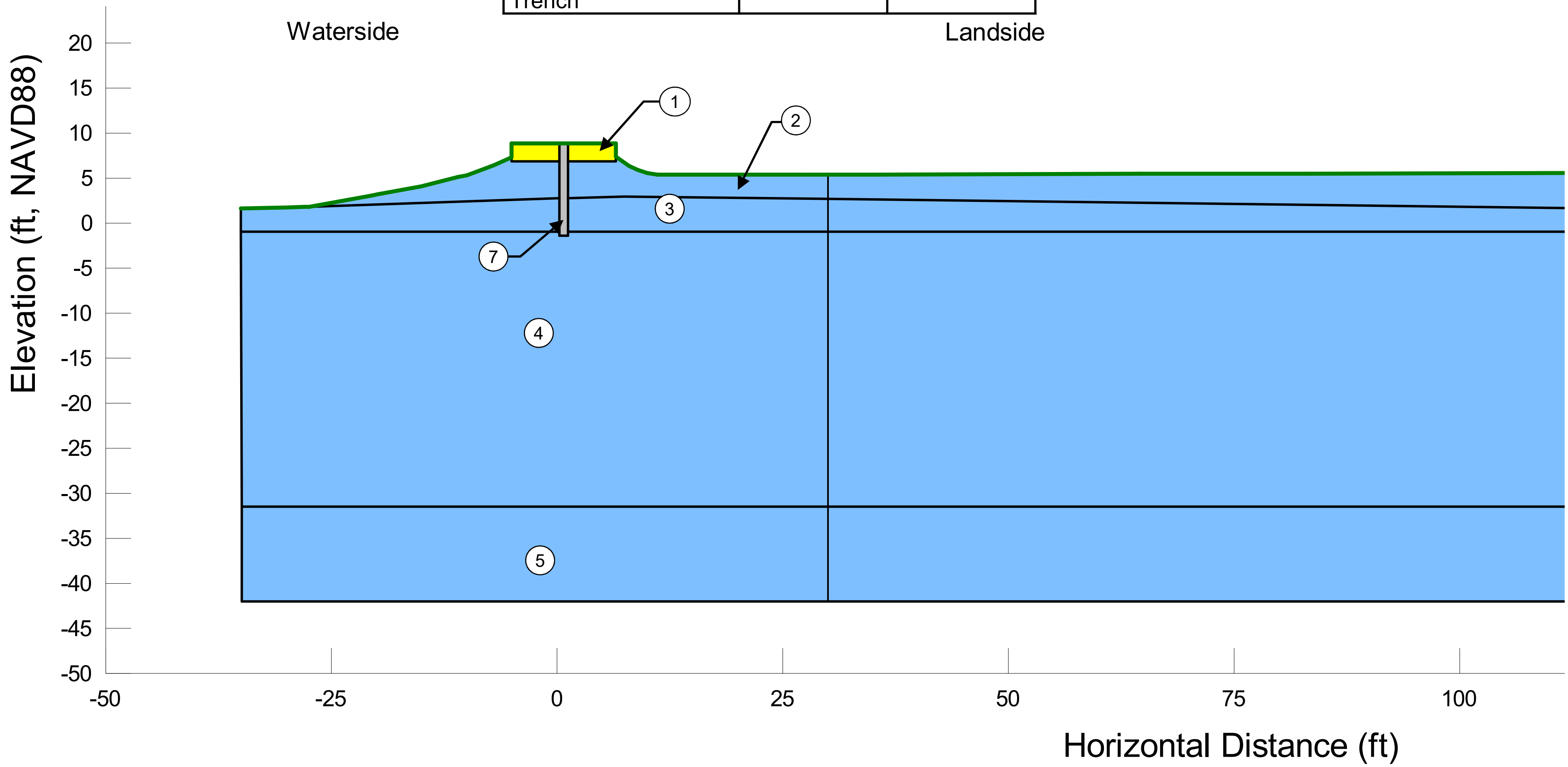
Marin County Flood Control and
Water Conservation District

GEI Consultants

BOTHIN MARSH HIGH GROUND
REACH 12 STATION 9+00 BM-L

Project 140457-0 | August 2015 | **Figure 22**

Layer Name	k_v (cm/sec)	k_h (cm/sec)
(1) GP-GM	1.0×10^{-2}	2.0×10^{-2}
(2) Sandy CL	2.5×10^{-6}	1.0×10^{-5}
(3) CH	1.0×10^{-7}	2.0×10^{-7}
(4) CH	1.0×10^{-7}	2.0×10^{-7}
(5) CL	1.0×10^{-7}	2.0×10^{-7}
(7) Bentonite Cut-Off Trench	1.0×10^{-6}	1.0×10^{-6}



Coyote Creek Levee Evaluation
Marin County

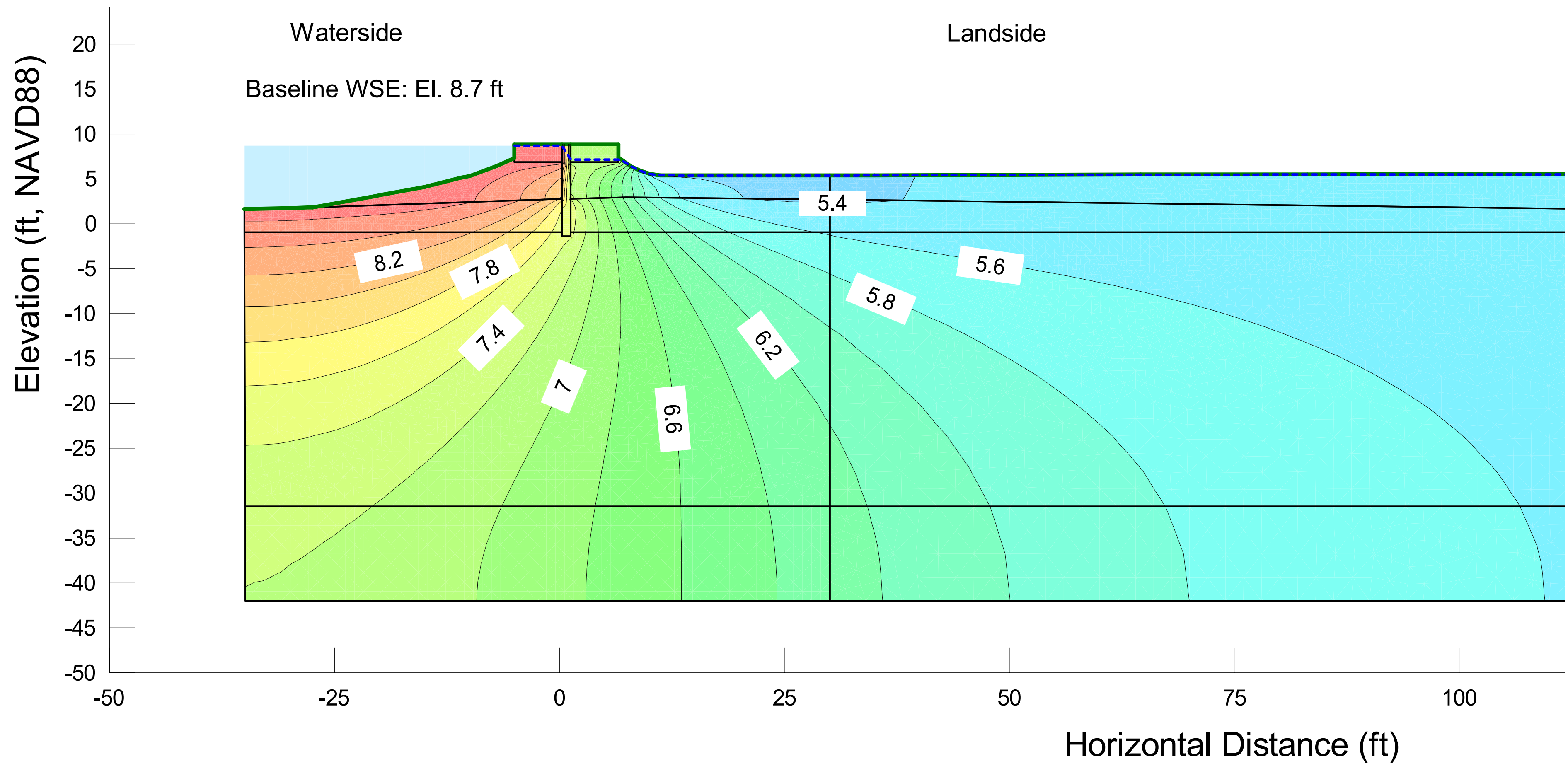
Marin County FCWCD

GEI Consultants

Reach 3 – Sta. 28+00 CC-L

Existing Conditions
Steady State Seepage Model

January 2016 Figure 23



Coyote Creek Levee Evaluation
Marin County

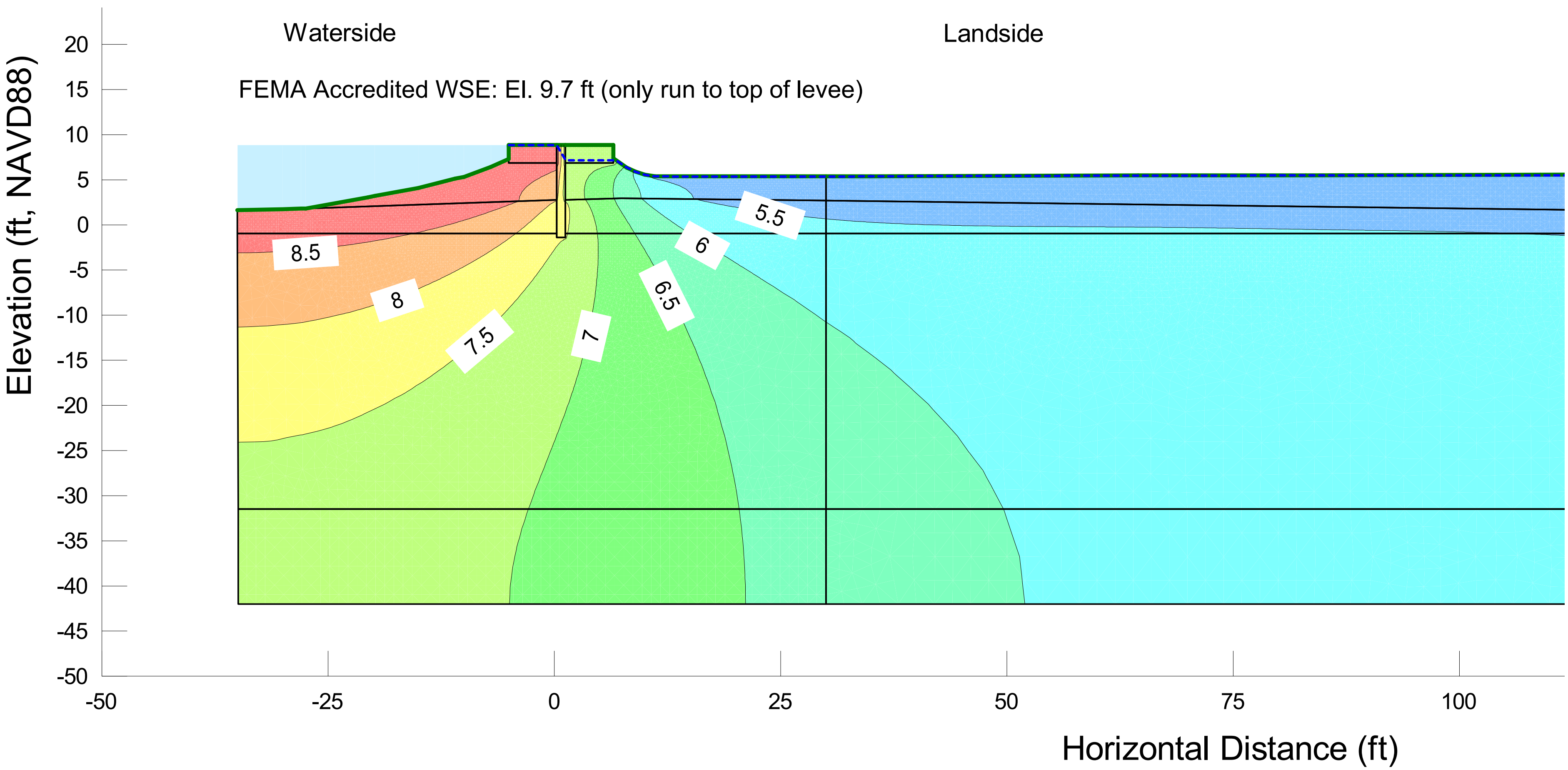
Marin County FCWCD

GEI Consultants

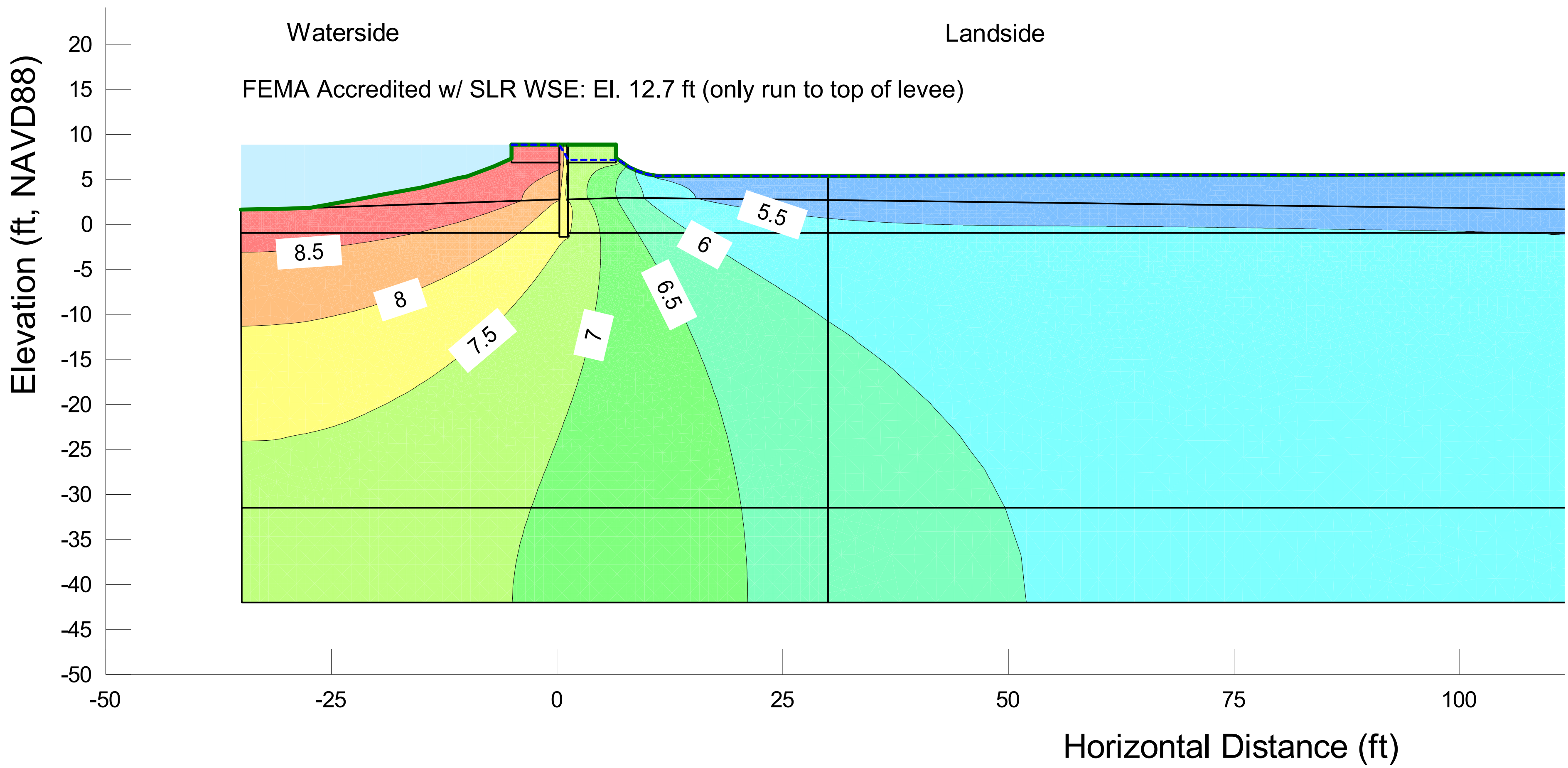
Reach 3 – Sta. 28+00 CC-L

Existing Conditions – Baseline
Steady State Seepage Results

January 2016 Figure 24

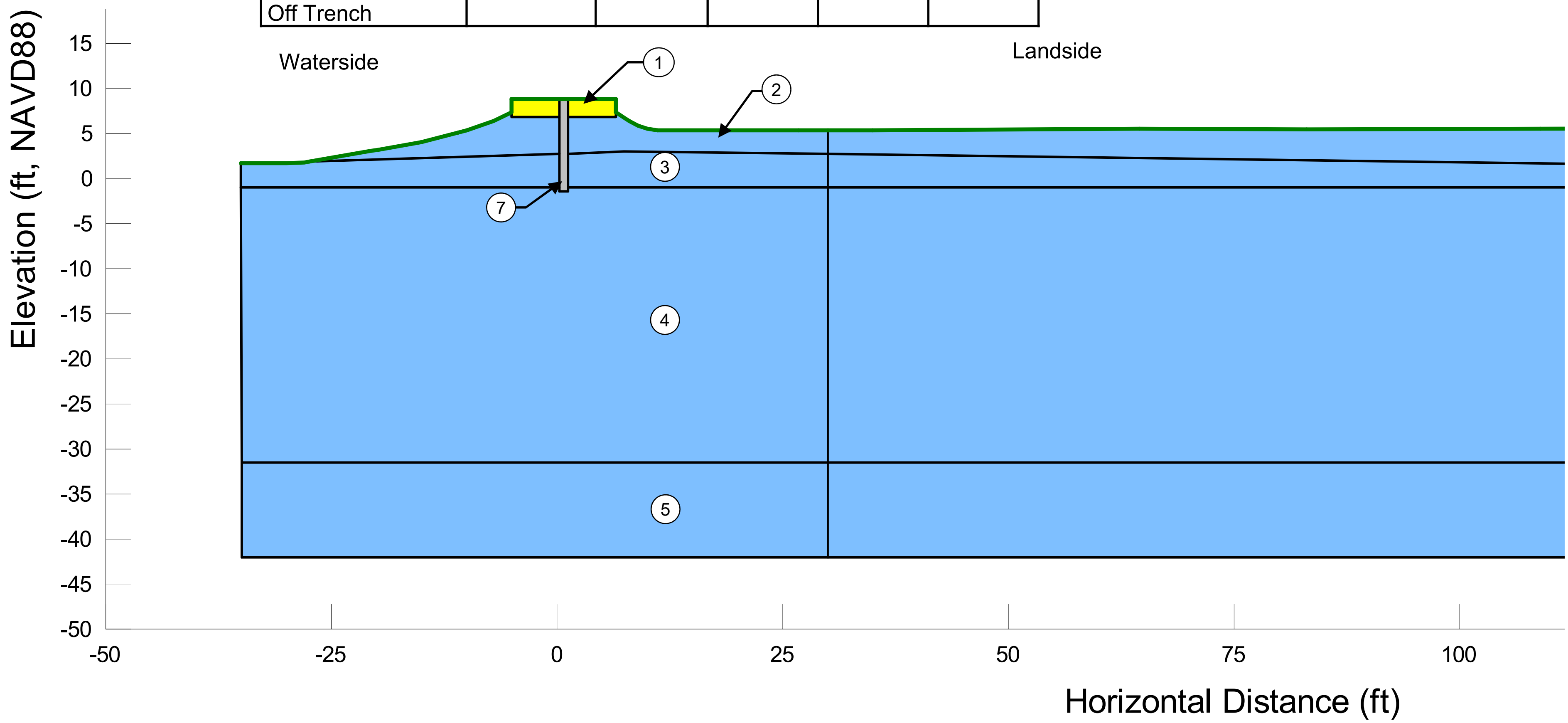


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions – FEMA Steady State Seepage Results
Marin County FCWCD		January 2016 Figure 25

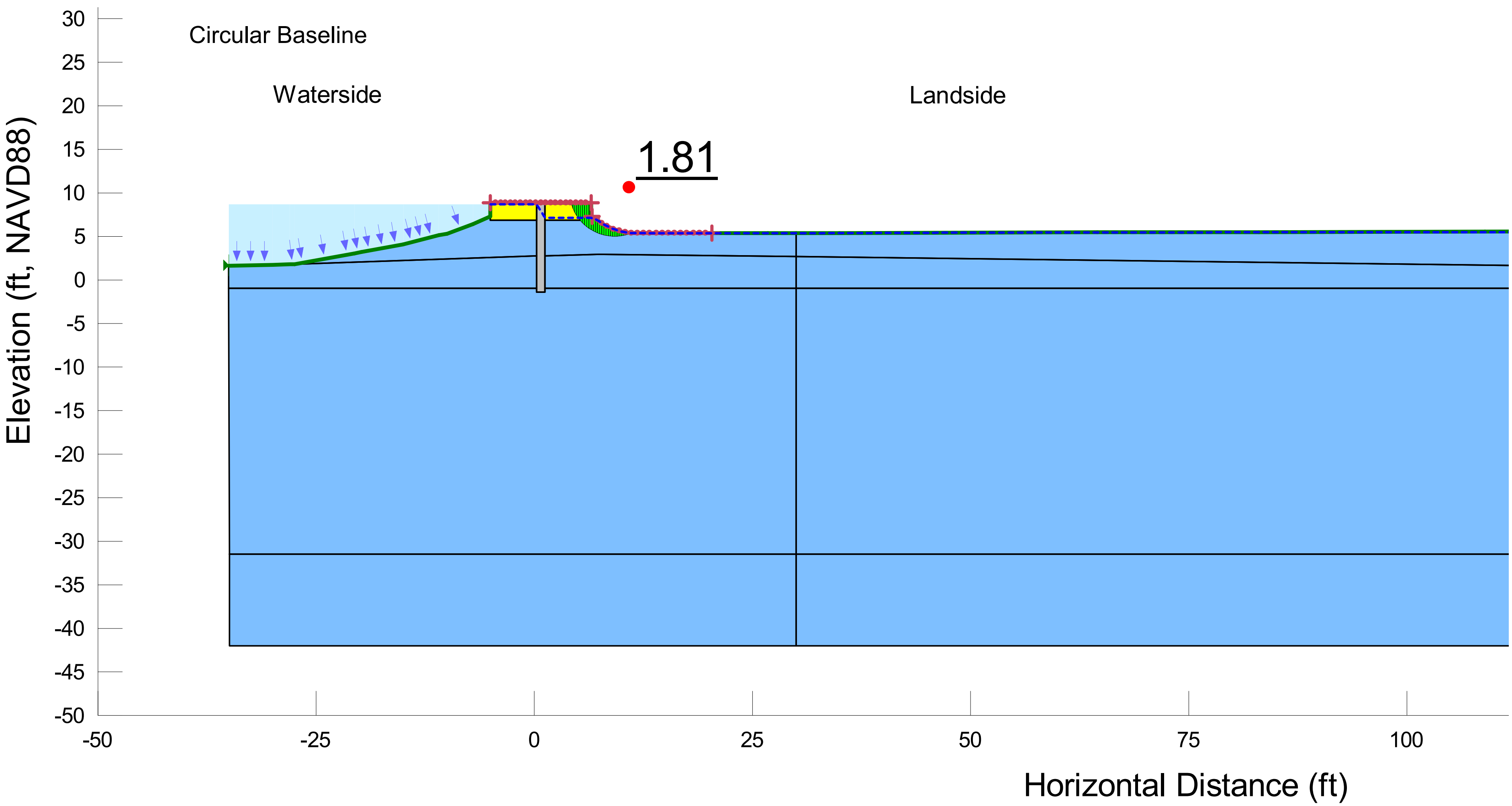


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions–FEMA w/ SLR Steady State Seepage Results
Marin County FCWCD		January 2016 Figure 26

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c'	ϕ'	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
(1) GP-GM	120	0	34	--	--
(2) Sandy CL	120	60	32	600	4
(3) CH	95	0	30	350	0
(4) CH	95	0	28	170	8
(5) CL	95	0	30	210	9
(7) Bentonite Cut-Off Trench	120	360	4	500	0

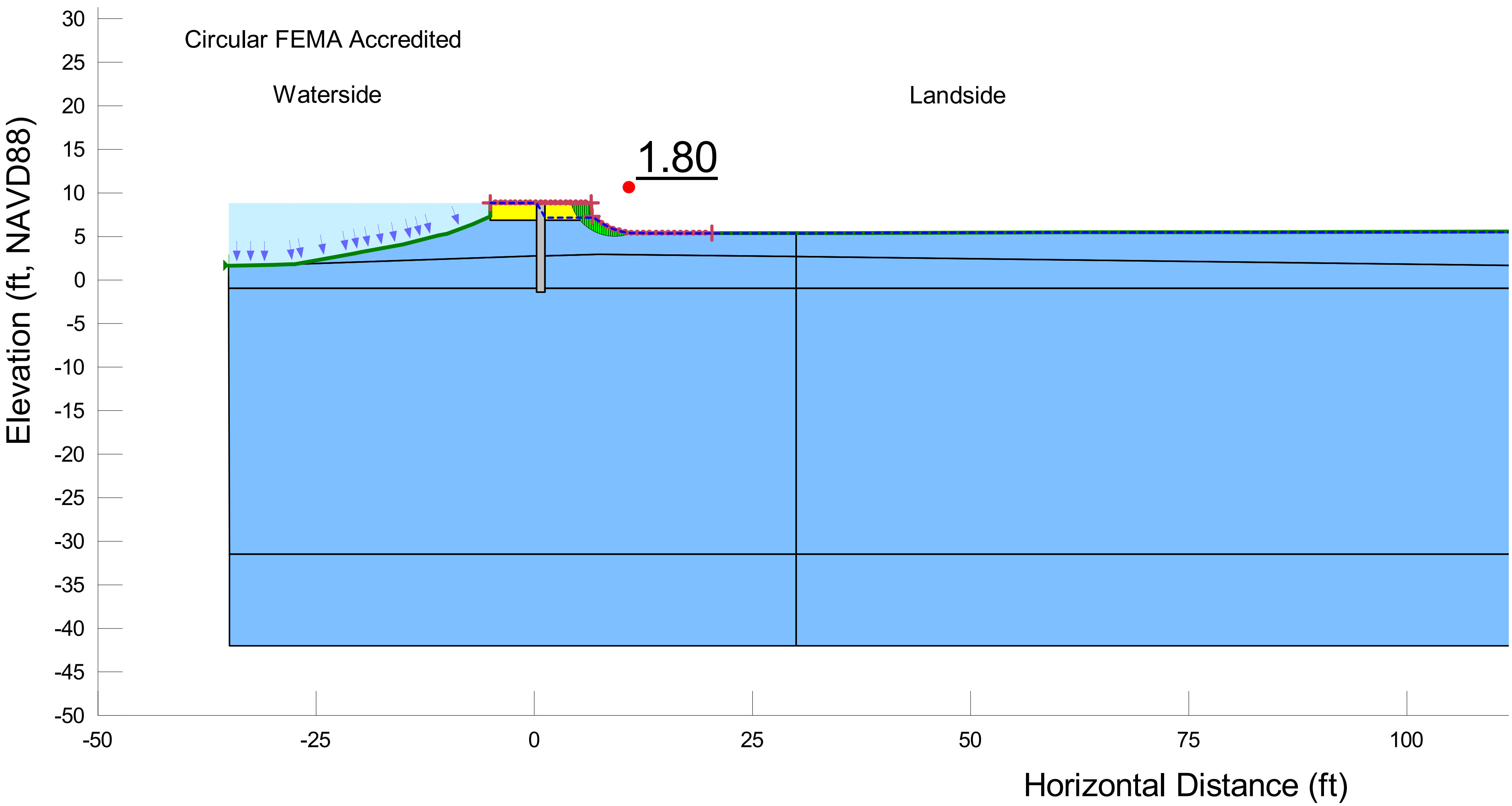


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions Stability Model
Marin County FCWCD		January 2016 Figure 27



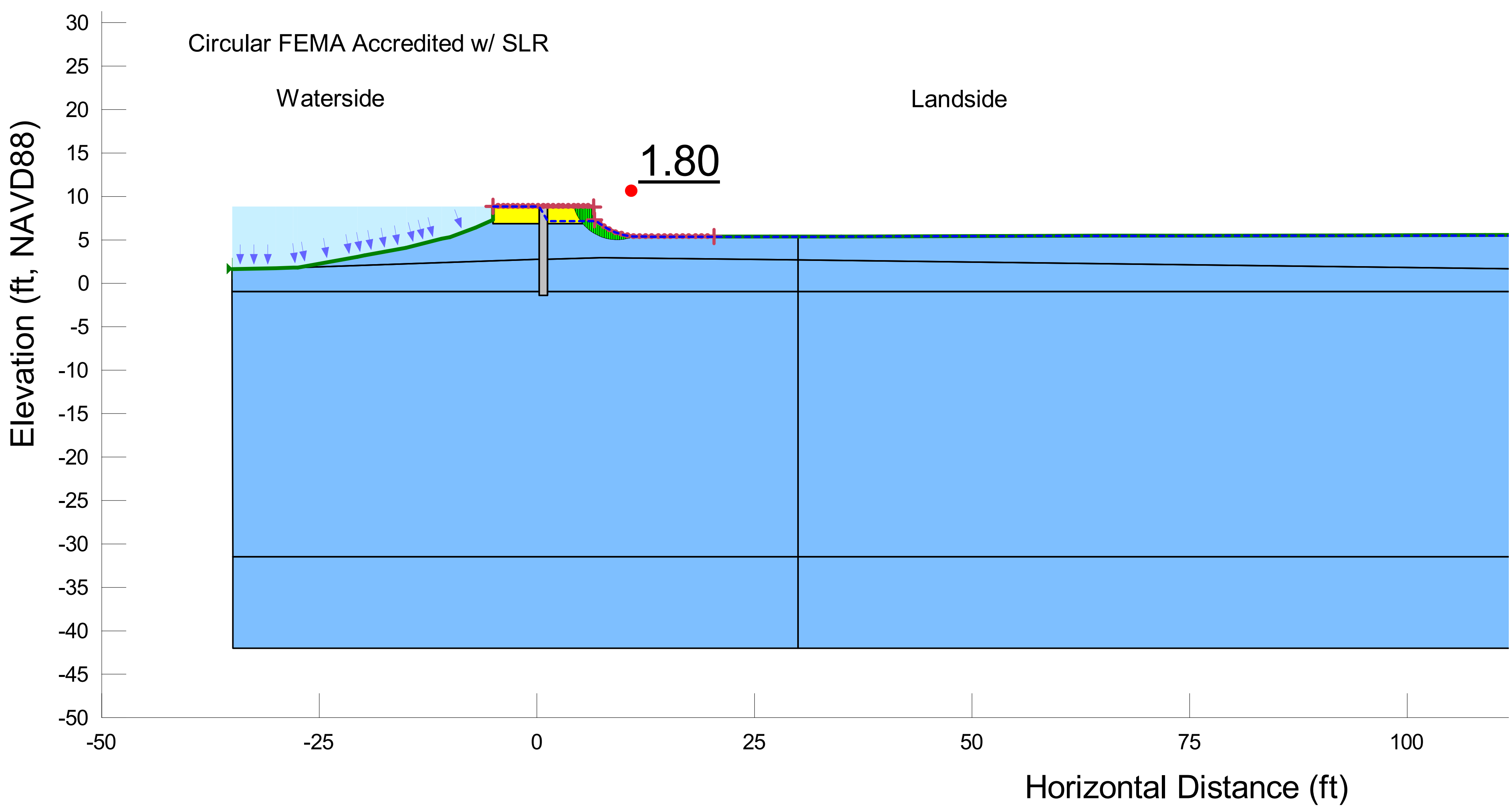
Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions – Baseline Landside Stability Results
Marin County FCWCD		January 2016 Figure 28



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions – FEMA Landside Stability Results
Marin County FCWCD		January 2016 Figure 29



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

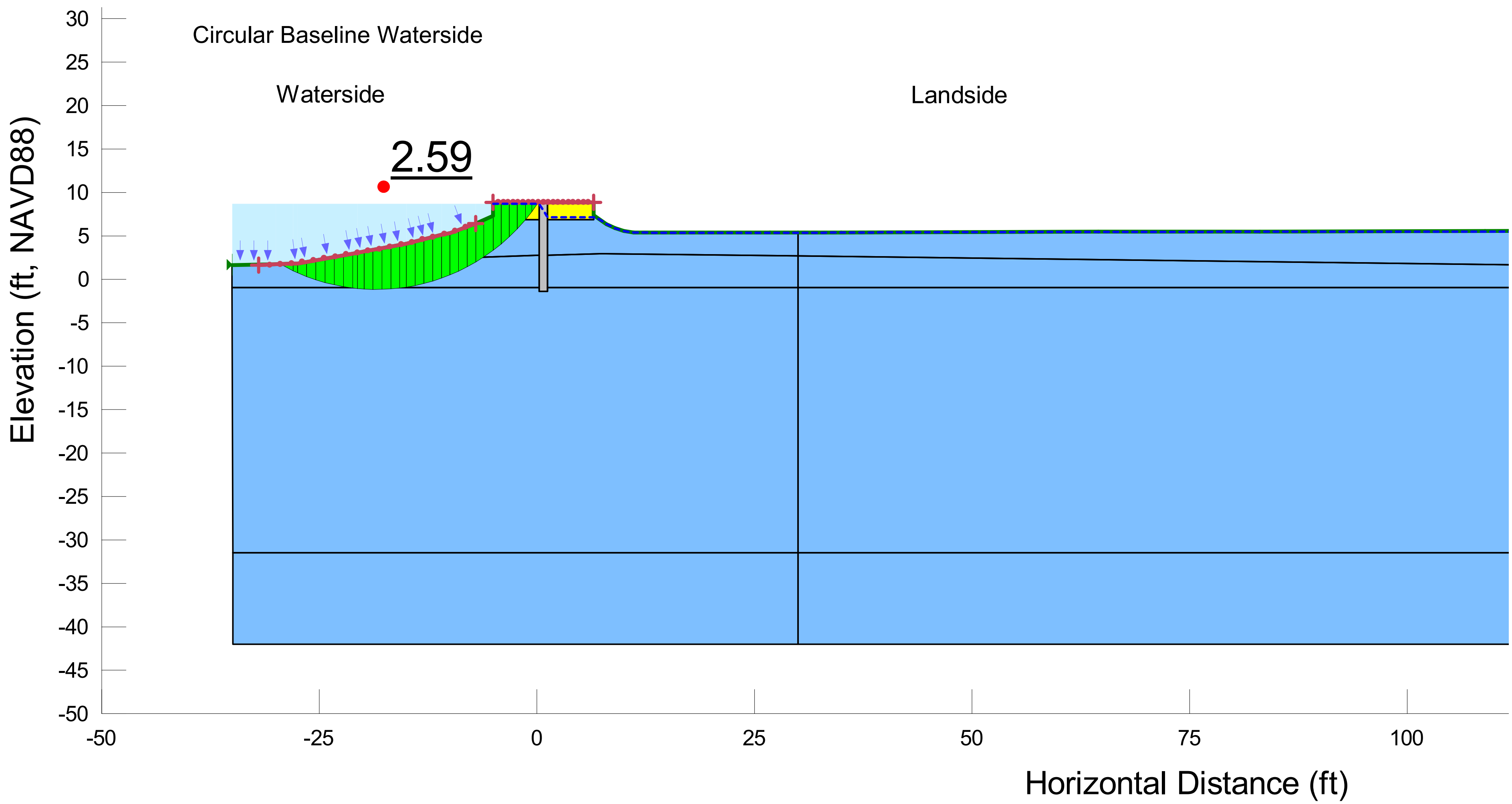
GEI Consultants

Reach 3 – Sta. 28+00 CC-L

Existing Conditions–FEMA w/ SLR
Landside Stability Results

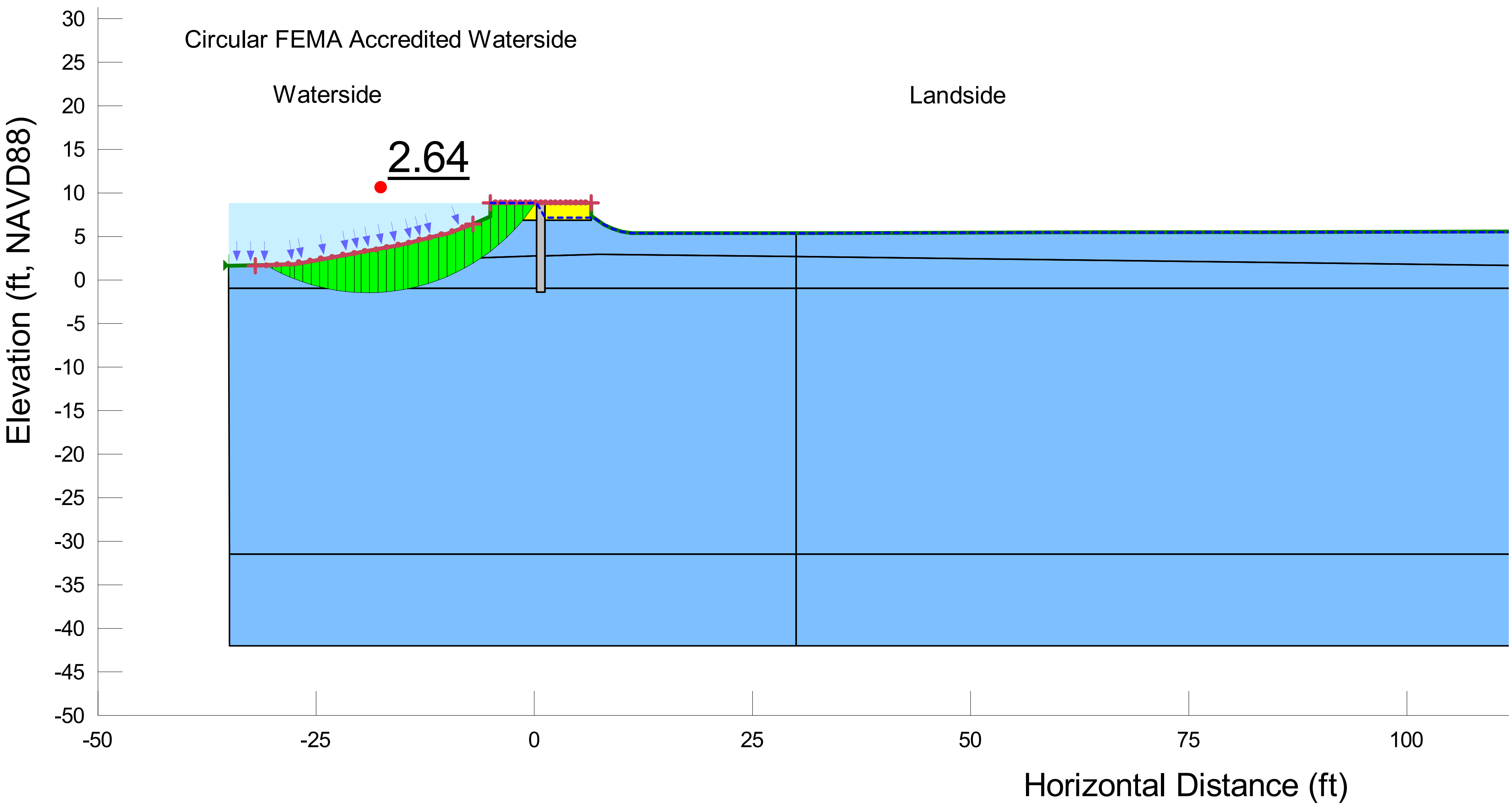
January 2016

Figure 30



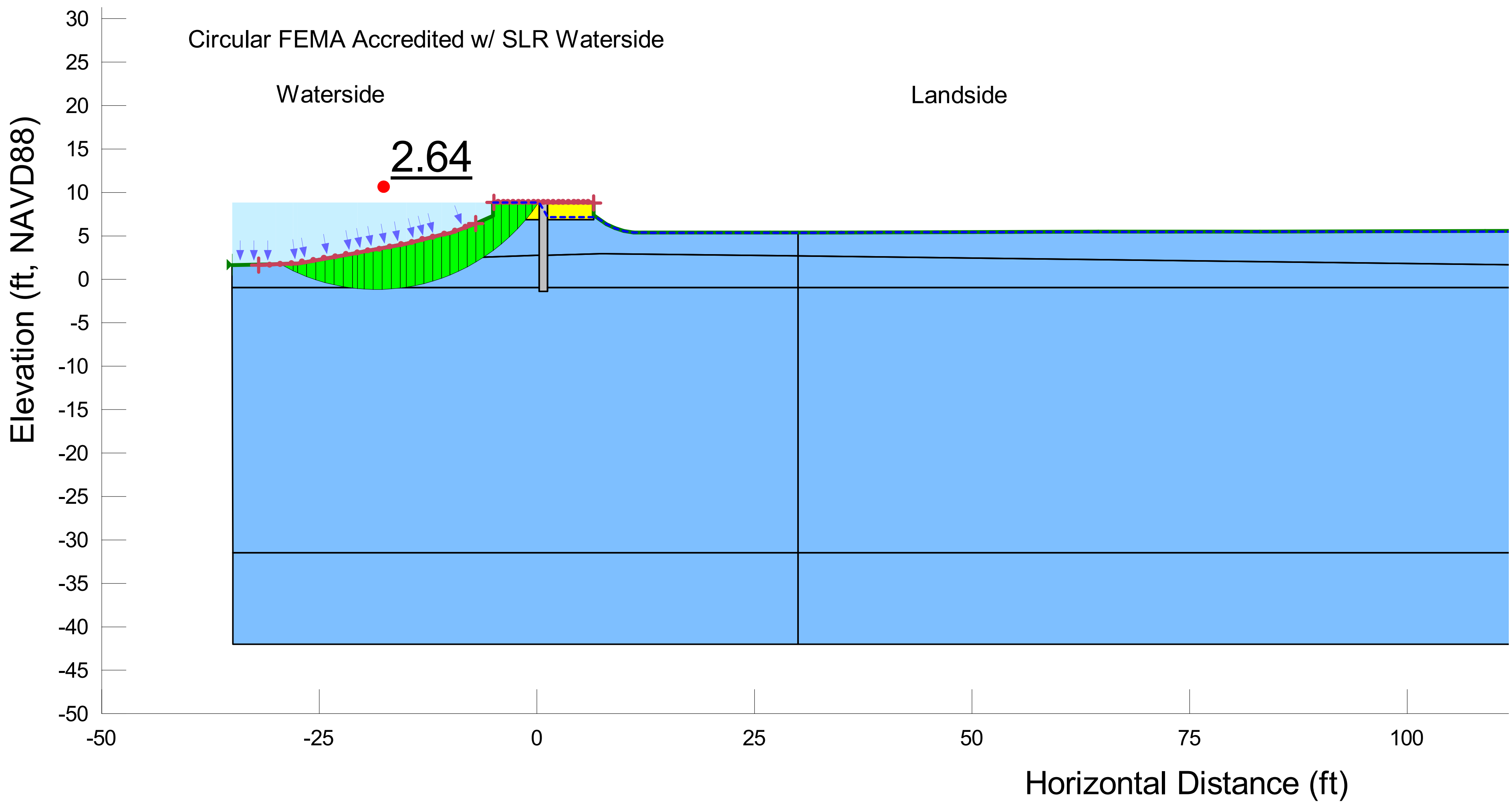
Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions – Baseline Waterside Stability Results
Marin County FCWCD		January 2016 Figure 31



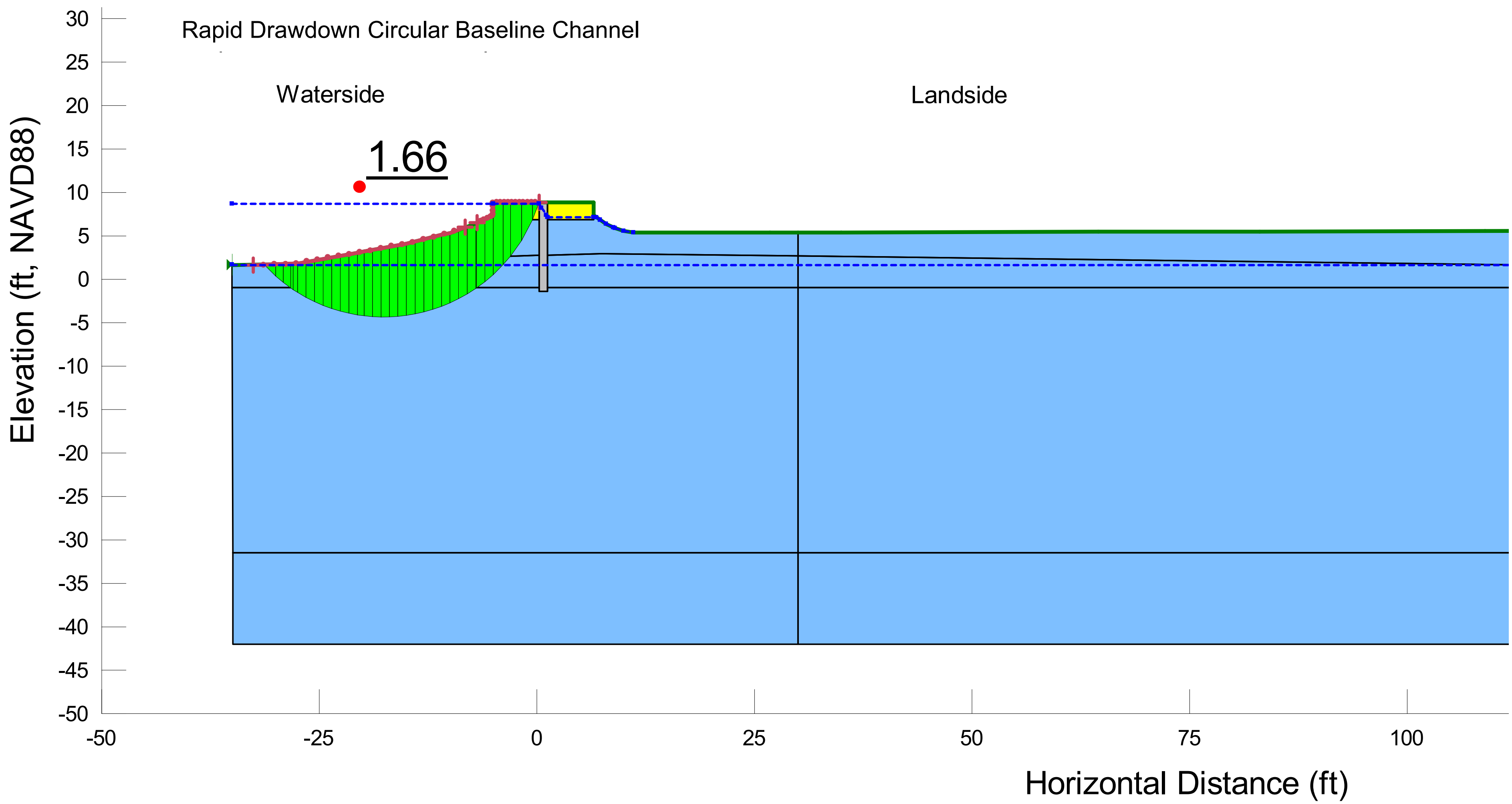
Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions – FEMA Waterside Stability Results
Marin County FCWCD		January 2016 Figure 32

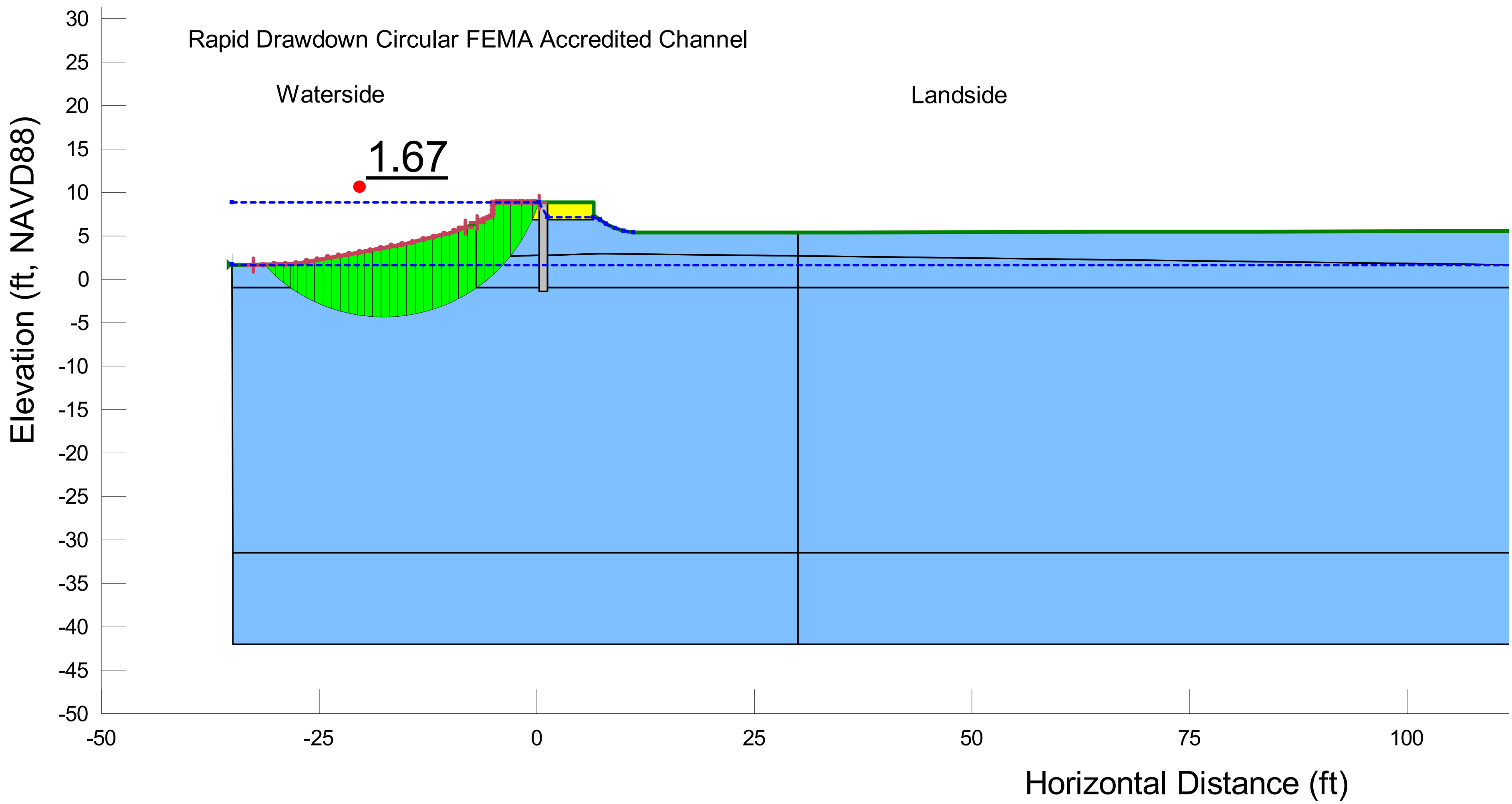


Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

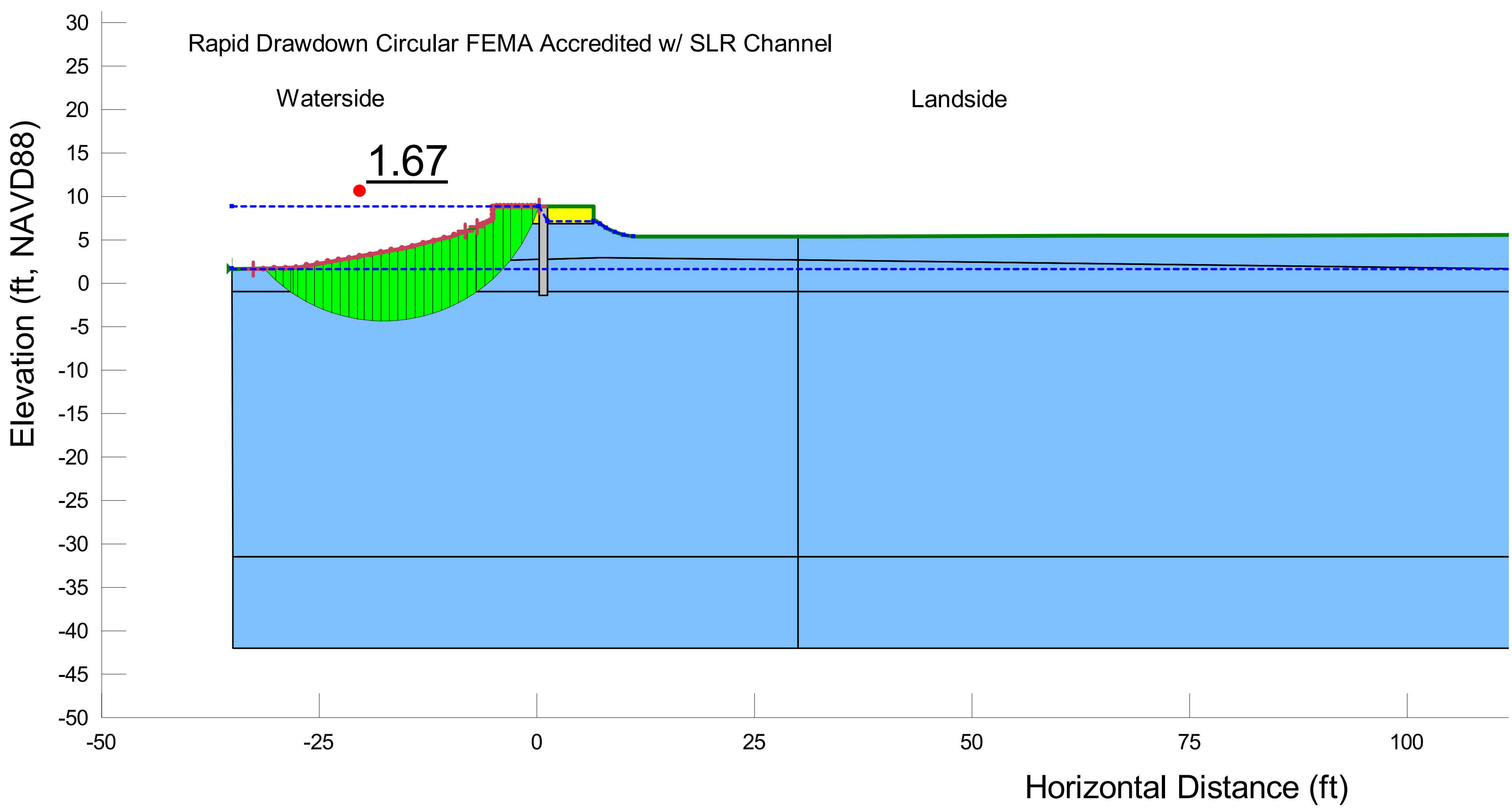
Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions–FEMA w/ SLR Waterside Stability Results
Marin County FCWCD		January 2016 Figure 33



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions – Baseline Total Flood Drawdown Results
Marin County FCWCD		January 2016 Figure 34

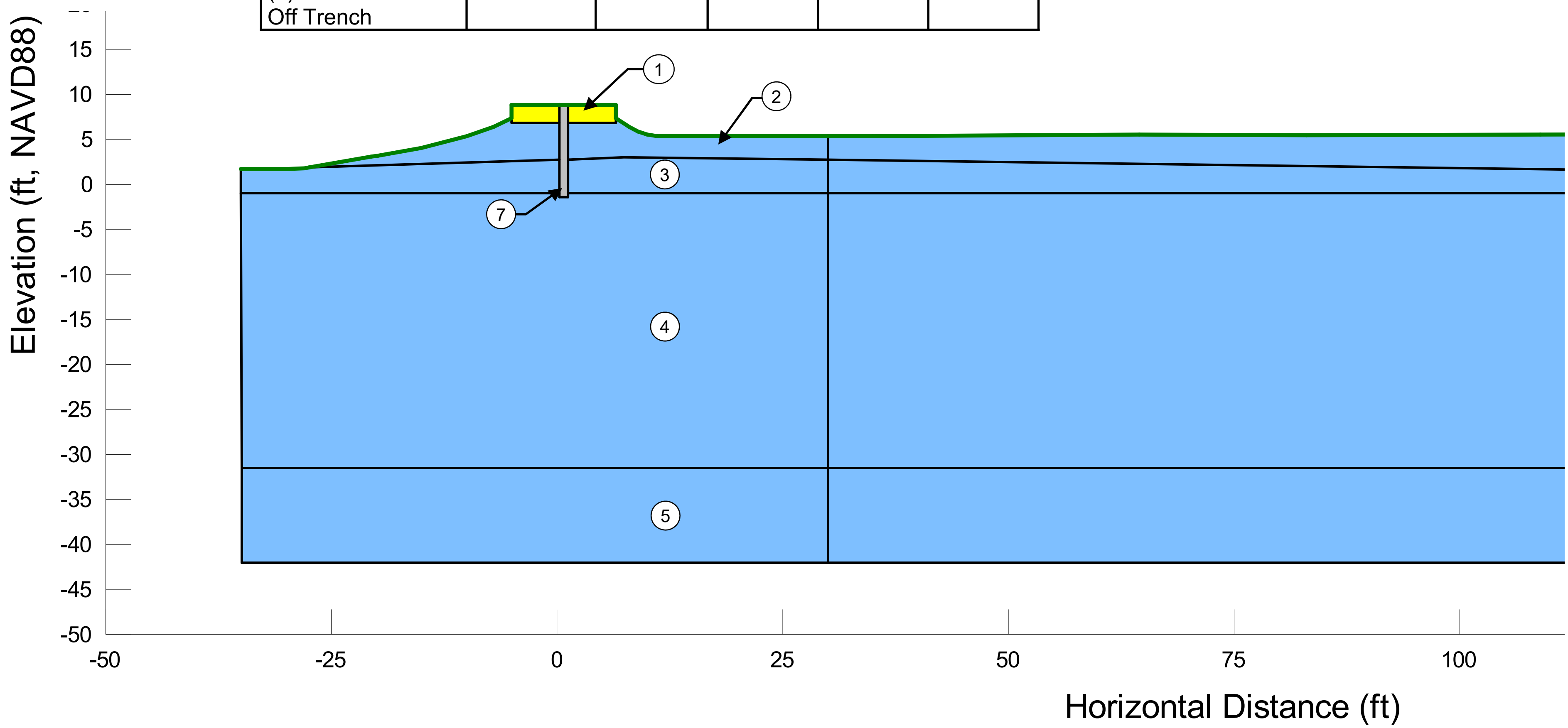


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions – FEMA Total Flood Drawdown Results
Marin County FCWCD		January 2016 — Figure 35



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 3 – Sta. 28+00 CC-L** Existing Conditions – FEMA w/ SLR Total Flood Drawdown Results
Marin County FCWCD		January 2016 Figure 36

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c'	ϕ'	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
(1) GP-GM	120	0	34	--	--
(2) Sandy CL	120	60	32	480	4
(3) CH	95	0	30	280	0
(4) CH	95	0	28	136	8
(5) CL	95	0	30	210	9
(7) Bentonite Cut-Off Trench	120	360	4	500	0



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

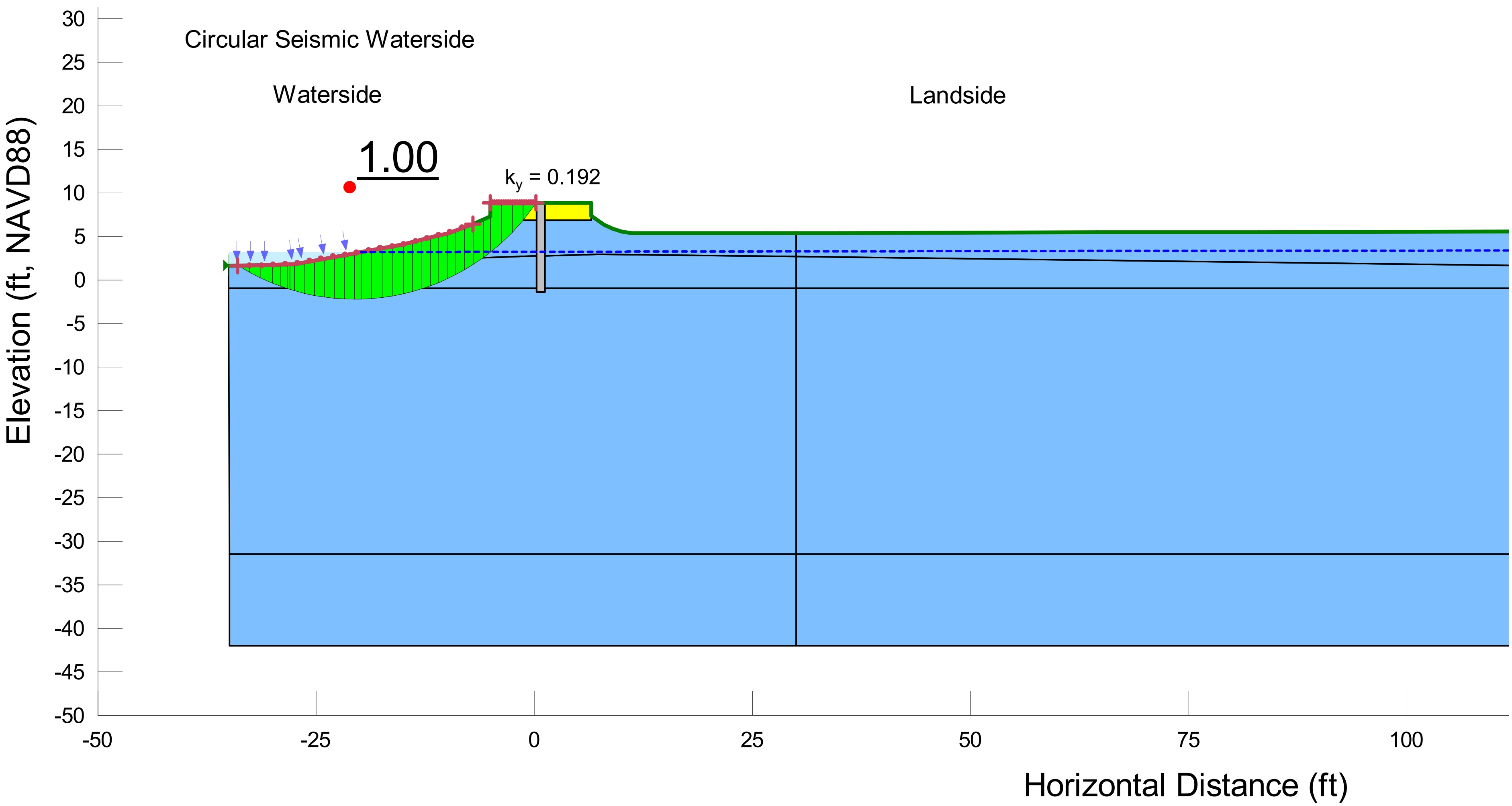
GEI Consultants

Reach 3 – Sta. 28+00 CC-L

Existing Conditions – Seismic Model

January 2016

Figure 37



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

GEI Consultants

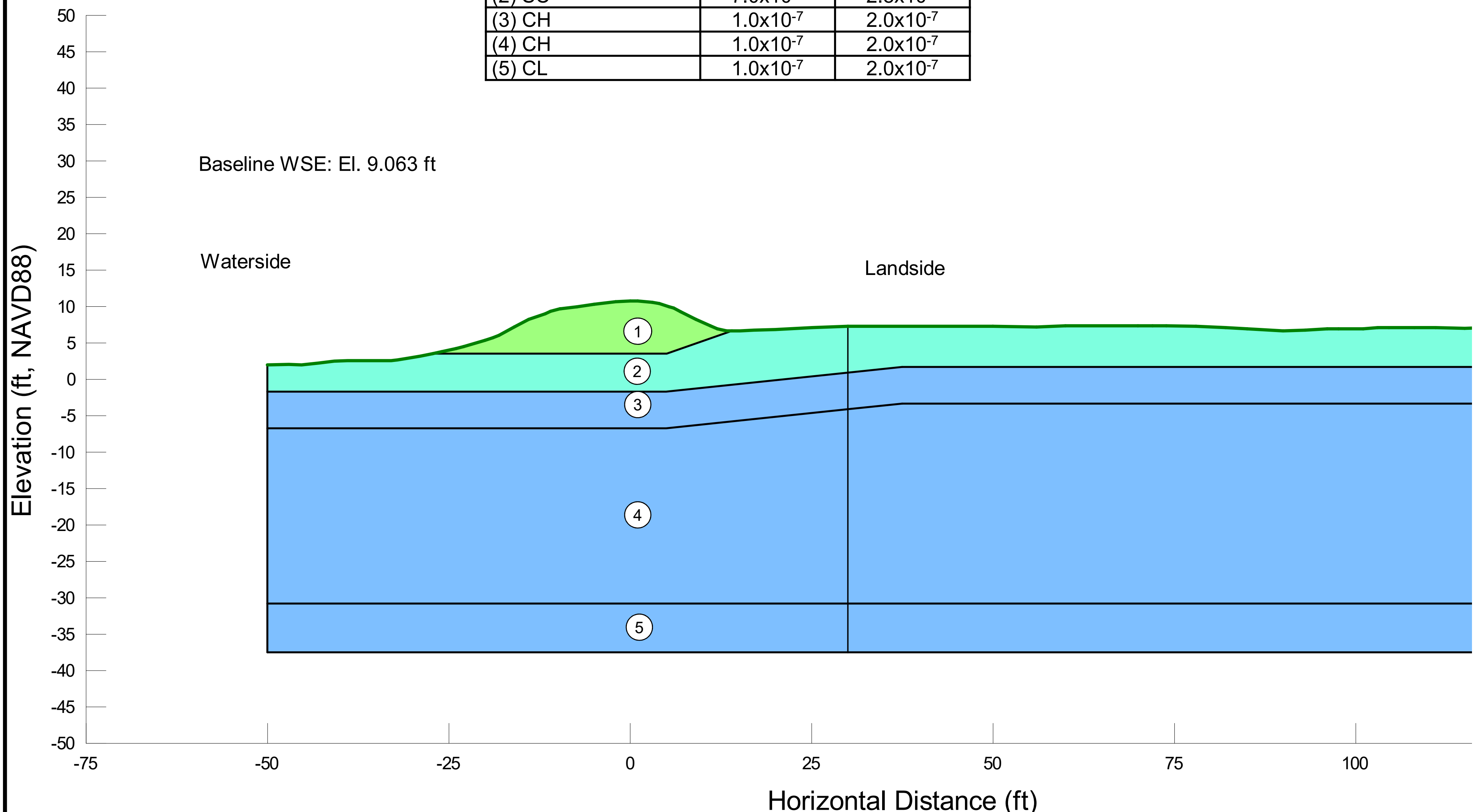
Reach 3 – Sta. 28+00 CC-L

Existing Conditions – Seismic Analysis Results

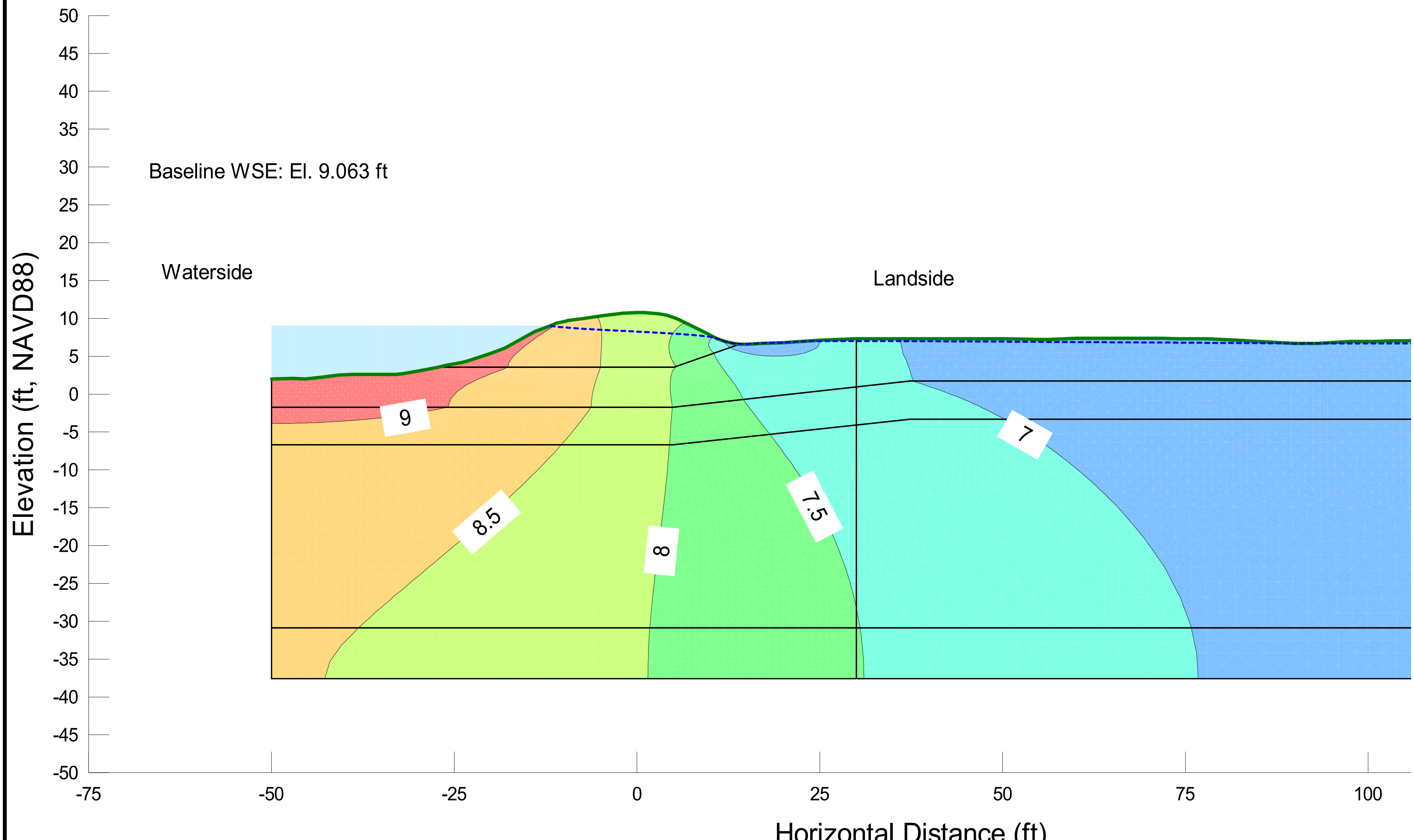
January 2016

Figure 38

Layer Name	k_v (cm/sec)	k_h (cm/sec)
(1) GP-GC	3.0×10^{-3}	1.2×10^{-2}
(2) SC	7.0×10^{-6}	2.8×10^{-5}
(3) CH	1.0×10^{-7}	2.0×10^{-7}
(4) CH	1.0×10^{-7}	2.0×10^{-7}
(5) CL	1.0×10^{-7}	2.0×10^{-7}



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions Steady State Seepage Model
Marin County FCWCD		January 2016 Figure 39



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

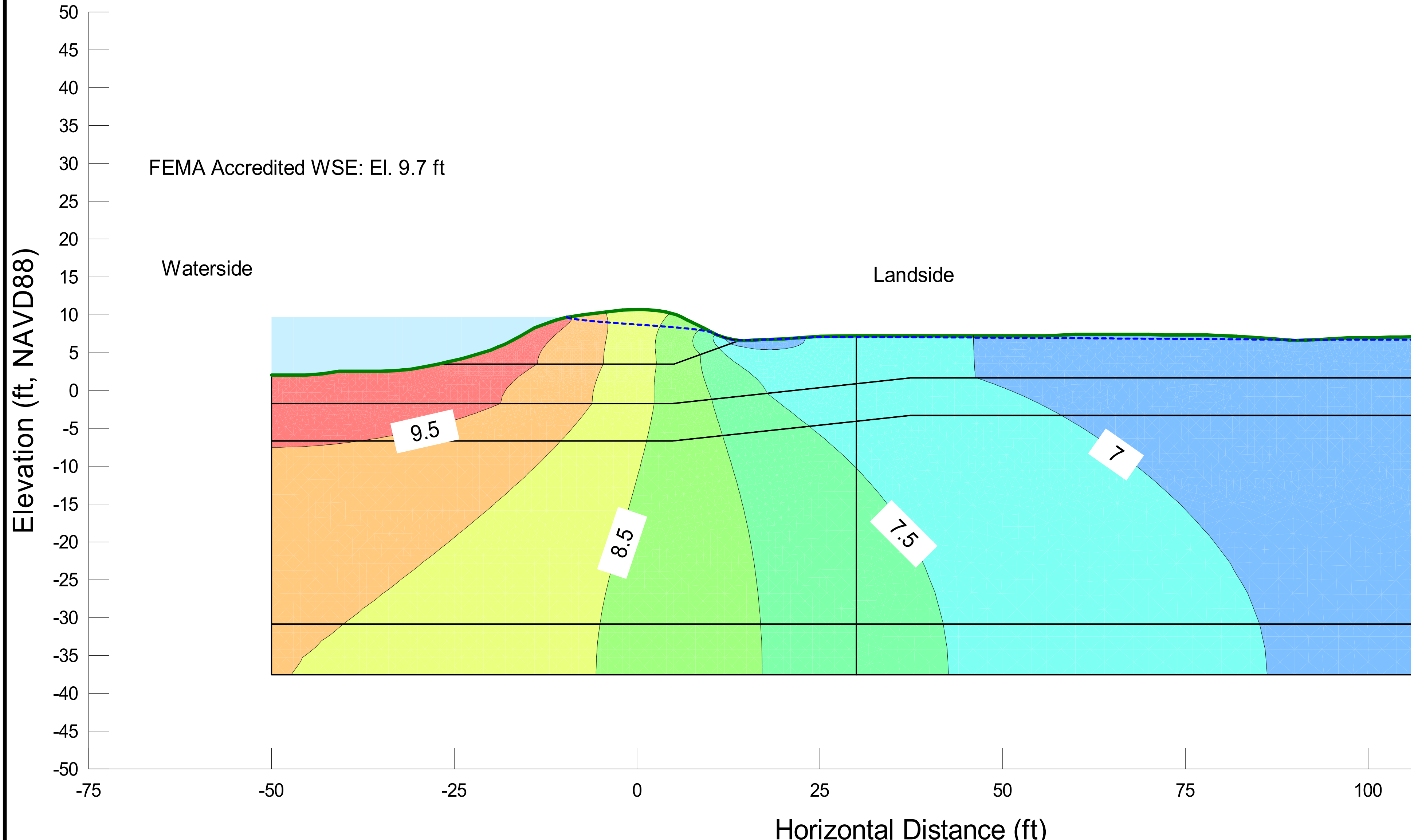
GEI Consultants

Reach 4 – Sta. 34+00 CC-L

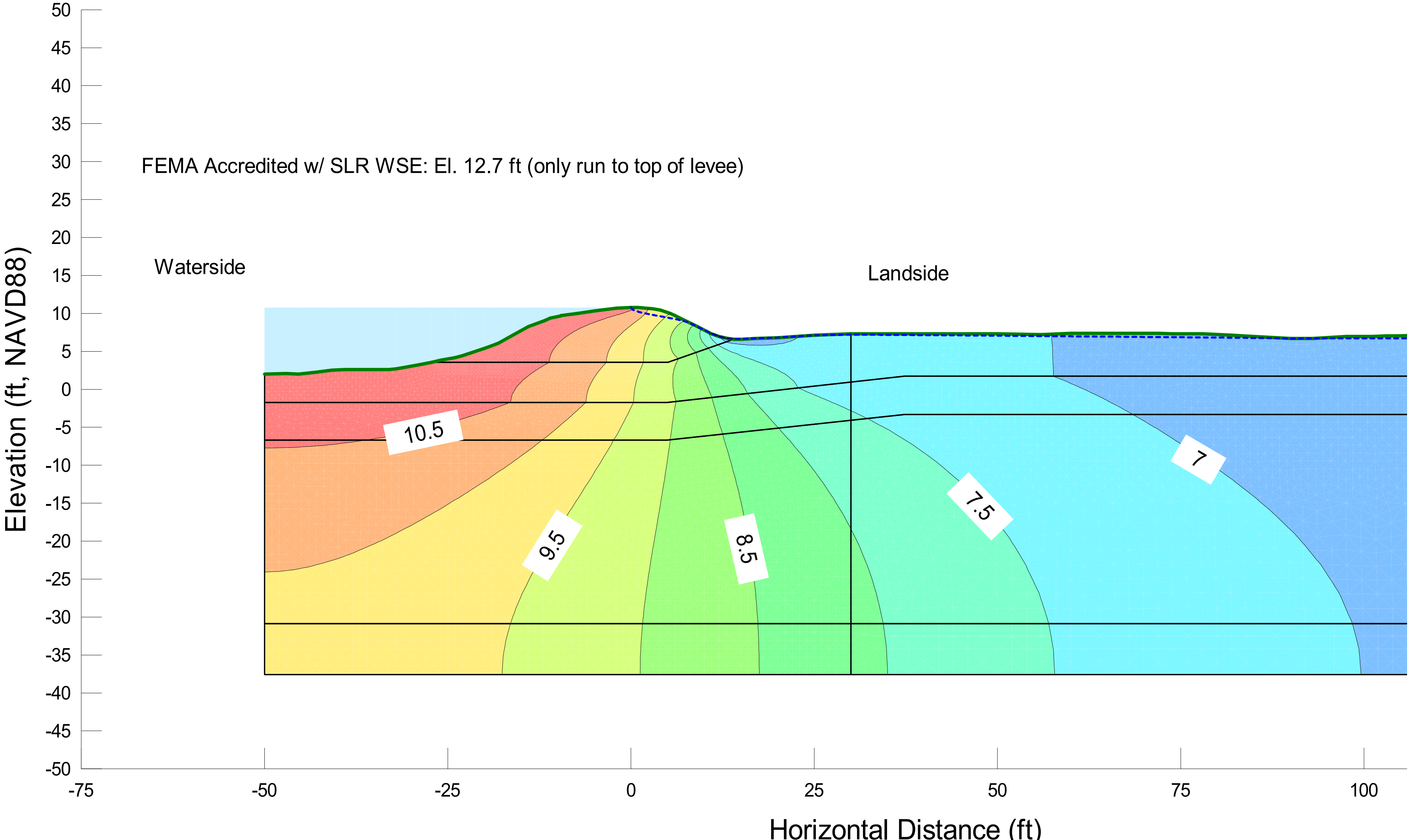
Existing Conditions – Baseline
Steady State Seepage Results

January 2016

Figure 40

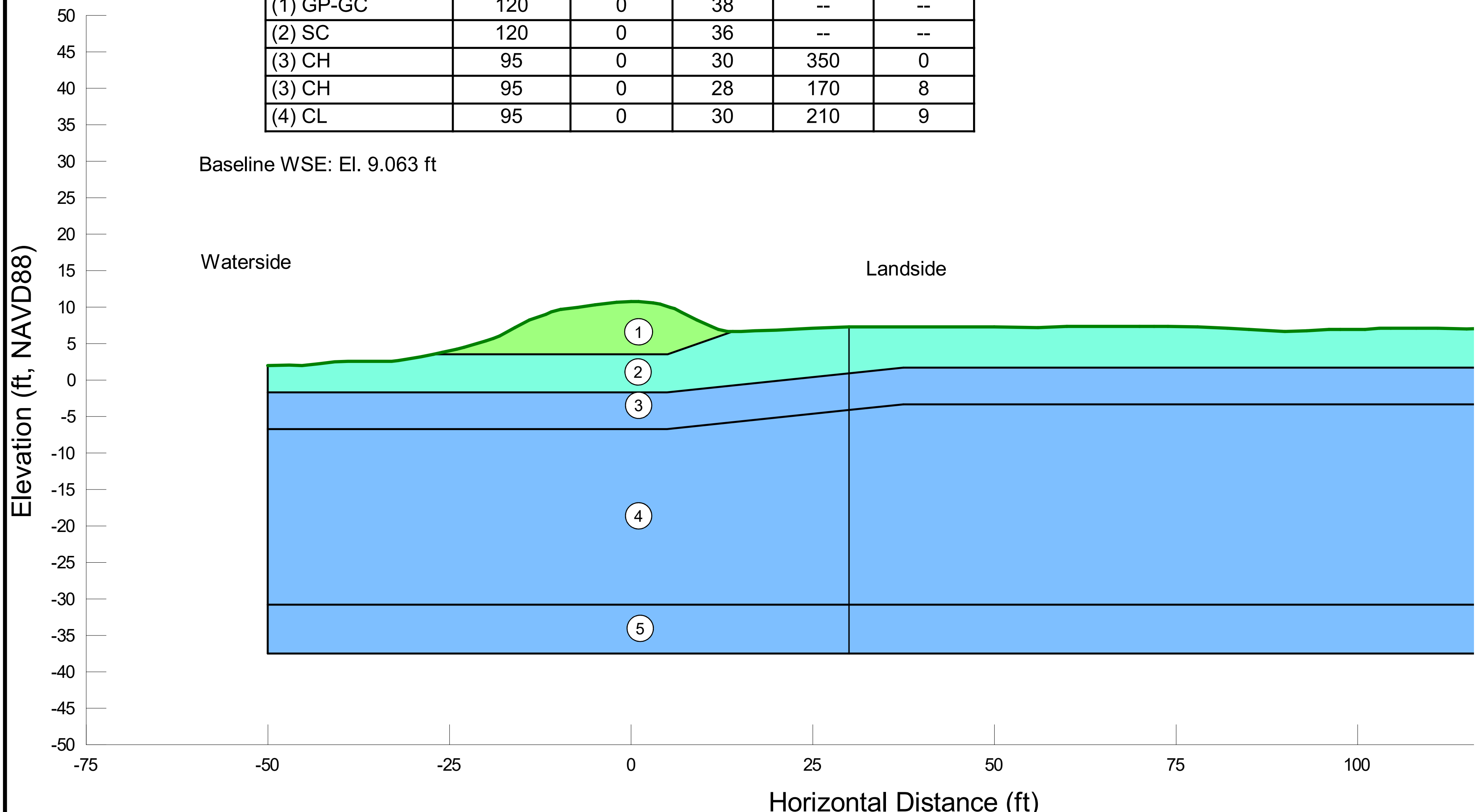


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions – FEMA Steady State Seepage Results
Marin County FCWCD		January 2016 Figure 41

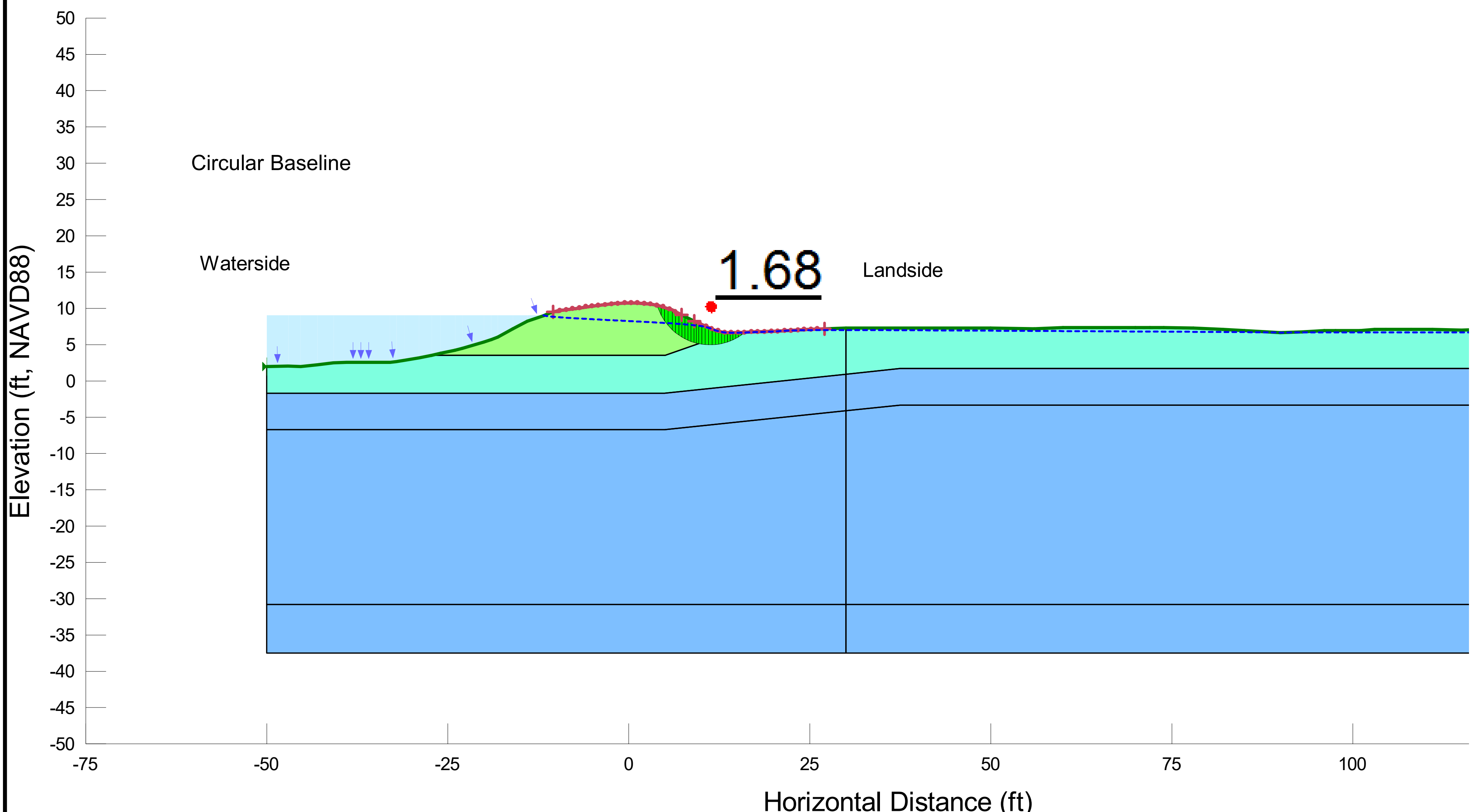


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions–FEMA w/ SLR Steady State Seepage Results
Marin County FCWCD		January 2016 Figure 42

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c'	ϕ'	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
(1) GP-GC	120	0	38	--	--
(2) SC	120	0	36	--	--
(3) CH	95	0	30	350	0
(3) CH	95	0	28	170	8
(4) CL	95	0	30	210	9

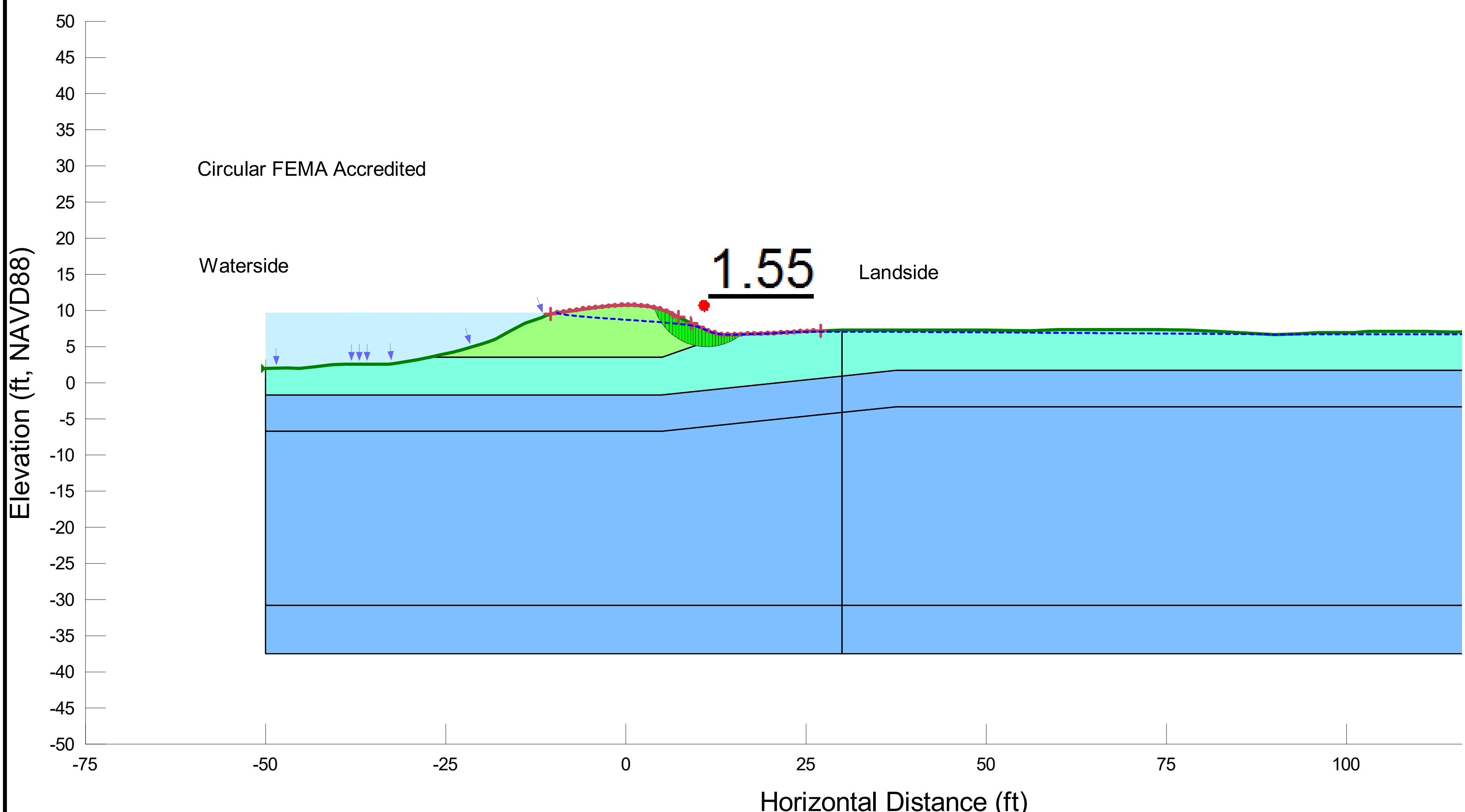


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions Steady State Seepage Model
Marin County FCWCD		January 2016 Figure 43



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions – Baseline Landside Stability Results
Marin County FCWCD		January 2016 Figure 44



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

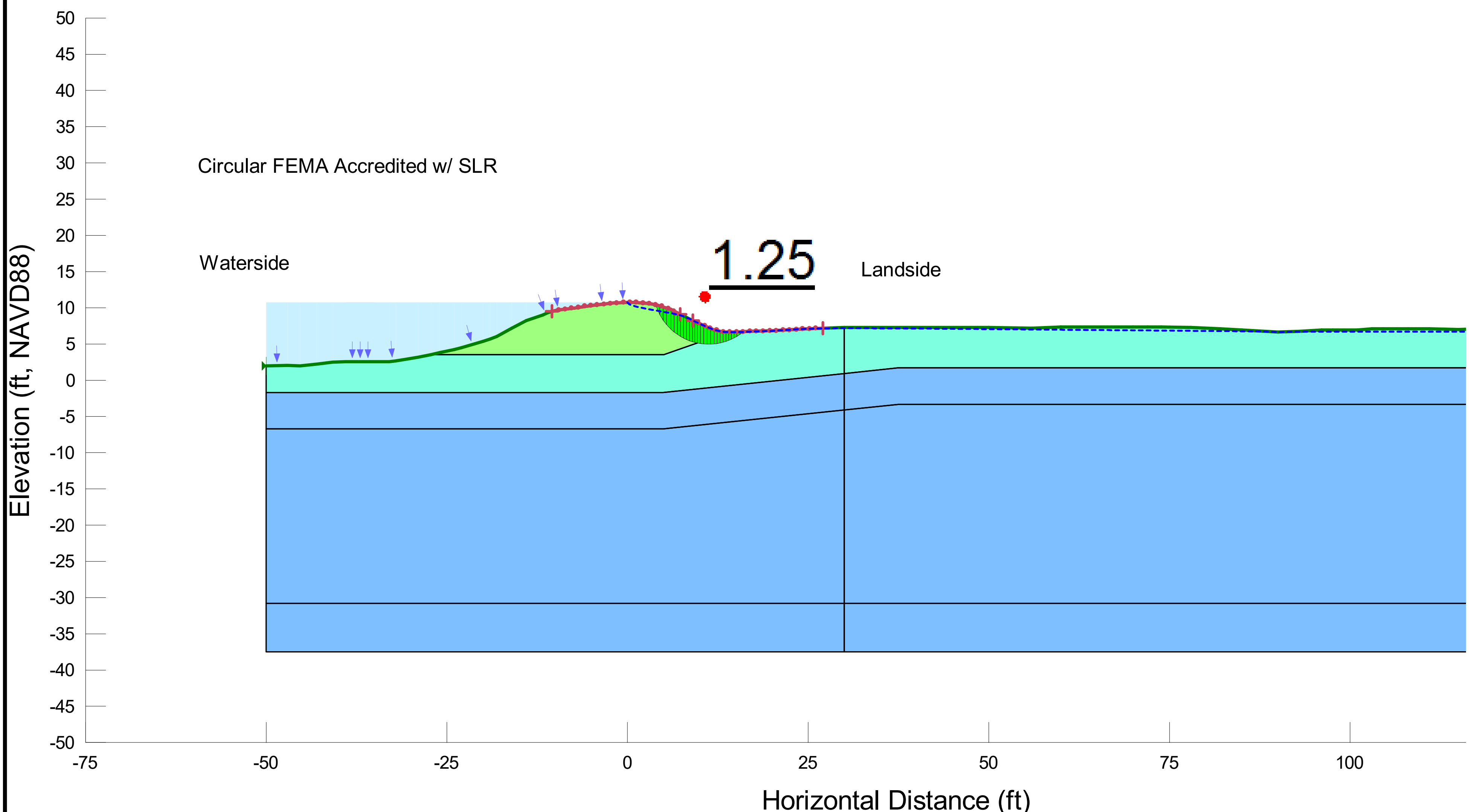
GEI Consultants

Reach 4 – Sta. 34+00 CC-L

Existing Conditions – FEMA Landside Stability Results

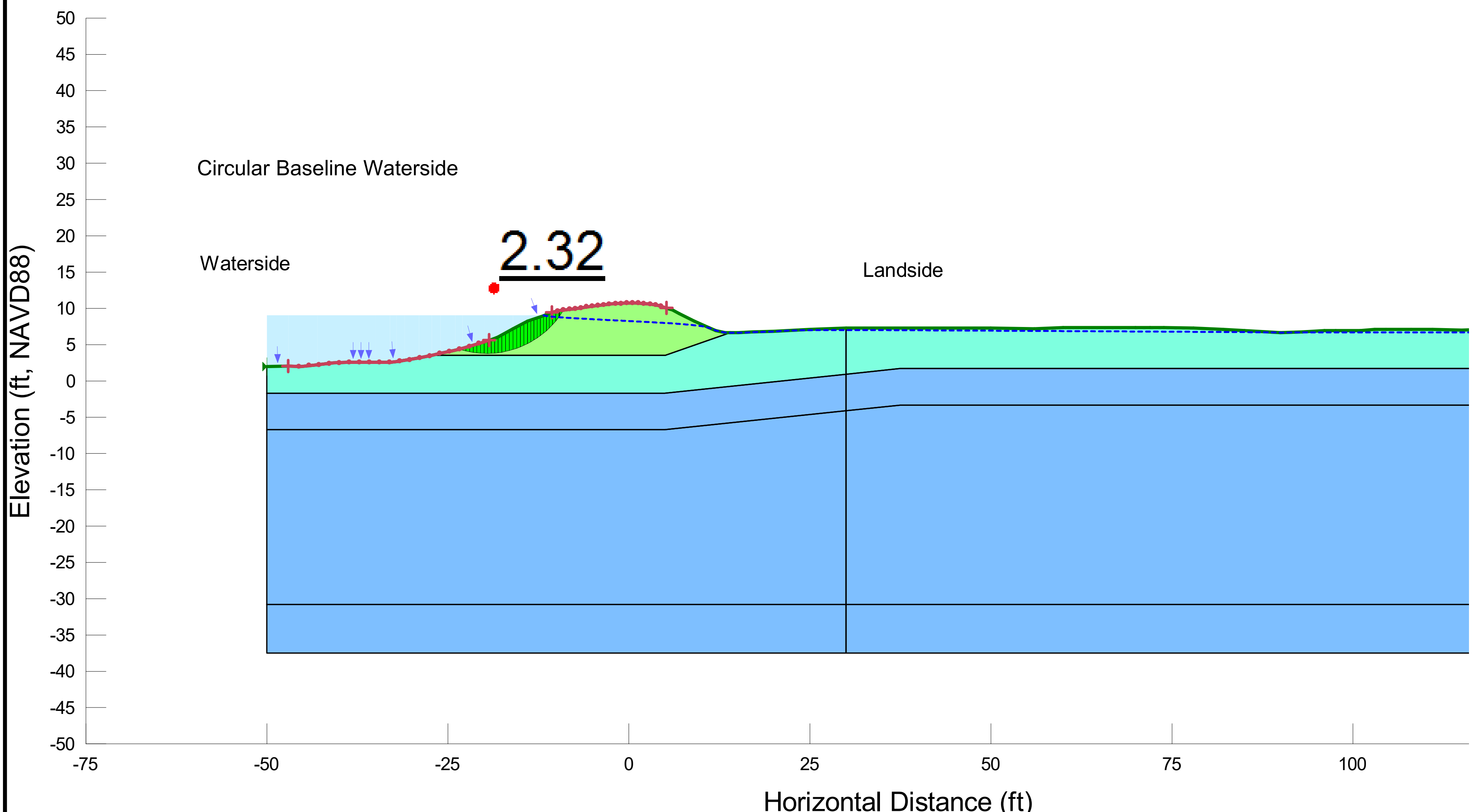
January 2016

Figure 45



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions–FEMA w/ SLR Landside Stability Results
Marin County FCWCD		January 2016 Figure 46



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

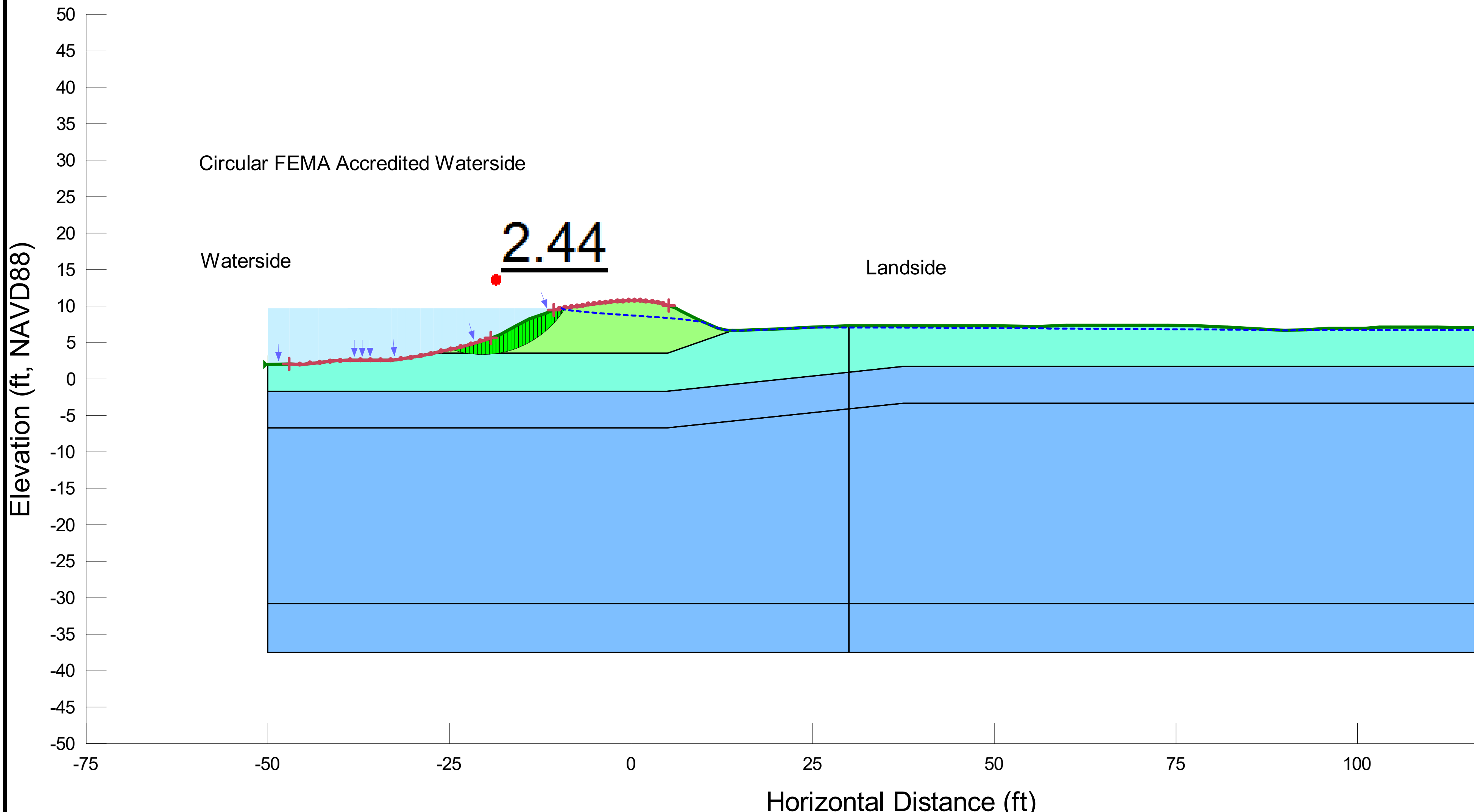
GEI Consultants

Reach 4 – Sta. 34+00 CC-L

Existing Conditions – Baseline
Waterside Stability Results

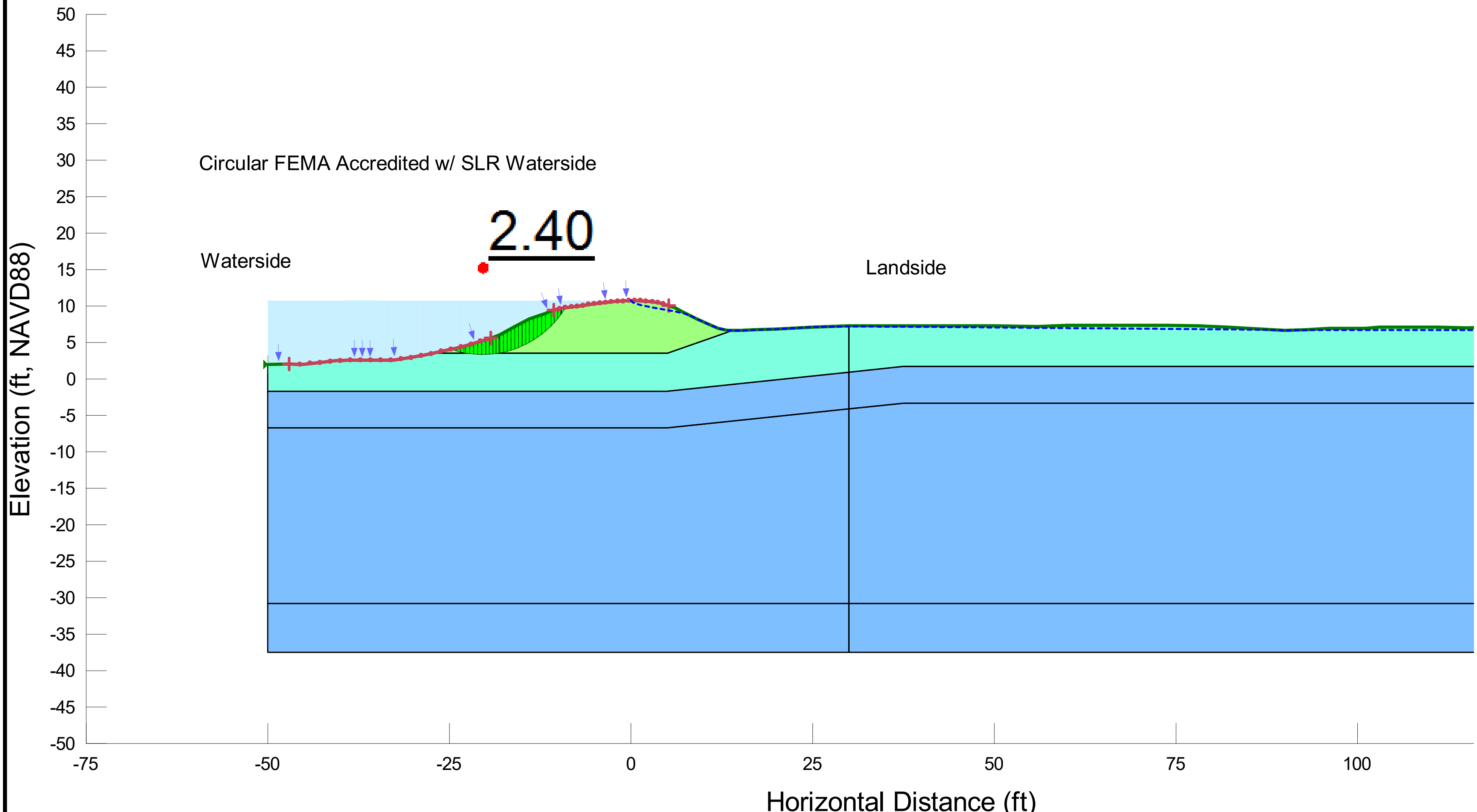
January 2016

Figure 47



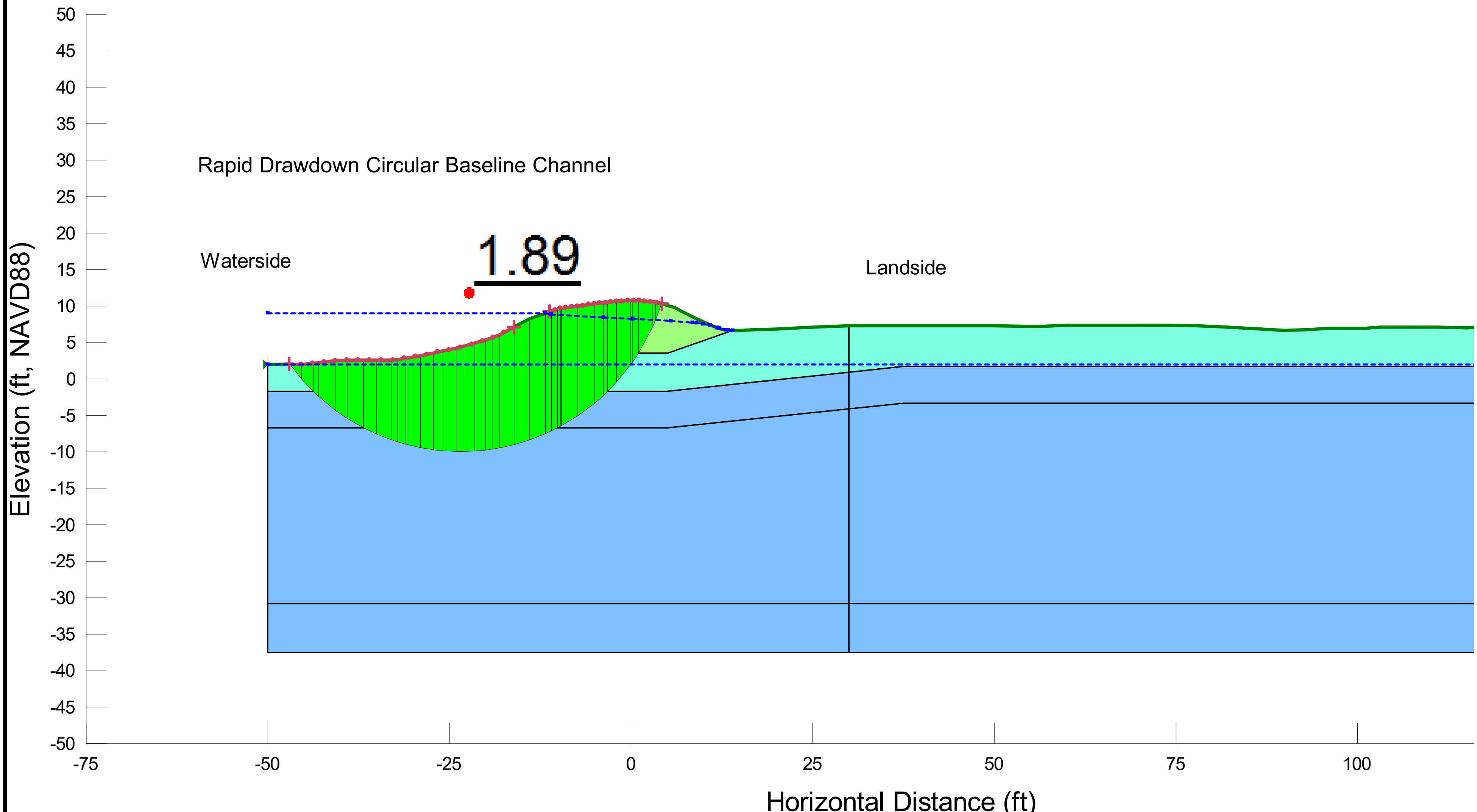
Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions – FEMA Waterside Stability Results
Marin County FCWCD		January 2016 Figure 48

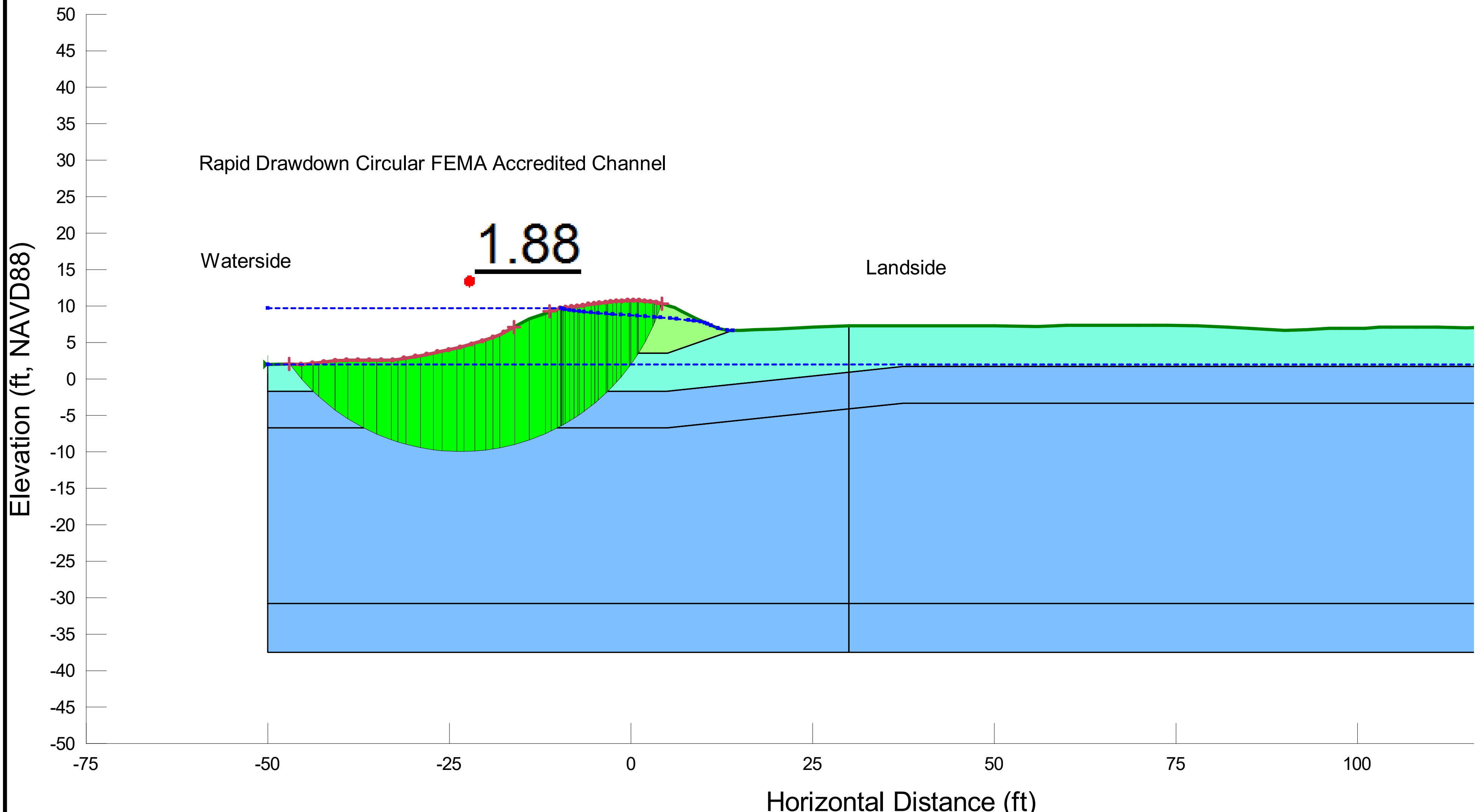


Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

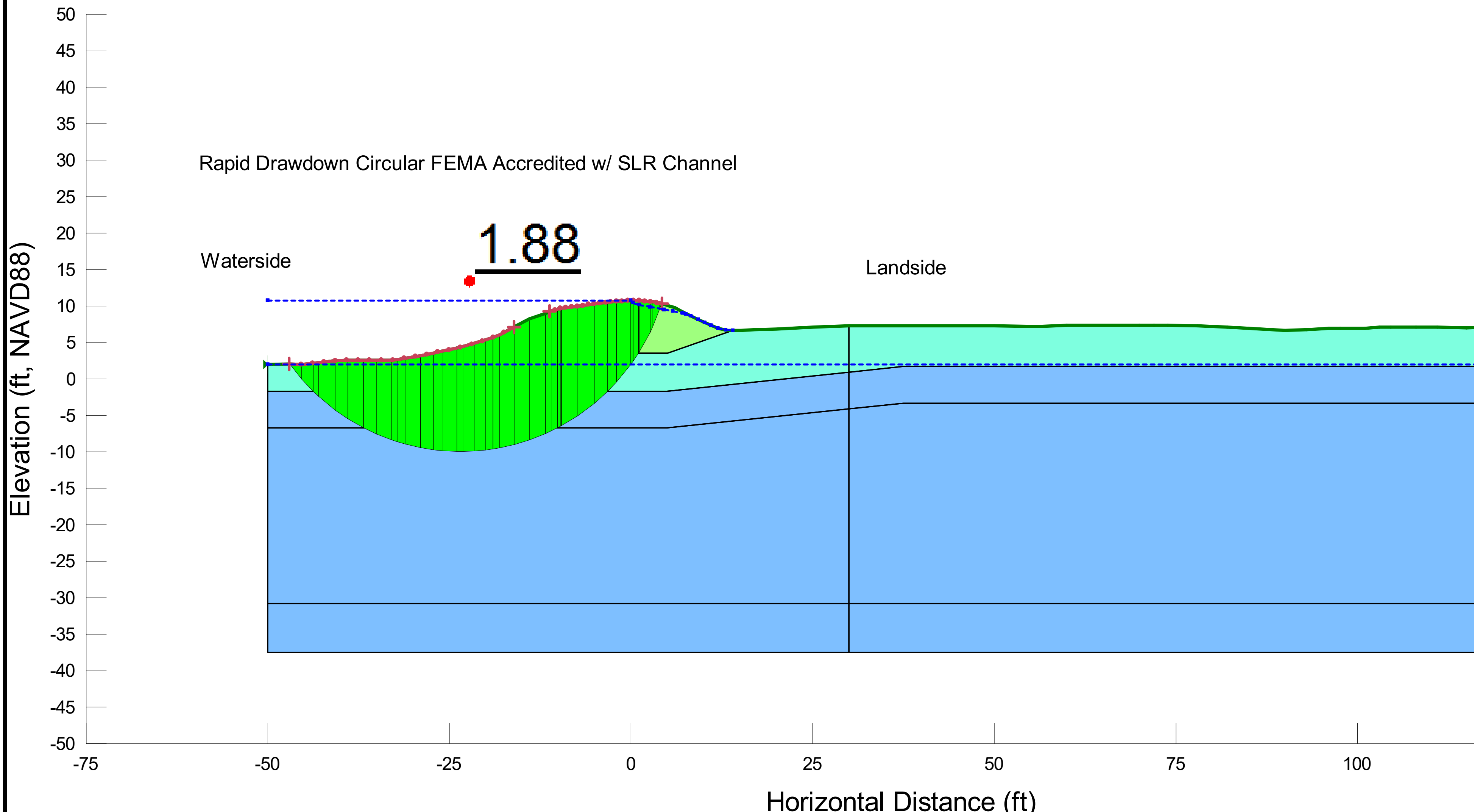
Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions–FEMA w/ SLR Waterside Stability Results
Marin County FCWCD		January 2016 Figure 49



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions – Baseline Total Flood Drawdown Results
Marin County FCWCD		January 2016 Figure 50

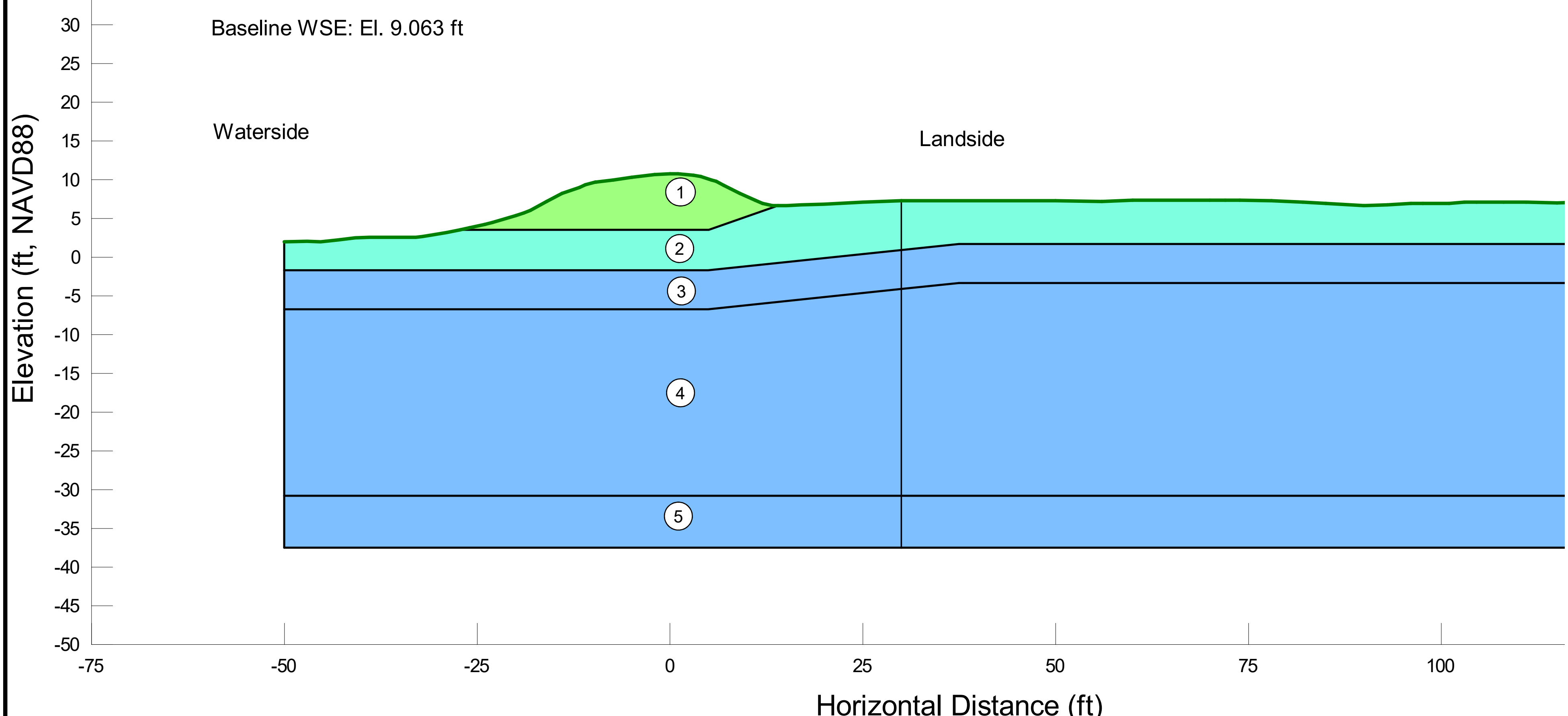


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions – FEMA Total Flood Drawdown Results
Marin County FCWCD		January 2016 Figure 51

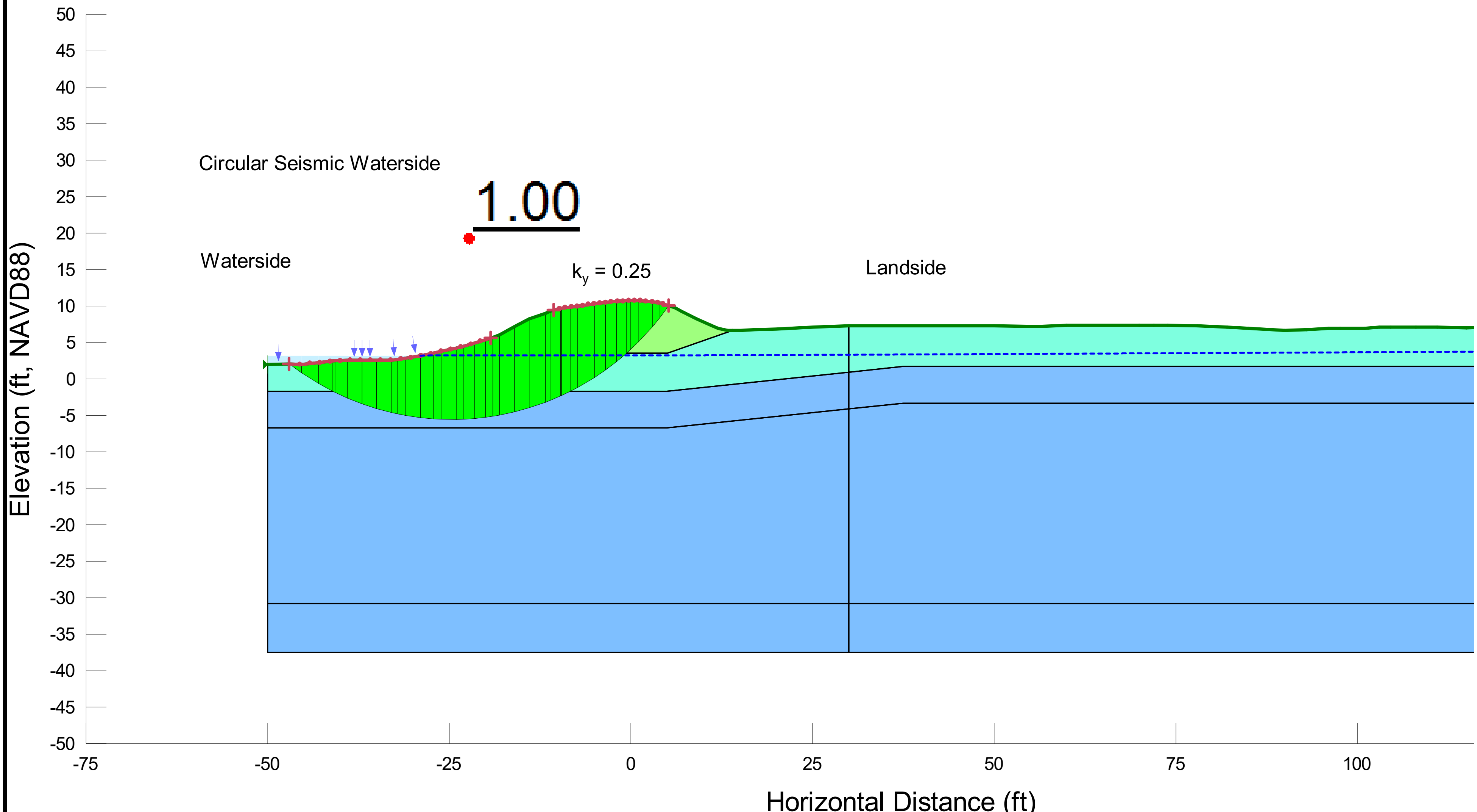


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions – FEMA w/ SLR Total Flood Drawdown Results
Marin County FCWCD		January 2016 Figure 52

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c’	ϕ’	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
(1) GP-GC	120	0	38	--	--
(2) SC	120	0	36	--	--
(3) CH	95	0	30	280	0
(4) CH	95	0	28	136	8
(5) CL	95	0	30	210	9



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 4 – Sta. 34+00 CC-L** Existing Conditions – Seismic Model
Marin County FCWCD		January 2016 Figure 53



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

GEI Consultants

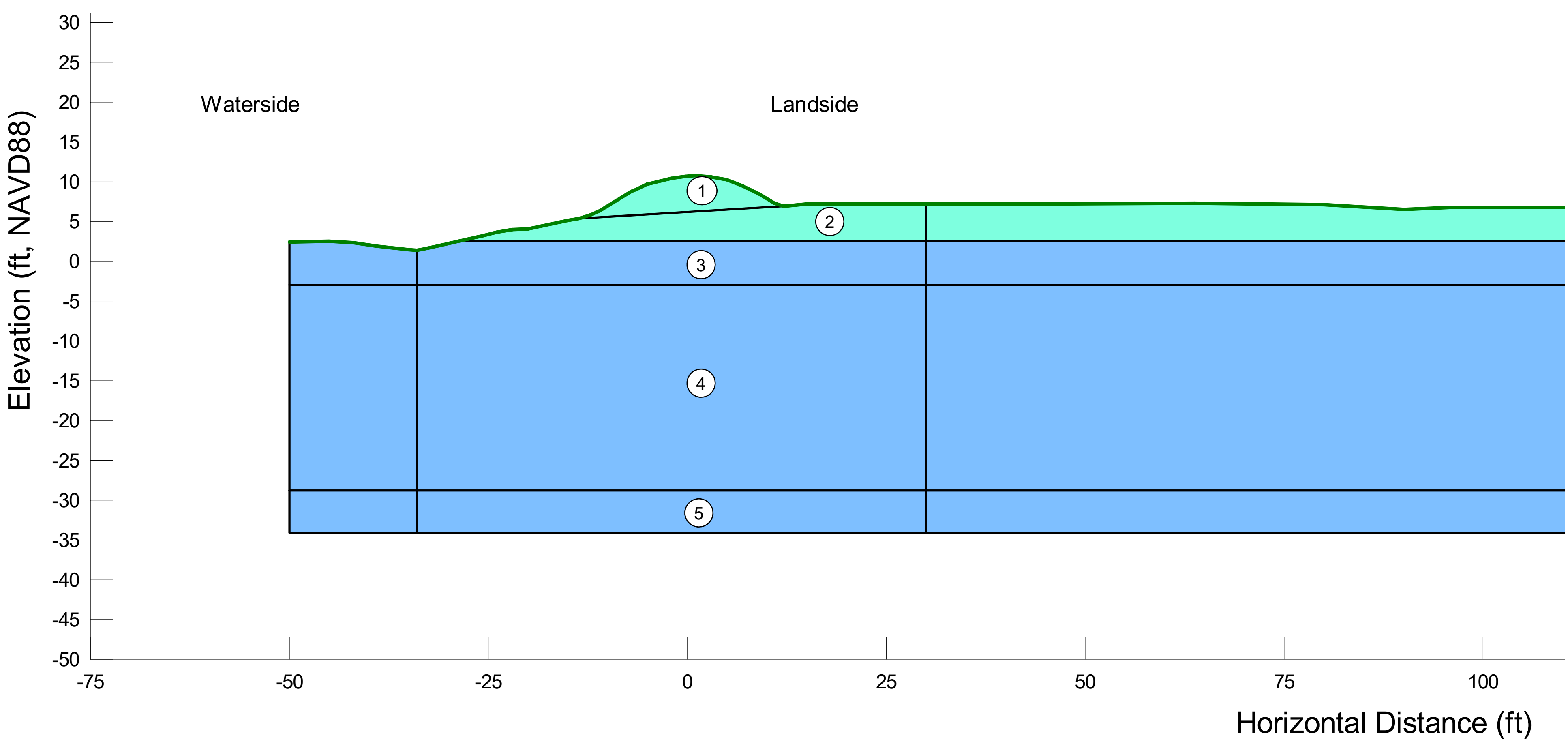
Reach 4 – Sta. 34+00 CC-L

Existing Conditions – Seismic Analysis Results

January 2016

Figure 54

Layer Name	k_v (cm/sec)	k_h (cm/sec)
(1) SC	$7.0x10^{-6}$	$2.8x10^{-5}$
(2) SC	$7.0x10^{-6}$	$2.8x10^{-5}$
(3) CH	$1.0x10^{-7}$	$2.0x10^{-7}$
(4) CH	$1.0x10^{-7}$	$2.0x10^{-7}$
(5) Sandy CH	$1.0x10^{-7}$	$2.0x10^{-7}$



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

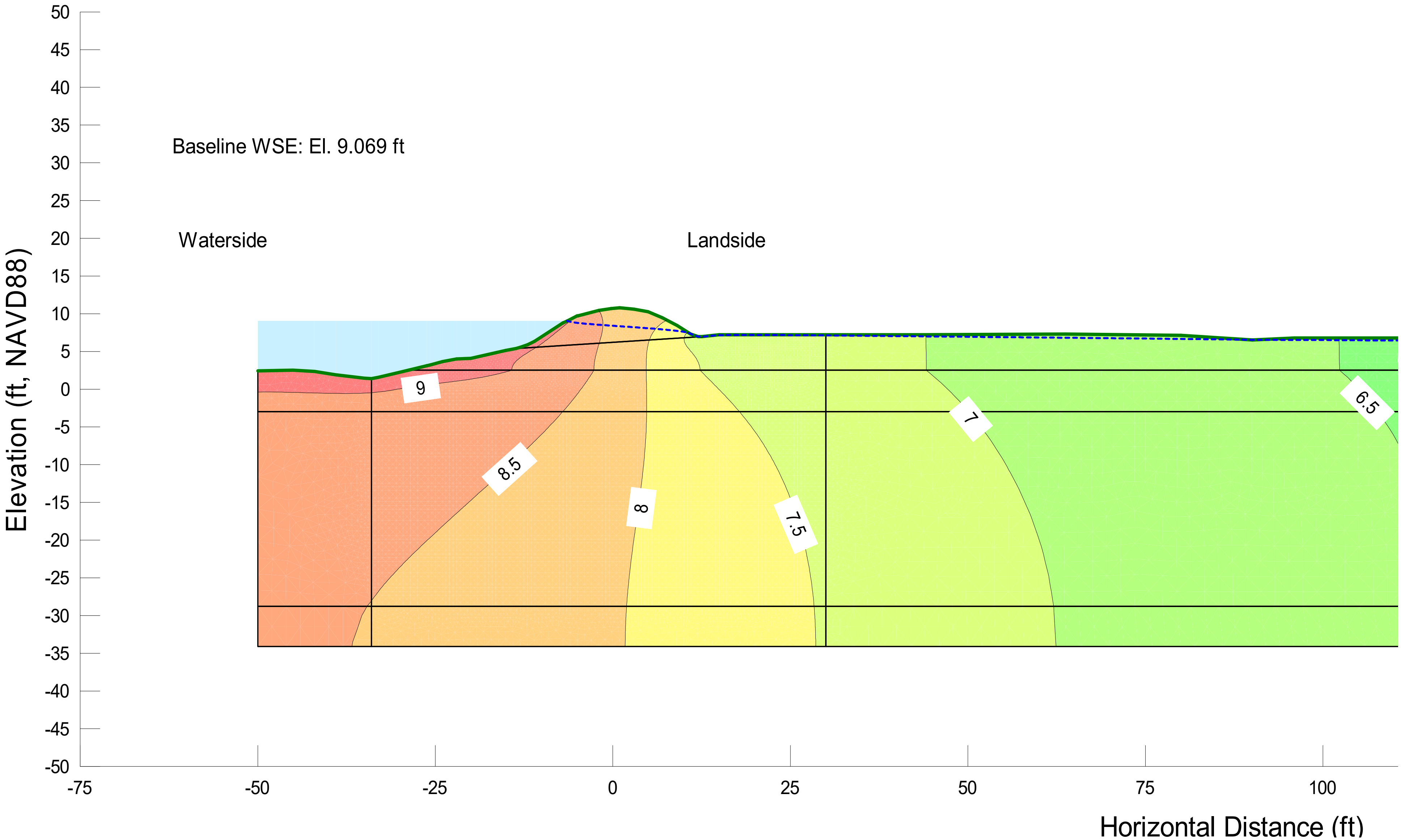
GEI Consultants

Reach 7 – Sta. 35+00 CC-R

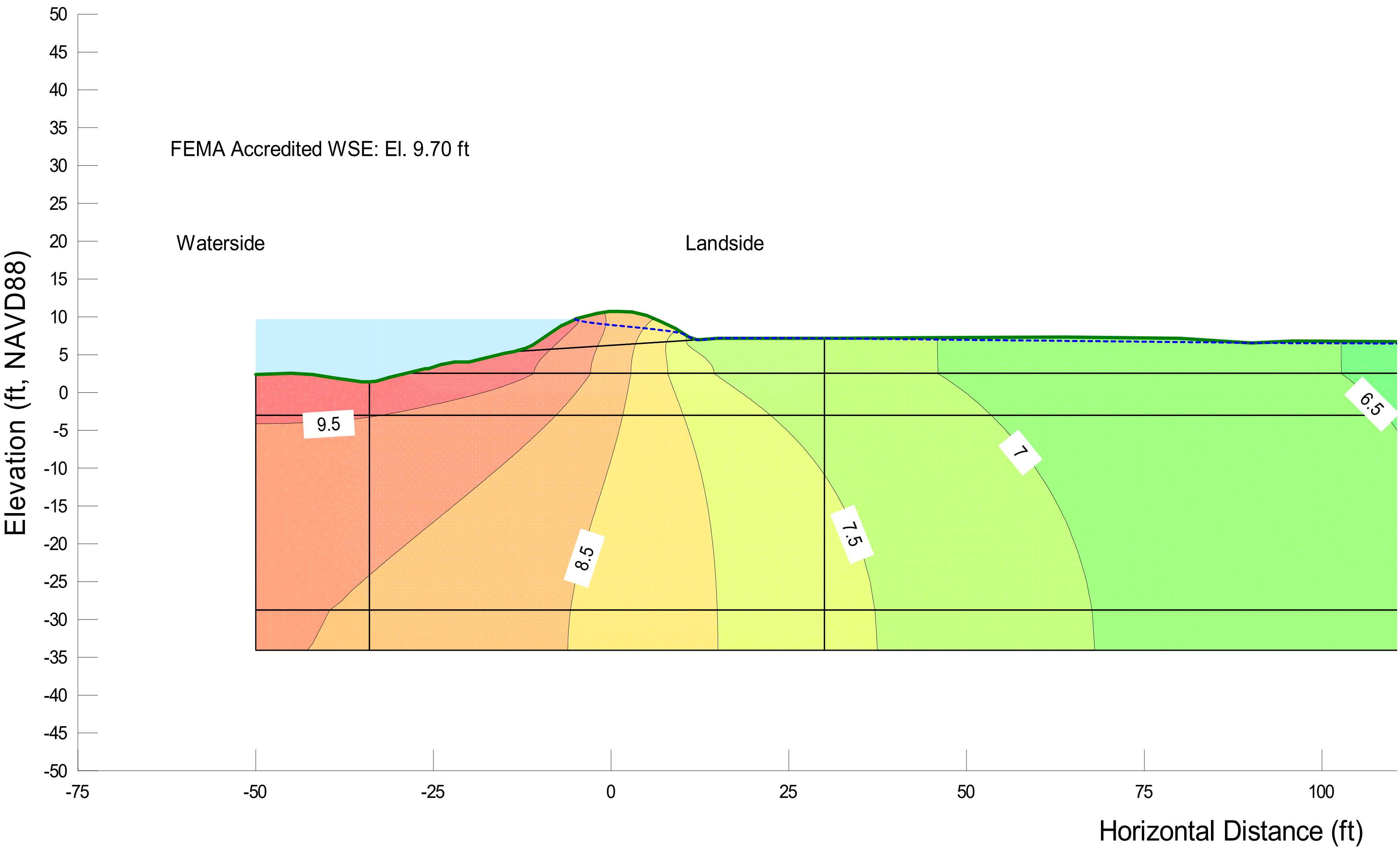
Existing Conditions
Steady State Seepage Model

January 2016

Figure 55



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions – Baseline Steady State Seepage Results
Marin County FCWCD		January 2016 Figure 56



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

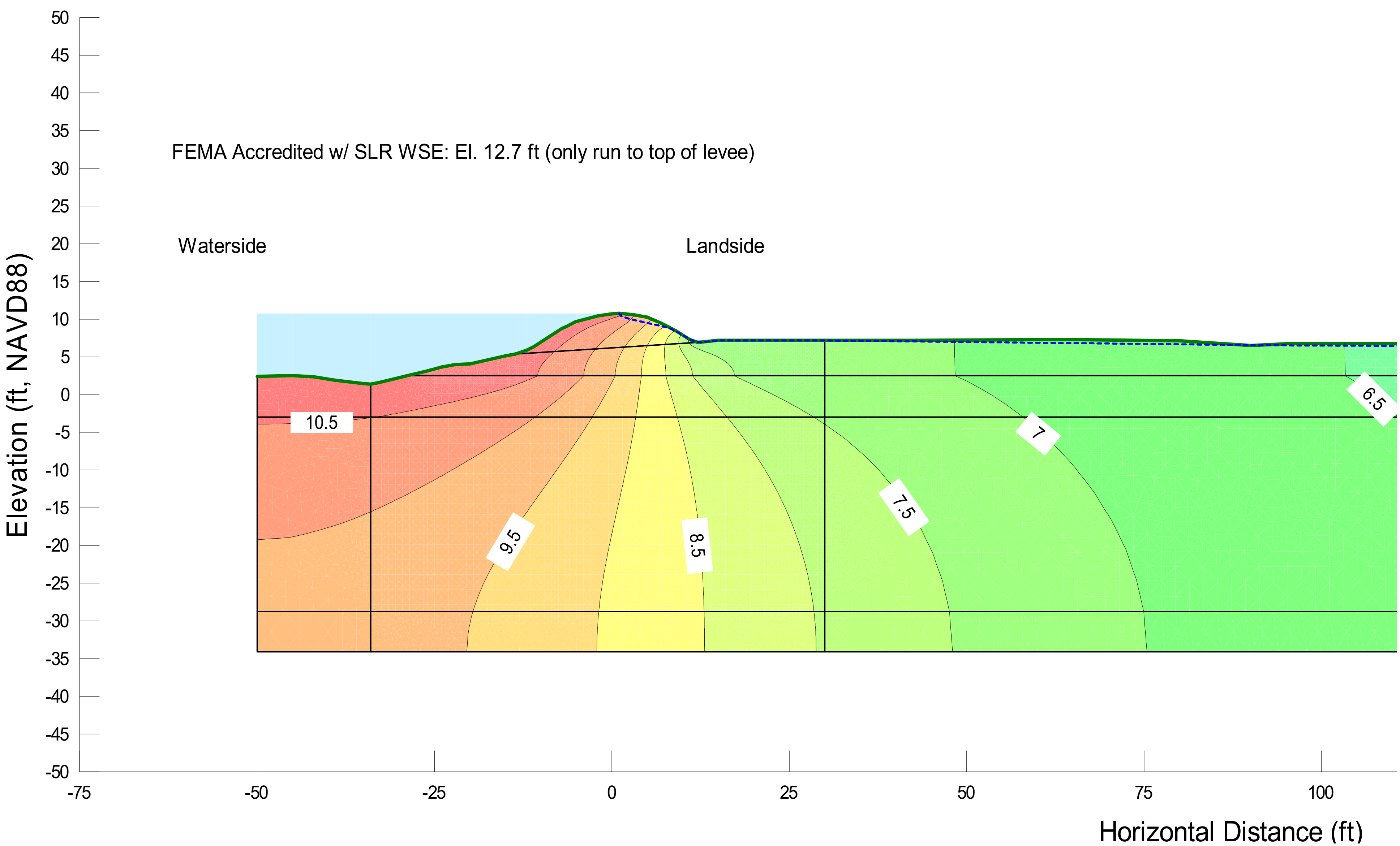
GEI Consultants

Reach 7 – Sta. 35+00 CC-R

Existing Conditions – FEMA
Steady State Seepage Results

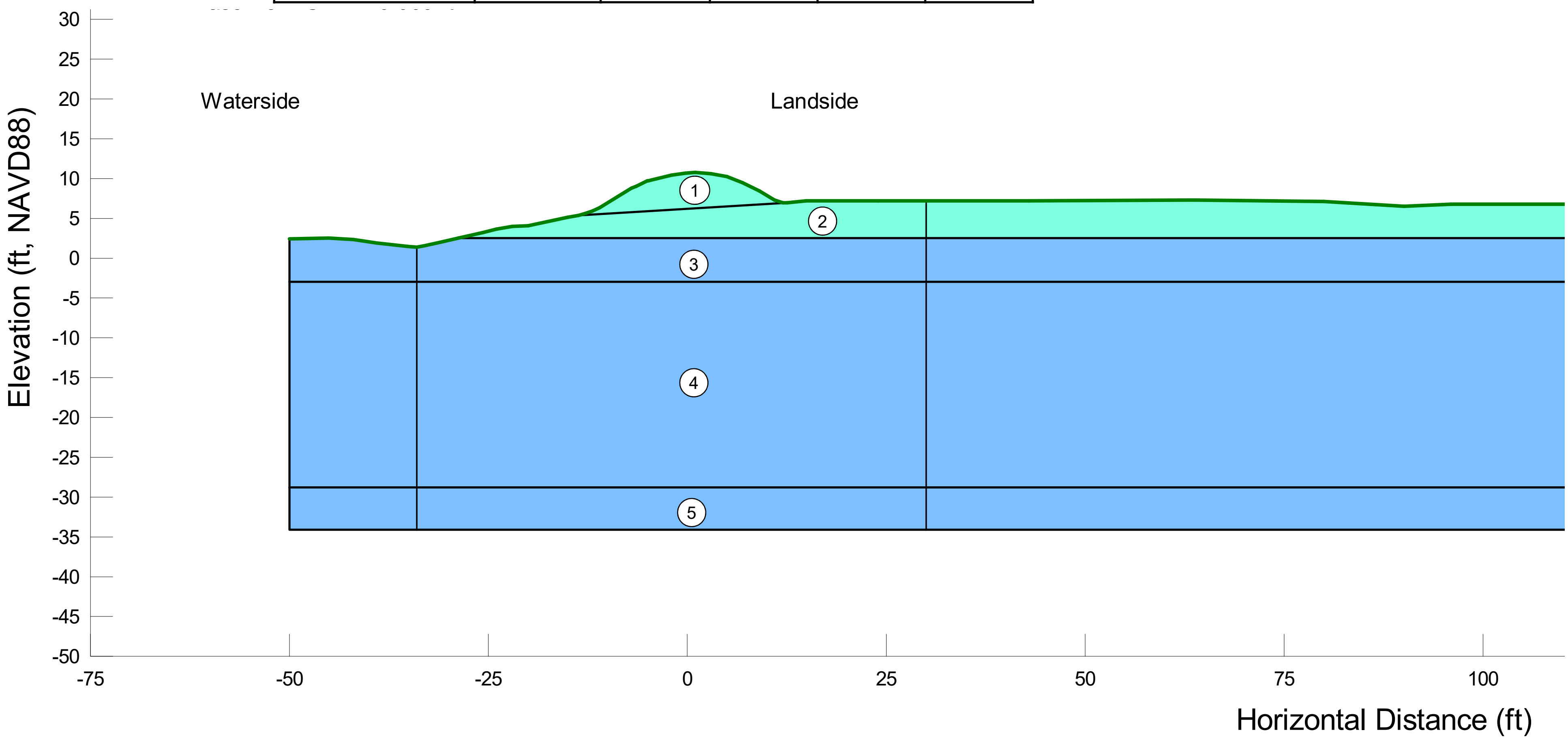
January 2016

Figure 57

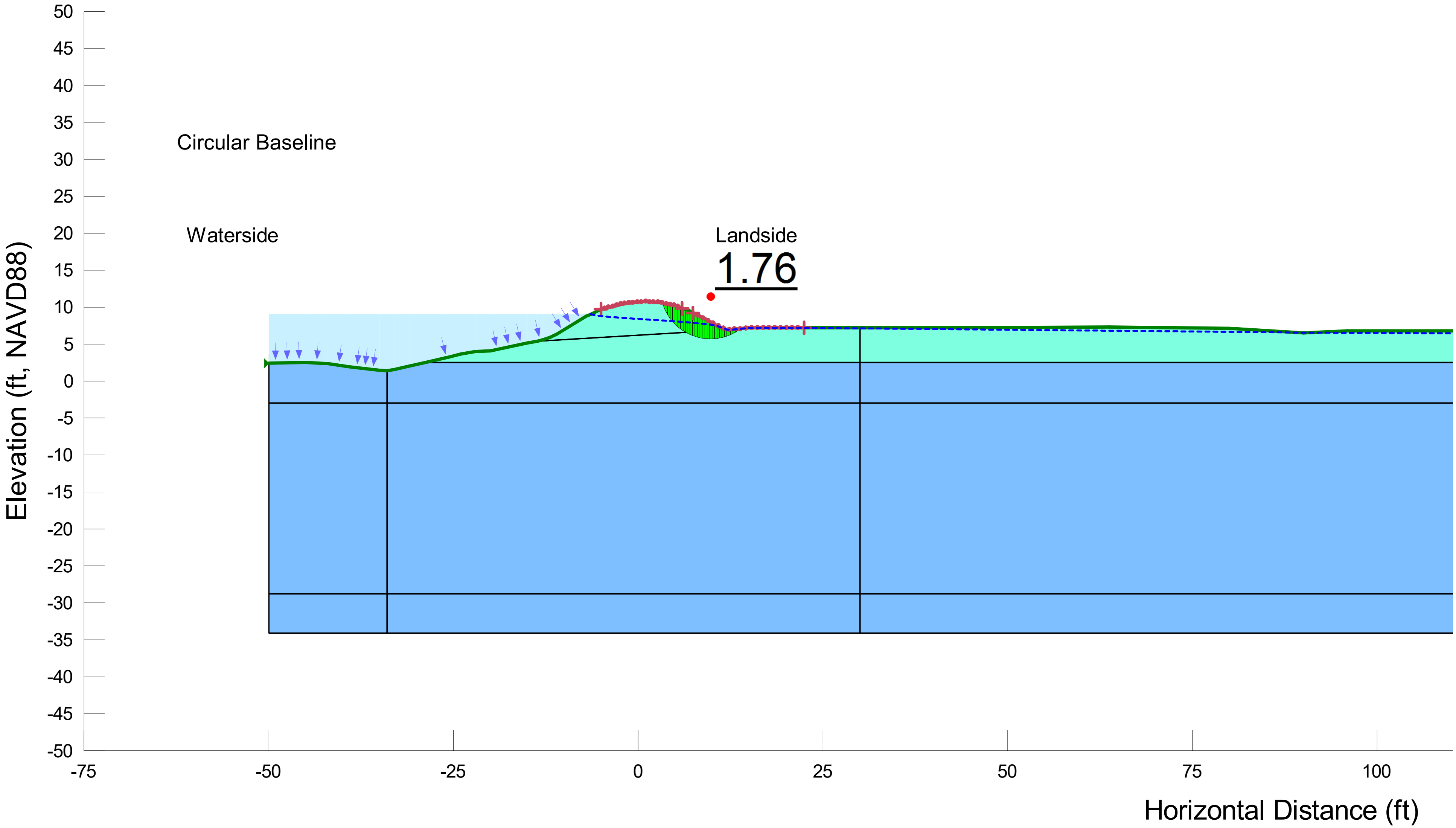


Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions–FEMA w/ SLR Steady State Seepage Results
Marin County FCWCD		January 2016 Figure 58

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c'	ϕ'	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
(1) SC	120	0	38	--	--
(2) SC	120	0	36	--	--
(3) CH	95	0	30	350	0
(4) CH	95	0	28	170	8
(5) Sandy CH	95	0	30	210	9



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions Stability Model
Marin County FCWCD		January 2016 Figure 59



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

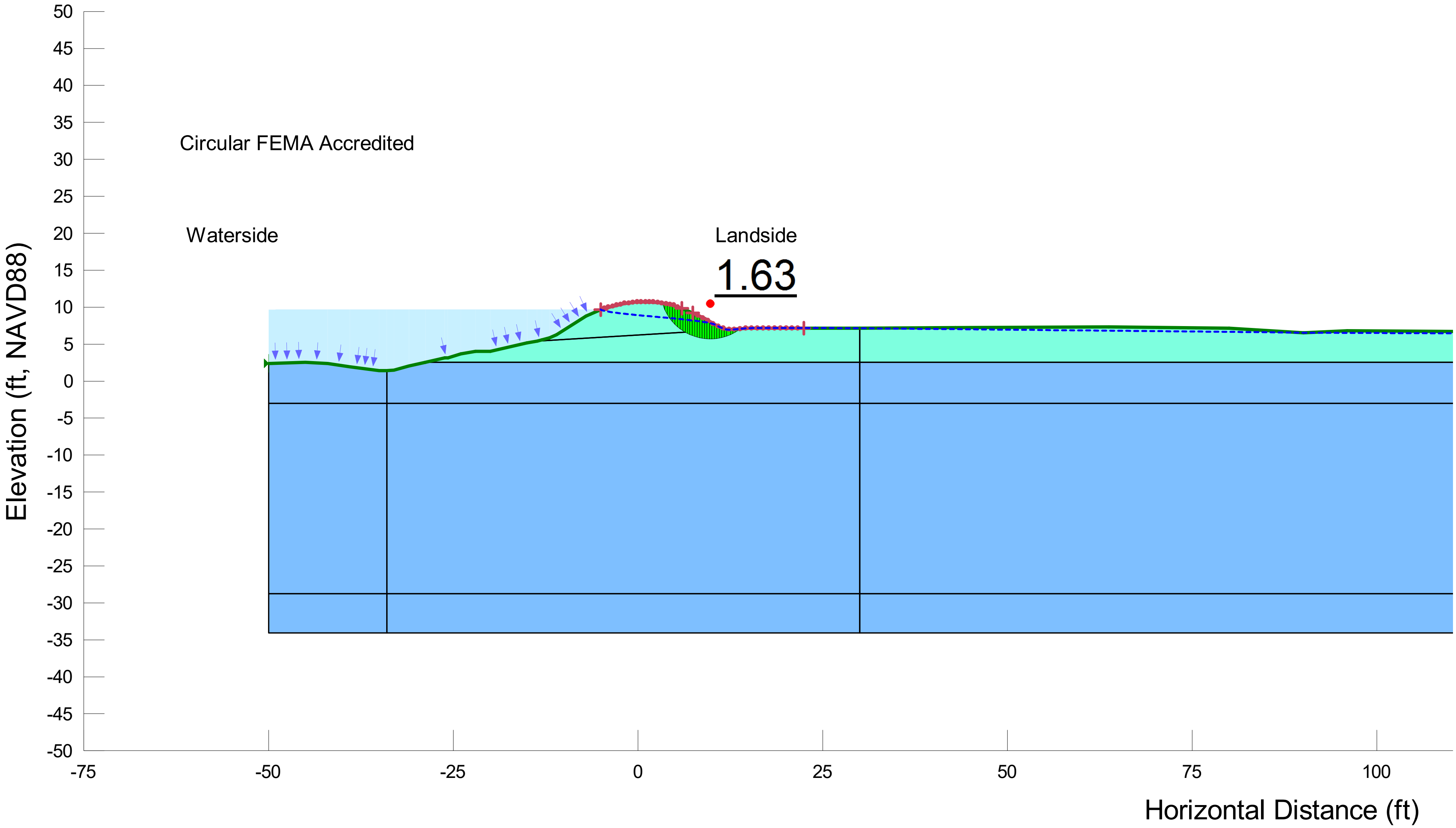
GEI Consultants

Reach 7 – Sta. 35+00 CC-R

Existing Conditions – Baseline
Landside Stability Results

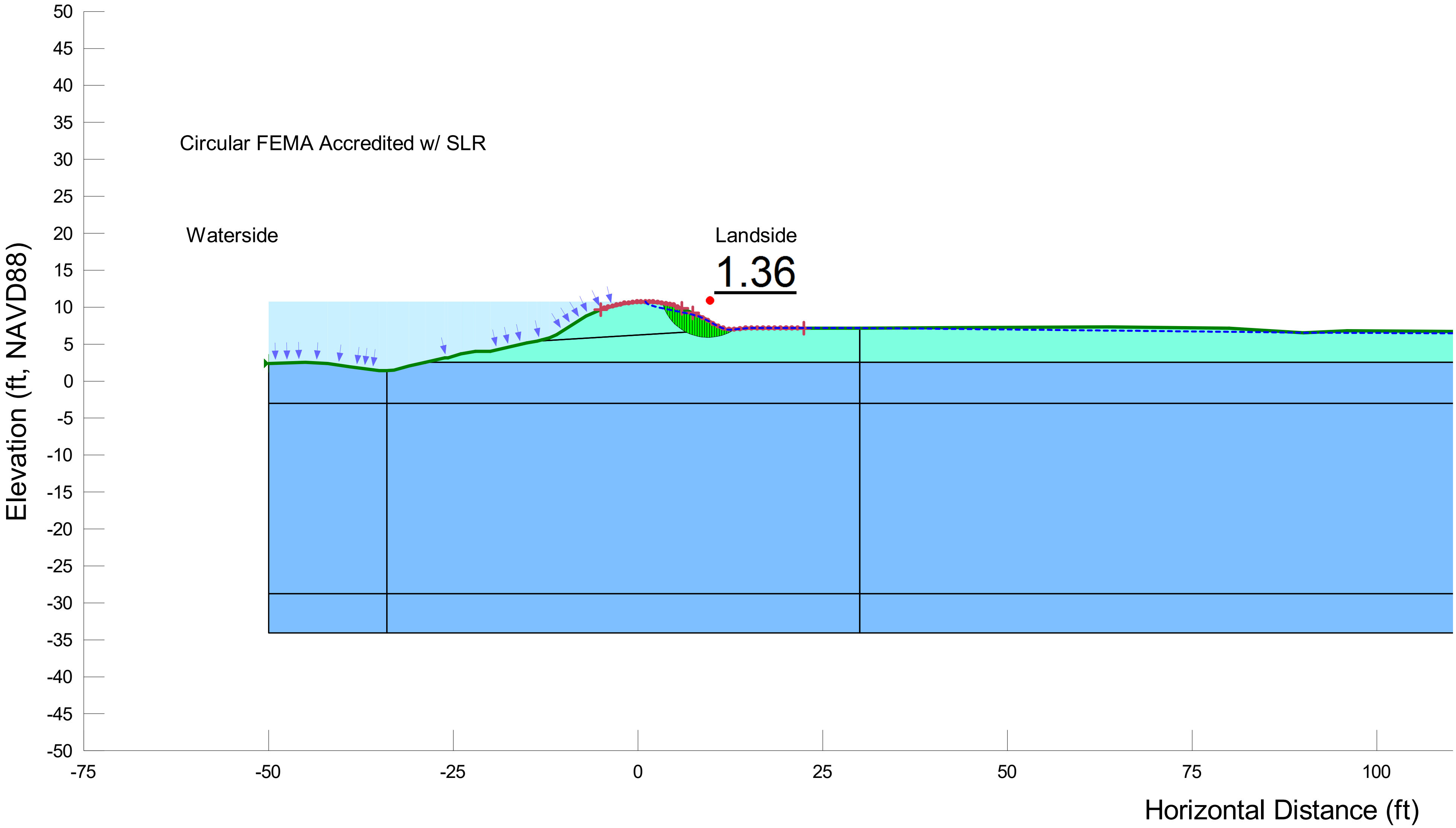
January 2016

Figure 60



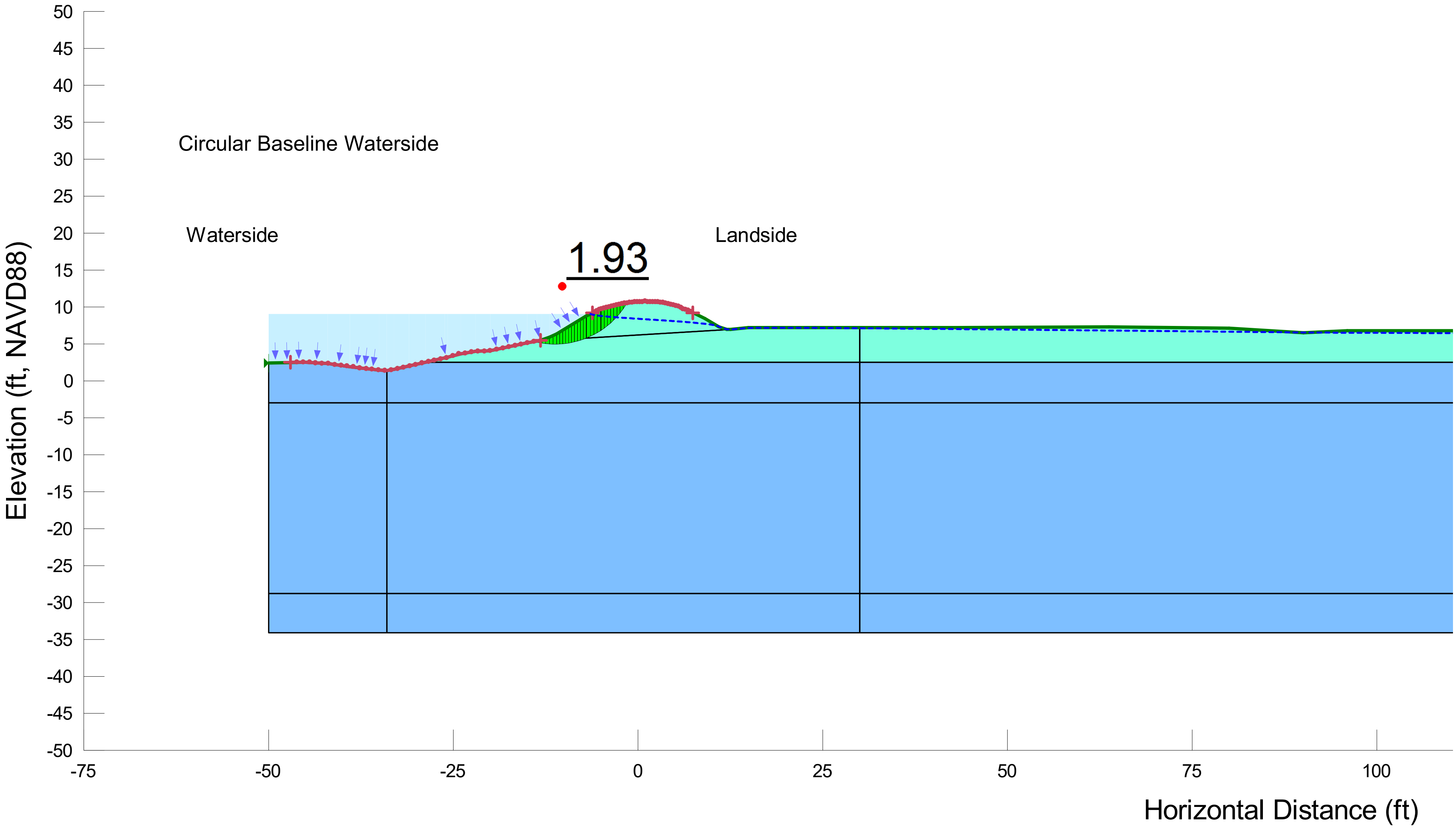
Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions – FEMA Landside Stability Results
Marin County FCWCD		January 2016 Figure 61



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions–FEMA w/ SLR Landside Stability Results
Marin County FCWCD		January 2016 Figure 62



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

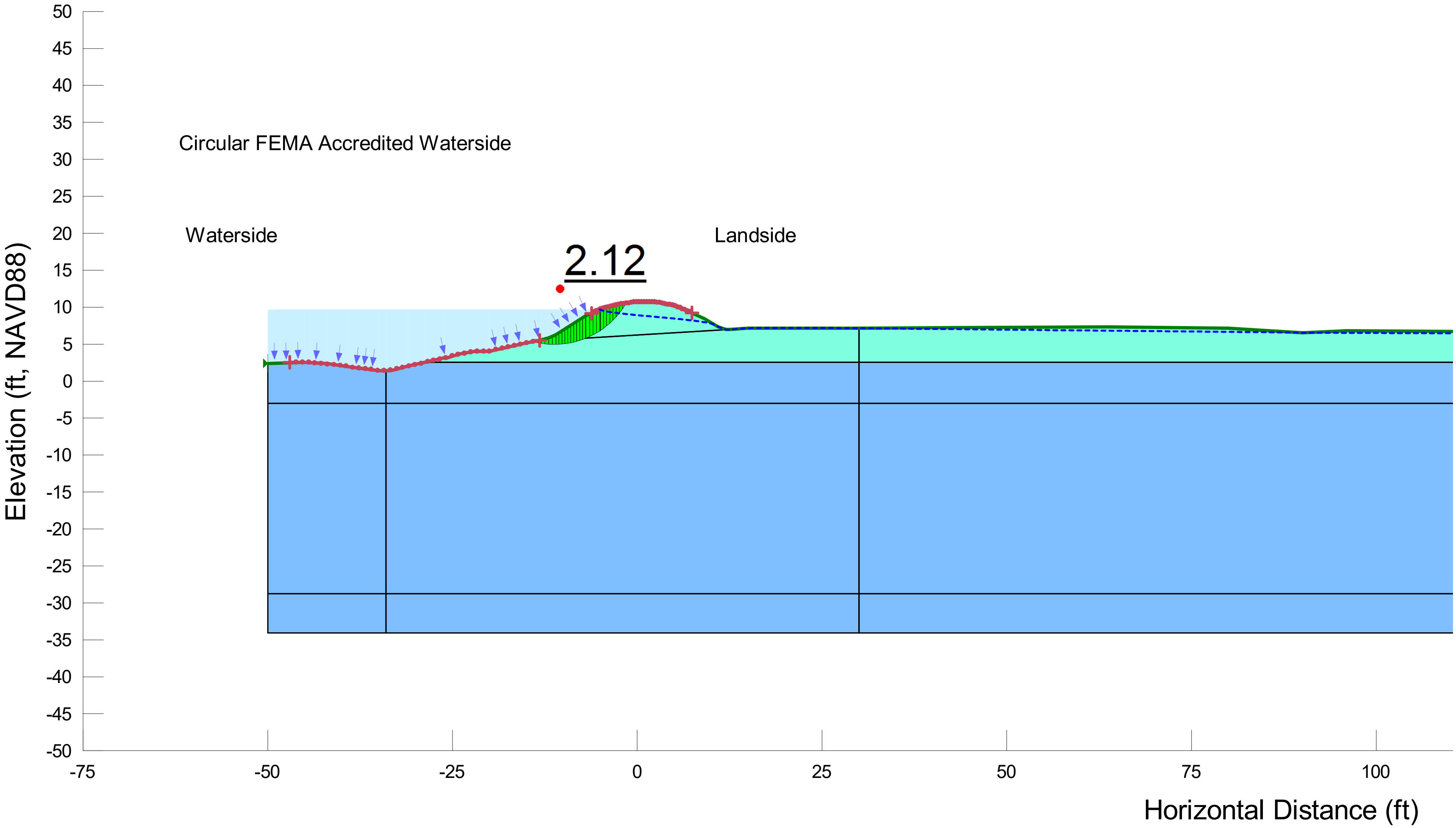
GEI Consultants

Reach 7 – Sta. 35+00 CC-R

Existing Conditions – Baseline Waterside Stability Results

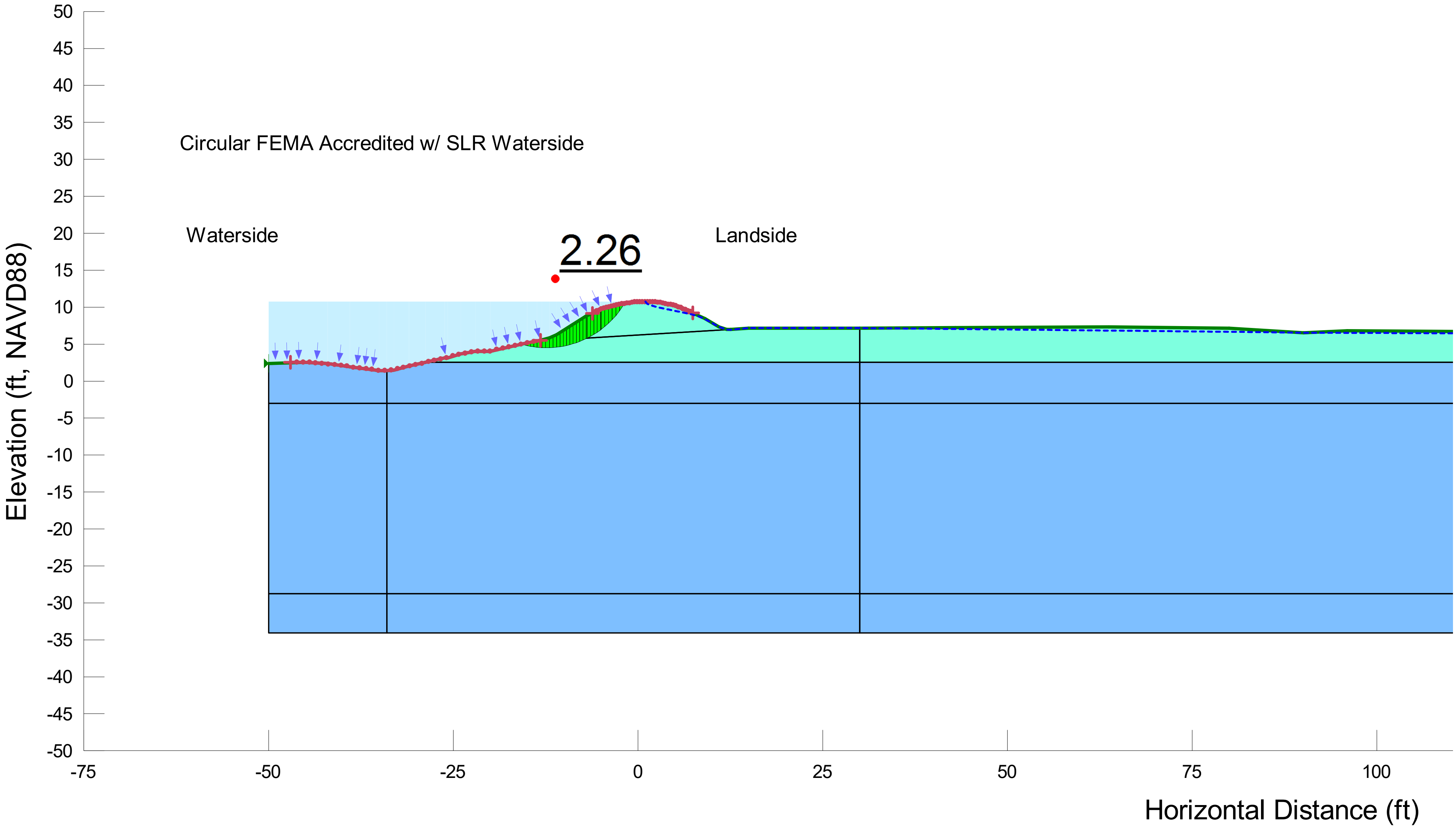
January 2016

Figure 63



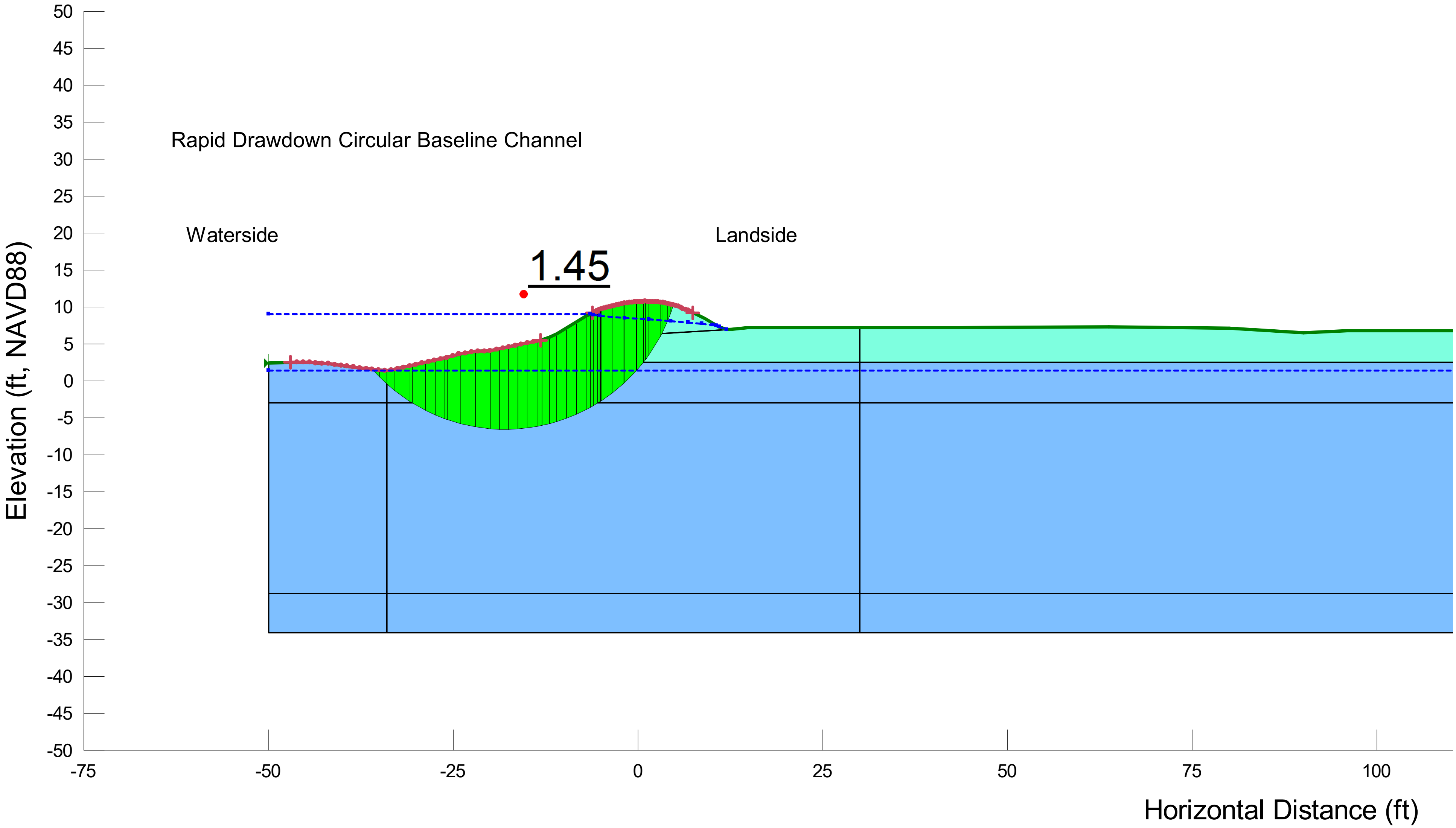
Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions – FEMA Waterside Stability Results
Marin County FCWCD		January 2016 Figure 64



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions–FEMA w/ SLR Waterside Stability Results
Marin County FCWCD		January 2016 Figure 65



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

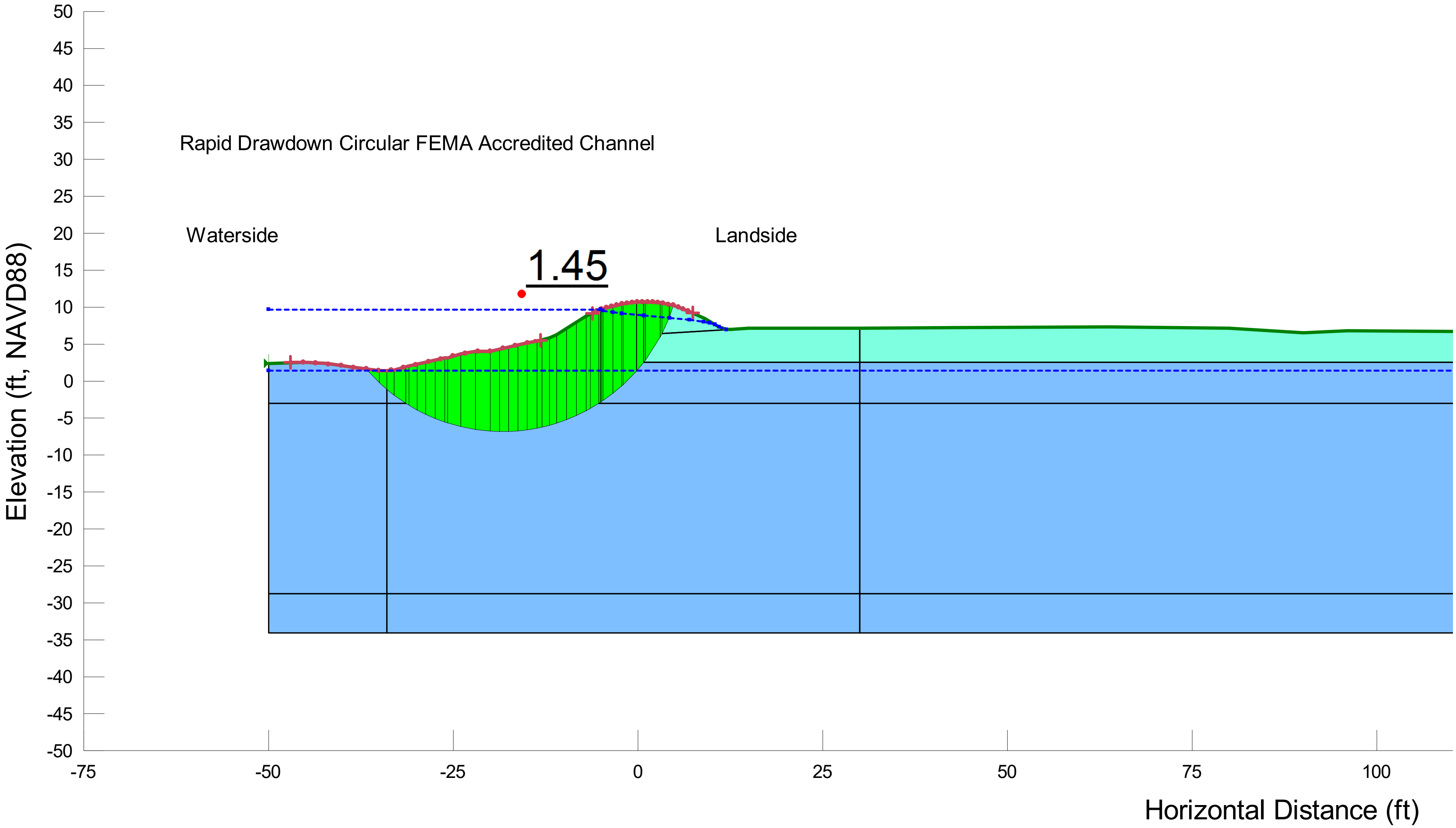
GEI Consultants

Reach 7 – Sta. 35+00 CC-R

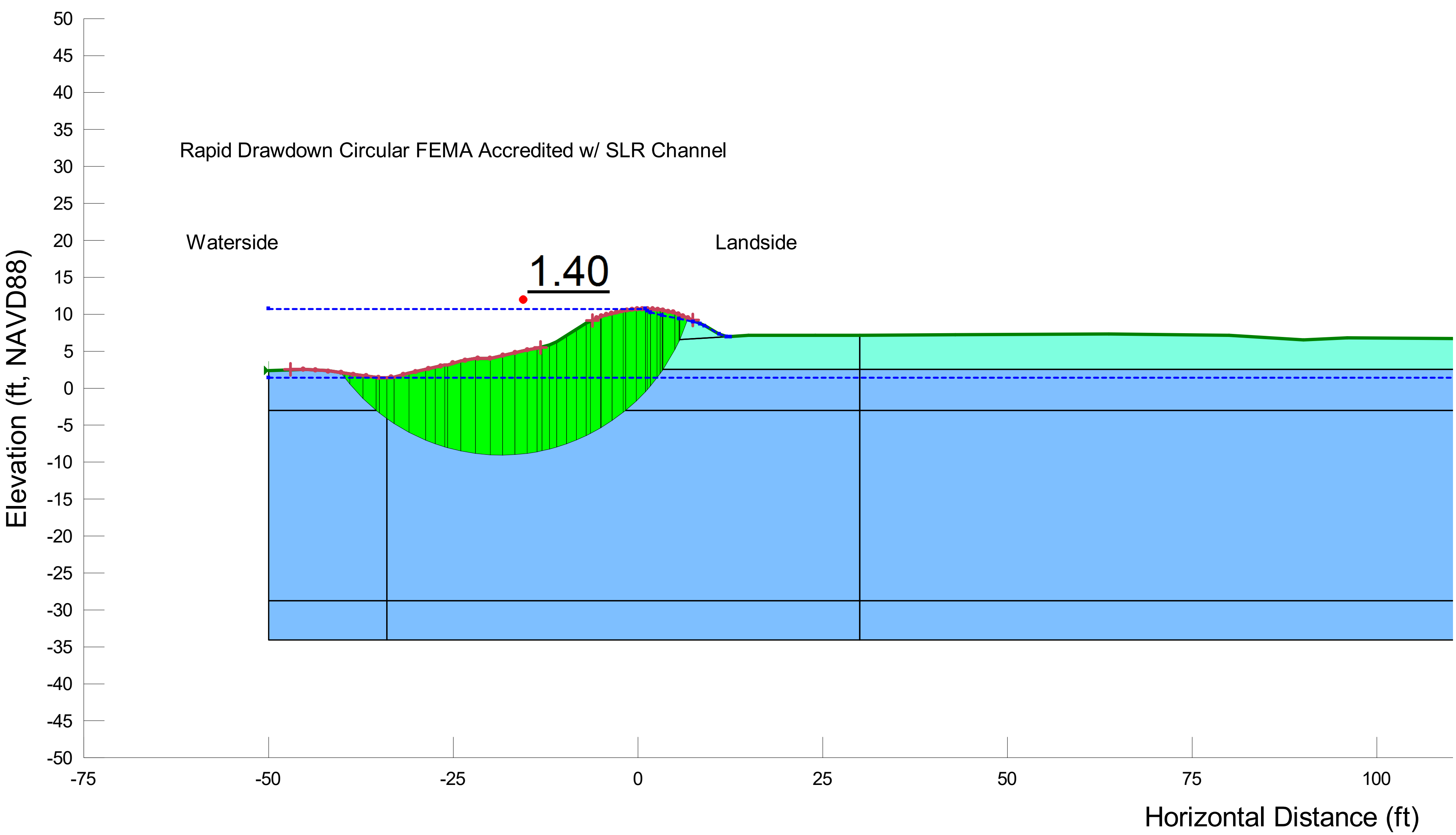
Existing Conditions – Baseline
Total Flood Drawdown Results

January 2016

Figure 66



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions – FEMA Total Flood Drawdown Results
Marin County FCWCD		January 2016 Figure 67



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

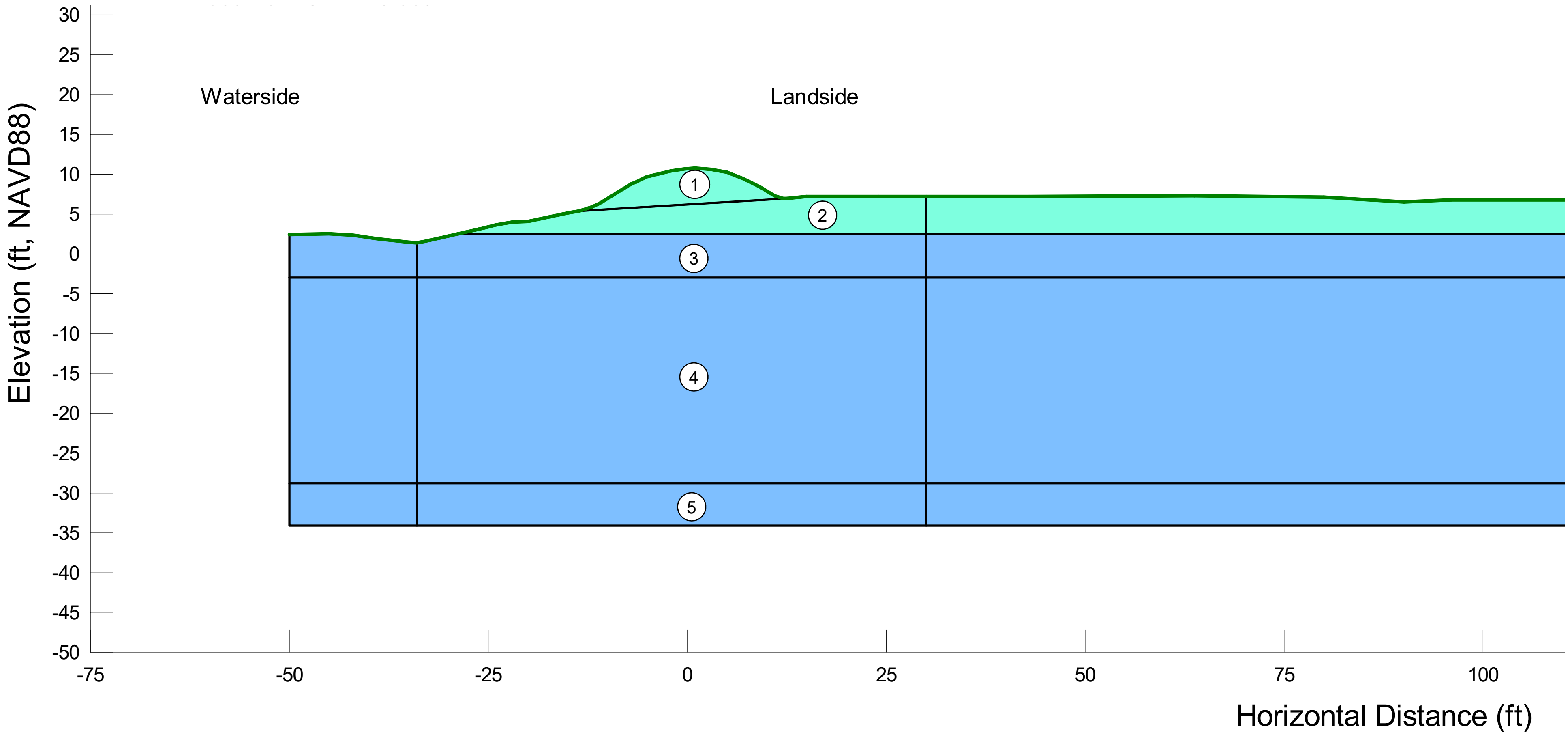
GEI Consultants

Reach 7 – Sta. 35+00 CC-R

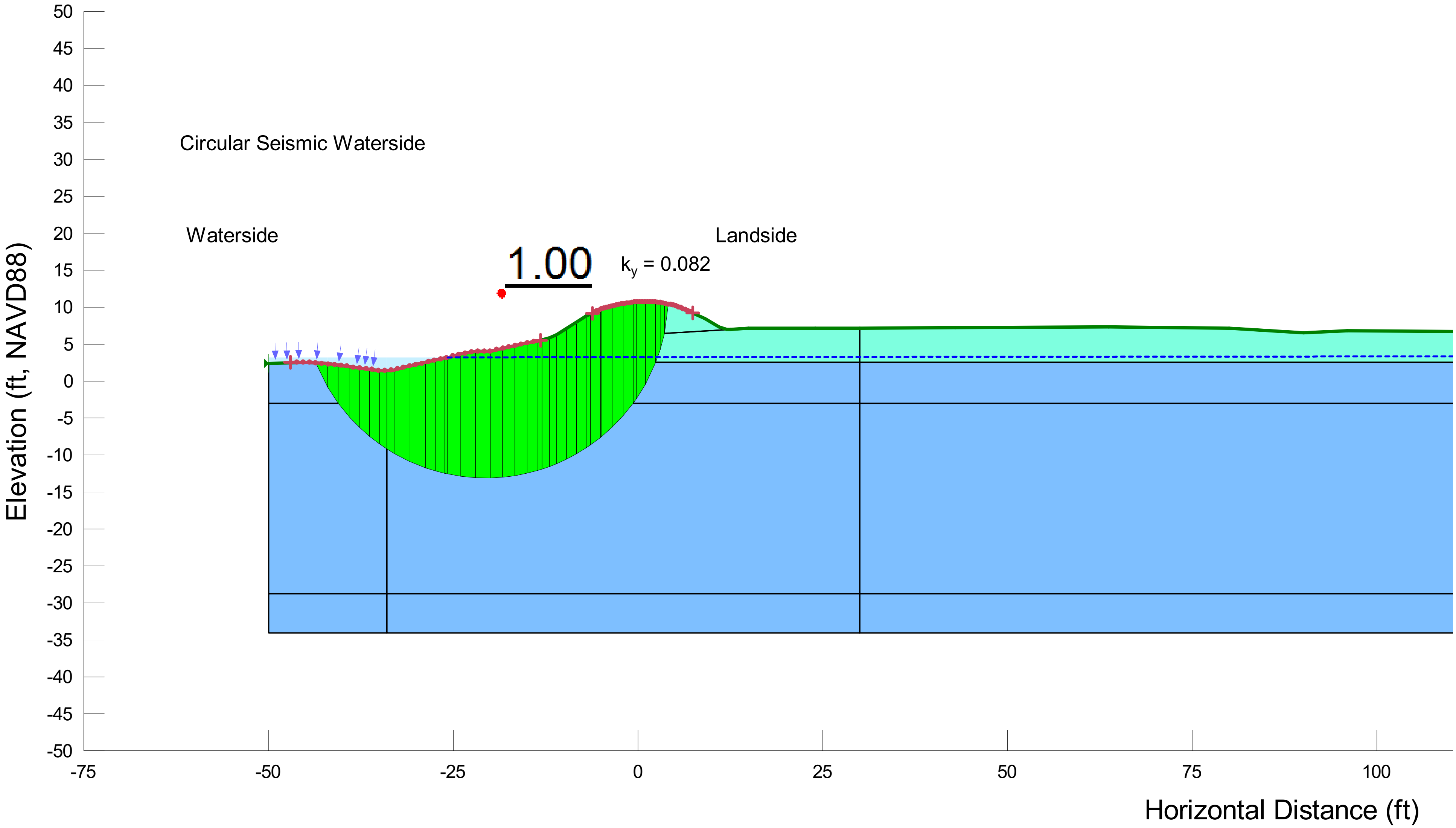
Existing Conditions – FEMA w/ SLR
Total Flood Drawdown Results

January 2016 Figure 68

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c’	ϕ’	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
(1) SC	120	0	38	--	--
(2) SC	120	0	36	--	--
(3) CH	95	0	30	280	0
(4) CH	95	0	28	136	8
(5) Sandy CH	95	0	30	210	9



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 7 – Sta. 35+00 CC-R** Existing Conditions – Seismic Model
Marin County FCWCD		January 2016 Figure 69



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

GEI Consultants

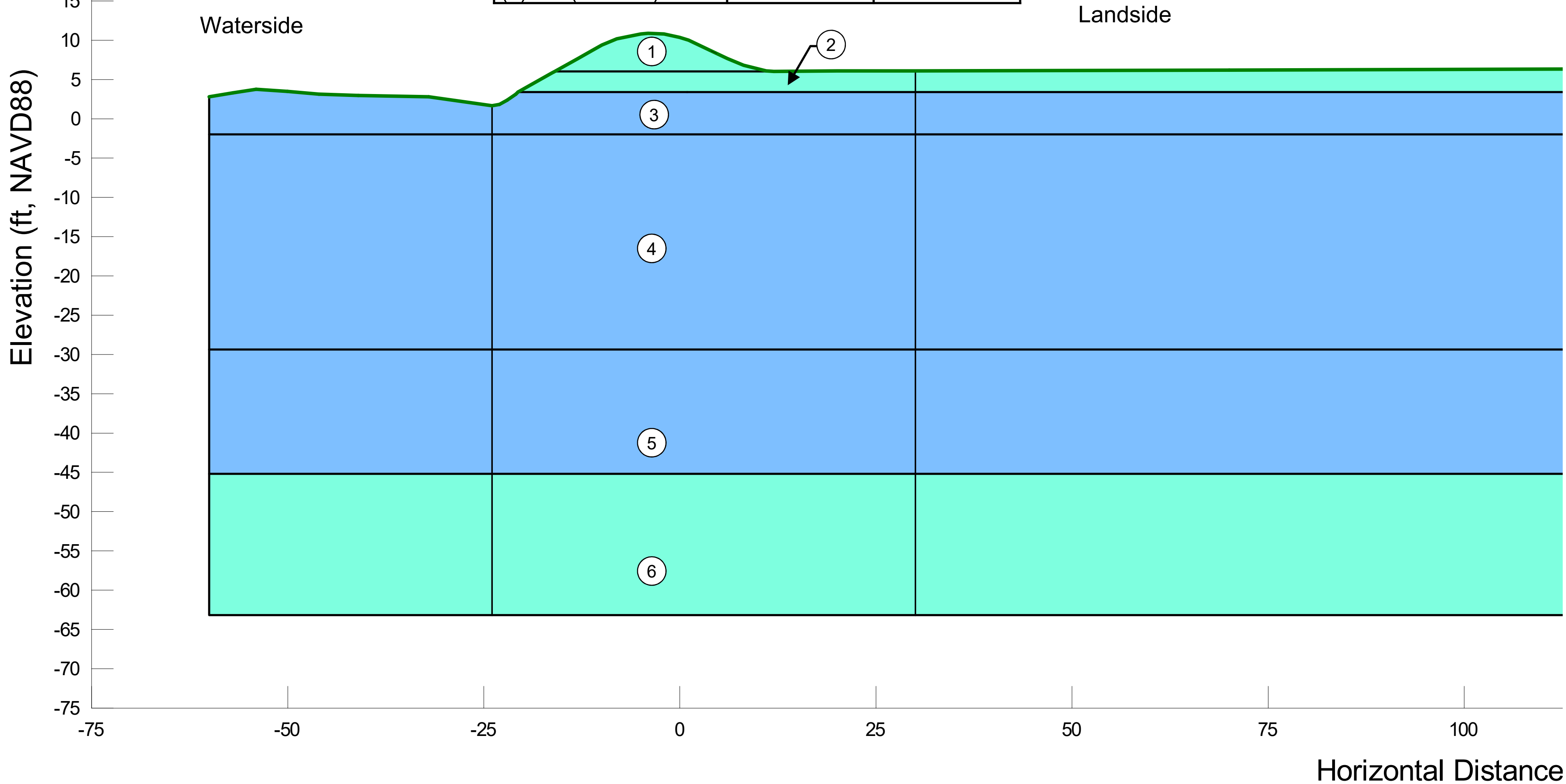
Reach 7 – Sta. 35+00 CC-R

Existing Conditions – Seismic Analysis Results

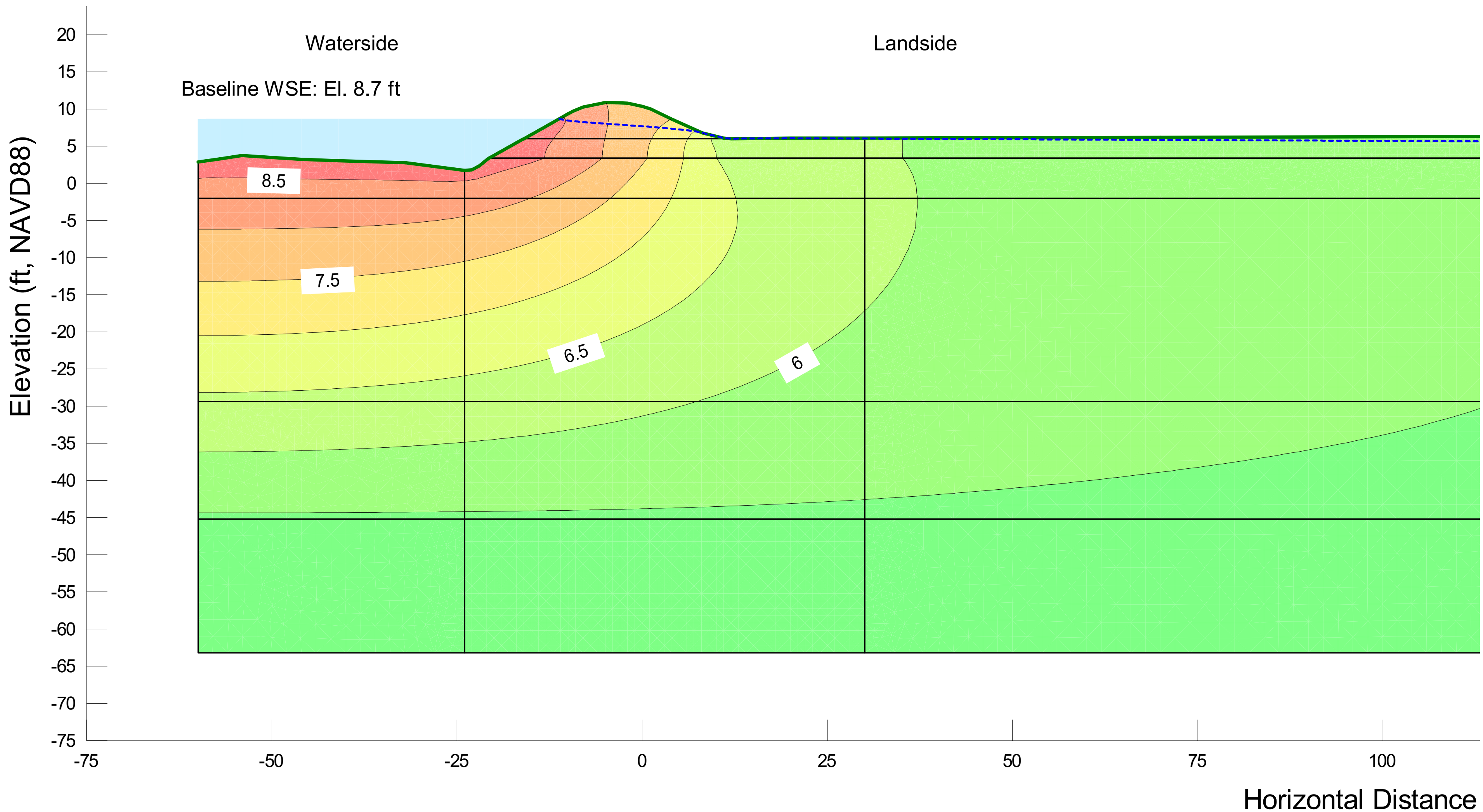
January 2016

Figure 70

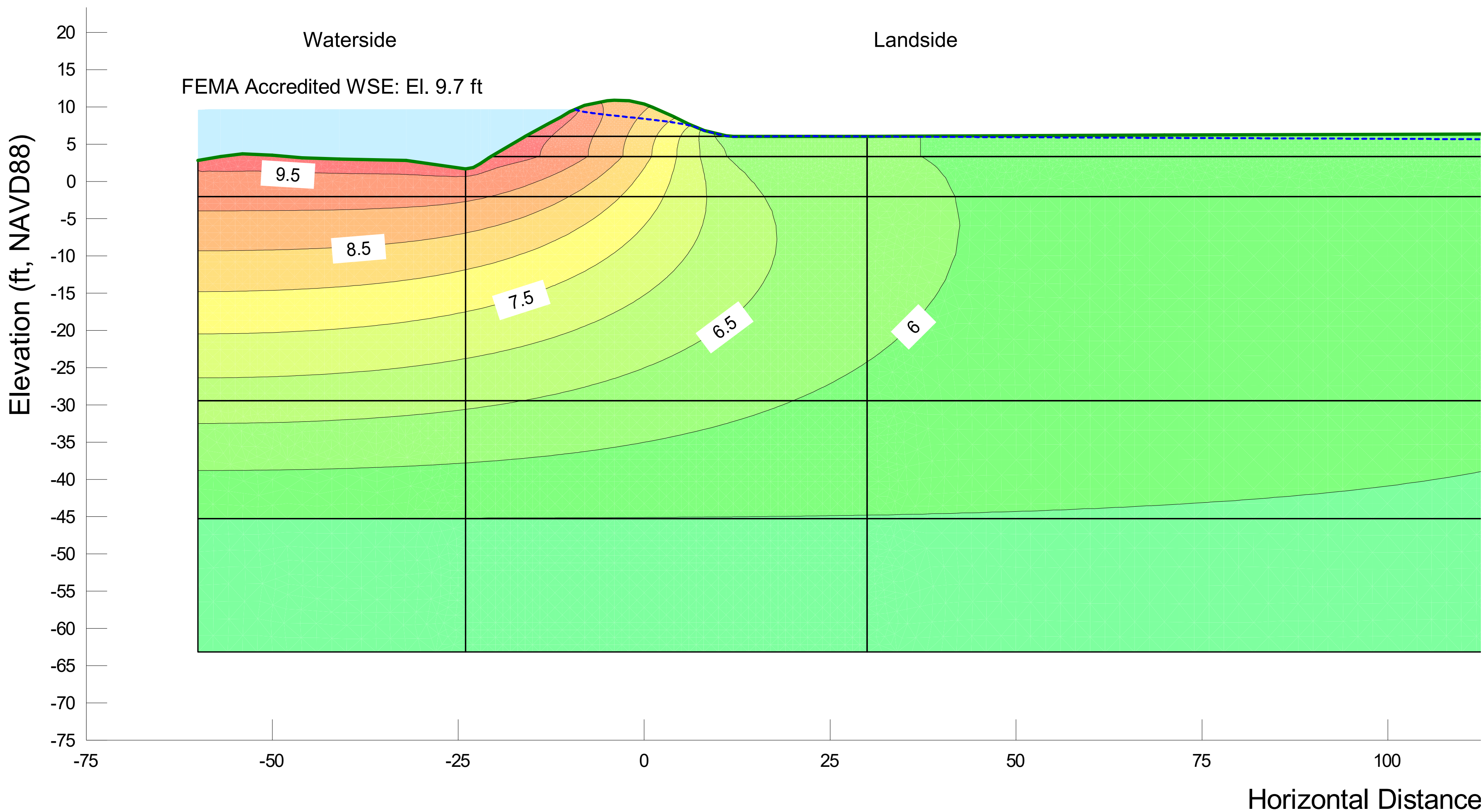
Layer Name	k_v (cm/sec)	k_h (cm/sec)
(1) SC (12-25%)	$4.0x10^{-5}$	$1.6x10^{-4}$
(2) SC (12-25%)	$4.0x10^{-5}$	$1.6x10^{-4}$
(3) CH	$1.0x10^{-7}$	$2.0x10^{-7}$
(4) CH	$1.0x10^{-7}$	$2.0x10^{-7}$
(5) Sandy CH	$1.0x10^{-7}$	$2.0x10^{-7}$
(6) SC (12-25%)	$4.0x10^{-5}$	$1.6x10^{-4}$



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 9 – Sta. 1+00 NC-L** Existing Conditions Steady State Seepage Model
Marin County FCWCD		January 2016 Figure 71



Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 9 – Sta. 1+00 NC-L** Existing Conditions – Baseline Steady State Seepage Results
Marin County FCWCD		January 2016 Figure 72



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

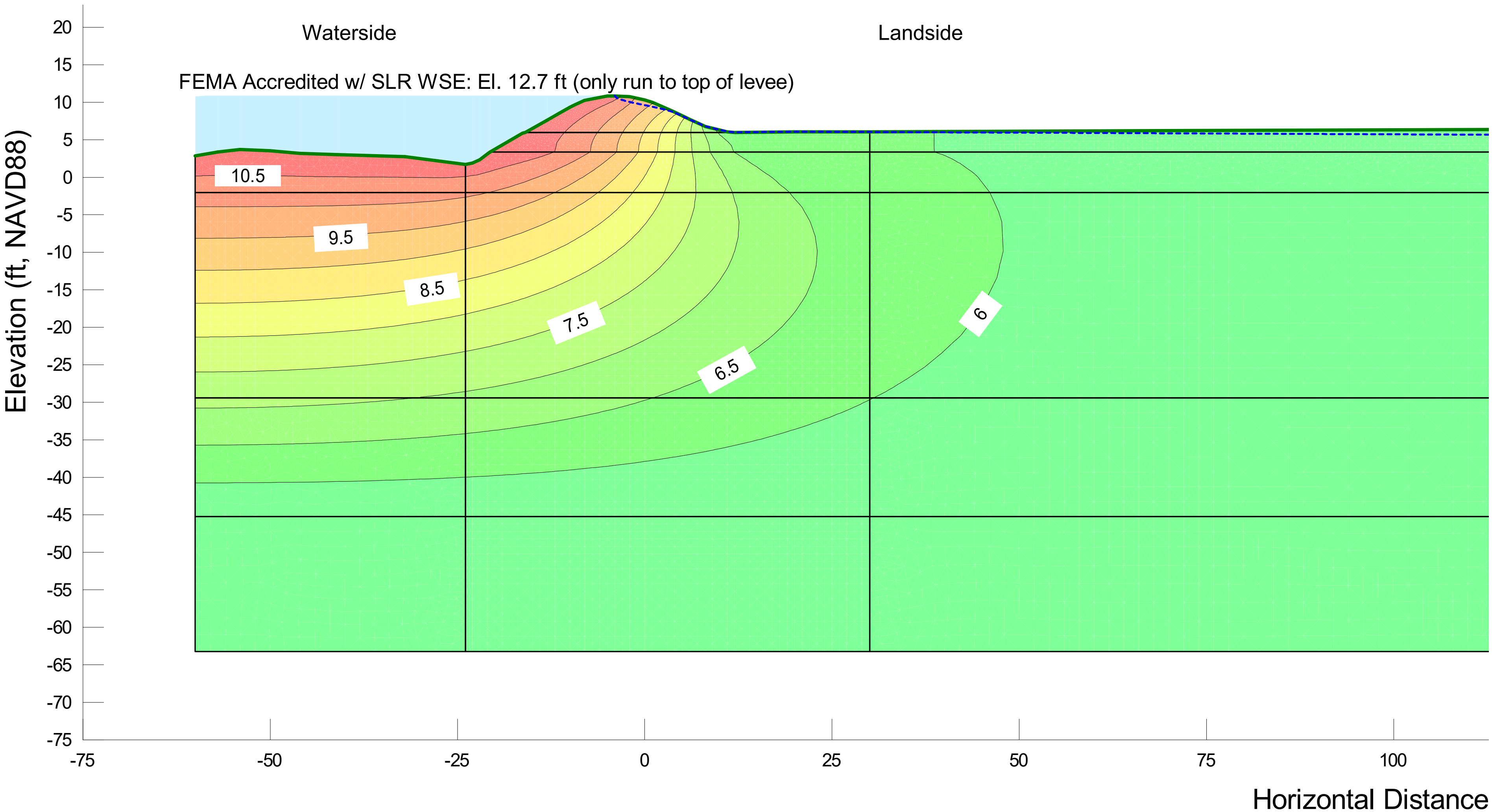
GEI Consultants

Reach 9 – Sta. 1+00 NC-L

Existing Conditions – FEMA
Steady State Seepage Results

January 2016

Figure 73



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

GEI Consultants

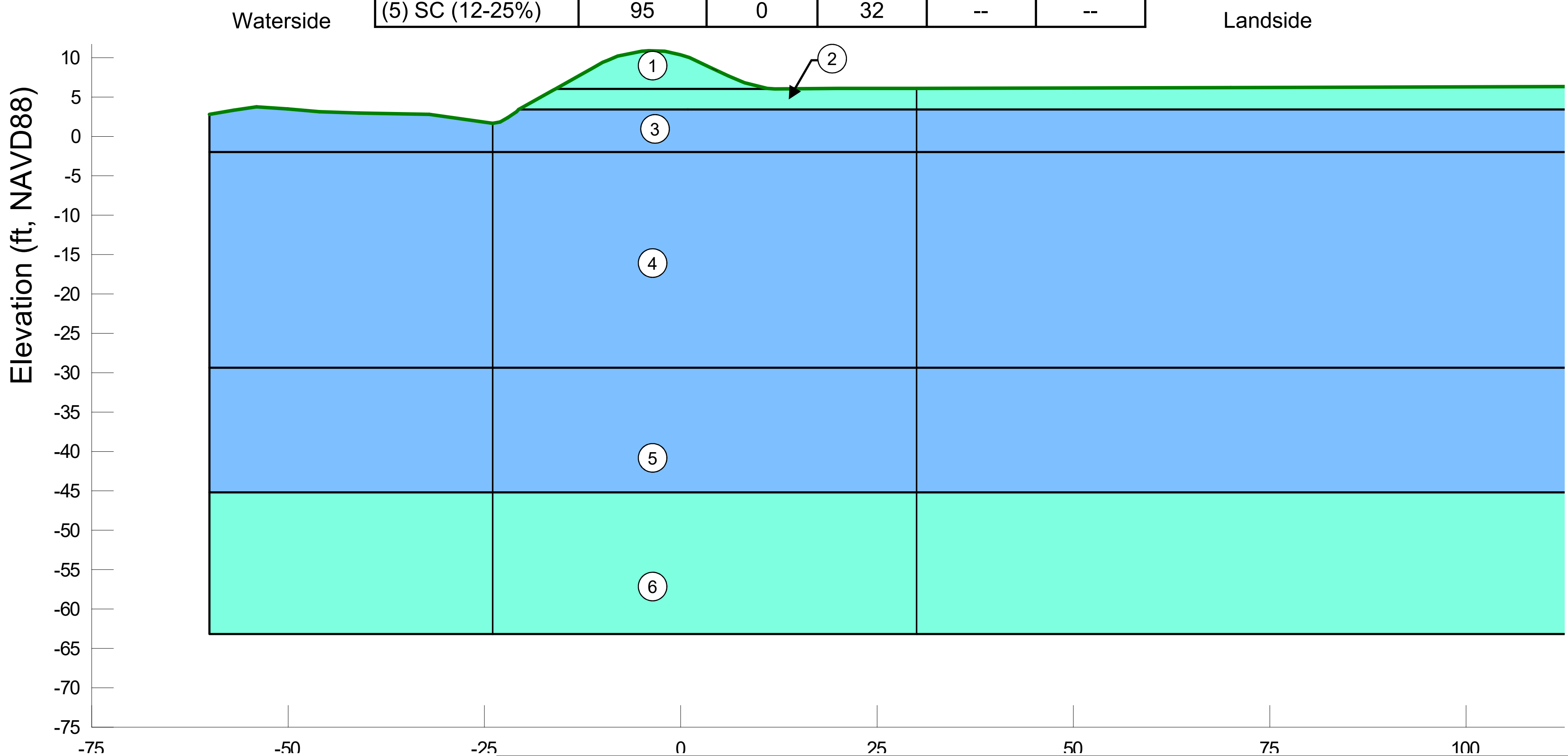
Reach 9 – Sta. 1+00 NC-L

Existing Conditions–FEMA w/ SLR
Steady State Seepage Results

January 2016

Figure 74

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c'	ϕ'	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
(1) SC (12-25%)	120	0	38	--	--
(2) SC (12-25%)	120	0	36	--	--
(3) CH	95	0	30	350	0
(3) CH	95	0	28	170	8
(4) Sandy CH	95	0	30	210	9
(5) SC (12-25%)	95	0	32	--	--



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

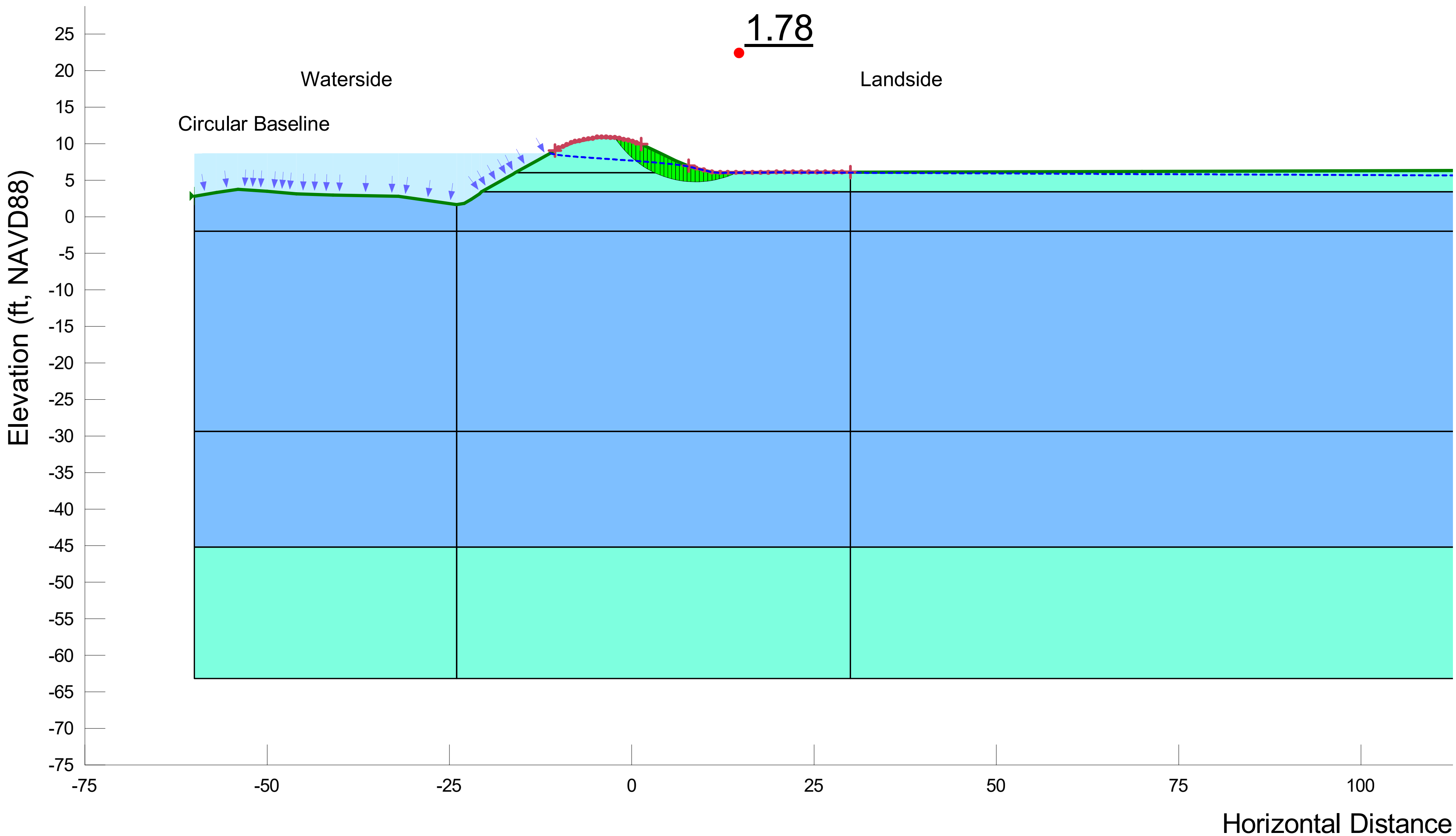
GEI Consultants

Reach 9 – Sta. 1+00 NC-L

Existing Conditions
Stability Model

January 2016

Figure 75



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

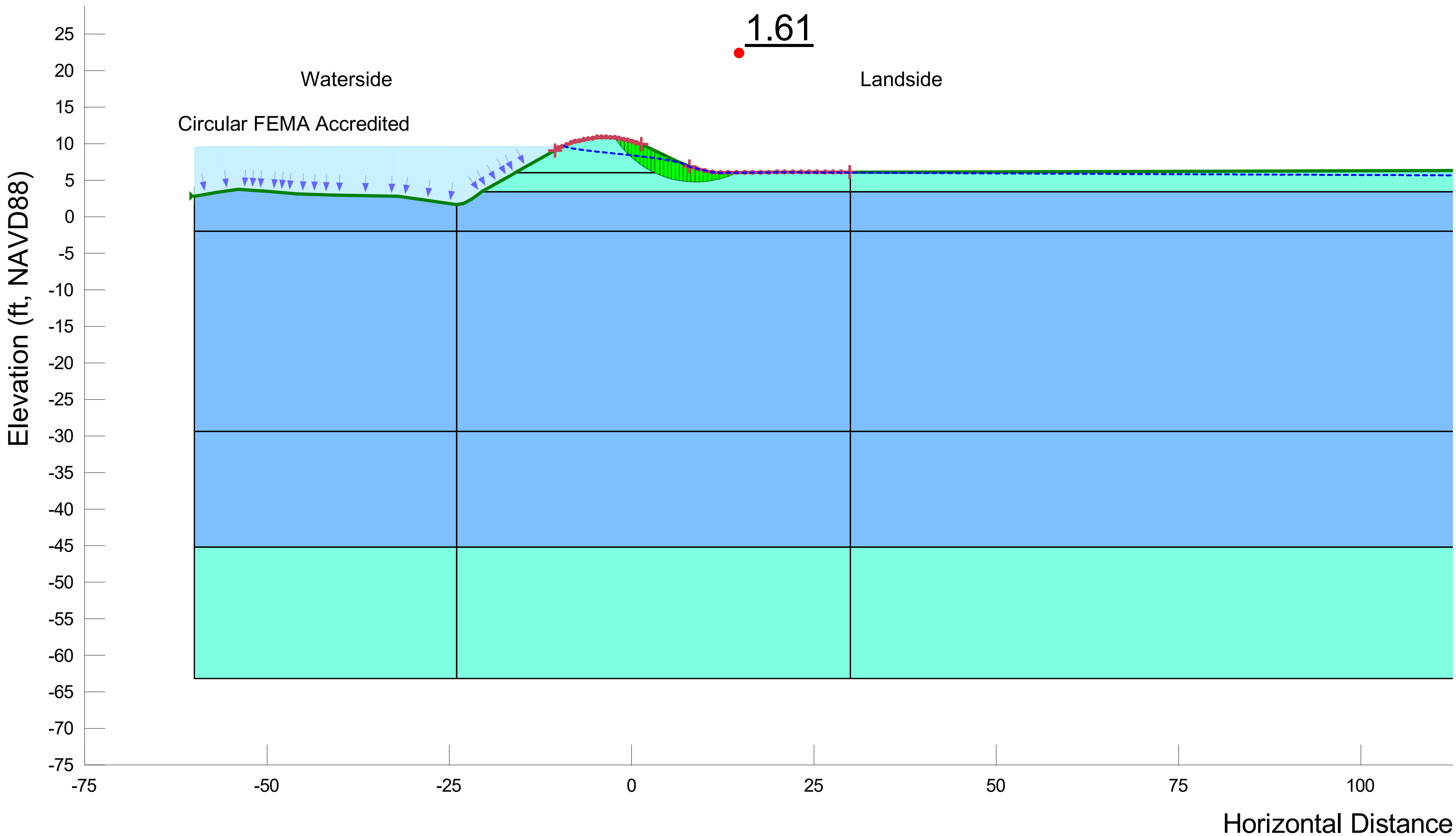
GEI Consultants

Reach 9 – Sta. 1+00 NC-L

Existing Conditions – Baseline
Landside Stability Results

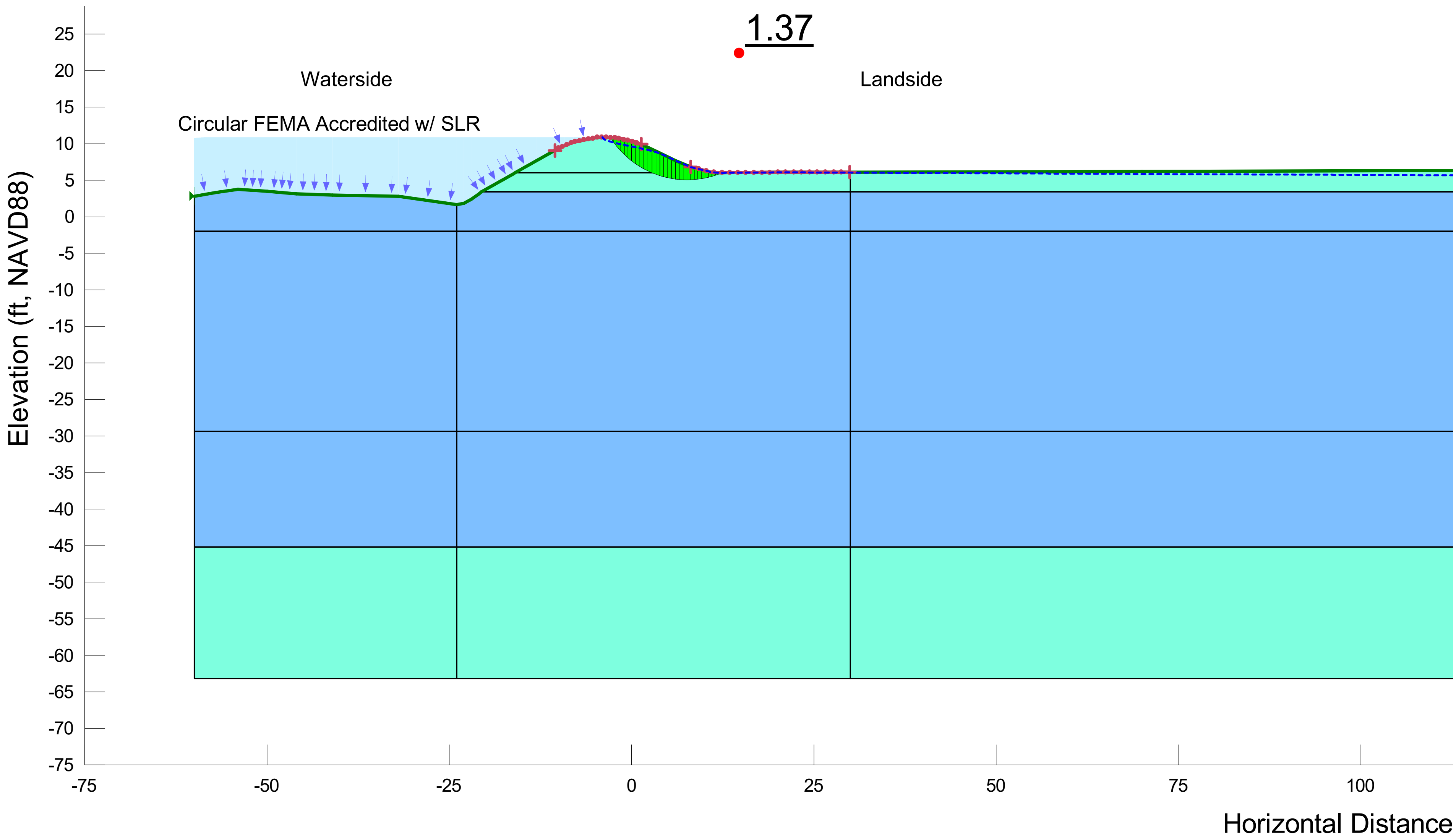
January 2016

Figure 76



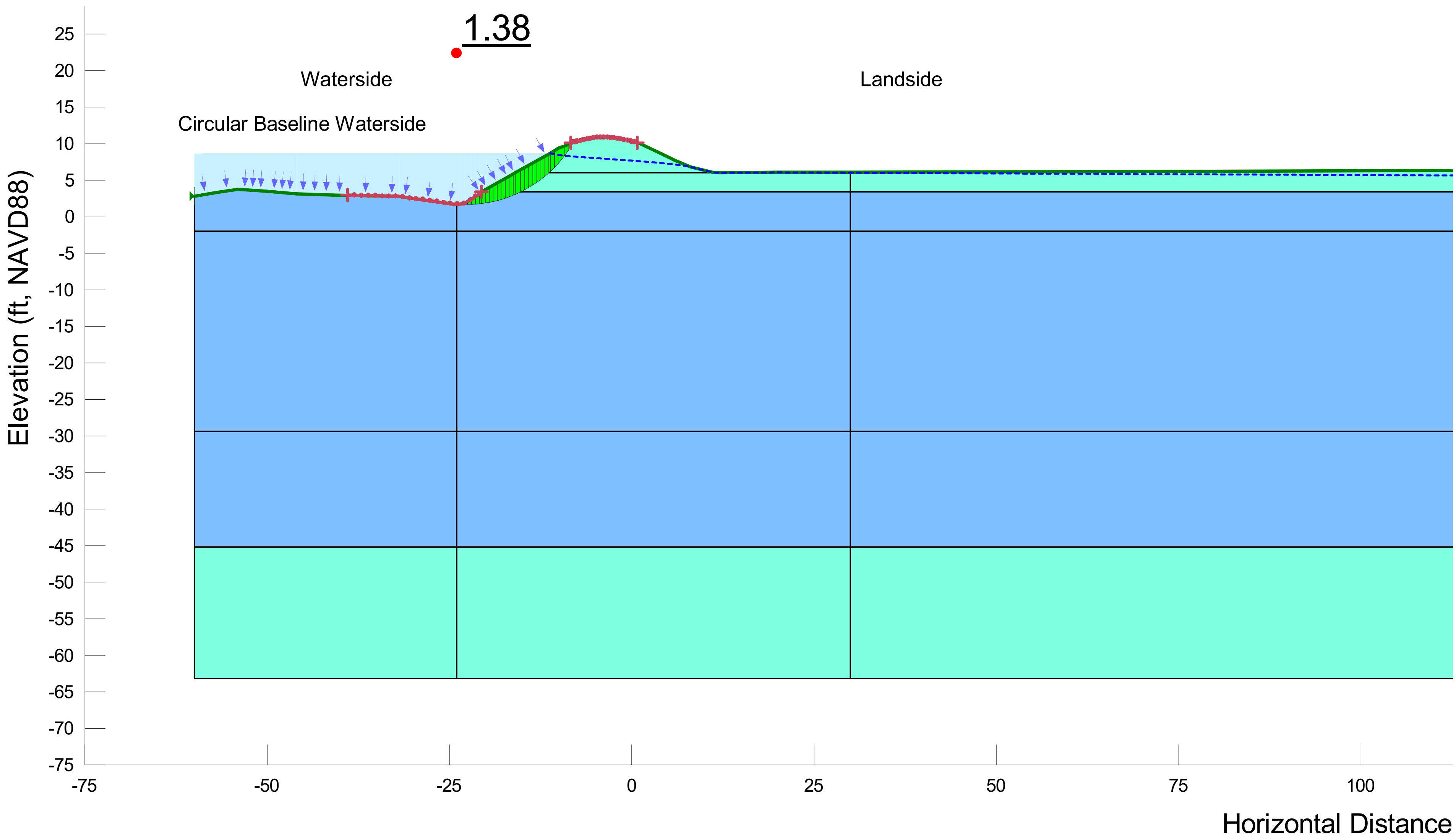
Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 9 – Sta. 1+00 NC-L** Existing Conditions – FEMA Landside Stability Results
Marin County FCWCD		January 2016 Figure 77



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 9 – Sta. 1+00 NC-L** Existing Conditions–FEMA w/ SLR Landside Stability Results
Marin County FCWCD		January 2016 Figure 78



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

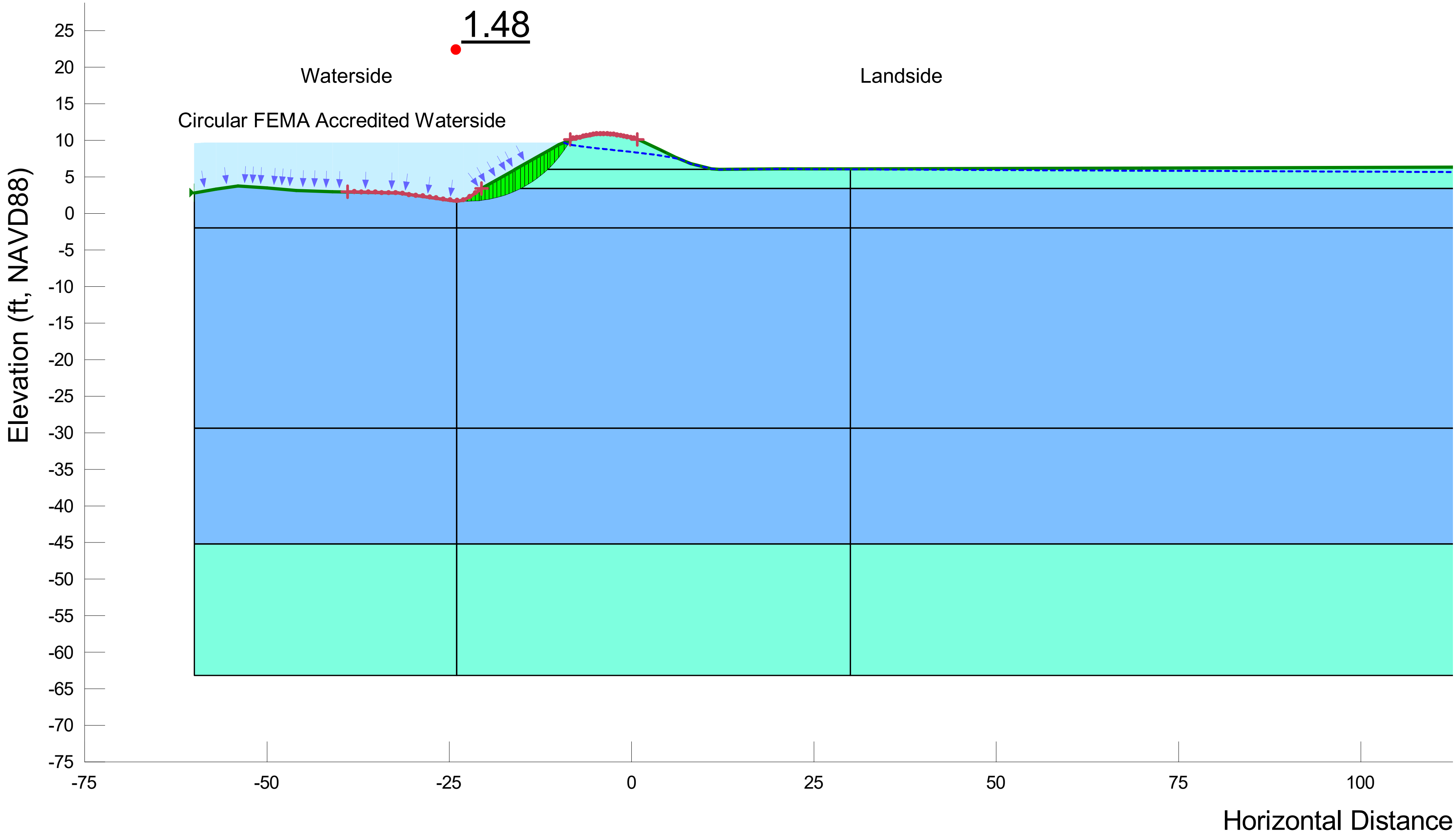
GEI Consultants

Reach 9 – Sta. 1+00 NC-L

Existing Conditions – Baseline Waterside Stability Results

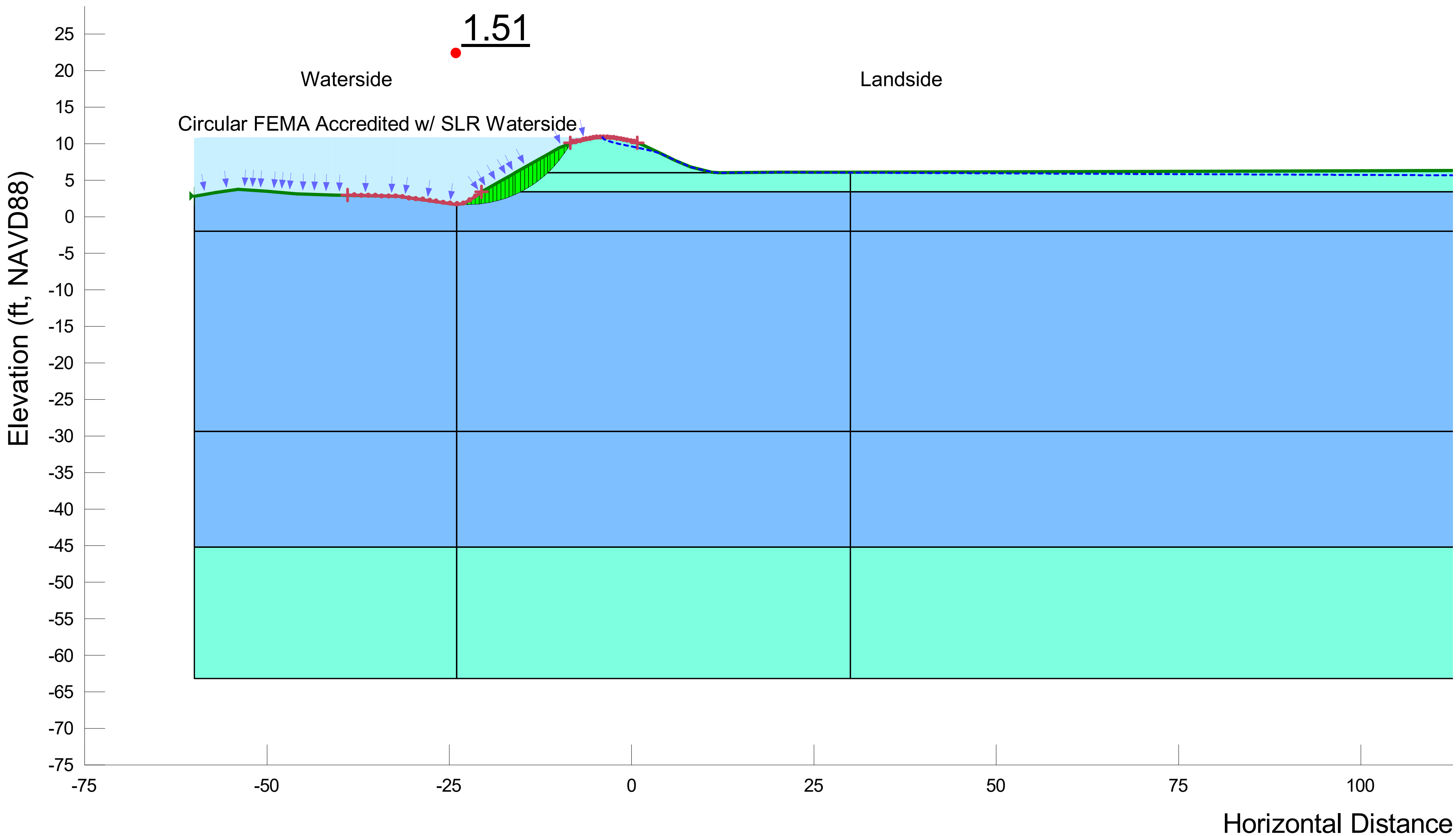
January 2016

Figure 79



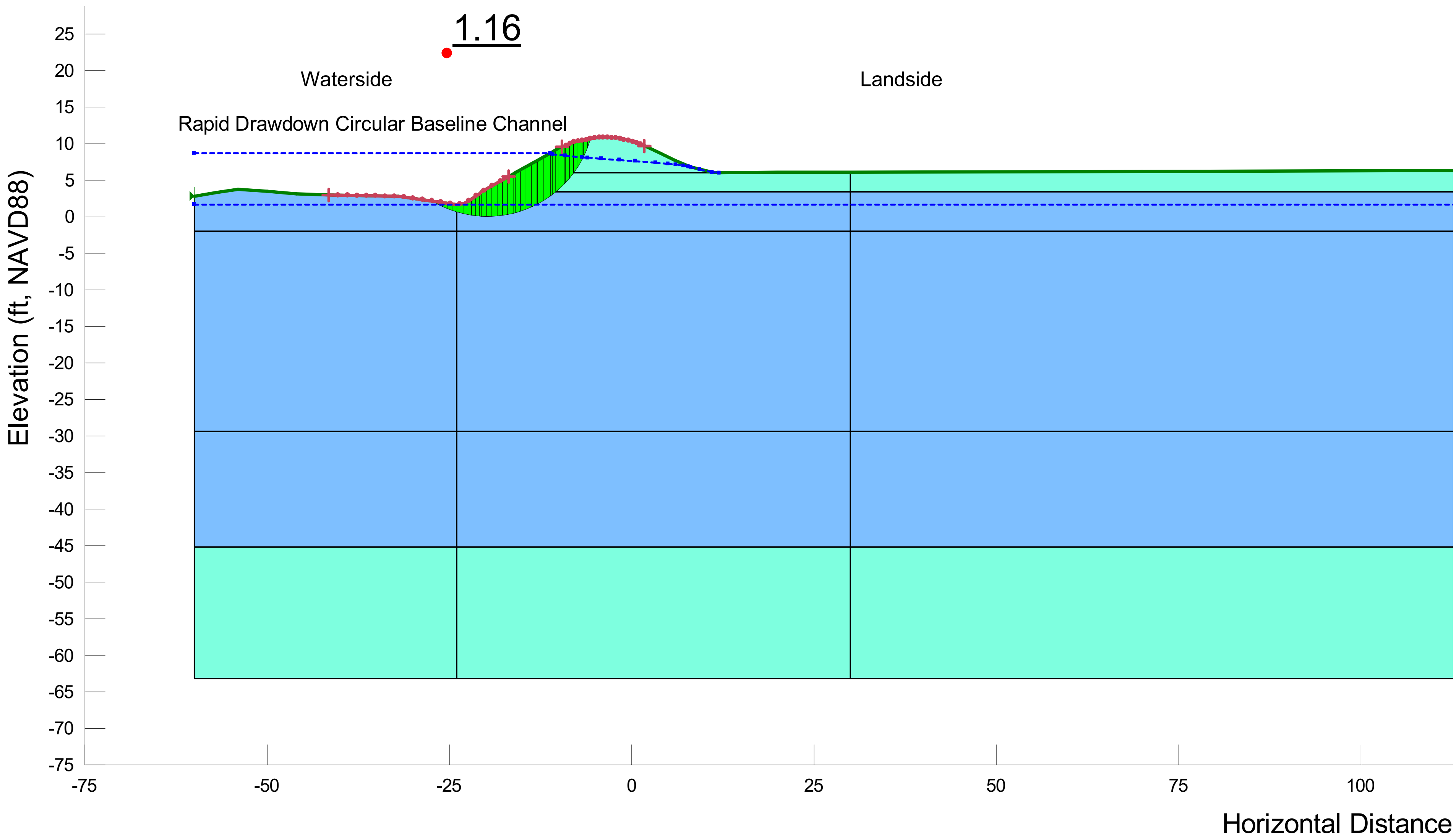
Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 9 – Sta. 1+00 NC-L** Existing Conditions – FEMA Waterside Stability Results
Marin County FCWCD		January 2016 Figure 80



Notes:
1. The Factor of Safety (FS) value shown is for the critical failure surface.

Coyote Creek Levee Evaluation Marin County	GEI Consultants	**Reach 9 – Sta. 1+00 NC-L** Existing Conditions–FEMA w/ SLR Waterside Stability Results
Marin County FCWCD		January 2016 Figure 81



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

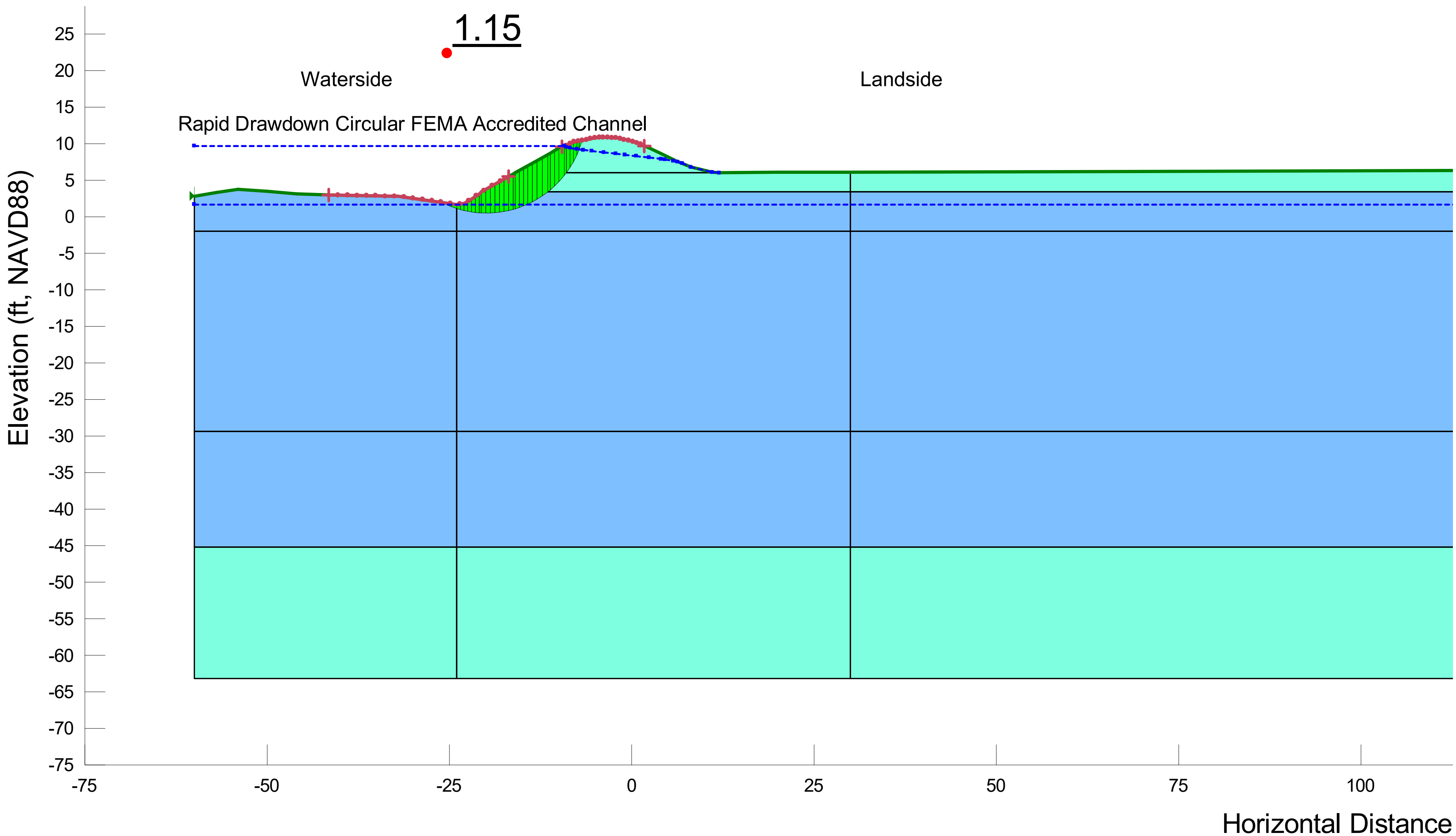
GEI Consultants

Reach 9 – Sta. 1+00 NC-L

Existing Conditions – Baseline
Total Flood Drawdown Results

January 2016

Figure 82



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

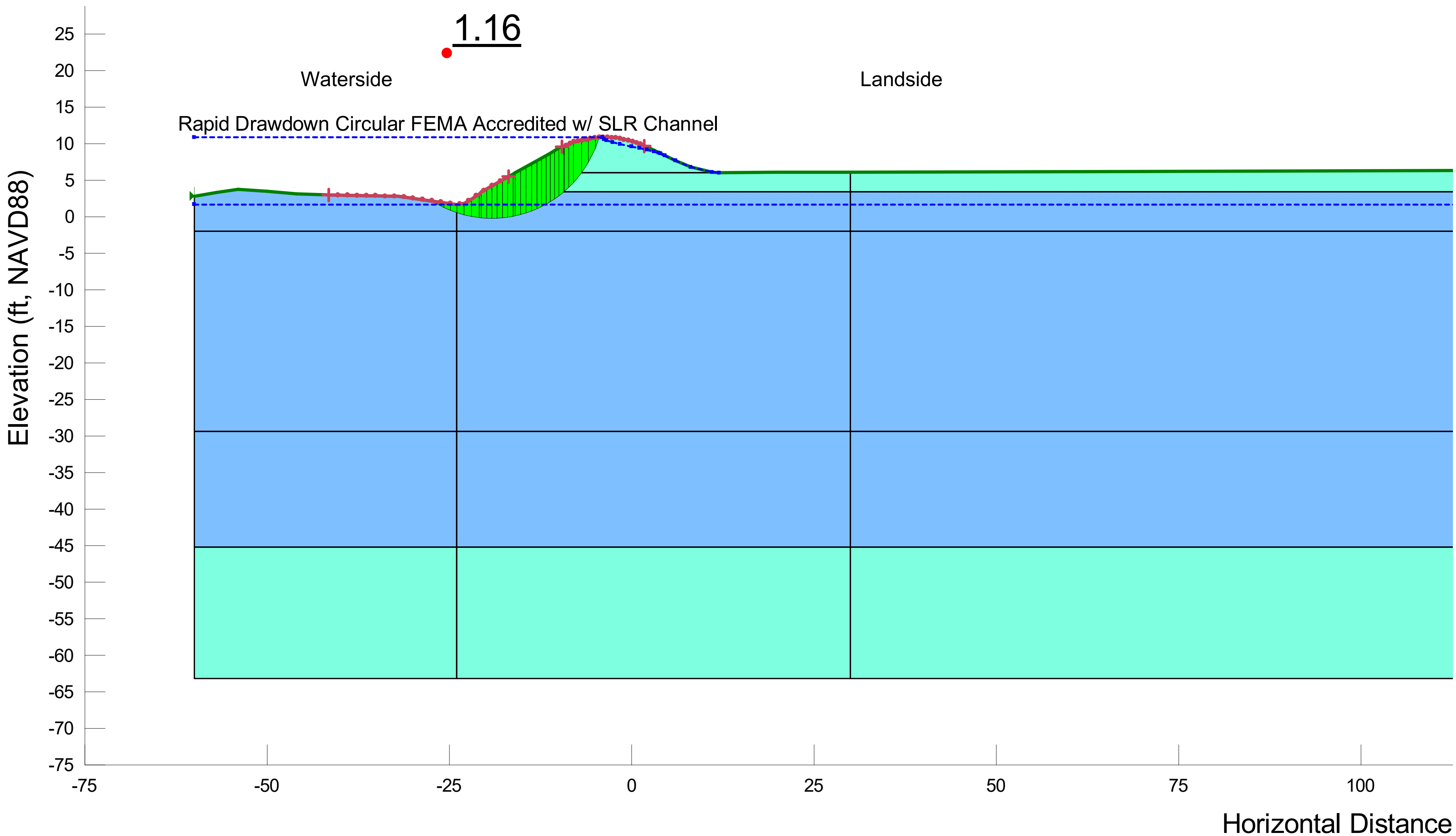
GEI Consultants

Reach 9 – Sta. 1+00 NC-L

Existing Conditions – FEMA
Total Flood Drawdown Results

January 2016

Figure 83



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

GEI Consultants

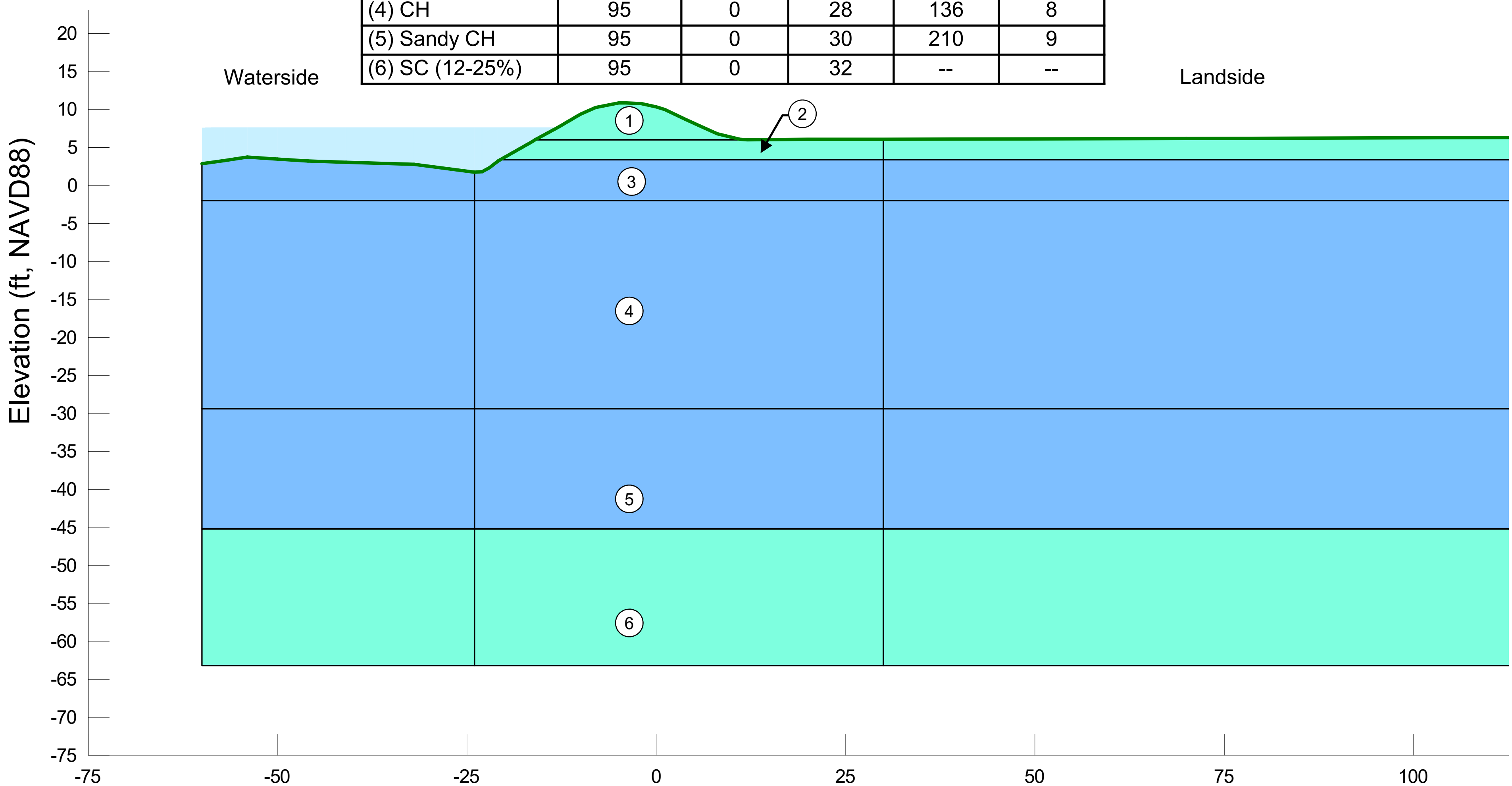
Reach 9 – Sta. 1+00 NC-L

Existing Conditions – FEMA w/ SLR
Total Flood Drawdown Results

January 2016

Figure 84

Layer Name	Saturated Unit Weight	Drained Parameters		Undrained Parameters	
		c’	ϕ’	c	ϕ
	(pcf)	(psf)	(deg.)	(psf)	(deg.)
(1) SC (12-25%)	120	0	38	--	--
(2) SC (12-25%)	120	0	36	--	--
(3) CH	95	0	30	280	0
(4) CH	95	0	28	136	8
(5) Sandy CH	95	0	30	210	9
(6) SC (12-25%)	95	0	32	--	--



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

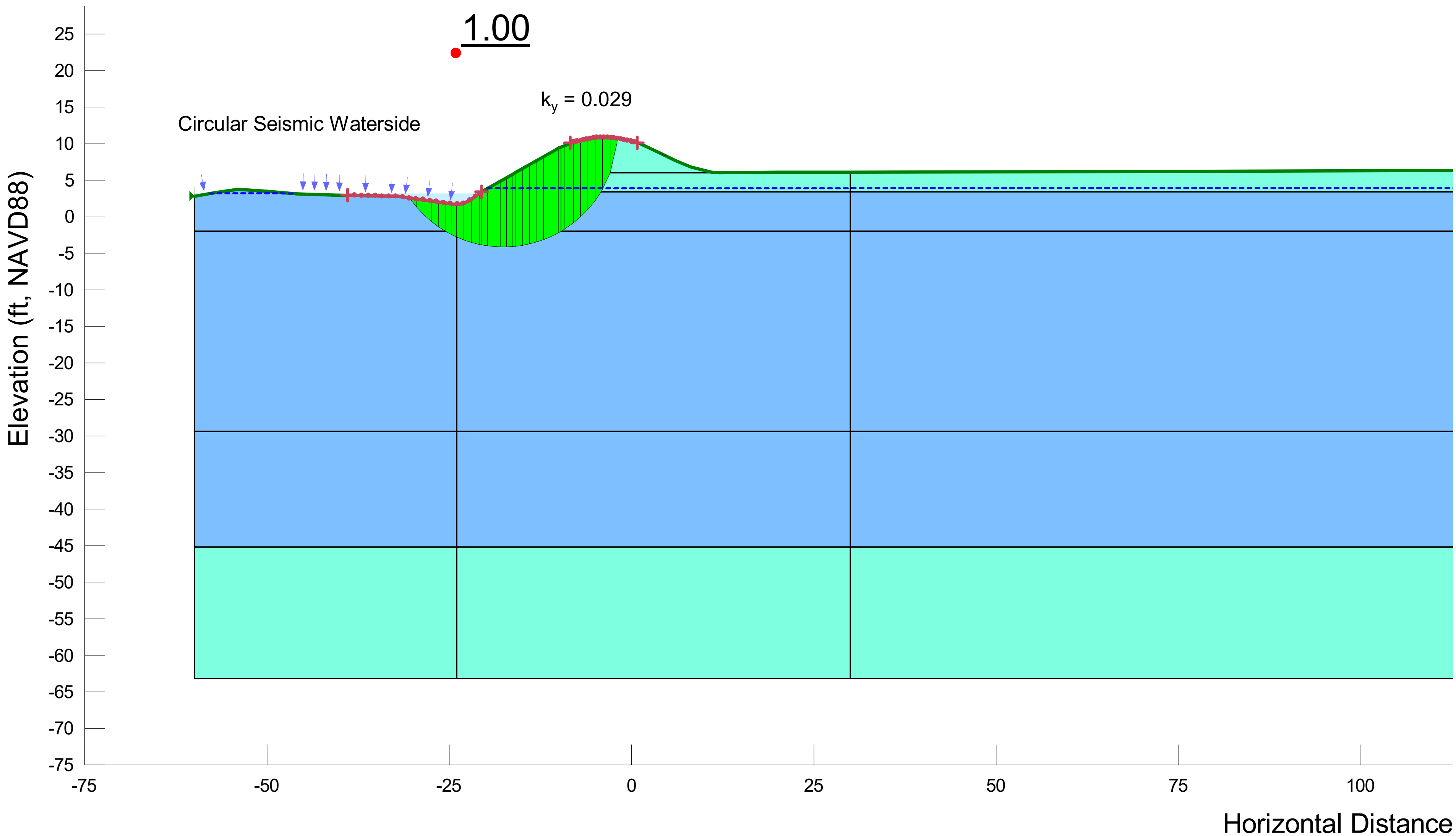
GEI Consultants

Reach 9 – Sta. 1+00 NC-L

Existing Conditions – Seismic Model

January 2016

Figure 85



Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

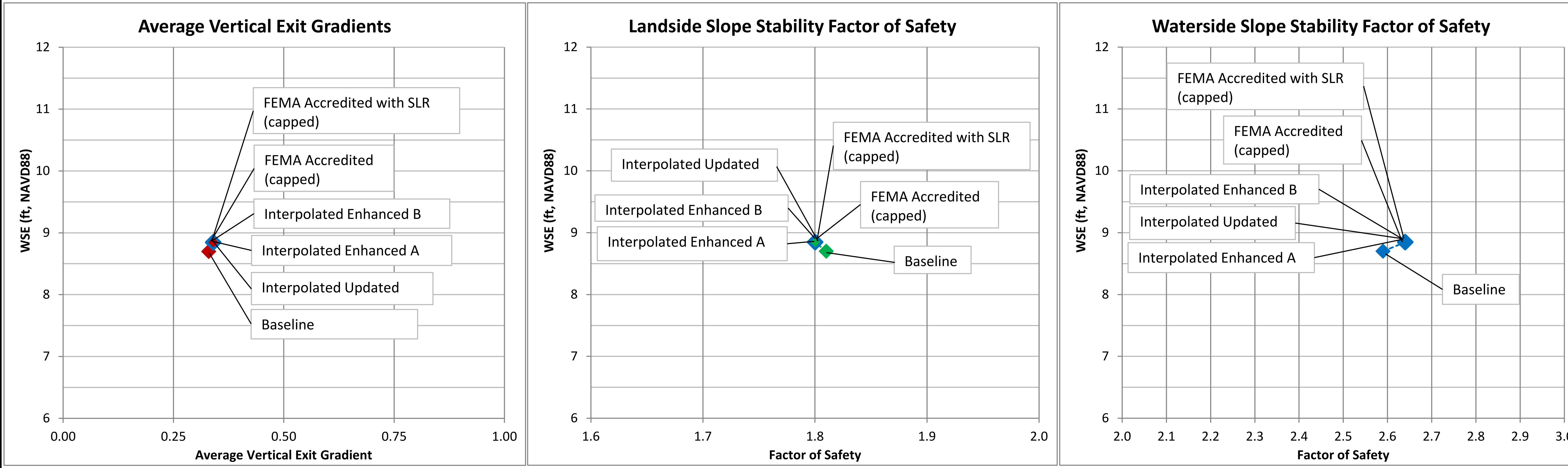
GEI Consultants

Reach 9 – Sta. 1+00 NC-L

Existing Conditions – Seismic Analysis Results

January 2016

Figure 86



Water Surface Scenario	Water Surface Elevation (NAVD88)	Underseepage	Stability Factor of Safety	
		Average Vertical Exit Gradient Over Upper 1-foot at Landside Toe	Landside	Waterside
Baseline	8.70	0.33	1.81	2.59
Updated	8.85	0.34	1.80	2.64
Enhanced A	8.85	0.34	1.80	2.64
Enhanced B	8.85	0.34	1.80	2.64
FEMA Accredited*	8.85	0.34	1.80	2.64
FEMA Accredited with SLR*	8.85	0.34	1.80	2.64

*Water surfaces during analysis were capped at top of levee elevation. Actual water surface elevation is 8.85 for FEMA Accredited and FEMA Accredited with SLR.

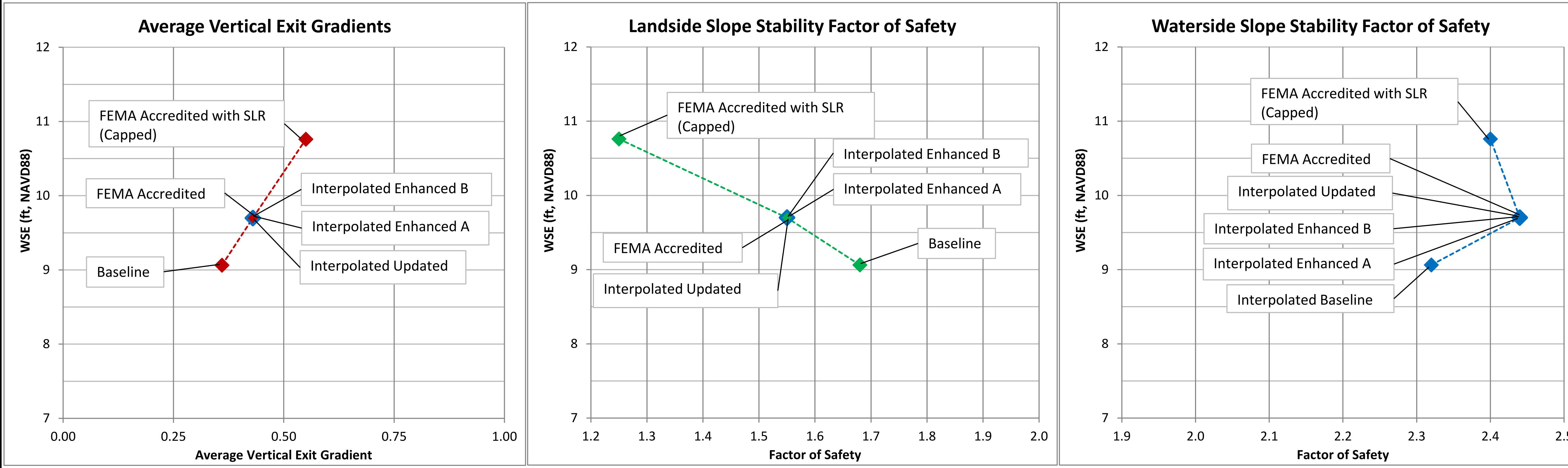
Interpolated underseepage and stability factor of safety

Coyote Creek Levee Evaluation
Marin County Flood control and Water Conservation District

GEI Consultants

Figure 87. Reach 3 28+00 CC-L Seepage and Stability Results

January 2016



Water Surface Scenario	Water Surface Elevation (NAVD88)	Underseepage	Stability Factor of Safety	
		Average Vertical Exit Gradient Over Upper 1-foot at Landside Toe	Landside	Waterside
Baseline	9.06	0.36	1.68	2.32
Updated	9.70	0.43	1.55	2.44
Enhanced A	9.70	0.43	1.55	2.44
Enhanced B	9.70	0.43	1.55	2.44
FEMA Accredited	9.70	0.43	1.55	2.44
FEMA Accredited with SLR*	10.76	0.55	1.25	2.40

*Water surfaces during analysis were capped at top of levee elevation. Actual water surface elevation is 10.76 for FEMA Accredited with SLR.

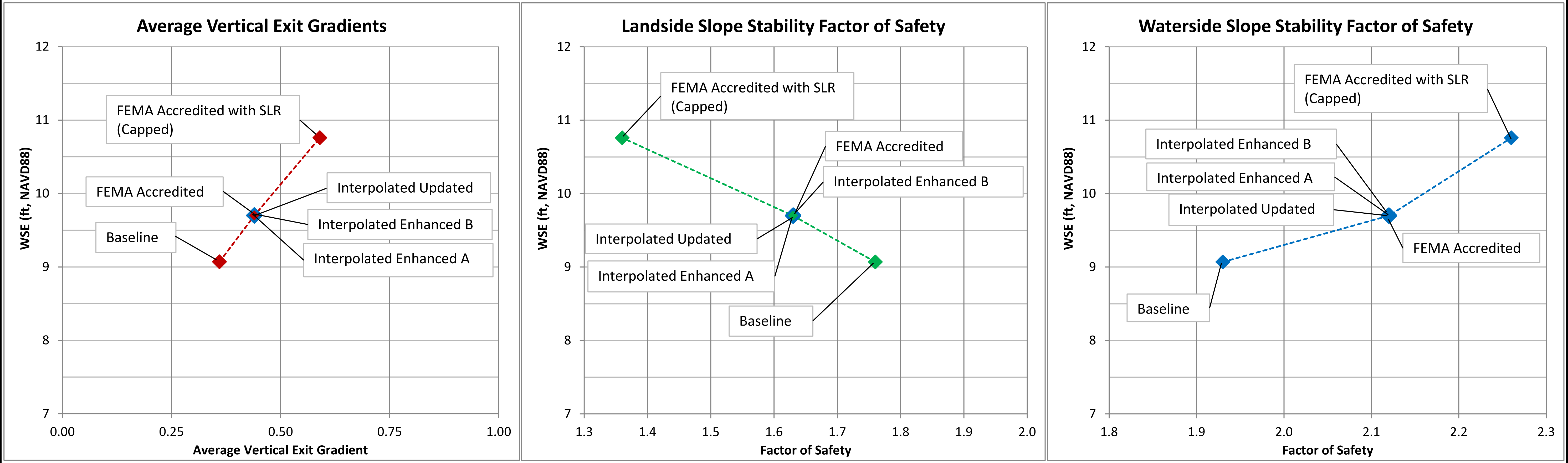
Interpolated underseepage and stability factor of safety

Coyote Creek Levee Evaluation
Marin County Flood control and Water Conservation District

GEI Consultants

Figure 88. Reach 4 34+00 CC-L Seepage and Stability Results

January 2016



Water Surface Scenario	Water Surface Elevation (NAVD88)	Underseepage: Average Vertical Exit Gradient Over Upper 1-foot at Landside Toe	Stability Factor of Safety: Landside	Stability Factor of Safety: Waterside
Baseline	9.07	0.36	1.76	1.93
Updated	9.70	0.44	1.63	2.12
Enhanced A	9.70	0.44	1.63	2.12
Enhanced B	9.70	0.44	1.63	2.12
FEMA Accredited	9.70	0.44	1.63	2.12
FEMA Accredited with SLR*	10.76	0.59	1.36	2.26

*Water surfaces during analysis were capped at top of levee elevation. Actual water surface elevations is 10.76 for FEMA Accredited with SLR.

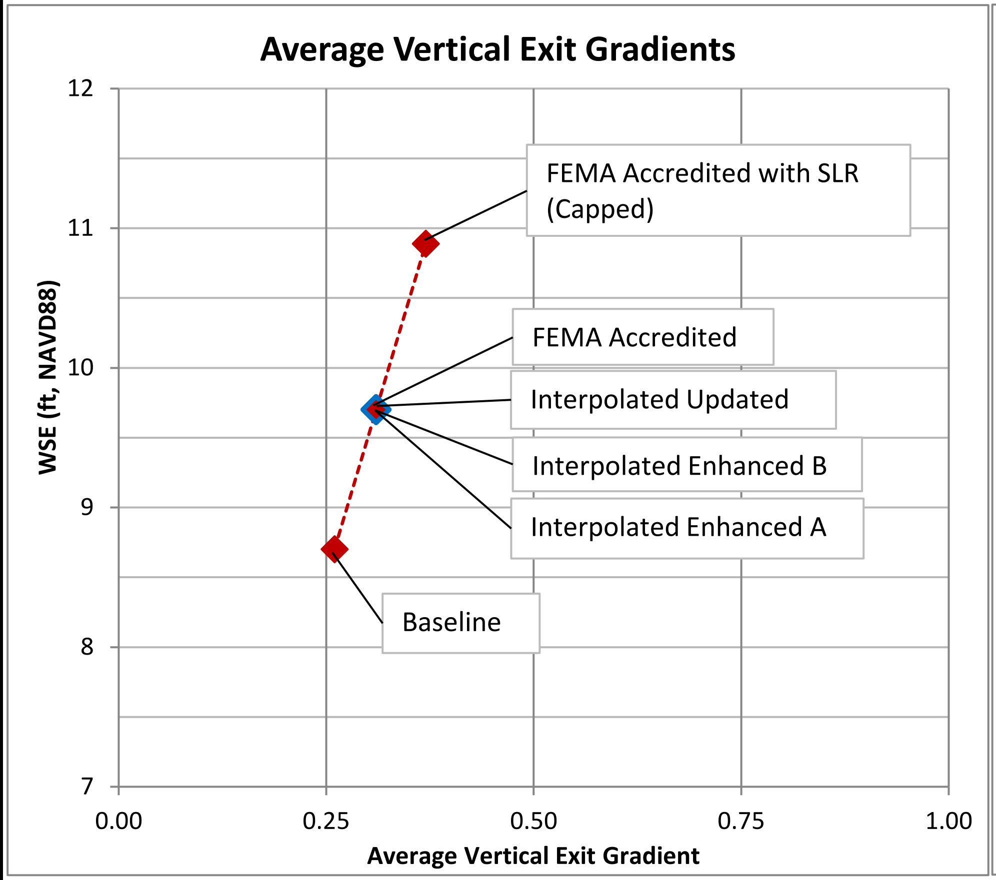
Interpolated underseepage and stability factor of safety

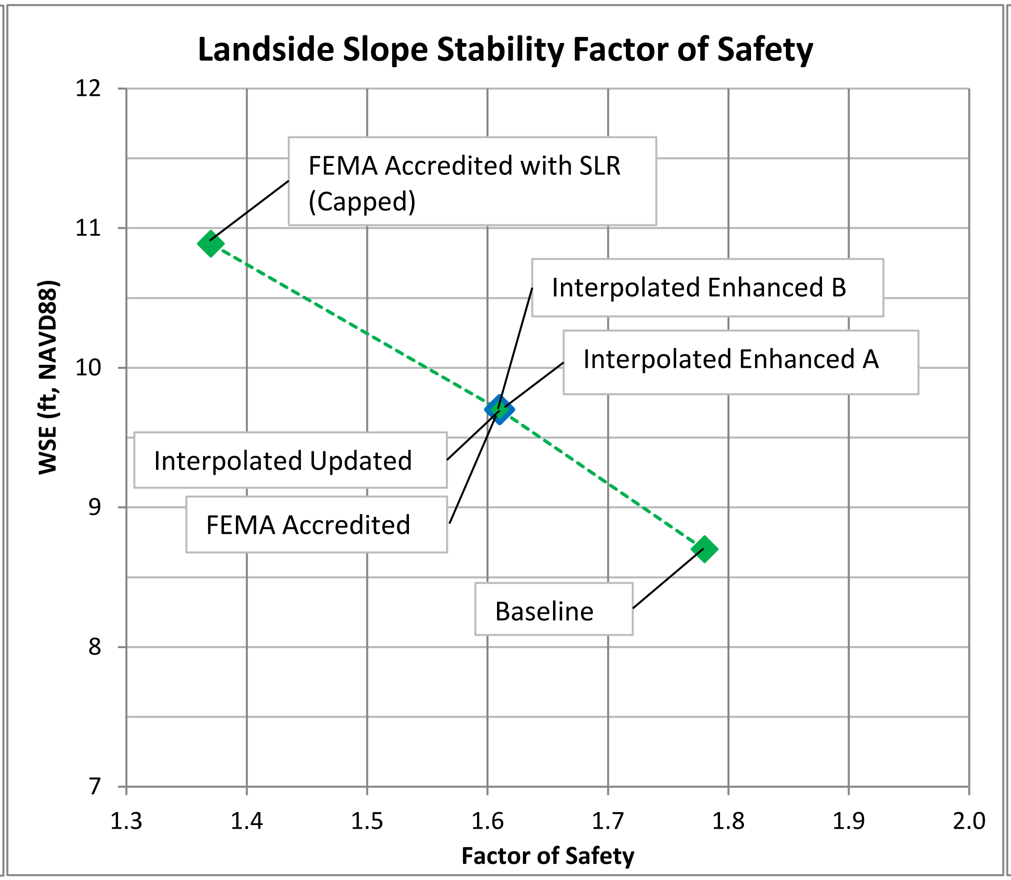
Coyote Creek Levee Evaluation
Marin County Flood control and Water Conservation District

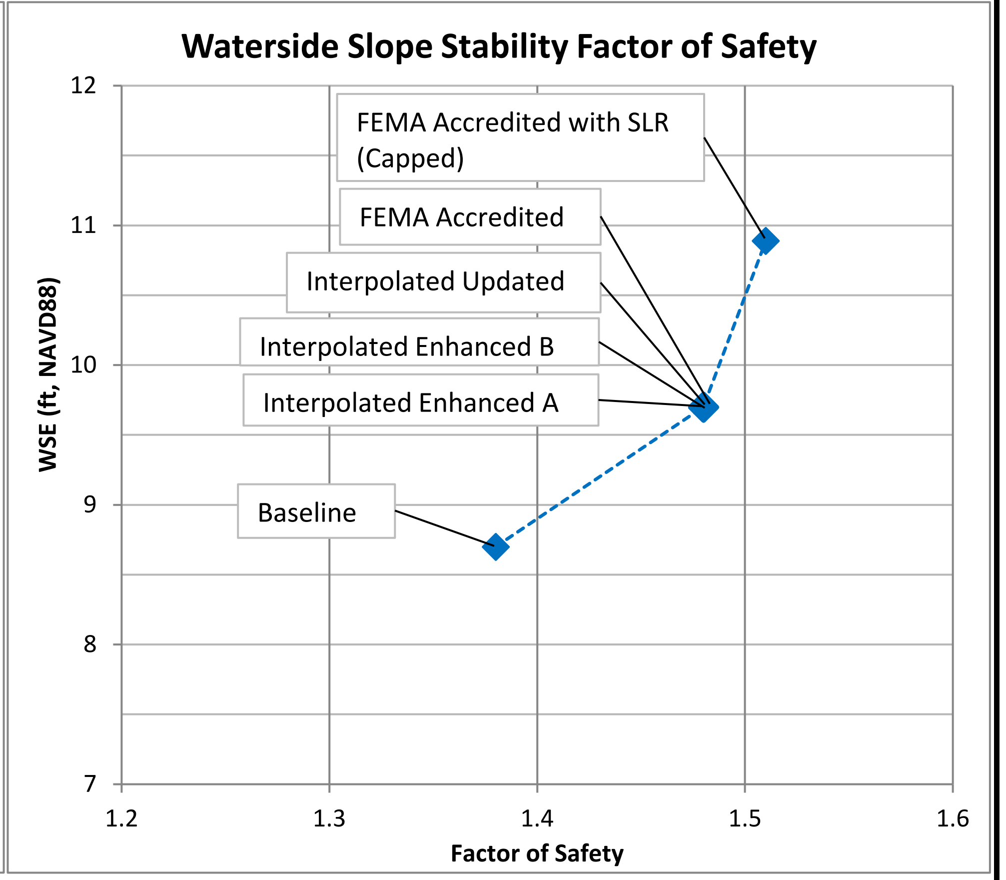
GEI Consultants

Figure 89. Reach 7 35+00 CC-R Seepage and Stability Results

January 2016







Water Surface Scenario	Water Surface Elevation (NAVD88)	Underseepage: Average Vertical Exit Gradient Over Upper 1-foot at Landside Toe	Stability Factor of Safety: Landside	Stability Factor of Safety: Waterside
Baseline	8.70	0.26	1.78	1.38
Updated	9.70	0.31	1.61	1.48
Enhanced A	9.70	0.31	1.61	1.48
Enhanced B	9.70	0.31	1.61	1.48
FEMA Accredited	9.70	0.31	1.61	1.48
FEMA Accredited with SLR*	10.89	0.37	1.37	1.51

*Water surfaces during analysis were capped at top of levee elevation. Actual water surface elevation is 10.89 for FEMA Accredited with SLR.

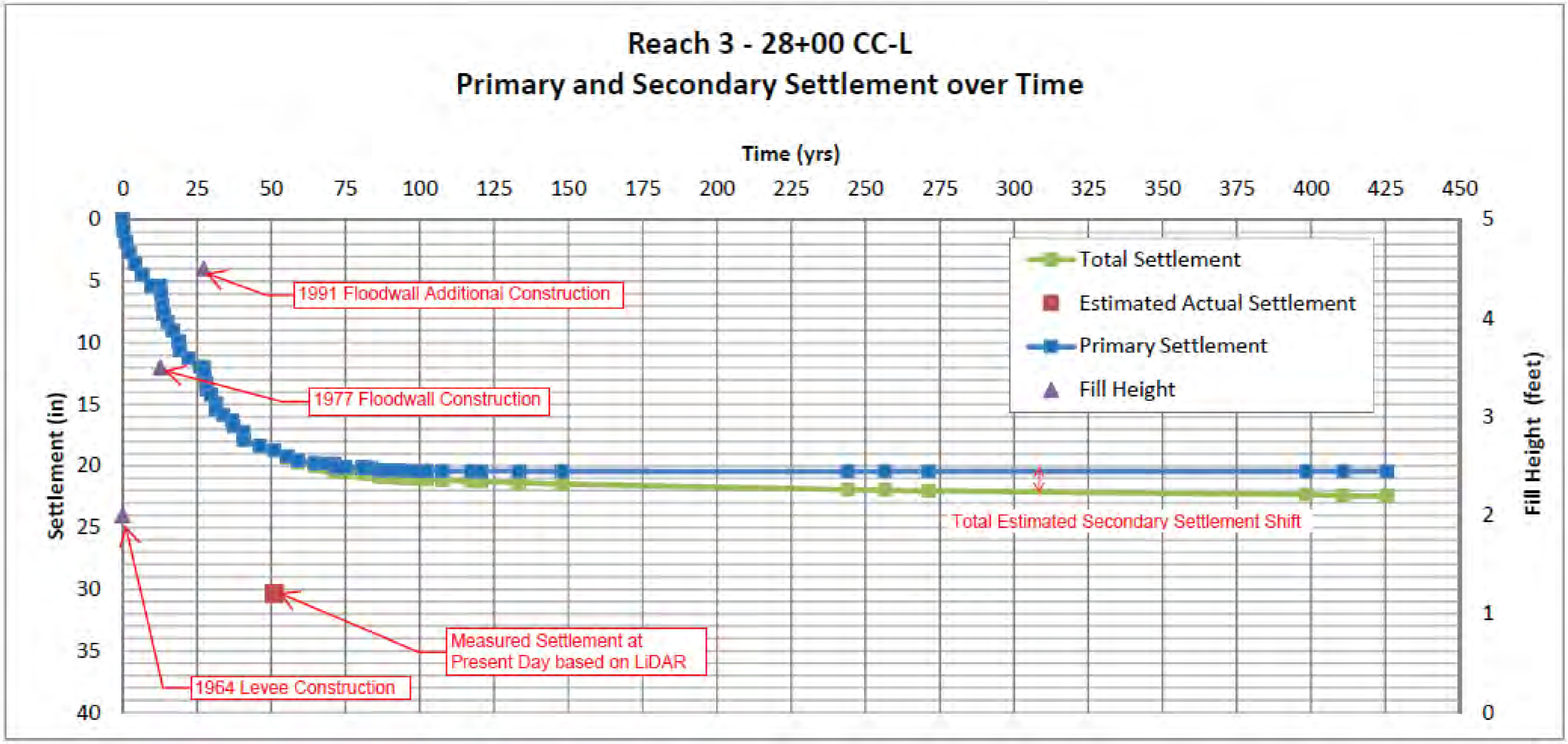
Interpolated underseepage and stability factor of safety

Coyote Creek Levee Evaluation
Marin County Flood control and Water Conservation District

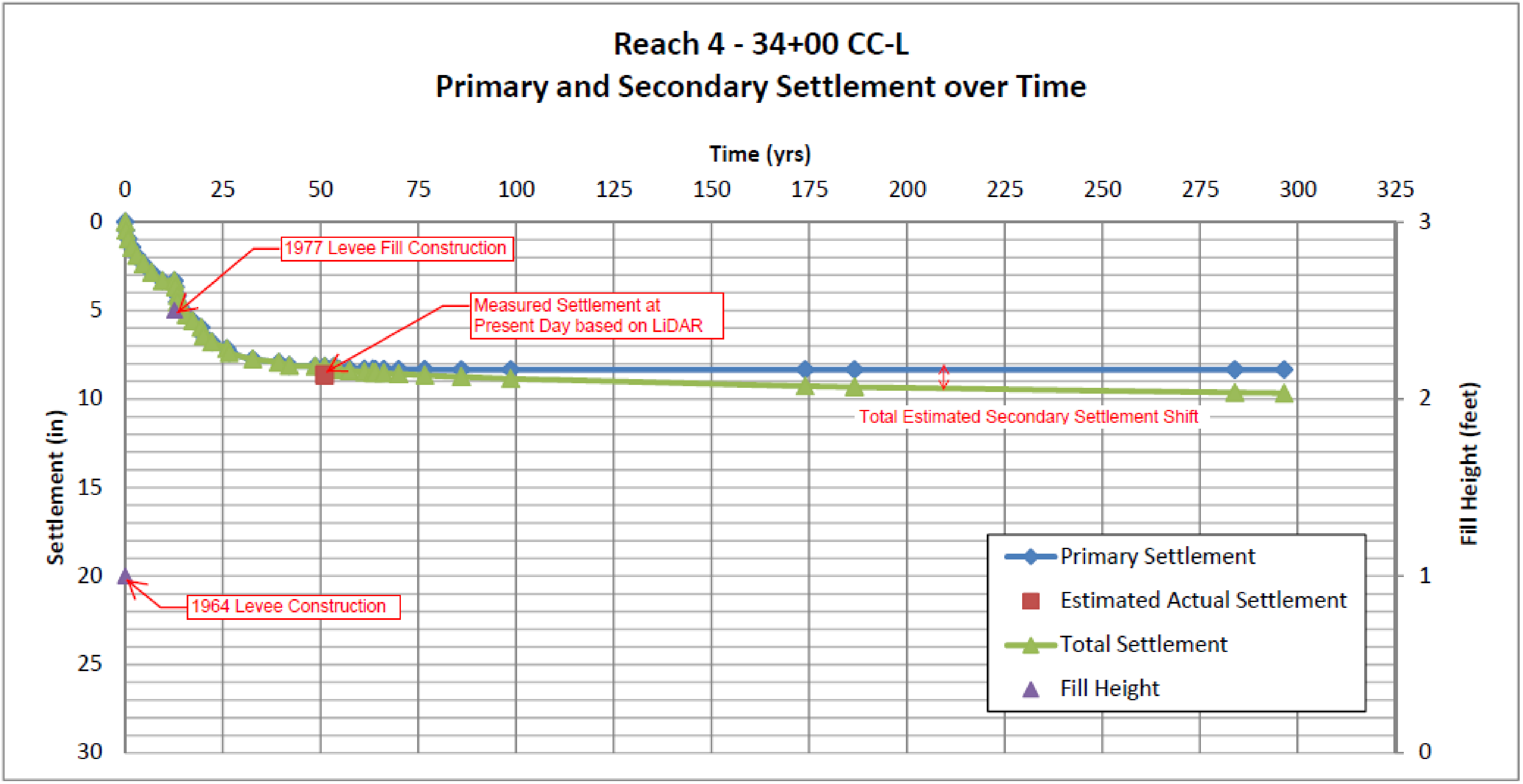
GEI Consultants

Figure 90. Reach 9 1+00 NC-L
Seepage and Stability Results

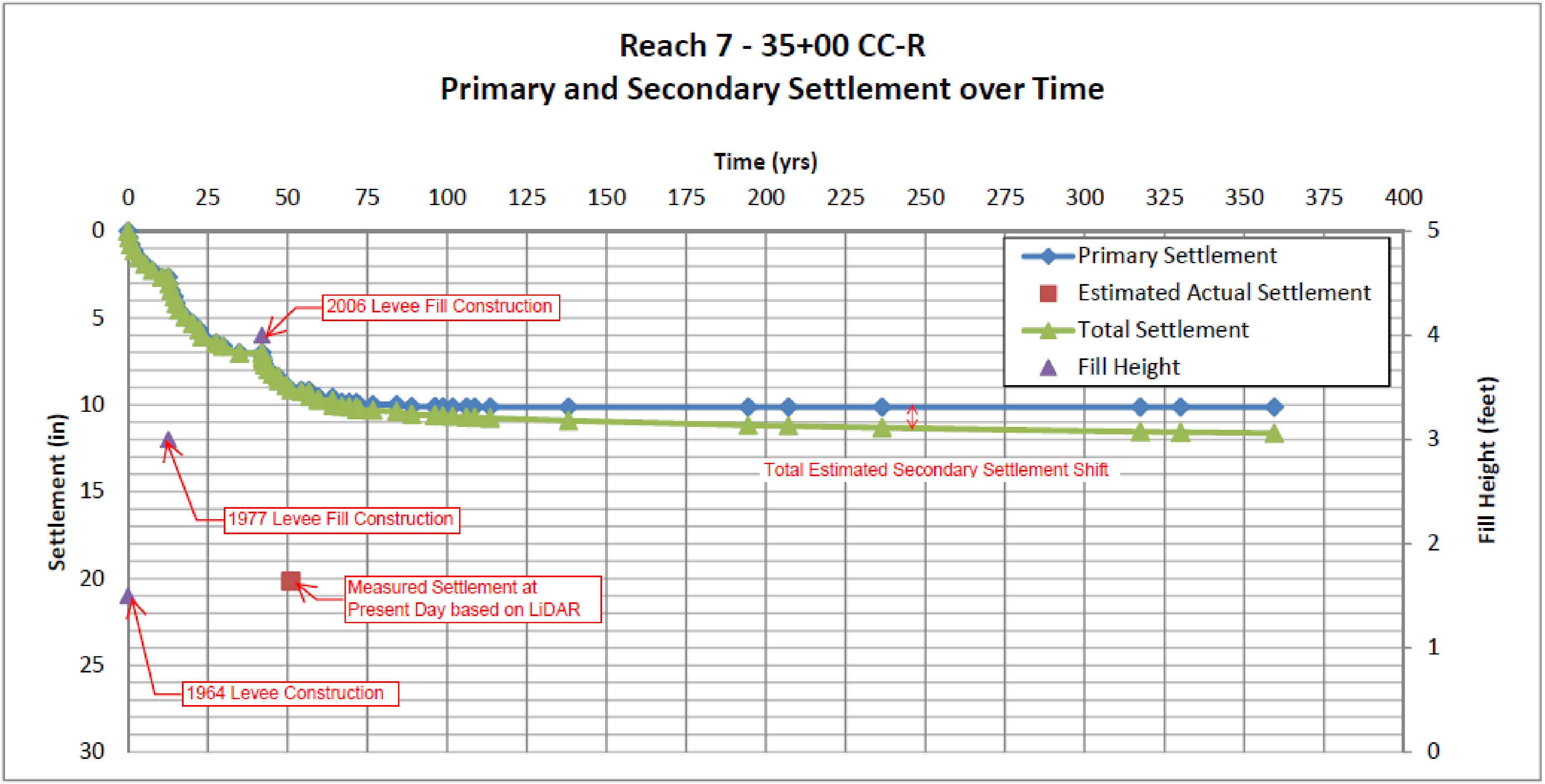
January 2016



Coyote Creek Levee Evaluation Marin County	GEI Consultants in association with HDR	**Reach 3 – Sta. 28+00 CC-L** Estimated Settlement Over Time
Marin County FCWCD		August 2015 Figure 91



Coyote Creek Levee Evaluation Marin County	GEI Consultants in association with HDR	**Reach 4 – Sta. 34+00 CC-L** Estimated Settlement Over Time
Marin County FCWCD		August 2015 Figure 92



Coyote Creek Levee Evaluation
Marin County

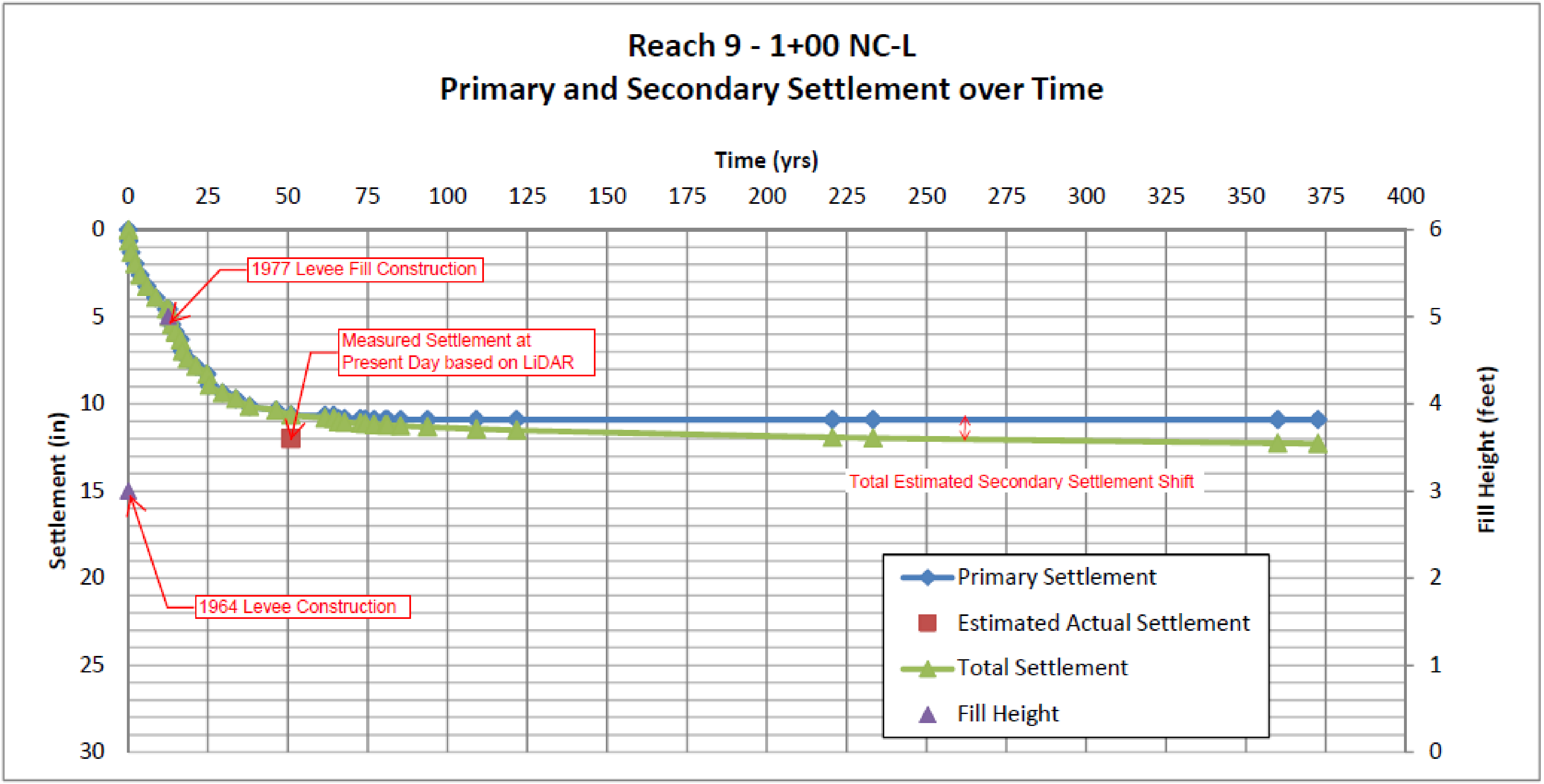
Marin County FCWCD

GEI Consultants in association with HDR

Reach 7 – Sta. 35+00 CC-R
Estimated Settlement Over Time

August 2015

Figure 93



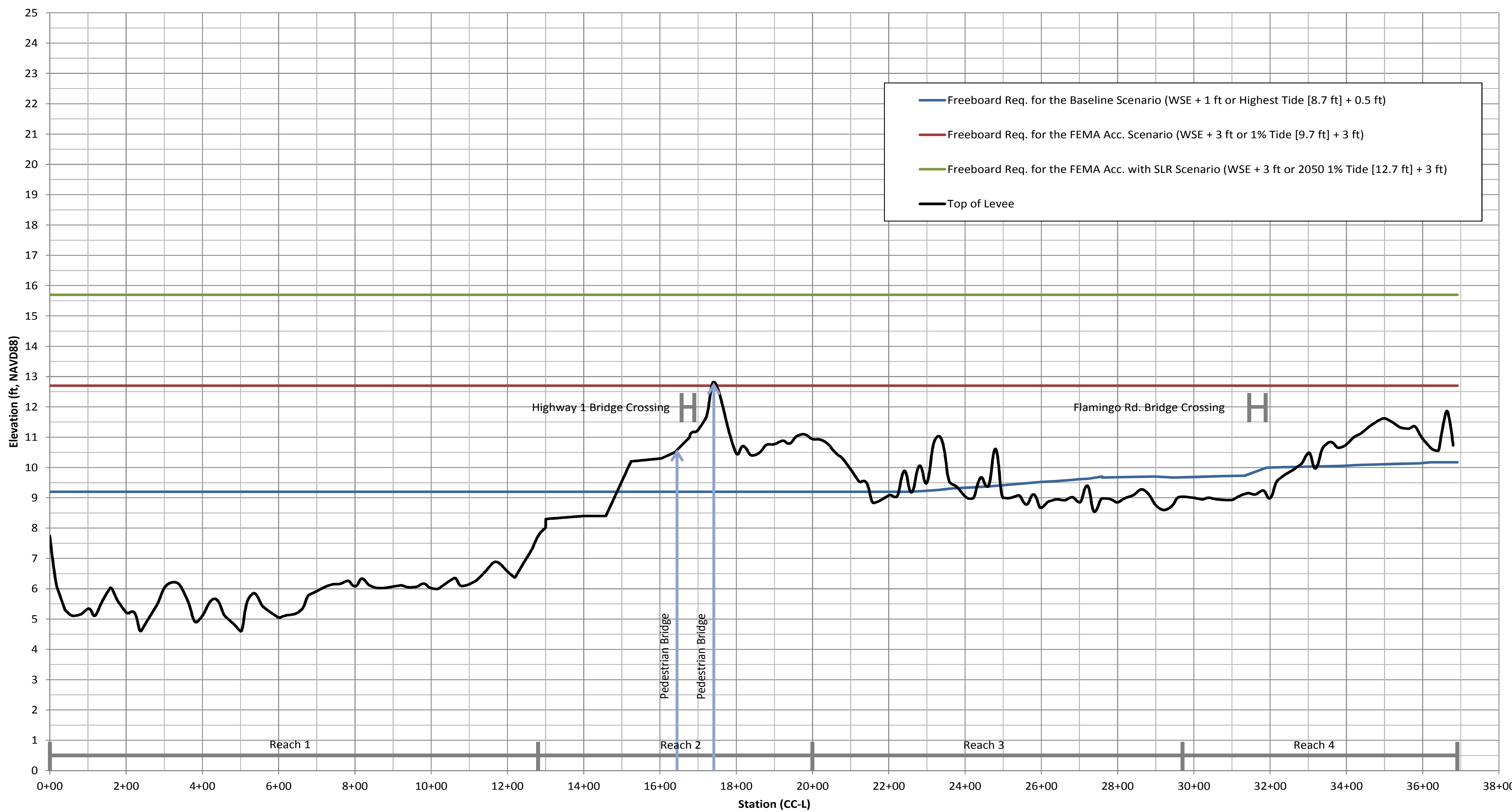
Coyote Creek Levee Evaluation
Marin County

Marin County FCWCD

GEI Consultants in association with HDR

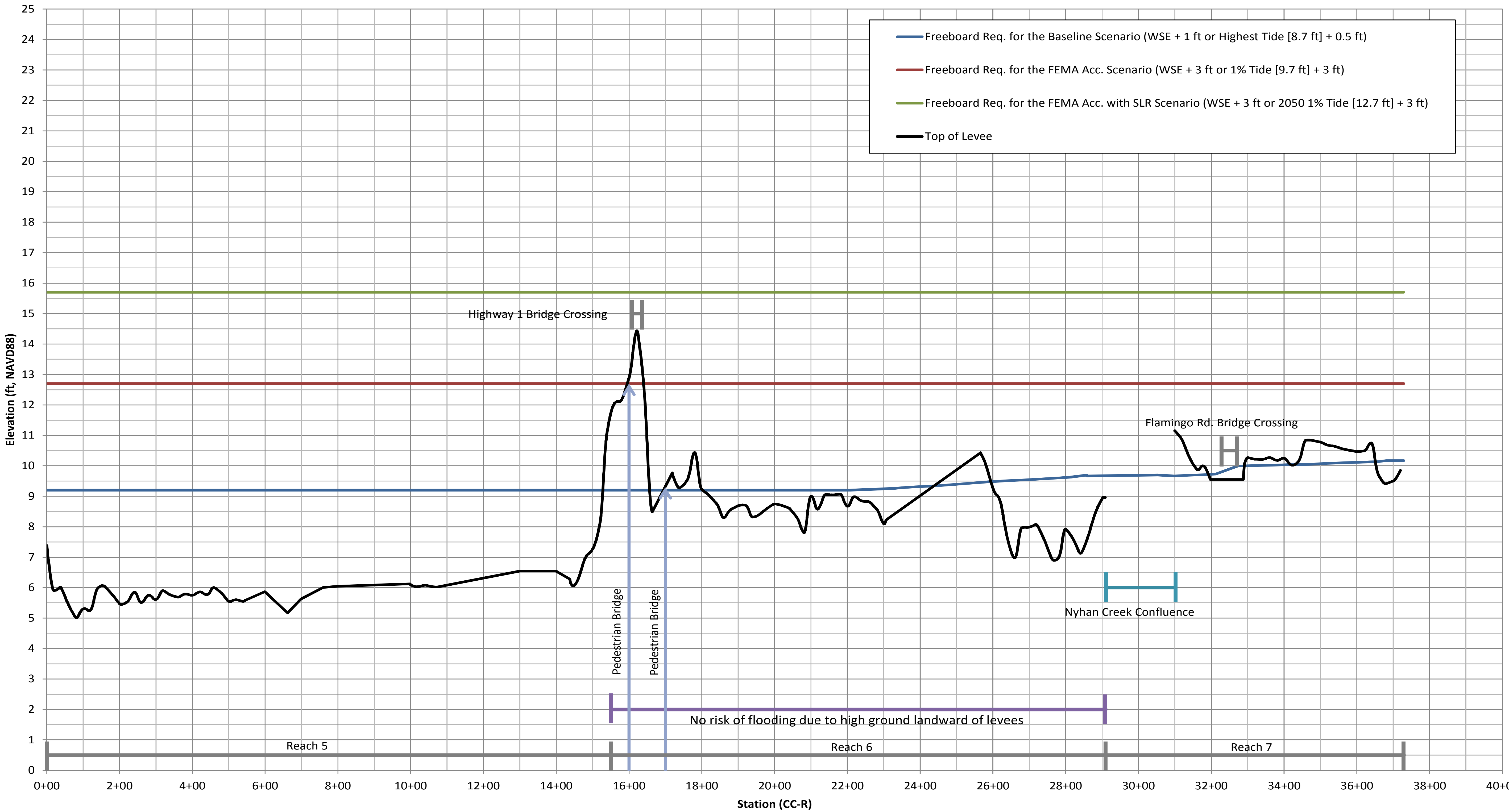
Reach 9 – Sta. 1+00 NC-L
Estimated Settlement Over Time

August 2015 Figure 94



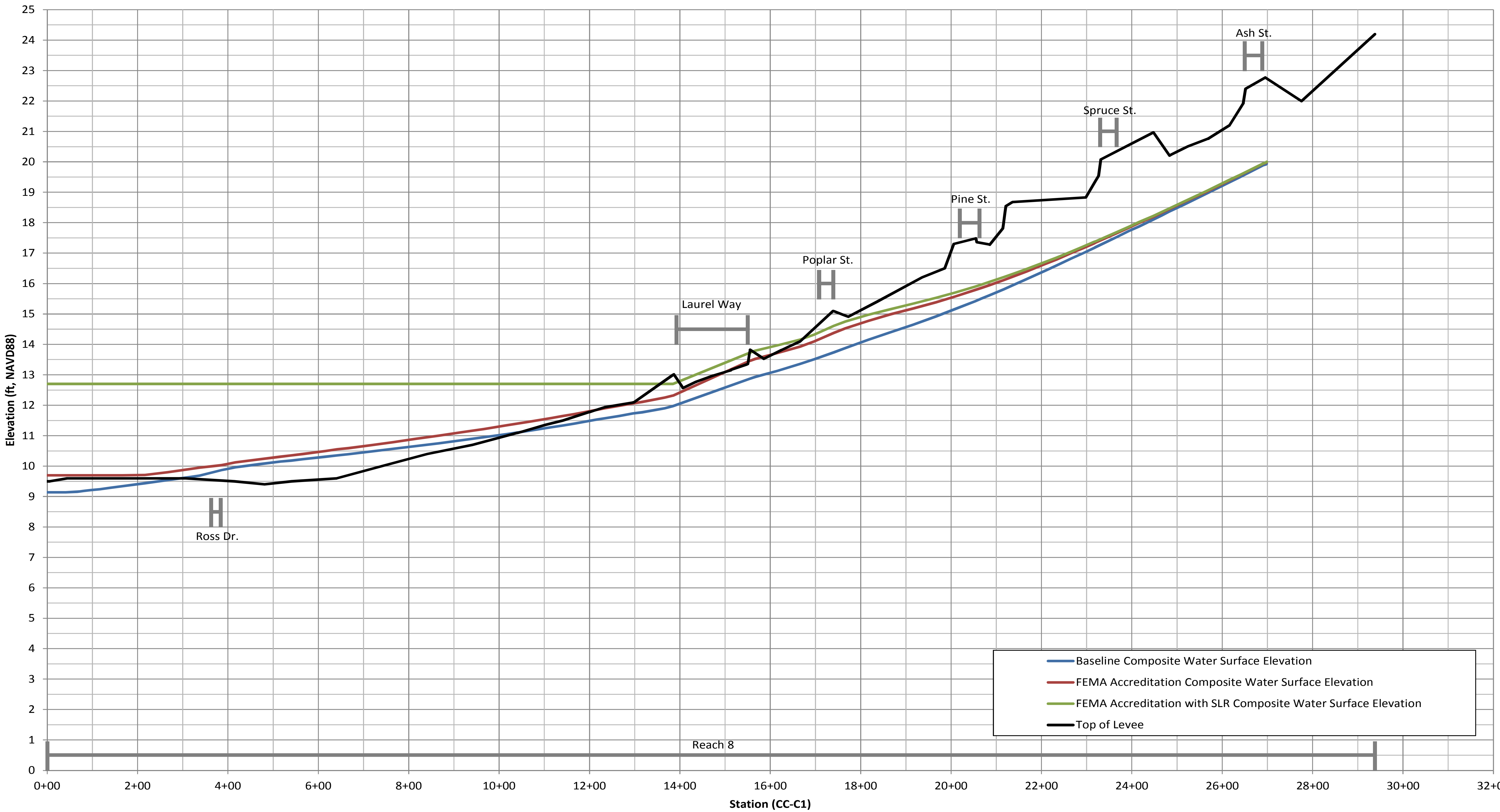
Note:
1. Approximate bridge locations estimated from available survey data.

Coyote Creek Levee Evaluation Project Marin County, California	GEI Consultants	Coyote Creek Left Bank (CC-L) Comparison of Levee Crown Profile to Freeboard Requirements
Marin County Flood Control and Water Conservation District	Project 1404570	January 2016 Figure 95



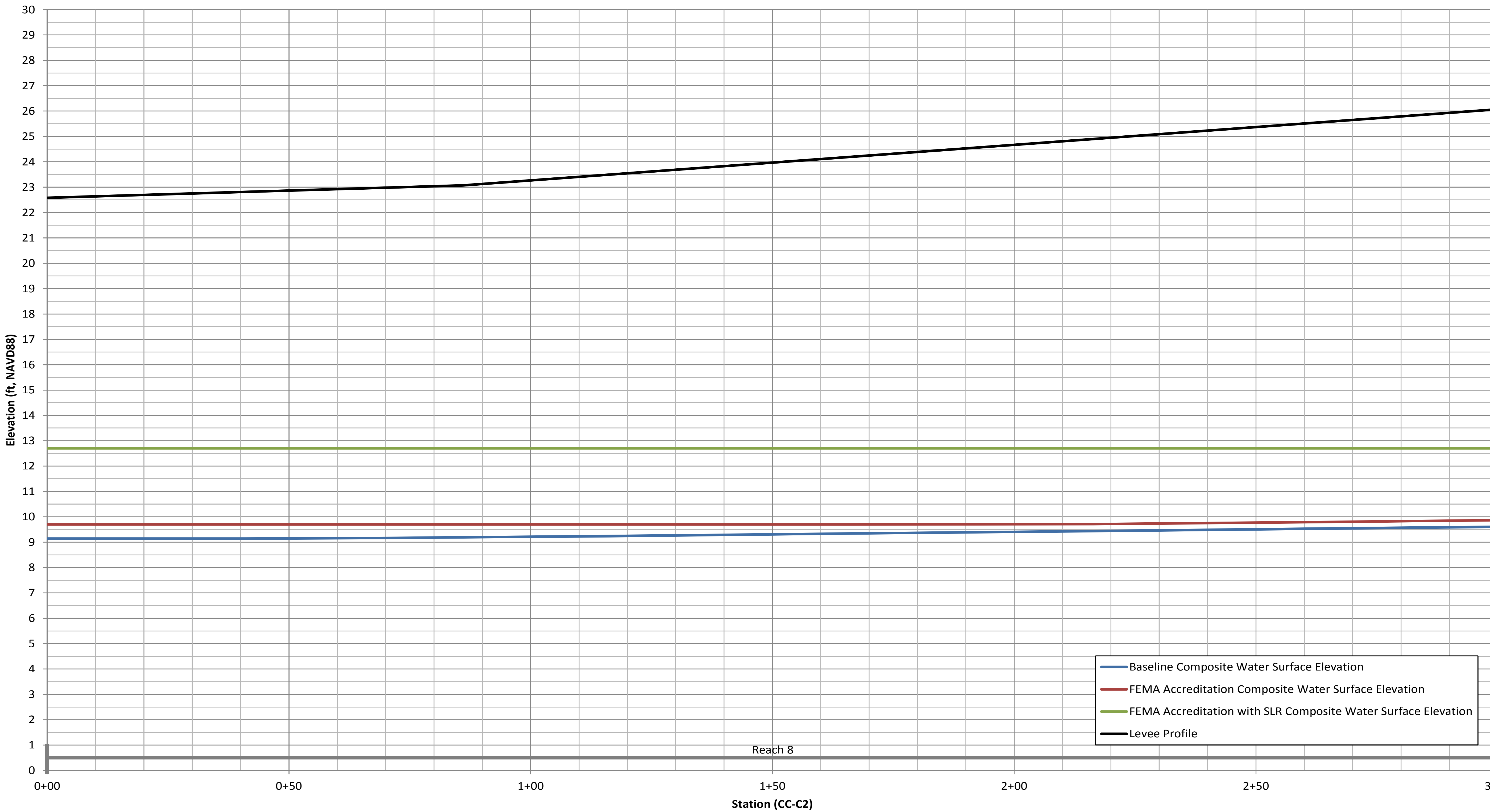
Note:
1. Approximate bridge locations estimated from available survey data.

Coyote Creek Levee Evaluation Project Marin County, California	GEI Consultants	Coyote Creek Right Bank (CC-R) Comparison of Levee Crown Profile to Freeboard Requirements
Marin County Flood Control and Water Conservation District	Project 1404570	January 2016 Figure 96



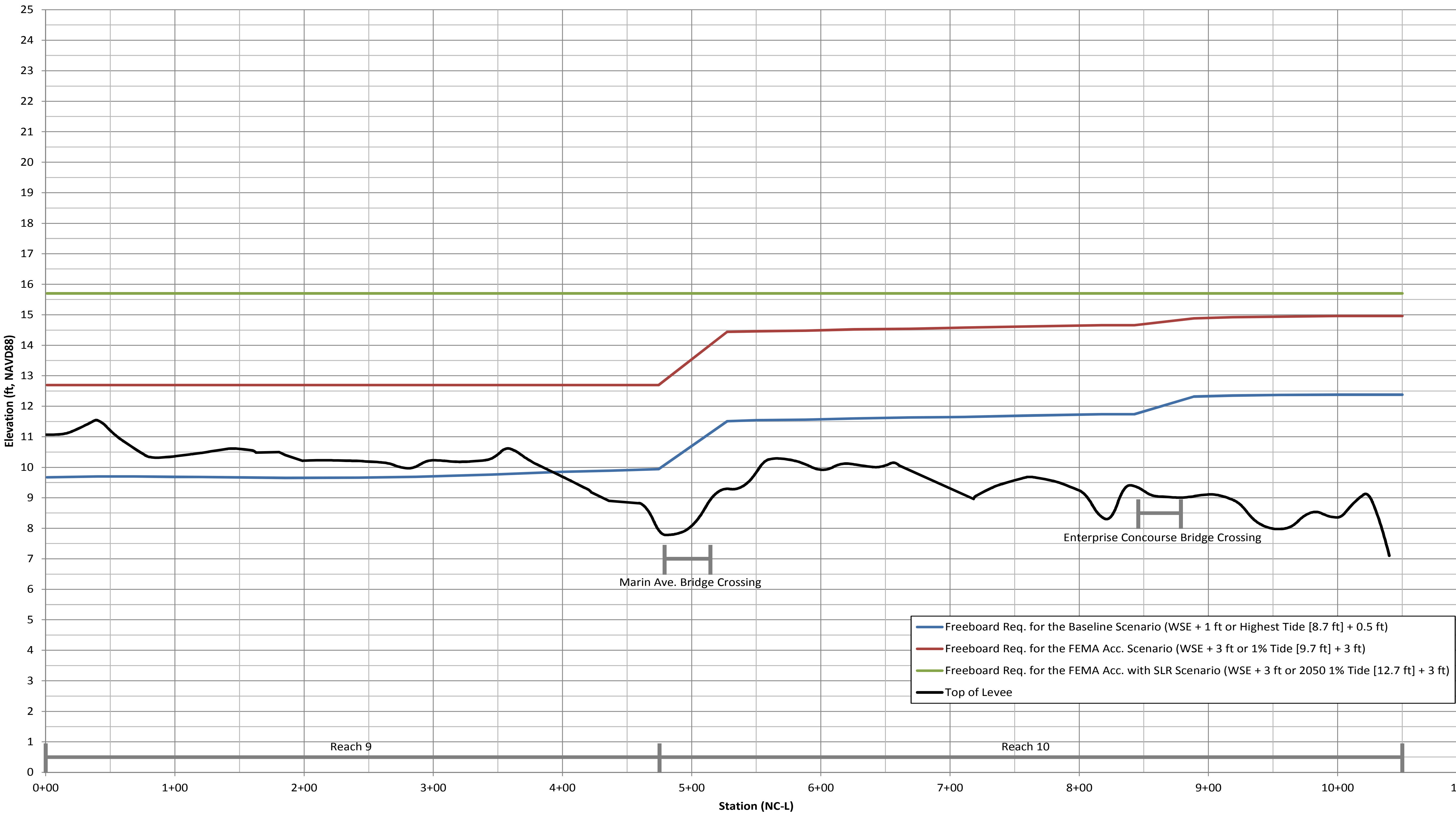
Notes:
1. Approximate locations of streets estimated from available survey data.
2. Assessment of concrete channel wall elevations within the incised channel is based on containment of the water surface profile. No freeboard is required.

Coyote Creek Levee Evaluation Project Marin County, California	GEI Consultants	Coyote Creek Concrete Channel 1 (CC-C1) Comparison of Levee Crown Profile to Composite Water Surface Elevations
Marin County Flood Control and Water Conservation District	Project 1404570	January 2016 Figure 97



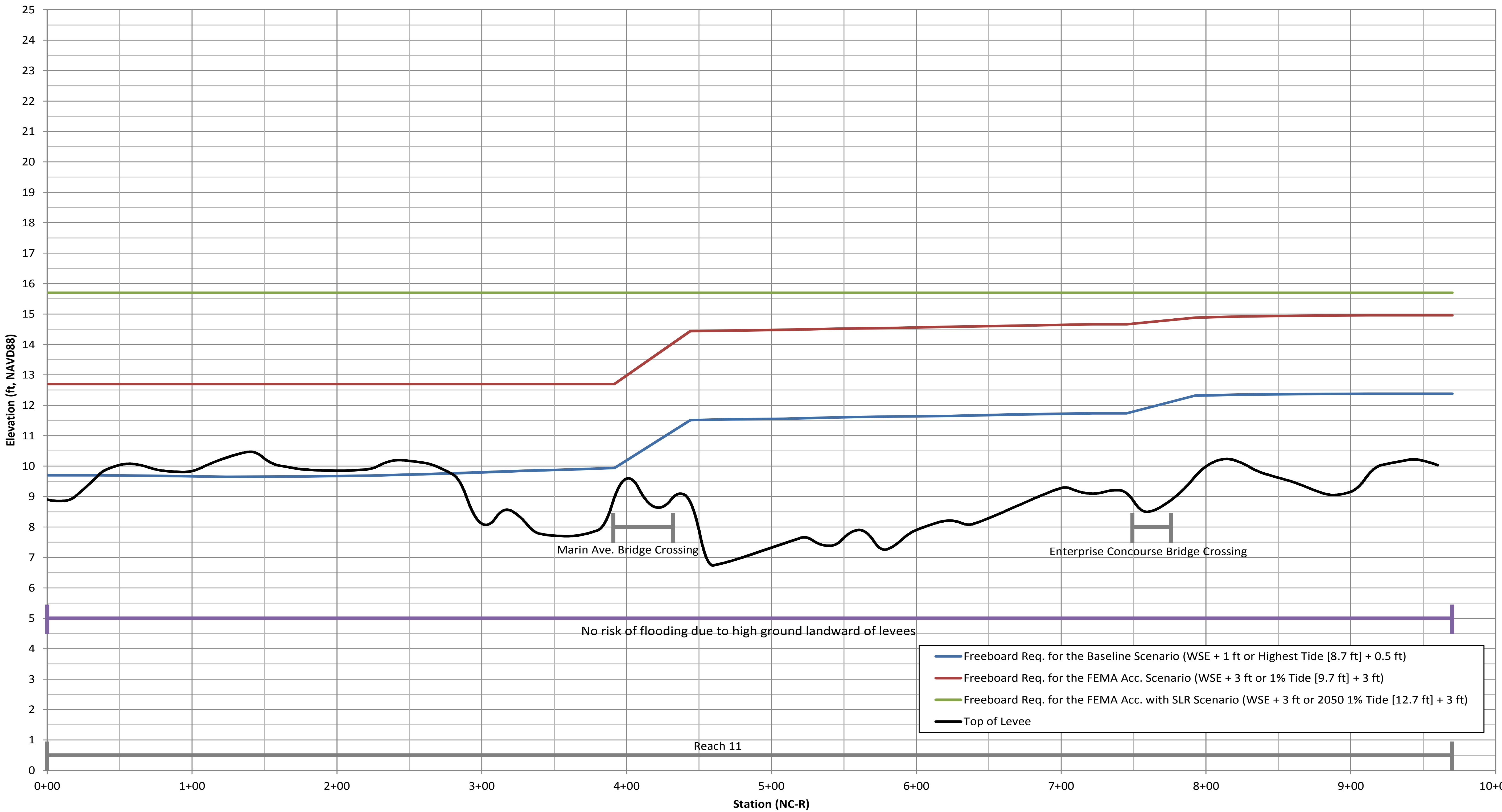
Note:
1. Assessment of concrete channel wall elevations within the incised channel is based on containment of the water surface profile. No freeboard is required.

Coyote Creek Levee Evaluation Project Marin County, California	GEI Consultants	Coyote Creek Concrete Channel 2 (CC-C2) Comparison of Levee Crown Profile to Composite Water Surface Elevations
Marin County Flood Control and Water Conservation District	Project 1404570	January 2016 Figure 98



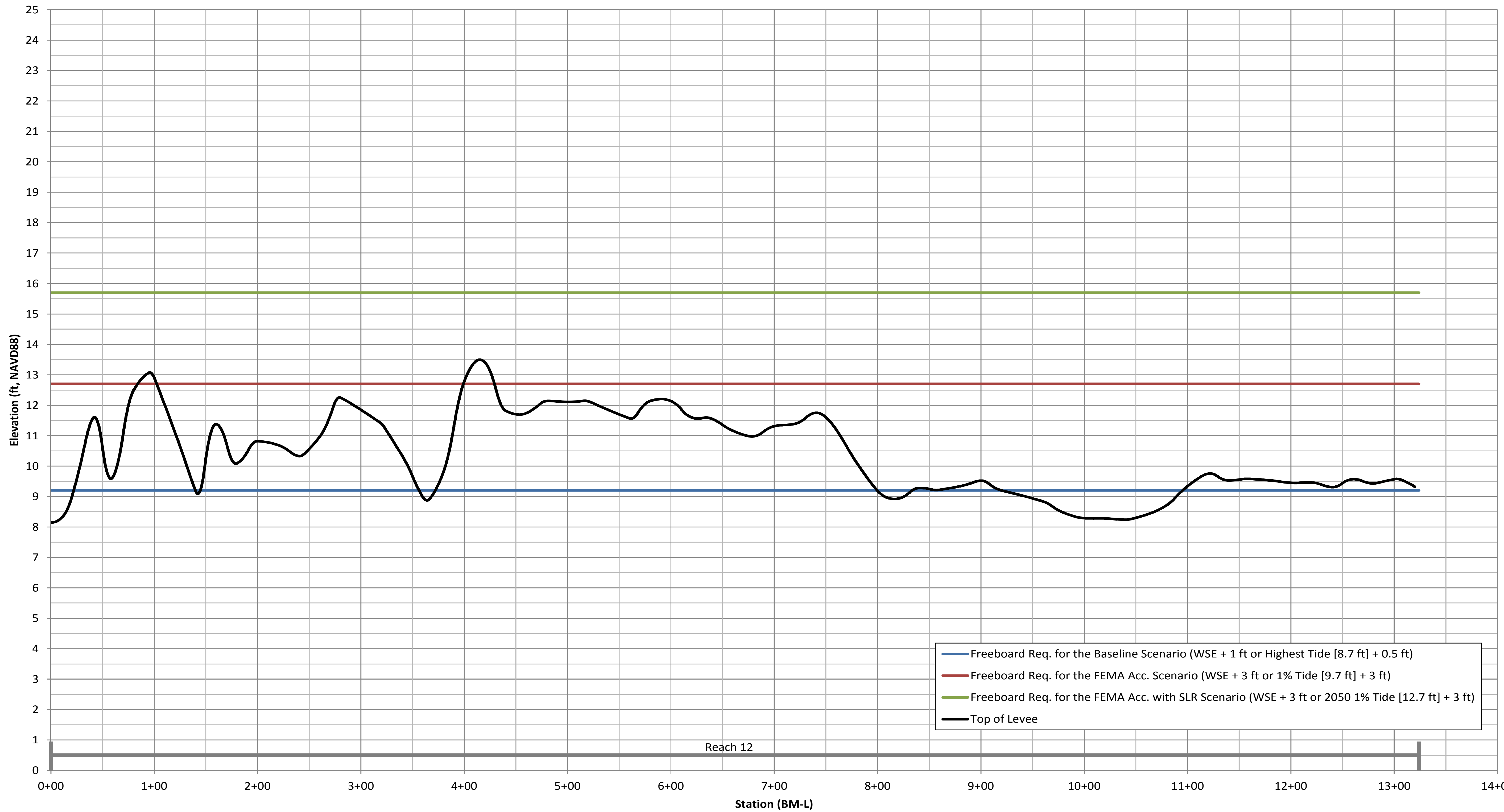
Note:
1. Approximate bridge locations estimated from available survey data.

Coyote Creek Levee Evaluation Project Marin County, California	GEI Consultants	Nyhan Creek Left Bank (NC-L) Comparison of Levee Crown Profile to Freeboard Requirements
Marin County Flood Control and Water Conservation District	Project 1404570	January 2016 Figure 99



Note:
1. Approximate bridge locations estimated from available survey data.

Coyote Creek Levee Evaluation Project Marin County, California	GEI Consultants	Nyhan Creek Right Bank (NC-R) Comparison of Levee Crown Profile to Freeboard Requirements	
Marin County Flood Control and Water Conservation District	Project 1404570	January 2016	Figure 100



Coyote Creek Levee Evaluation Project Marin County, California	GEI Consultants	Bothin Marsh Left Bank (BM-L) Comparison of Levee Crown Profile to Freeboard Requirements
Marin County Flood Control and Water Conservation District	Project 1404570	January 2016 Figure 101

Appendices

Appendix A

Hydraulic Conductivity Write-Up

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

Figure 1: Assigned Vertical Hydraulic Conductivities versus San Francisco Bay specific testing and established literature
Figure 2: Vertical Hydraulic Conductivities – SP and SP-SM (0 to 12% fines)
Figure 3: Vertical Hydraulic Conductivities –SM (12 to 49% fines)
Figure 4: Vertical Hydraulic Conductivities – SP-SC, SC, and SC-SM
Figure 5: Vertical Hydraulic Conductivities – ML, CL-ML, and CL
Figure 6: Vertical Hydraulic Conductivities – San Francisco Bay Mud

In Figures 2 through 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 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 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 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 1: Coyote Creek Hydraulic Conductivities Summary

Initial Analytical Hydraulic Conductivities

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)
Cutoff Walls		Soil-bentonite		1.0.E-06	2.83E-03	1	1.0.E-06	2.83E-03
		Soil-Cement-Bentonite		1.0.E-06	2.83E-03	1	1.0.E-06	2.83E-03
		Cement-Bentonite		1.0.E-06	2.83E-03	1	1.0.E-06	2.83E-03
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
Sandy Silt (50-70% fines)	ML	Embankment		1.5.E-05	4.25E-02	4	6.0.E-05	1.70E-01
		Natural Deposits		1.5.E-05	4.25E-02	4	6.0.E-05	1.70E-01
Sand	SP, SW	< 5% fines		2.0.E-02	5.67E+01	1	2.0.E-02	5.67E+01
	SP-SM, SW-SM	5-12% fines		4.0.E-03	1.13E+01	2	8.0.E-03	2.27E+01
	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
	SP-SC, SW-SC	5-12% fines		2.0.E-04	5.67E-01	4	8.0.E-04	2.27E+00
	SC	Natural Deposits / Embankment - Uncontrolled Placement	12-25% fines	4.0.E-05	1.13E-01	4	1.6.E-04	4.54E-01
			25-49% fines	7.0.E-06	1.98E-02	4	2.8.E-05	7.94E-02
		Embankment - Controlled Placement	12-49% fines	4.0.E-06	1.13E-02	4	1.6.E-05	4.54E-02
Gravel	GP, GW	< 5% fines		1.0.E-01	2.83E+02	1	1.0.E-01	2.83E+02
	GP-GM, GW-GM	5-12% fines		1.0.E-02	2.83E+01	2	2.0.E-02	5.67E+01
	GM	12-25% fines		1.0.E-03	2.83E+00	4	4.0.E-03	1.13E+01
	GM	25-35% fines		3.0.E-04	8.50E-01	4	1.2.E-03	3.40E+00
	GM	35-49% fines		1.0.E-04	2.83E-01	4	4.0.E-04	1.13E+00
	GP-GC, GW-GC	5-12% fines		3.0.E-03	8.50E+00	4	1.2.E-02	3.40E+01
	GC	12-25% fines		5.0.E-04	1.42E+00	4	2.0.E-03	5.67E+00
	GC	25-49% fines		7.0.E-05	1.98E-01	4	2.8.E-04	7.94E-01

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

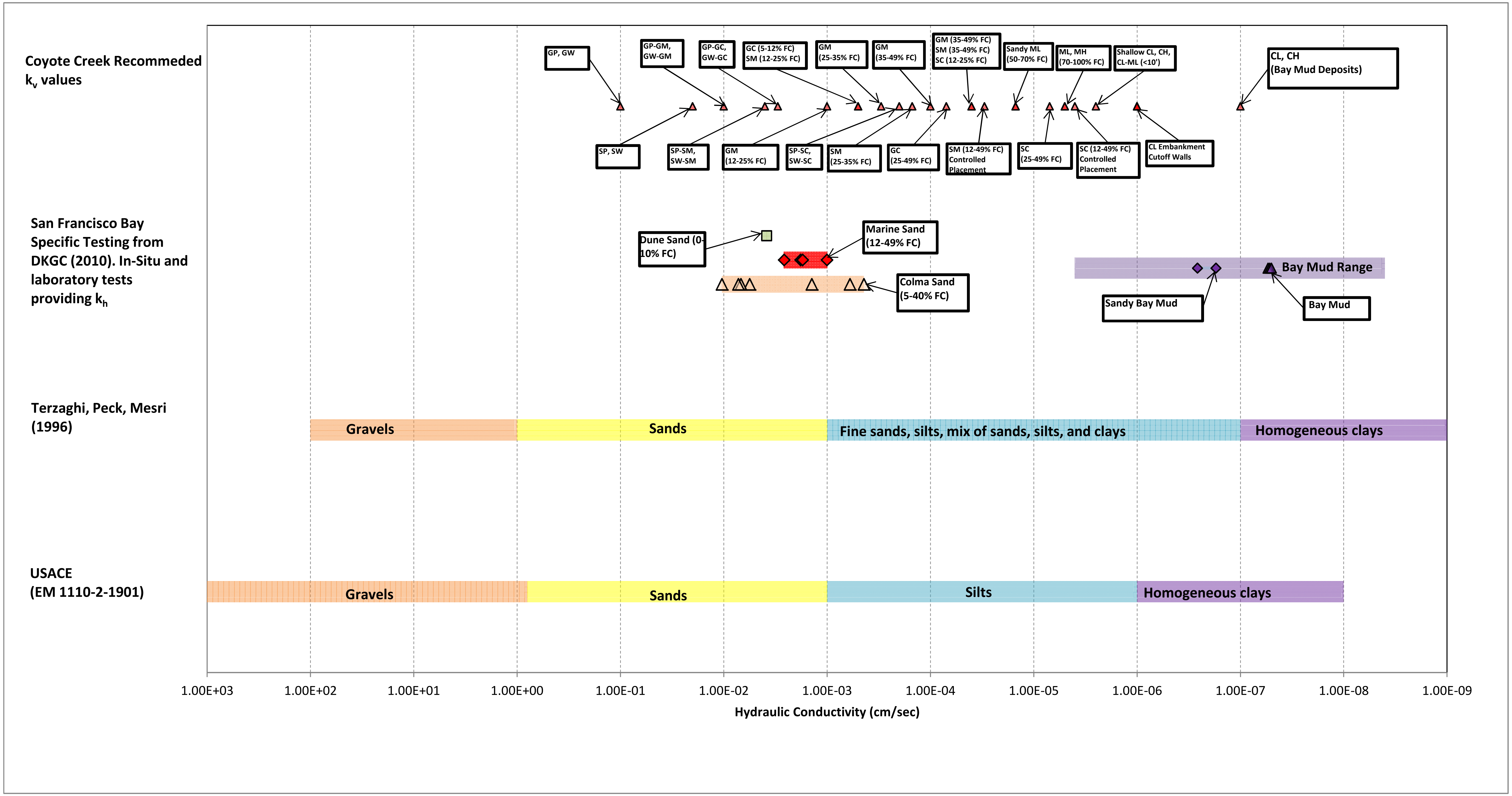
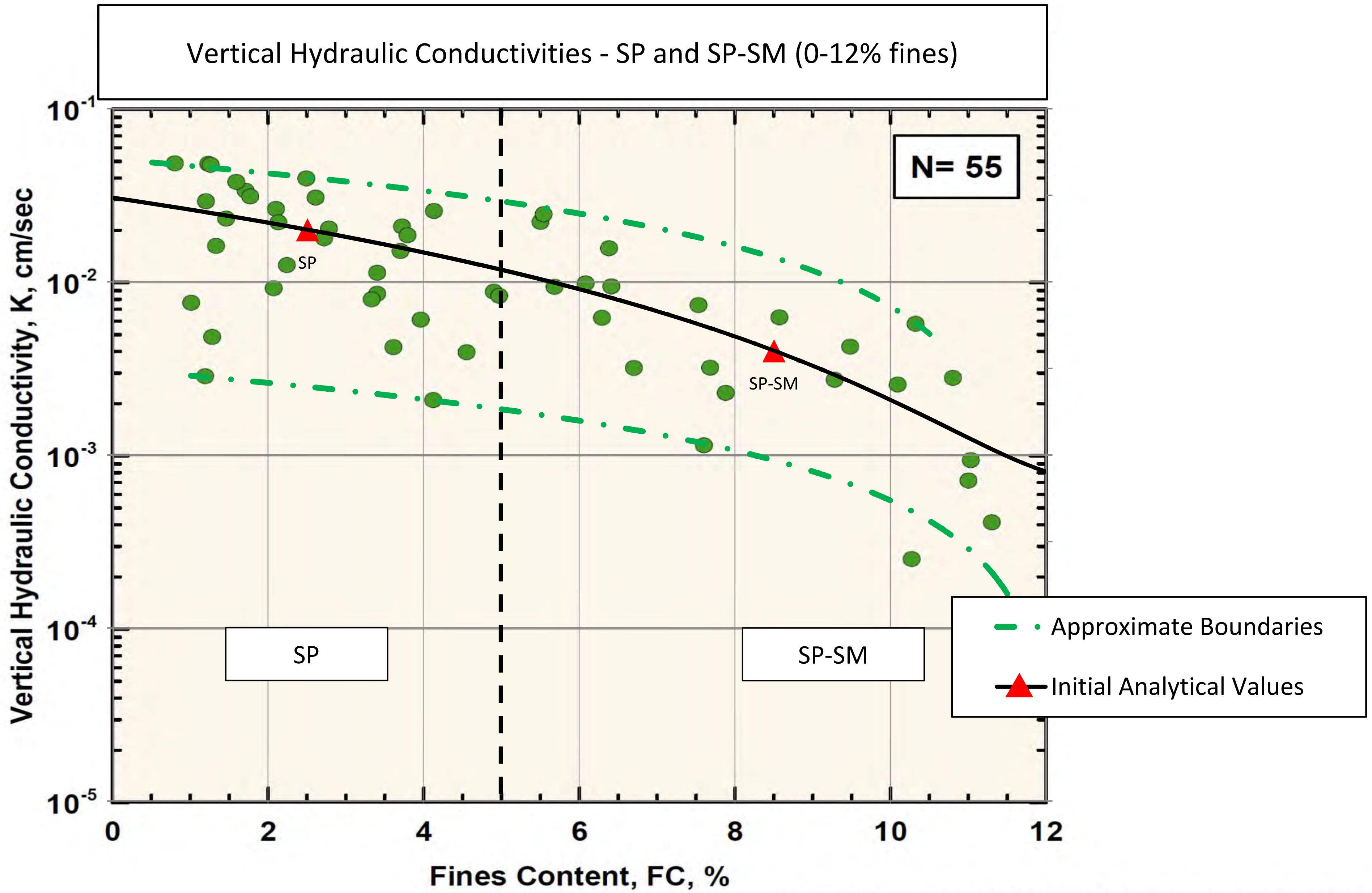


Figure 1: Assigned Vertical Hydraulic Conductivities versus San Francisco Bay Specific Testing and Established Literature



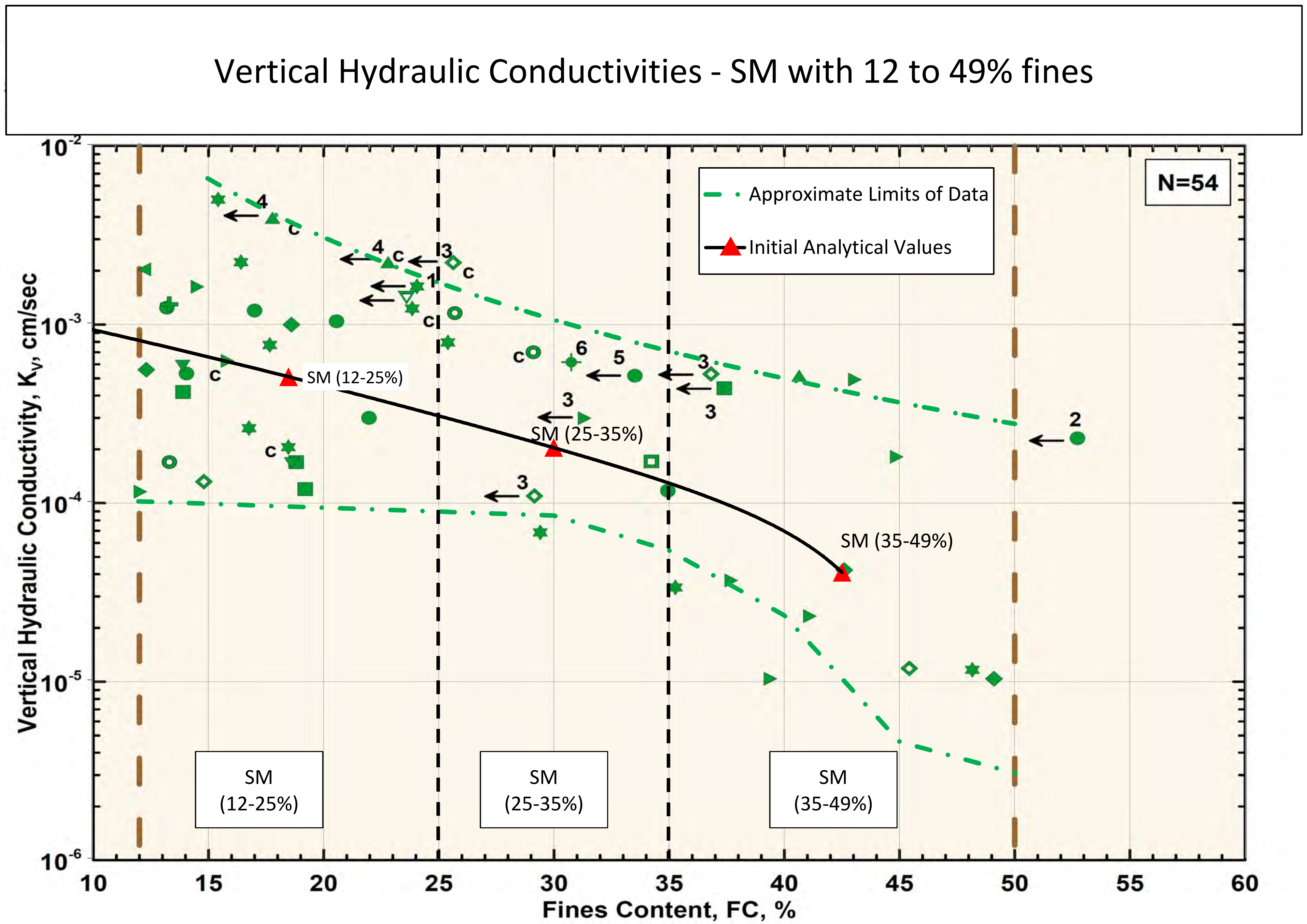
VARIATION OF HYDRAULIC CONDUCTIVITY WITH FINES CONTENT: RESULTS OF TESTS ON HOMOGENEOUS, RELATIVELY CLEAN SANDS (SP, SP-SM): ULE PROGRAM

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 A2-3

Figure 2: Vertical Hydraulic Conductivities SP and SP-SM (0 to 12% 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

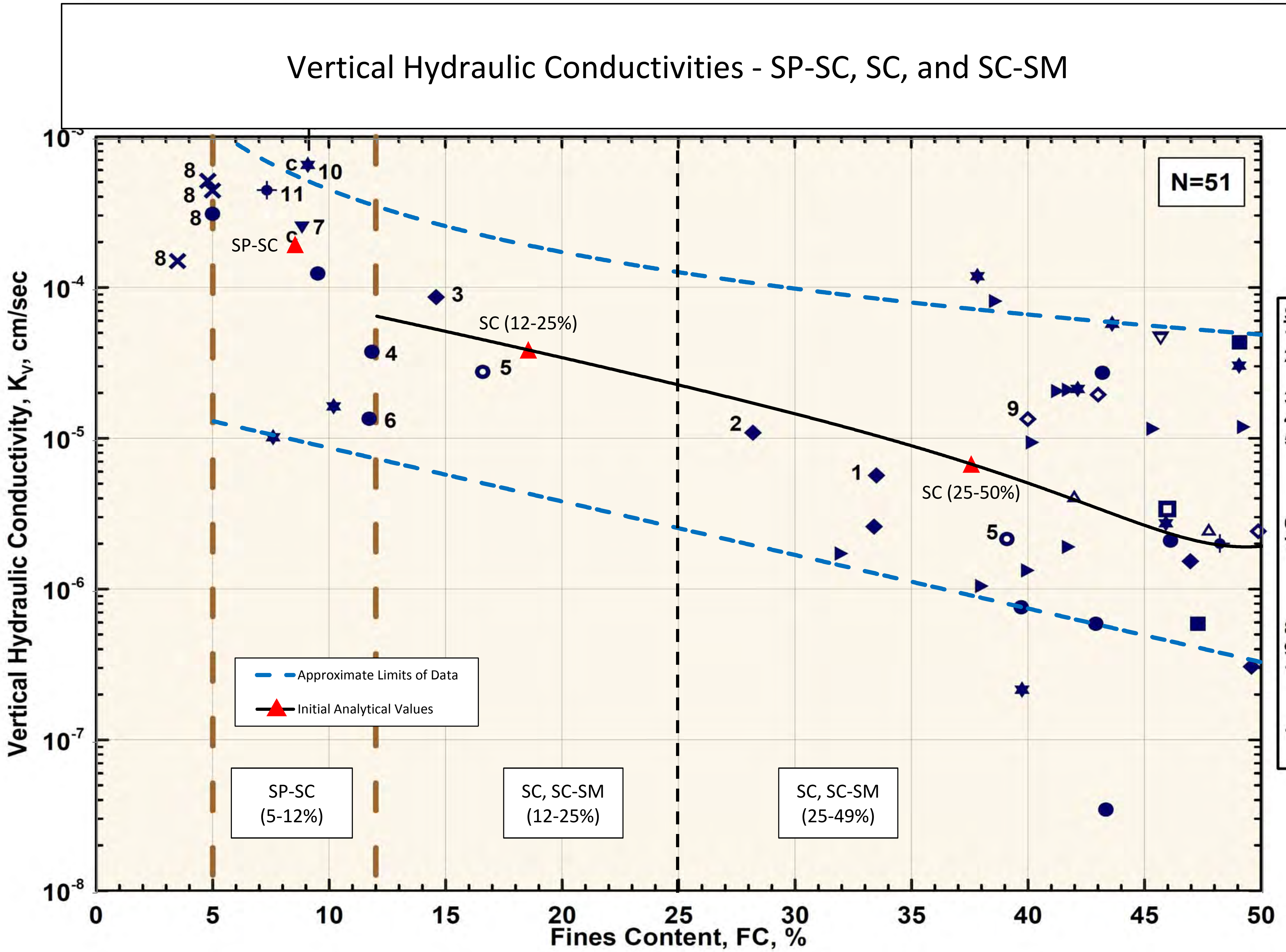
Note:
Fines Content = Percent Passing the No. 200 Sieve

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

Q:\132217\4 Internal Project Data\4-05 Reports & Narratives\2013-01 Appendix A-2 DWR Guidance Document\Appendix A-2 New Version\Figures\11X17 K vs FC Silty sand.grf

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



Symbol	Study Area	Symbol	Study Area
×	Walthall Slough	○	Rough and Ready Island
▼	Woodland	✦	South Sacramento Streams
▽	Davis	▶	Sacramento River
□	Shima Tract	c	Other tests performed using the constant head method
●	Bear Creek		
◇	Smith Canal		
■	Brookside		
◆	Lincoln Village		
★	RD 17		

Special conditions of numbered tests:

1. Interbedded clean sand, silt and clay
2. Interbedded medium to fine sand, silt and clay
3. Clean sand, with thin silt lenses
4. SP-SM with a streak of cohesive material
5. The hydraulic conductivity of this specimen of silty sand was controlled by the presence of thin seams of silt / clay that extended over the entire cross section of the specimen. Behaves like SC material.
6. SP-SM with two subhorizontal streaks of cohesive material
7. This sample had two thin streaks of fine grained materials that caused lower hydraulic conductivity than if the specimen was actually uniform SP-SM material. Consequently it behaves like SP-SC material.
8. SP soil with streaks of fine grained soil (behaves like SP-SC)
9. SP over ML soil
10. Hydraulic conductivity of this SP-SM specimen is lower than expected because of the presence of thin stringers of fine grained material (behaves like SP-SC)
11. Streak of black material through this SP-SM specimen indicating possibly the presence of cohesive fines

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

VARIATION OF VERTICAL HYDRAULIC CONDUCTIVITY
WITH FINES CONTENT: CLAYEY SANDS (SP-SC, SM-SC AND SC)

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

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

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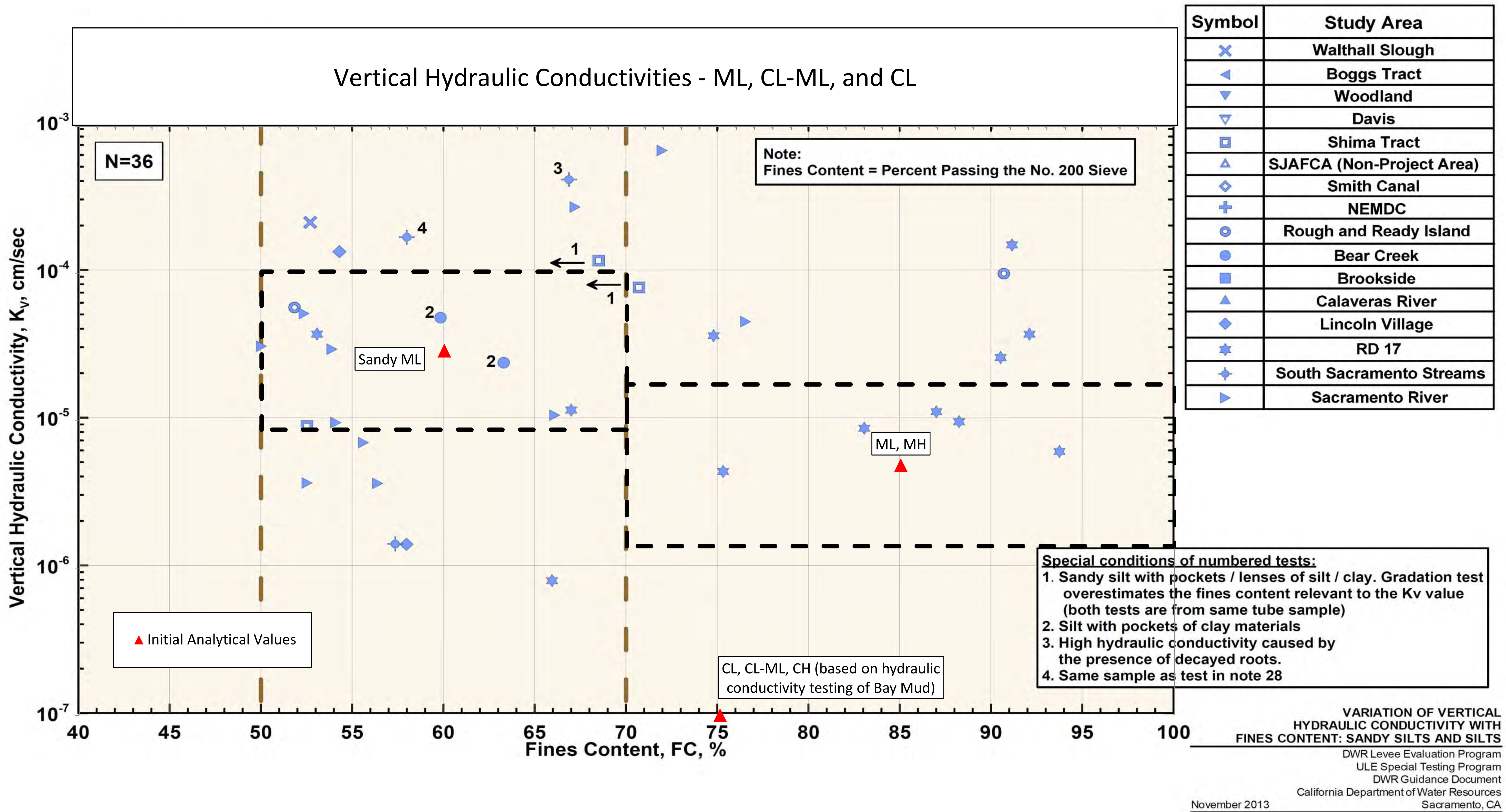
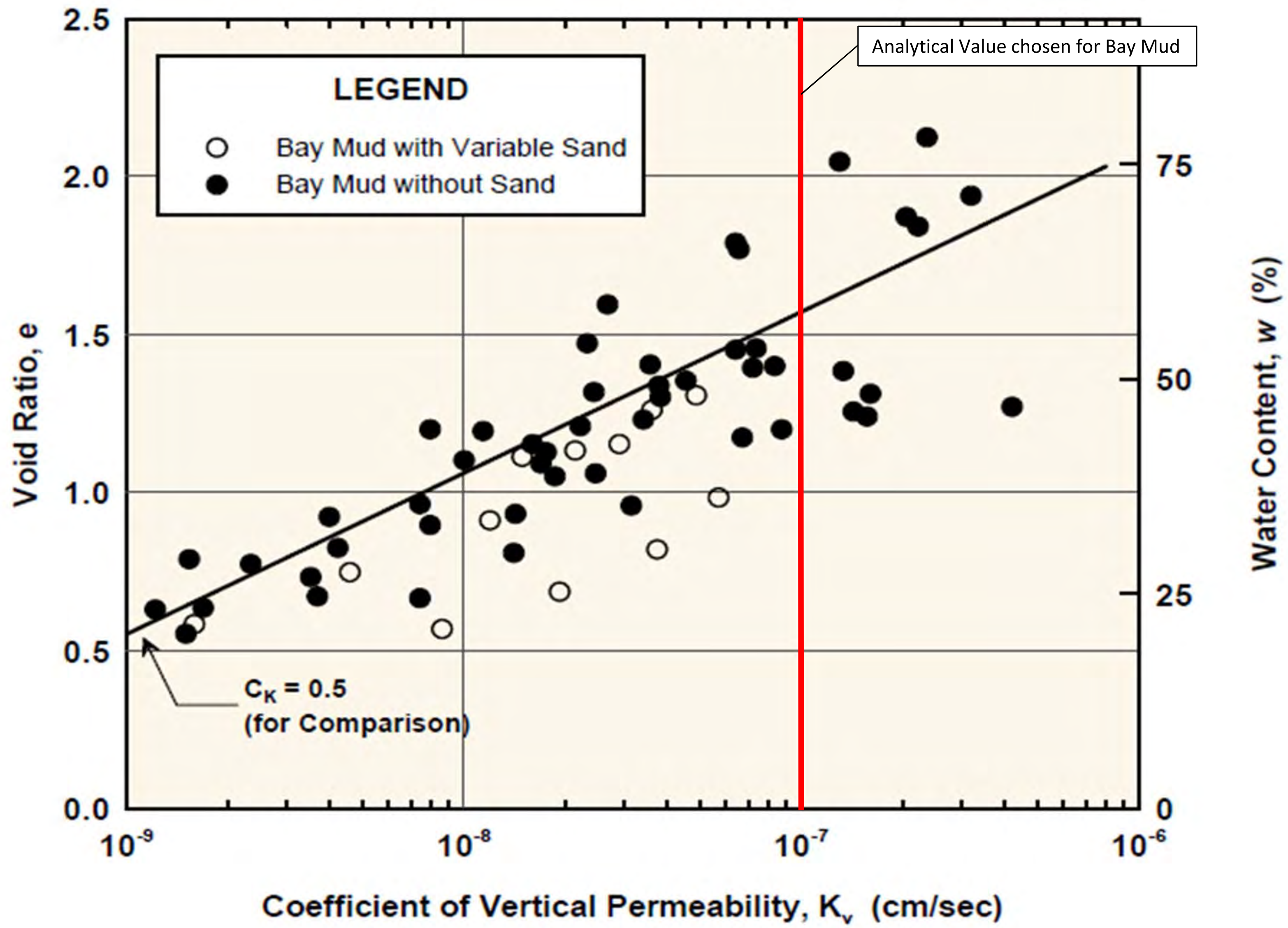


Q:\132217\4 Internal Project Data\4-05 Reports & Narratives\2013-01 Appendix A-2 DWR Guidance Document\Appendix A-2 New Version\Figures\11X17 K vs FC Silts.grf

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

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

Q:\132217\4 Internal Project Data\4-05 Reports & Narratives\2011-07 Appendix A\Figures\Fig A-24 - Dames&Moore Permeability Bay Mud.grf

Figure 6: Vertical Hydraulic Conductivities San Francisco Bay Mud

Appendix B

Strength Parameter Write-Up

Approach for Development of Strength Parameters

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.

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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.

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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.

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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

GEI Consultants in association with HDR

Client:	Marin County FCWCD	**Prepared By:**	I. Maki
Project:	Coyote Creek Levee Evaluation Project	**Date:**	06/24/2015
Project No.:	1404570	**Checked By:**	G. Bradner & M. Stanley
	Approach for Development of Strength Parameters	**Date:**	06/24/2015

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 Coyote Creek 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, CPT 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 Bay Mud (see Figures 1 and 2). 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 will perform 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

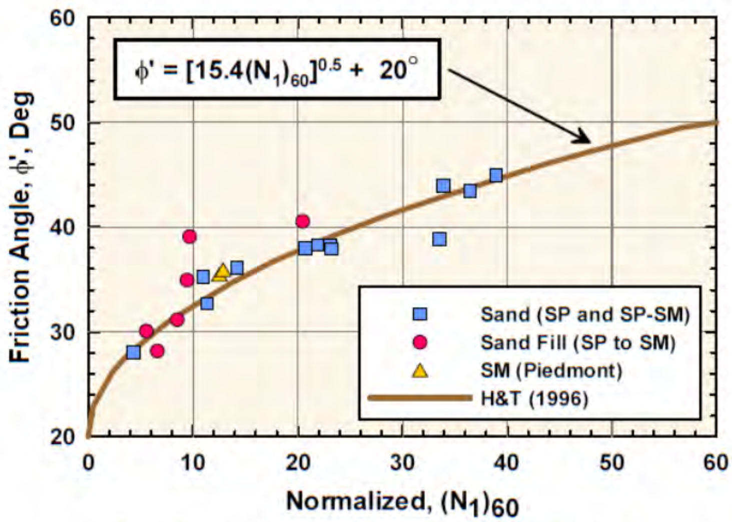
We estimated drained shear strengths of predominantly coarse-grained soils using empirical correlations to SPT N-values and CPT normalized tip resistance.

As recommended in the CA DWR (2014), Urban Levee Geotechnical Evaluations Program, Guidance Document for Geotechnical Analyses, Version 14 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).

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, Pitcher Drilling Company provided recent hammer energy correlations for both drill rigs used during the exploration program. For historical borings, we used the hammer energy or the N_{60} noted on the boring logs. If neither the hammer energy nor N_{60} were available, we assumed that the hammer energy was equal to 75% if automatic hammer was noted. We assumed a hammer energy equal to 60% (no correction) for all other historical borings.

$N_{1(60)}$ Overburden Correction

An overburden correction factor C_N 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:

$$C_N = \sqrt{\frac{P_a}{\sigma'_{v0}}}$$

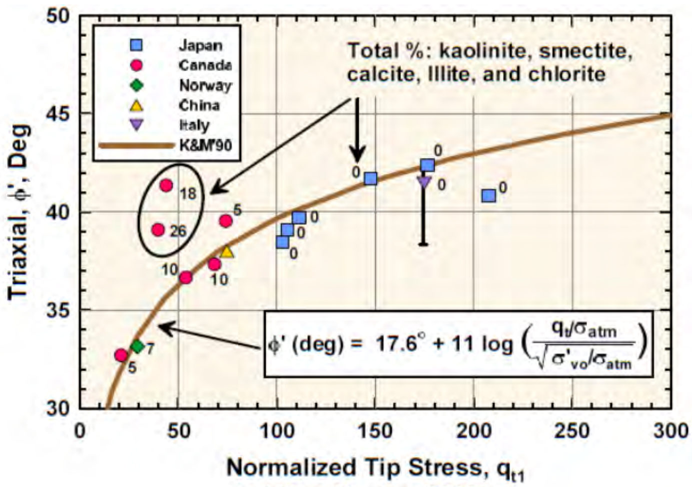
Where P_a is the atmospheric pressure (equal to 2,116 psf) and σ'_{vo} is the in situ effective overburden stress.

To simplify the calculation of the overburden correction factor, we assumed a total unit weight of 120 pcf for the soil above the bay mud and 95 pcf for the younger and older bay mud. If noted, we used groundwater depths on the boring logs. Otherwise, we assumed a depth to groundwater based on the groundwater elevations of nearby borings. 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. Appropriate correction factors will be used in other applications, such as liquefaction triggering analyses where correlations were developed with $N_{1(60)}$ values corrected for these factors.

<u>CPT Correlation:</u>

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

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



(After Mayne, 2007)

We selected representative drained shear strengths for each coarse grained soil layer by estimating the mean 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, incremental load consolidation (ILC) tests (ASTM D2435), and unconsolidated undrained triaxial compression (UU) tests (ASTM D2850) on undisturbed samples taken from the fine-grained soil layers (predominantly bay mud). Our testing program consisted of 2 CIUC tests, 7 ILC tests, and 8 UU tests on younger bay mud. Additional details and test data of the strength testing program were provided in the Geotechnical Data Report (*GDR*) issued in May of 2015 (GEI, 2015).

Several Atterberg limit tests were performed on embankment and shallow blanket material, with plasticity index (PI) values ranging from 5 to 12. Literature on bay mud strengths and soil indices were also reviewed for bay mud (Bonaparte and Mitchell, 1979). Bonaparte and Mitchell (1979) suggest an average PI value for bay mud to be 40.

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 a correlation of drained cohesion to maximum past pressure presented in CA DWR (2013), Urban Levee Geotechnical Evaluations Program, Guidance Document for Geotechnical Analyses, Version 13, and using available correlations of drained friction angle to plasticity index.

A table from CA DWR (2013) 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 x σ'_p ≤ 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 x σ'_p ≤ 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 σ'_p ≤ 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 x σ'_p ≤ 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 σ'_p ≤ 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 x σ'_p ≤ 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 σ'_p ≤ 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 σ'_p ≤ 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

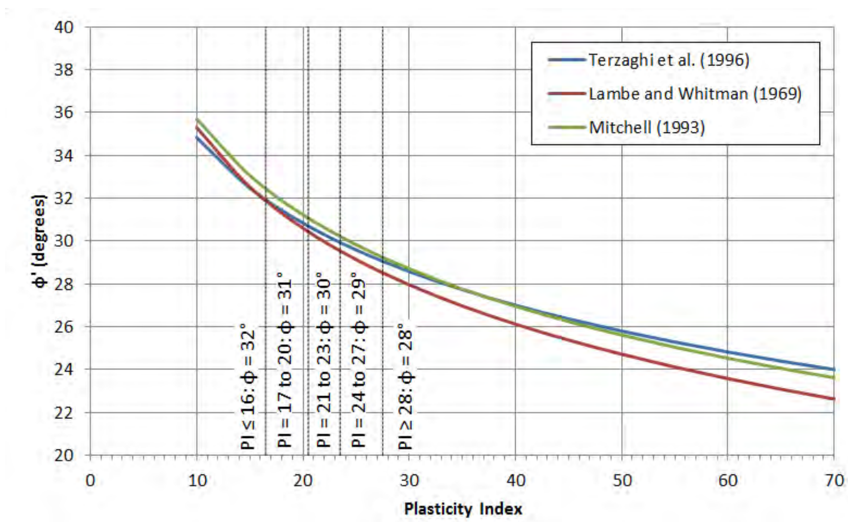
Notes:
1 See Figure 5-1 for definition of soil groups.
2 ULE analysts must consider construction processes and potential variability that might not have been captured by available tests when selecting parameters for stability.
3 For embankment soils, ULE analysts should consider the effects of construction processes and potential variability of the placement conditions when selecting conservative parameters for 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.

We estimated the drained friction angle (ϕ') of predominantly fine-grained soils using a correlation to plasticity index provided in Lambe and Whitman (1969). Correlations to plasticity index (PI) are also provided in Terzaghi et al. (1996) and Mitchell (1976). As shown below, the correlations plot very close to each other.



We developed the following table to simplify the selection of drained friction angle from PI as outlined above:

Plasticity Index (PI)	Drained Friction Angle (ϕ') Degrees
0 – 16	32
17 – 20	31
21 – 23	30
24 – 27	29
28+	28

Based on this table, the drained friction angle of bay mud was determined to be 28 degrees. No Atterberg limit data was available for the older bay mud. It is expected that this layer still contains a high plasticity index, but has also been shown to be stiffer and generally slightly overconsolidated based on in-situ testing. Therefore, the drained friction angle was generally estimated to be 30 degrees unless other site-specific data indicated otherwise.

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.

We developed soil strength parameters (S and m) for the bay mud by evaluating the ICU tests, UU tests, and ICL tests at a range of effective stresses in the bay mud and fitting the soil strength parameters to the laboratory testing. For this project an <u>S value of 0.25 and m value of 0.8 were</u> chosen.

4b) Calibrate Undrained Shear Strength (S_u) 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, q_t 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).

We modified this formula for cases where CPT data is available above the water table, where soil may be unsaturated. We assumed that cone penetration is a drained process above the water table. In order to estimate the undrained strength of silts and clays from drained CPT data, we divided the net tip resistance by a factor Q_{drn}/Q_{und} which is the ratio of drained net cone tip resistance to undrained net cone tip resistance. Based on a database of soils (DeJong et al. 2012), Q_{drn}/Q_{und} typically ranges from 2 to 3 for fine grained soils as were observed within the project site. The equation for undrained shear strength above the water table then becomes:

$$S_u = \frac{\dfrac{(q_t - \sigma_{vo})}{Q_{drn} / Q_{und}}}{N_{kt}}$$

To calibrate N_{kt} and Q_{drn}/Q_{und}, we identified 4 locations of laboratory measured S_u from ICU and UU tests nearby CPT q_t measurements. Based on the test data, we selected N_{kt} = 13 and Q_{drn}/Q_{und} = 2.0 to estimate undrained shear strengths from CPT data. Comparisons of laboratory test data and CPT data are provided in Figures 3 through 10 of the attachments.

4c) Undrained Shear Strength (S_u) Estimates from Pocket Penetration Data and Torvane Data

CPT data was not available in several reaches. For these reaches pocket penetration data and torvane data taken from borings within the reach was utilized, generally in conjunction with UU tests to better understand the undrained shear strength within the bay mud.

Based on literature regarding the shear strength of deep sea and marine deposits from Blum, 1997, the pocket penetration test determines shear failure of the soil in the same manner as UU tests, providing the deviator stress ($\Delta\sigma_f$). The shear strength is then determined by dividing $\Delta\sigma_f$ by 2.

Torvane tests provide the shear strength of fine grained soils under similar loading conditions as direct simple shear tests (DSS). As such, the undrained shear strength (S_u) measured from torvane requires normalizing the measured S_u values by a ratio dependent on the drained friction angle in order to compare to UU, ICU, and pocket penetration test loading conditions. According to EPRI (1990) the measured S_u of the torvane (DSS loading) should be multiplied by approximately 1.5 for a drained friction angle of 28 degrees to convert to isotropically consolidated undrained triaxial test (ICU) loading. Therefore, 1.5 was used as a multiplier for torvane measurements in bay mud deposits.

4d) 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 0.2 was used for younger bay mud and a k_{OCR} value of 0.4 for older bay mud. As shown in Figures 3 through 10 of the attachments (where CPT data is available) a k_{OCR} value of 0.2 generally results in a normally to lightly overconsolidated soil for the younger bay mud, and a k_{OCR} value of 0.4 generally results in a lightly overconsolidated to overconsolidated soil for the older bay mud.

4e) Estimate Maximum Past Pressure (σ'_{vm}) for each fine-grained soil layer

We prepared a figure of maximum past pressure and overconsolidation ratio for each fine-grained soil layer at each Reach. These figures are shown in Figures 3 through 10 of the attachment. The plot includes available CPT data, laboratory consolidation test data, and the maximum past pressure determined by the calculated pocket penetration and torvane undrained shear strength using the SHANSEP relationship.

4e) Estimate Range of Effective Stresses (σ'_{vo}) in each fine-grained soil layer during flood

We developed a plot of effective stress during the analysis flood event for each cross section (see Figures 3 through 10 of the attachment). Effective stresses will be lowest during the flood, when the river water level and soil pore pressures are highest. Therefore, this condition was selected to represent the lower bound range of operating stresses for the stability evaluation. We assumed at this point, that the water level is located at the crest elevation or in cases that do not currently contain a levee, at the elevation of high ground.

4f) 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 (Su) 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 or pocket penetration 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 2012 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:

$$\tau_{ff} = c + \sigma'_v * \tan(\varphi)$$

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):

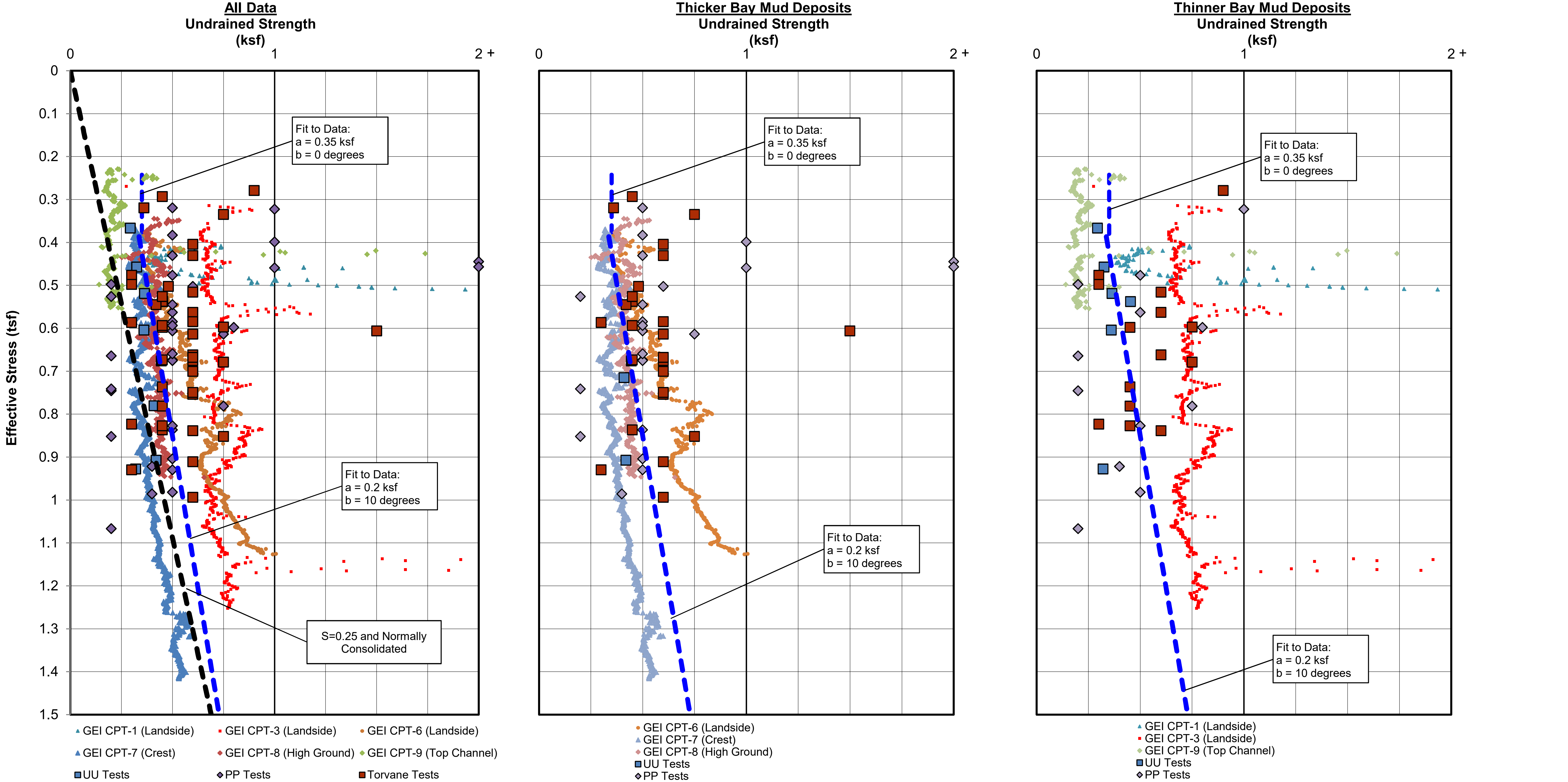
$$\phi = \sin^{-1}\left(\frac{\tan(b)}{\cos(\phi') + \tan(b)}\right)$$

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

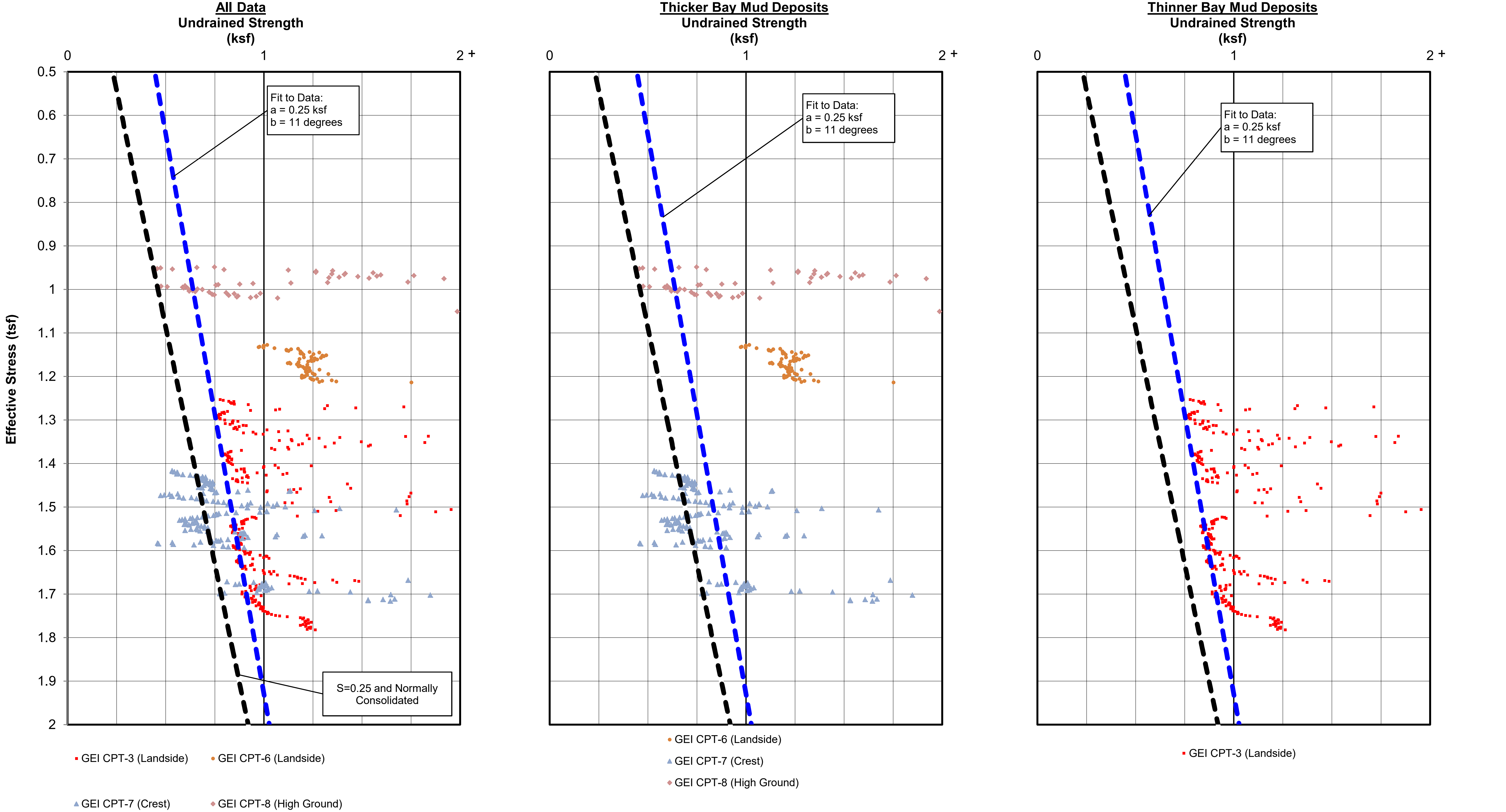
We have verified this procedure with the developers of SLOPE/W via personal communication.

Client:	Marin County FCWCD	**Prepared By:**	I. Maki
Project:	Coyote Creek Levee Evaluation Project	**Date:**	06/24/2015
Project No.:	1404570	**Checked By:**	G. Bradner & M. Stanley
	Approach for Development of Strength Parameters	**Date:**	06/24/2015

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 2012. For laterally continuous soil units, such as younger and older Bay Mud, all available undrained strength data determined by in-situ and laboratory testing was plotted versus vertical effective stress to understand the consistency of undrained soil strength (see Figures 1 and 2). This plot provides a project-specific fit of undrained strength versus effective stress for younger bay mud and older bay mud that is applied consistently to every Reach. The undrained shear strength data was also filtered by depositional environment (thicker and thinner bay mud deposits). The data from the thicker deposits generally suggests a smaller deviation of data than the thinner bay mud deposits. However, both depositions can be fit with the same function of "a" and "b" parameters. 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, laboratory 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. Remedial conditions analysis will also consider changes in soil properties due to changing conditions occurring during construction, directly after construction, and long-term after construction.



Coyote Creek Levee Evaluation Project Marin County, California	GEI Consultants	Younger Bay Mud Undrained Strength Data VS. Elevation
Marin County Flood Control and Water Conservation District	Project 140457-0	June 2015 Fig. 1



Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

Older Bay Mud Undrained Strength Data VS. Elevation

Project 140457-0 | June 2015 | Fig. 2

Reach 3, Sta. 28+00 CC-L

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) GP-GM	130	0	38	NA	NA
(2) Sandy CL	120	60	32	600	4
(3) CH	95	0	30	350	0
(4) CH	95	0	28	170	8
(5) CL	95	0	30	210	9
(6) Siltstone	NA	NA	NA	NA	NA
(7) Bentonite Cut-Off Trench	120	360	4	500	0

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

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

Exploration	Station	Offset (ft)	Location	Ground Surface Elevation (ft, NAVD 88)	Drilled Depth (ft)
GEI_B-9	28+00	0	Crest	9.1	28.4
GEI_CPT-3	30+75	40	Landside	6.2	66.3

Note: Offset is reported as the perpendicular distance landward (positive) or waterward (negative) of the levee centerline

Fig. 3 is a summary of the data from these explorations.

Coarse-Grained Soils:

Layer 1 (GP-GM)

Layer 1 is an embankment fill layer generally consisting of poorly graded gravel with silt. No SPT $N_{1(60)}$ values were available in this layer. The drained friction angle is estimated from engineering judgement for material type as 38°.

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

Fine-Grained Soils:

Layer 2 (Sandy CL)

Layer 2 is a foundation layer generally consisting of sandy clay and clayey sand. The SPT $N_{1(60)}$ value in this layer was 10 blows per foot (bpf). No Atterberg limits tests were performed in this layer. The estimated drained friction angle for the clayey sand based on SPT $N_{1(60)}$ 32°. The c' value was estimated from the maximum past pressure (discussed below) to be 60 psf.

The maximum past pressure was estimated from nearby GEI CPT-3 to be approximately 4 ksf.

The undrained strength values for a and b were determined by fitting the parameters to undrained strengths determined with CPT data across a range of effective stresses. The values of a and b were determined to be 0.65 ksf and 4 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 32° and c' = 60 psf.</u>
<u>We used total strength parameters of b = 4 ° and a = 650 psf, which for analysis purposes converts to: ϕ = 4° and c = 600 psf.</u>

Layer 3 (CH)
Layer 3 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ values in this layer is 1 bpf. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle is 30° based on conclusions drawn by Bonaparte and Mitchell (1979) for bay mud and higher $N_{1(60)}$ and q_t values than in lower layer of CH material. The drained cohesion is assumed to be equal to 0.

The maximum past pressure was estimated from nearby GEI CPT-3 to be approximately 1.1 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.35 ksf and 0 degrees respectively.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 0° and a = 350 psf, which for analysis purposes convert to: ϕ = 0° and c = 350 psf.</u>

Layer 4 (CH)
Layer 4 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ values in this layer ranged from 0 bpf to 4 bpf. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle based on correlations to plasticity index is 28. The drained cohesion is assumed to be equal to 0. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated from nearby GEI CPT-3 to range from approximately 1.1 to 1.5 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.25 ksf and 10 degrees, respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 28° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 0° and a = 350 psf, which for analysis purposes convert to: ϕ = 8° and c = 170 psf.</u>

Layer 5 (CL)
Layer 4 is a foundation layer generally consisting of CL (older bay mud). No SPT $N_{1(60)}$ values existed in this layer. No Atterberg limits tests were performed in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general plasticity index (PI) value of 40 and liquidity index (LI) value of 1 for bay mud. Note that this material is stiffer than the younger bay mud above and may have a lower PI and LI value, but is still close to normally consolidated. The estimated drained friction angle based on correlations to plasticity index, due to being lightly overconsolidated, and engineering judgement is 30°. The drained cohesion is assumed to be equal to 0. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated from nearby GEI CPT-3 to range from approximately 3.5 to 5.0 ksf, and assumed to be 3.8 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 2). This plot provides a project-specific fit of undrained strength versus effective stress for older bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.25 ksf and 11 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 11° and a = 250 psf, which for analysis purposes convert to: ϕ = 9° and c = 210 psf.</u>

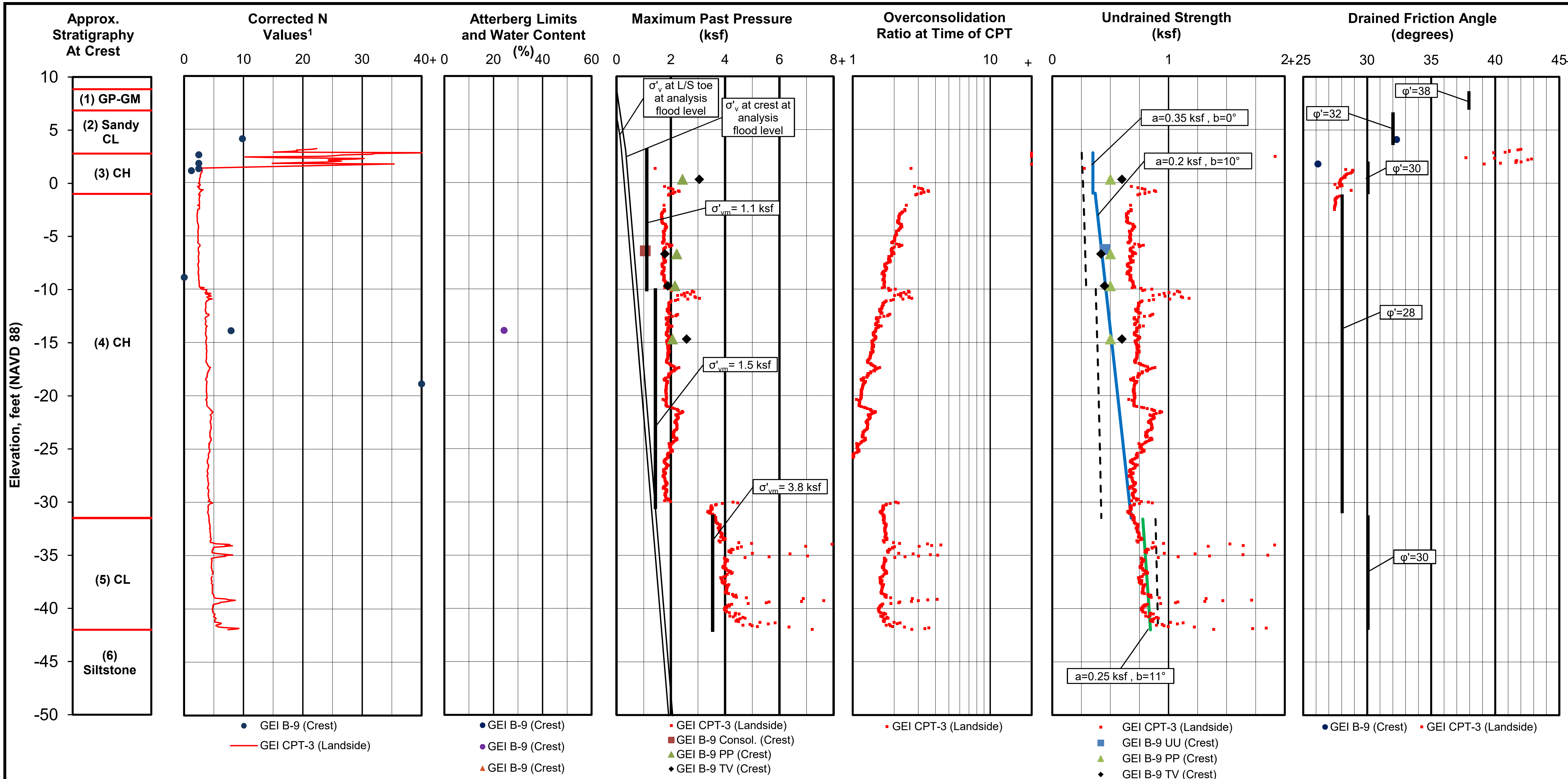
Layer 6 (Siltstone)
Layer 5 is bedrock and analyzed as the bottom of model (BOM).

<u>**Bentonite Cut-off Trench:**</u>

Layer 7 (Bentonite Cutoff Wall)
The properties for the Soil-Cement-Bentonite Cutoff Wall are assumed based on recommended properties from the DWR Guidance Document for Geotechnical Analyses (DWR, 2013).

<u>We used drained strength parameters of ϕ' = 4° and c' = 360 psf.</u>

We used total strength parameters of $\phi = 0°$ and $c = 500$ psf.



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5	6
Drained Parameters	c' (psf)	0	60	0	0	0	NA
	ϕ' (deg)	38	32	30	28	30	NA
Undrained Parameters	c (psf)	NA	600	350	170	210	NA
	ϕ (deg)	NA	4	0	8	9	NA

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. LL=Liquid Limit, w=As Received Water Content, PL=Plastic Limit.

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and
Water Conservation District

GEI Consultants

SOIL PROPERTIES VS. ELEVATION
STA. 28+00 CC-L (REACH 3)

Project 140457-0 | June 2015 | Fig. 3

Reach 4, Sta. 34+00 CC-L

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) GP-GC	130	0	38	NA	NA
(2) SC (35-49%)	120	0	36	NA	NA
(3) CH	95	0	30	350	0
(4) CH	95	0	28	170	8
(5) CL	95	0	30	210	9
(6) Siltstone	NA	NA	NA	NA	NA

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

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

Exploration	Station	Offset (ft)	Location	Ground Surface Elevation (ft, NAVD 88)	Drilled Depth (ft)
GEI_B-8	28+00	0	Crest	9.3	15.0
GEI_CPT-3	30+75	40	Landside	6.2	66.3
GEI_GP-2	30+75	40	Landside	6.4	6.0

Note: Offset is reported as the perpendicular distance landward (positive) or waterward (negative) of the levee centerline

Fig. 4 is a summary of the data from these explorations.

Coarse-Grained Soils:

Layer 1 (GP-GC)

Layer 1 is an embankment fill layer generally consisting of poorly graded gravel with clay. The SPT $N_{1(60)}$ values in this layer ranged from 49 blows per foot (bpf) to 128 bpf, indicating a dense to very dense soil. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) is larger than 45°, but this correlation was determined for sands and did not account for gravels such as exist in this soil. For this reason the drained friction angle was capped at 38°.

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

Layer 2 (SC 35-49%)

Layer 2 is a foundation layer generally consisting of clayey sand. The SPT $N_{1(60)}$ value in this layer was 18 bpf, indicating a medium dense soil. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) is 36°.

Based on the correlation to tip resistance provided in DWR (2013), the friction angle estimated from CPT soundings ranges from 37° to 43°.

<u>We used drained strength parameters of ϕ' = 36° and c' = 0 psf.</u>

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

Layer 3 (CH)
Layer 3 is a foundation layer generally consisting of high plasticity clay (bay mud). No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle is 30° based on conclusions drawn by Bonaparte and Mitchell (1979) for bay mud and higher torvane, pocket penetrometer, and q_t values than in the lower layer of CH material. The drained cohesion is assumed to be equal to 0.

The maximum past pressure was estimated from nearby GEI CPT-3 to range to be approximately 1.1 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.35 ksf and 0 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 0° and a = 350 psf, which for analysis purposes convert to: ϕ = 0° and c = 350 psf.</u>

Layer 4 (CH)
Layer 4 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ value in this layer was 0 bpf. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle based on correlations to plasticity index is 28°. The drained cohesion is assumed to be equal to 0. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated from nearby GEI CPT-3 to range from approximately 1.1 to 1.5 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.2 ksf and 10 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 28° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 10° and a = 200 psf, which for analysis purposes convert to: ϕ = 8° and c = 170 psf.</u>

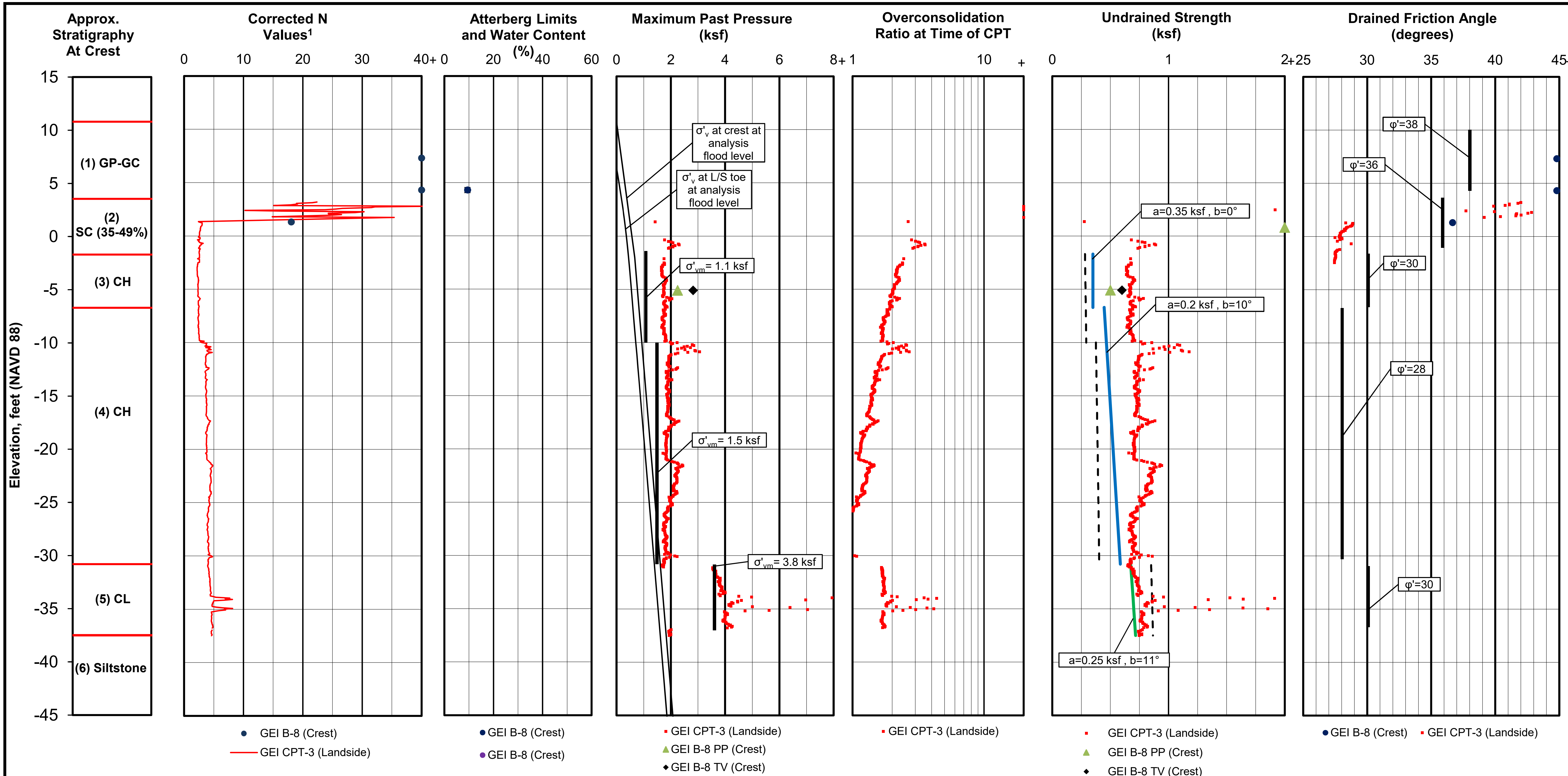
Layer 5 (CL)
Layer 4 is a foundation layer generally consisting of CL (older bay mud). No SPT $N_{1(60)}$ values existed in this layer. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general plasticity index (PI) value of 40 and liquidity index (LI) value of 1 for bay mud. Note that this material is stiffer than the younger bay mud above and may have a lower PI and LI value, but is still close to normally consolidated. The estimated drained friction angle based on correlations to plasticity index, due to being lightly overconsolidated, and engineering judgement is 30°. The drained cohesion is assumed to be equal to 0. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated from nearby GEI CPT-3 to range from approximately 3.5 to 5.0 ksf, and assumed to be 3.8 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 2). This plot provides a project-specific fit of undrained strength versus effective stress for older bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.25 ksf and 11 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 11° and a = 250 psf, which for analysis purposes convert to: ϕ = 9° and c = 210 psf.</u>

Layer 6 (Siltstone)
Layer 5 is bedrock and analyzed as the bottom of model (BOM).



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5	6
Drained Parameters	c' (psf)	0	0	0	0	0	NA
	φ' (deg)	38	36	30	28	30	NA
Undrained Parameters	c (psf)	NA	NA	350	170	210	NA
	φ (deg)	NA	NA	0	8	9	NA

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. LL=Liquid Limit, w=As Received Water Content, PL=Plastic Limit.

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and Water Conservation District

GEI Consultants

SOIL PROPERTIES VS. ELEVATION
STA. 34+00 CC-L (REACH 4)

Project 140457-0 | June 2015 | Fig. 4

Reach 5, Sta. 5+00 CC-R

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) SC (35-49%)	120	0	30	NA	NA
(2) CH	95	0	30	350	0
(3) CH	95	0	28	170	8
(4) Sandy CH	95	0	30	210	9
(5) Siltstone	NA	NA	NA	NA	NA

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

We primarily considered the following boring in our evaluation of the cross section.

Exploration	Station	Offset (ft)	Location	Ground Surface Elevation (ft, NAVD 88)	Drilled Depth (ft)
GEI_B-5	4+20	55.0	Landside Levee Toe	9.0	101.5

Note: Offset is reported as the perpendicular distance landward (positive) or waterward (negative) of the levee centerline

Fig. 5 is a summary of the data from these explorations.

Coarse-Grained Soils:

Layer 1 (SC 35-49%)

Layer 1 is a foundation layer generally consisting of clayey sand. The SPT $N_{1(60)}$ value in this layer was 6 bpf, indicating a loose soil. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) is 30°.

We used drained strength parameters of ϕ' = 30° and c' = 0 psf.

Fine-Grained Soils:

Layer 2 (CH)

Layer 2 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ value in this layer is 14 bpf. No Atterberg limits tests were performed in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle is 30° based on conclusions drawn by Bonaparte and Mitchell (1979) for bay mud and higher $N_{1(60)}$ values than in the lower layers of CH material. The drained cohesion is assumed to be equal to 0.

The maximum past pressure was estimated from pocket penetrometer data, one consolidation test, and engineering judgement in nearby boring GEI B-5 to range from approximately 1 ksf to 2 ksf (normally consolidated). An average value of 1.5 ksf was applied to the layer.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.35 ksf and 0 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 0° and a = 350 psf, which for analysis purposes convert to: ϕ = 0° and c = 350 psf.</u>

Layer 3 (CH)

Layer 3 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ values in this layer range from 0 bpf to 2 bpf. No Atterberg limits tests were performed in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle based on correlations to plasticity index is 28°. The drained cohesion is assumed to be equal to 0. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated from pocket penetrometer data, one consolidation test, and engineering judgement in nearby boring GEI B-5 to range from approximately 1ksf to 2 ksf (normally consolidated). Average values of 1.5 ksf and 2.0 ksf were applied to the layer depending on depth.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.2 ksf and 10 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 28° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 10° and a = 200 psf, which for analysis purposes convert to: ϕ = 8° and c = 170 psf.</u>

Layer 4 (Sandy CH)

Layer 4 is a foundation layer generally consisting of sandy CH (older bay mud). No SPT $N_{1(60)}$ values existed in this layer. No Atterberg limits tests were perform in this layer, but existing literature

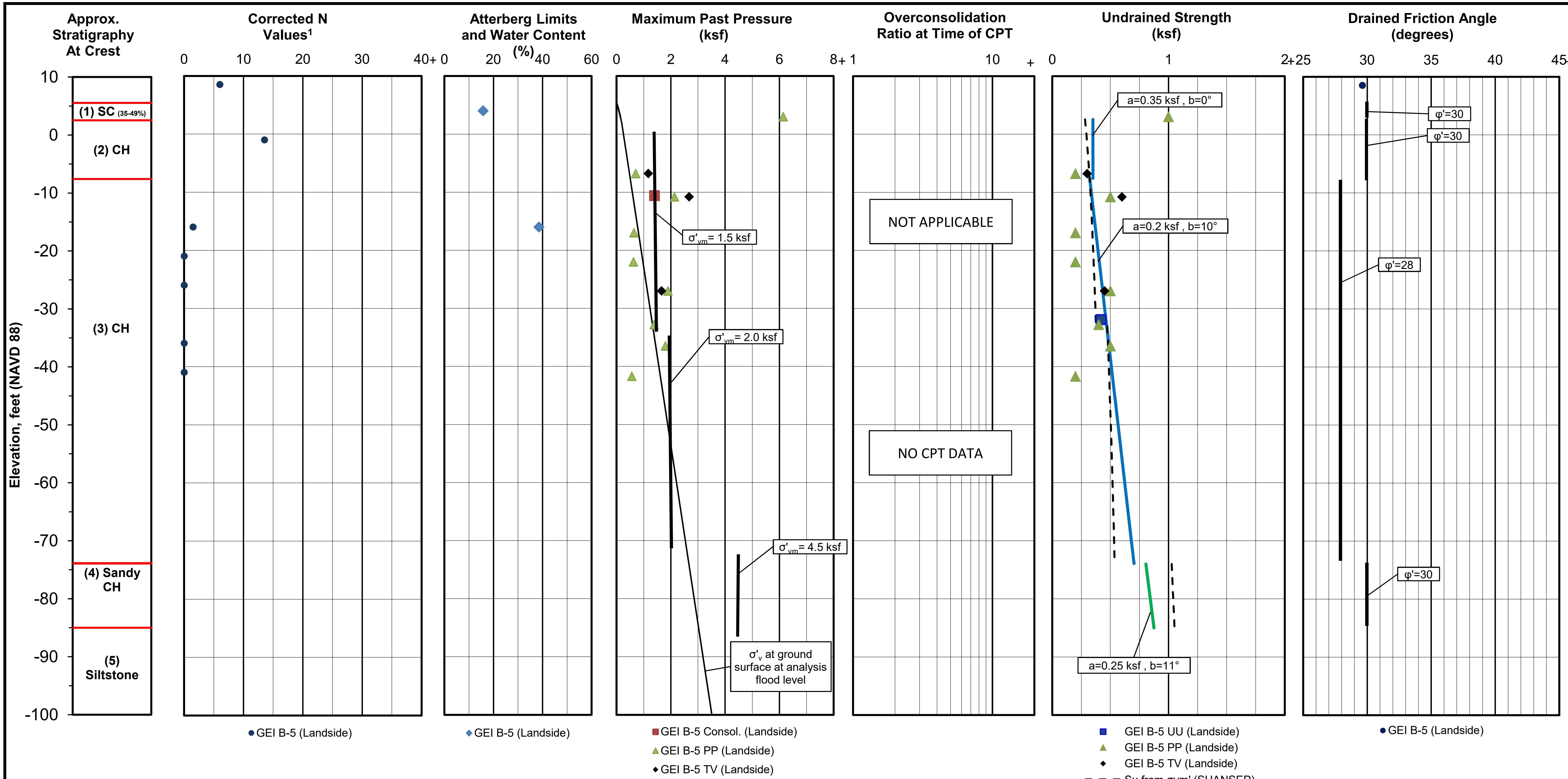
(Bonaparte and Mitchell, 1979) indicate a general plasticity index (PI) value of 40 and liquidity index (LI) value of 1 for bay mud. Note that this material is stiffer than the younger bay mud above and may have a lower PI and LI value, but appears to still be close to normally consolidated. The estimated drained friction angle based on correlations to plasticity index, due to being normally consolidated to lightly overconsolidated, and engineering judgement is 30°. The drained cohesion value is assumed to be 0 psf. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated from pocket penetrometer data in nearby boring GEI B-5 to range from 1 ksf to 4 ksf (normally consolidated) in the CH layer above. Nearby reaches indicate a lightly overconsolidated to normally consolidated soil in the lower older bay mud layer. Based on this, an average value of 4.5 ksf (normally consolidated at this depth) is assumed for this layer.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 2). This plot provides a project-specific fit of undrained strength versus effective stress for older bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.25 ksf and 11 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ’ = 30° and c’ = 0 psf.</u>
<u>We used total strength parameters of b = 11° and a = 250 psf, which for analysis purposes convert to: ϕ = 9° and c = 210 psf.</u>

Layer 5 (Siltstone)
Layer 4 is bedrock and analyzed as the bottom of model (BOM).



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5
Drained Parameters	c' (psf)	0	0	0	0	NA
	ϕ' (deg)	30	30	28	30	NA
Undrained Parameters	c (psf)	NA	350	170	210	NA
	ϕ (deg)	NA	0	8	9	NA

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. LL=Liquid Limit, w=As Received Water Content, PL=Plastic Limit.

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SOIL PROPERTIES VS. ELEVATION
STA. 5+00 CC-R (REACH 5)

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Reach 7, Sta. 35+00 CC-R

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) SC (25-35%)	120	0	38	NA	NA
(2) SC (35-49%)	120	0	36	NA	NA
(3) CH	95	0	30	350	0
(4) CH	95	0	28	170	8
(5) Sandy CH	95	0	30	210	9
(6) Siltstone	NA	NA	NA	NA	NA

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

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

Exploration	Station	Offset (ft)	Location	Ground Surface Elevation (ft, NAVD 88)	Drilled Depth (ft)
GEI_B-2	35+00	0	Crest	10.7	45.3

Note: Offset is reported as the perpendicular distance landward (positive) or waterward (negative) of the levee centerline

Fig. 6 is a summary of the data from these explorations.

Coarse-Grained Soils:

Layer 1 (SC 25-35%)

Layer 1 is an embankment layer generally consisting of clayey sand. The SPT $N_{1(60)}$ value in this layer was greater than 40 bpf, indicating a very dense soil. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) is larger than 45°, but this correlation was determined for sands and did not account for gravels such as exist in portions of this soil. For this reason the drained friction angle was capped at 38°.

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

Layer 2 (SC 35-49%)

Layer 2 is a foundation layer generally consisting of clayey sand. The SPT $N_{1(60)}$ value in this layer ranged between 1 bpf and 25 bpf, indicating a very loose to medium dense soil. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) ranges from approximately 25° to 38°. The lower blow count is influenced by the soft younger bay mud below, and should not be weighted equally with the higher blow count above. For this reason the drained friction angle was chosen as 36°.

<u>We used drained strength parameters of ϕ' = 36° and c' = 0 psf.</u>

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

Layer 3 (CH)

Layer 3 is a foundation layer generally consisting of high plasticity clay (bay mud). No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle is 30° based on conclusions drawn by Bonaparte and Mitchell (1979) and higher torvane and pocket penetrometer values than in the lower layer of CH material. The drained cohesion value is assumed to be equal to 0 psf.

The maximum past pressure was estimated based on the cross section on the other side of Coyote Creek (Reach 4) using nearby GEI CPT-3. The maximum past pressure was estimated to be 1.1 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.35 ksf and 0 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 0° and a = 350 psf, which for analysis purposes convert to: ϕ = 0° and c = 350 psf.</u>

Layer 4 (CH)

Layer 4 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ value in this layer was 0 bpf. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle based on correlations to plasticity index is 28°. The drained cohesion value is assumed to be 0 psf. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated based on the cross section on the other side of Coyote Creek (Reach 4) using nearby GEI CPT-3. The maximum past pressure was estimated to range from 1.1 to 1.5 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.2 ksf and 10 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

We used drained strength parameters of ϕ' = 28° and c' = 0 psf.
We used total strength parameters of b = 10° and a = 200 psf, which for analysis purposes convert to: ϕ = 8° and c = 170 psf.

Layer 5 (Sandy CH)

Layer 5 is a foundation layer generally consisting of Sandy CH (older bay mud). No SPT $N_{1(60)}$ values existed in this layer. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general plasticity index (PI) value of 40 and liquidity index (LI) value of 1 for bay mud. Note that this material is stiffer than the younger bay mud above and may have a lower PI and LI value, but appears to still be close to normally consolidated. The estimated drained friction angle based on correlations to plasticity index, due to being lightly overconsolidated, and engineering judgement is 30°. The drained cohesion value is assumed to be 0 psf. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

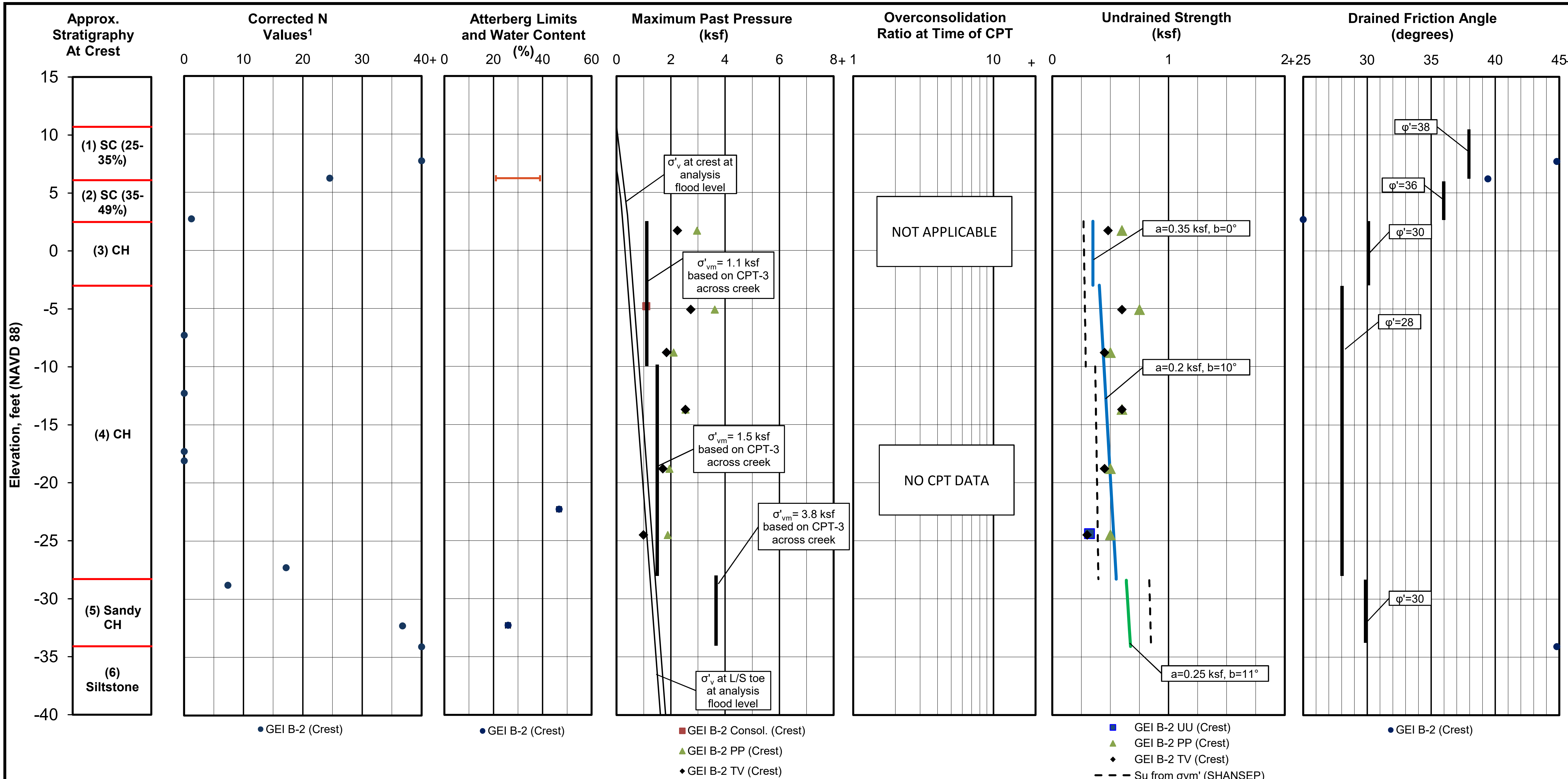
The maximum past pressure was estimated based on the cross section on the other side of Coyote Creek (Reach 4) using nearby GEI CPT-3. The maximum past pressure was estimated from nearby GEI CPT-3 to be approximately 3.8 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 2). This plot provides a project-specific fit of undrained strength versus effective stress for older bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.25 ksf and 11 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

We used drained strength parameters of ϕ' = 30° and c' = 0 psf.
We used total strength parameters of b = 11° and a = 250 psf, which for analysis purposes convert to: ϕ = 9° and c = 210 psf.

Layer 6 (Siltstone)

Layer 6 is bedrock and analyzed as the bottom of model (BOM).



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5	6
Drained Parameters	c' (psf)	0	0	0	0	0	NA
	ϕ' (deg)	38	36	30	28	30	NA
Undrained Parameters	c (psf)	NA	NA	350	170	210	NA
	ϕ (deg)	NA	NA	0	8	9	NA

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. LL=Liquid Limit, w=As Received Water Content, PL=Plastic Limit.

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STA. 35+00 CC-R (REACH 7)

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Fig. 6

Reach 8, Sta. 4+00 CC-C

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) SC (12-25%)	120	0	35	NA	NA
(2) CL	120	0	32	0	12
(3) CH	95	0	28	170	8
(4) Sandy CL	95	50	32	210	9
(5) Siltstone	NA	NA	NA	NA	NA

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

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

Exploration	Station	Offset (ft)	Location	Ground Surface Elevation (ft, NAVD 88)	Drilled Depth (ft)
GEI_B-1	3+75	60.0	Landside of Channel	7.5	21.3

Note: Offset is reported as the perpendicular distance landward (positive) or waterward (negative) of the levee centerline

Fig. 7 is a summary of the data from these explorations.

Coarse-Grained Soils:

Layer 1 (SC 12-25%)
Layer 1 is a foundation layer generally consisting of clayey sand. No SPT $N_{1(60)}$ values were available in this layer. The drained friction angle is estimated from engineering judgement for material type as 35°.

We used drained strength parameters of ϕ' = 35° and c' = 0 psf.

Fine-Grained Soils:

Layer 2 (CL)
Layer 2 is a foundation layer generally consisting of low plasticity clay. The SPT $N_{1(60)}$ value in this layer was 0 bpf. Atterberg limits tests were perform in this layer, with a PI value of 12. The estimated drained friction angle based on site-specific correlations to plasticity index is 32°. The drained cohesion is assumed to be equal to 0.

The maximum past pressure is assumed to be 0.25 ksf, based on the low blow counts and assumption that the layer is normally consolidated.

The undrained strength values for a and b were determined by using the SHANSEP relationship with values of S=0.25 and m=0.8. The layer is assumed to be normally consolidated which leads to an a value of 0 and a b value of 14 degrees.

<u>We used drained strength parameters of ϕ' = 32° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 14° and a = 0 psf, which for analysis purposes convert to: ϕ = 12° and c = 0 psf.</u>

Layer 3 (CH)

Layer 3 is a foundation layer generally consisting of high plasticity clay (bay mud). There were no SPT $N_{1(60)}$ values in this layer. No Atterberg limits tests were performed in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle based on correlations to plasticity index is 28°. The c' is assumed equal to 0 psf. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated based on pocket penetration data and engineering judgement. The maximum past pressure was estimated to be approximately 2.0 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.2 ksf and 10 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 28° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 10° and a = 200 psf, which for analysis purposes convert to: ϕ = 8° and c = 170 psf.</u>

Layer 4 (Sandy CL)

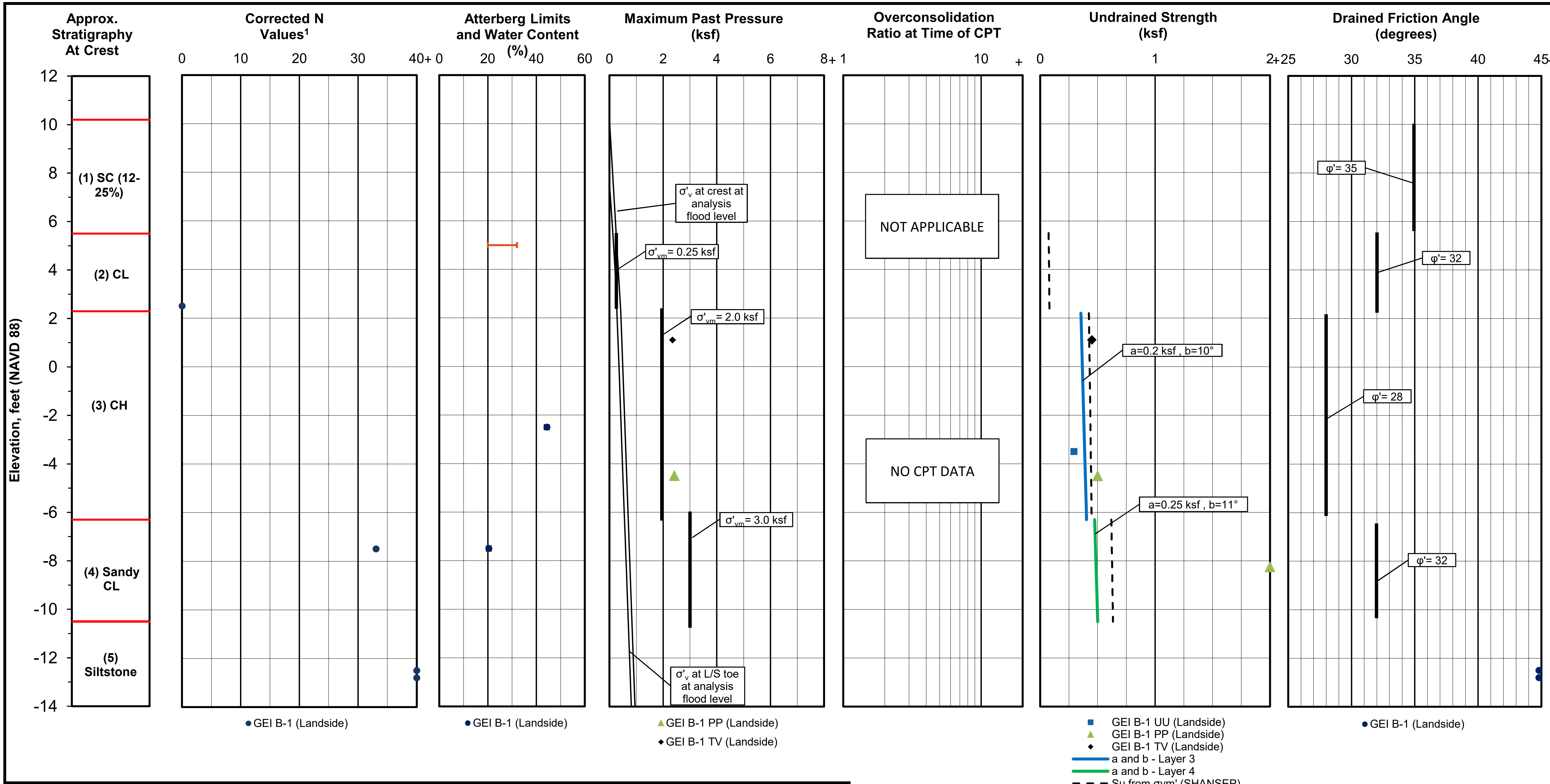
Layer 4 is a foundation layer generally consisting of Sandy CL (older bay mud). The SPT $N_{1(60)}$ value in this layer was 33. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general plasticity index (PI) value of 40 and liquidity index (LI) value of 1 for bay mud. However, this material is stiffer than the younger bay mud located above based on the measured SPT $N_{1(60)}$ value and may have a lower PI and LI value. The estimated drained friction angle based on correlations to plasticity index and due to distinctly higher blow counts is 32°. The c' is assumed equal to 50 psf based on the measured SPT $N_{1(60)}$ value.

The maximum past pressure was estimated based on pocket penetration data and engineering judgement. The maximum past pressure was estimated to be overconsolidated, and approximately 3.0 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 2). This plot provides a project-specific fit of undrained strength versus effective stress for older bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.25 ksf and 11 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ’ = 32° and c’ = 50 psf.</u>
<u>We used total strength parameters of b = 11° and a = 250 psf, which for analysis purposes convert to: ϕ = 9° and c = 210 psf.</u>

Layer 5 (Siltstone)
Layer 5 is bedrock and analyzed as the bottom of model (BOM).



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5
Drained Parameters	c' (psf)	0	0	0	50	NA
	ϕ' (deg)	35	32	28	32	NA
Undrained Parameters	c (psf)	NA	0	170	210	NA
	ϕ (deg)	NA	12	8	9	NA

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. LL=Liquid Limit, w=As Received Water Content, PL=Plastic Limit.

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SOIL PROPERTIES VS. ELEVATION
STA. 4+00 CC-C (REACH 8)

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Reach 9, Sta. 1+00 NC-L

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) SC (12-25%)	120	0	38	NA	NA
(2) SC (12-25%)	120	0	38	NA	NA
(3) CH	95	0	30	350	0
(3) CH	95	0	28	170	8
(4) Sandy CH	95	0	30	210	9
(5) SC (12-25%)	120	0	32	NA	NA
(6) Siltstone	NA	NA	NA	NA	NA

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

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

Exploration	Station	Offset (ft)	Location	Ground Surface Elevation (ft, NAVD 88)	Drilled Depth (ft)
GEI_B-10	2+00	0	Crest	9.9	73.1

Note: Offset is reported as the perpendicular distance landward (positive) or waterward (negative) of the levee centerline

Fig. 8 is a summary of the data from these explorations.

Coarse-Grained Soils:

Layer 1 (SC 12-25%)

Layer 1 is an embankment layer generally consisting of clayey sand. SPT $N_{1(60)}$ values in this layer ranged between 25 and 34 bpf. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) ranges between 39° and 43°. Based on the measured blow counts and engineering judgement, the drained friction angle was determined to be 38°.

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

Layer 2 (SC 12-25%)

Layer 2 is a foundation layer generally consisting of clayey sand. SPT $N_{1(60)}$ values in this layer ranged between 25 and 34 bpf. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) ranges between 39° and 43°. Based on the measured blow counts and engineering judgement, the drained friction angle was determined to be 38°.

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

Layer 6 (SC 12-25%)
Layer 5 is a foundation layer generally consisting of clayey sand. SPT $N_{1(60)}$ values in this layer ranged between 6 and 15 bpf. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) ranges between 30° and 35°. Based on the measured blow counts and engineering judgement, the drained friction angle was determined to be 32°.

We used drained strength parameters of ϕ' = 32° and c' = 0 psf.

Fine-Grained Soils:

Layer 3 (CH)
Layer 3 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ value in this layer is 4 bpf. No Atterberg limits tests were performed in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle is 30° based on conclusions drawn by Bonaparte and Mitchell (1979) for bay mud and higher torvane, pocket penetrometer, and $N_{1(60)}$ values than in lower layers of CH material. The drained cohesion value is assumed to be 0 psf.

The maximum past pressure was estimated based on pocket penetration data and one consolidation test. The maximum past pressure was estimated to be approximately 1.0 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.35 ksf and 0 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

We used drained strength parameters of ϕ' = 30° and c' = 0 psf.
We used total strength parameters of b = 0° and a = 350 psf, which for analysis purposes convert to: ϕ = 0° and c = 350 psf.

Layer 4 (CH)
Layer 4 is a foundation layer generally consisting of high plasticity clay (bay mud). SPT $N_{1(60)}$ values in this layer range between 0 and 4 bpf. No Atterberg limits tests were performed in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle based on correlations to plasticity index is 28°. The c' is assumed equal to 0 psf. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated based on pocket penetration data and one Consolidation test. The maximum past pressure was estimated to range from approximately 1.0 to 1.5 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.2 ksf and 10 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 28° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 10° and a = 200 psf, which for analysis purposes convert to: ϕ = 8° and c = 170 psf.</u>

Layer 5 (Sandy CH)

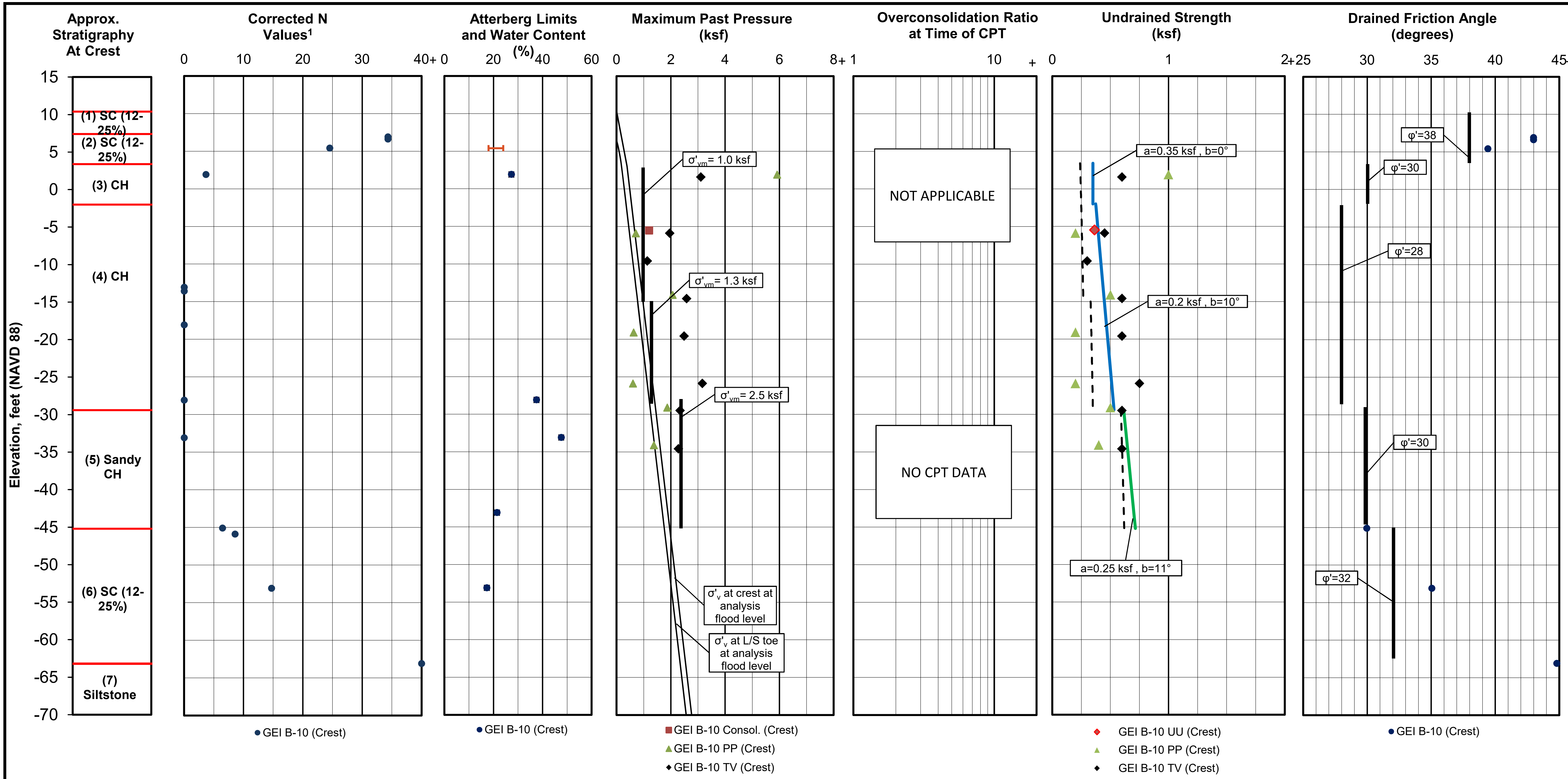
Layer 4 is a foundation layer generally consisting of Sandy CH (older bay mud). The SPT $N_{1(60)}$ value in this layer was 0. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general plasticity index (PI) value of 40 and liquidity index (LI) value of 1 for bay mud. However, this material is stiffer than the younger bay mud located above and may have a lower PI and LI value. The estimated drained friction angle based on correlations to plasticity is 30°. The c' is assumed equal to 0 psf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 2). This plot provides a project-specific fit of undrained strength versus effective stress for older bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.25 ksf and 11 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 11° and a = 250 psf, which for analysis purposes convert to: ϕ = 9° and c = 210 psf.</u>

Layer 7 (Siltstone)

Layer 6 is bedrock and analyzed as the bottom of model (BOM).



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5	6	7
Drained Parameters	c' (psf)	0	0	0	0	0	0	NA
	ϕ' (deg)	38	38	30	28	30	32	NA
Undrained Parameters	c (psf)	NA	NA	350	170	210	NA	NA
	ϕ (deg)	NA	NA	0	8	9	NA	NA

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. LL=Liquid Limit, w=As Received Water Content, PL=Plastic Limit.

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Reach 10, Sta. 7+00 NC-L

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) GC	130	0	36	NA	NA
(2) CL	120	50	32	200	3
(3) CH	95	0	30	350	0
(4) CH	95	0	28	170	8
(5) SC (12-25%)	120	0	38	NA	NA
(6) Siltstone	NA	NA	NA	NA	NA

Layers 1 and 4 are considered coarse-grained soils. Layers 2 and 3 are considered fine-grained soils.

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

Exploration	Station	Offset (ft)	Location	Ground Surface Elevation (ft, NAVD 88)	Drilled Depth (ft)
GEI_B-7	6+50	0	Top of Channel Bank	10.0	31.5
GEI_CPT-9	9+00	-10	Top of Channel Bank	9.0	30.2

Note: Offset is reported as the perpendicular distance landward (positive) or waterward (negative) of the levee centerline

Fig. 9 is a summary of the data from these explorations.

Coarse-Grained Soils:

Layer 1 (GC)

Layer 1 is an embankment layer generally consisting of clayey gravel. SPT $N_{1(60)}$ values in this layer were larger than 40 bpf. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) is larger than 45°. Based on the correlation to tip resistance provided in DWR (2013), the friction angle estimated from CPT soundings ranges from 36° to 42°. Based on the measured blow counts and CPT, the drained friction angle was determined to be 36°.

We used drained strength parameters of ϕ' = 36° and c' = 0 psf.

Layer 5 (SC 12-25%)

Layer 5 is a foundation layer generally consisting of clayey sand. SPT $N_{1(60)}$ values in this layer were larger than 40 bpf. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) is larger than 45°. Based on the correlation to tip resistance provided in DWR (2013), the friction angle estimated from CPT soundings is approximately 38° to 39°. Based on the measured blow counts and CPT, the drained friction angle was determined to be 38°.

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

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

Layer 2 (CL)

Layer 2 is a foundation layer generally consisting of low plasticity clay. Atterberg limits tests were perform in this layer, with a PI value of 5. The estimated drained friction angle based on site-specific correlations to plasticity index is 32°. The drained cohesion is assumed to be equal to 50.

The maximum past pressure is estimated to be 0.7 ksf, based on the CPT data. This indicates a lightly overconsolidated soil.

The undrained strength parameters were determined using the maximum past pressure described above and the SHANSEP relationship with values of S=0.25 and m=0.8.The values of a and b were determined to be 0.2 ksf and 3 degrees respectively.

<u>We used drained strength parameters of ϕ' = 32° and c' = 50 psf.</u>
<u>We used total strength parameters of b = 3° and a = 200 psf, which for analysis purposes convert to: ϕ = 3° and c = 200 psf.</u>

Layer 3 (CH)

Layer 3 is a foundation layer generally consisting of high plasticity clay (bay mud). No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle is 30° based on conclusions drawn by Bonaparte and Mitchell (1979) for bay mud and higher torvane and pocket penetrometer values. The drained cohesion is assumed to be 0.

The maximum past pressure was estimated from nearby GEI CPT-9 to be approximately 0.8 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.35 ksf and 0 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 0° and a = 350 psf, which for analysis purposes convert to: ϕ = 0° and c = 350 psf.</u>

Layer 4 (CH)

Layer 4 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ value in this layer was 0 bpf. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle based on correlations to plasticity index is 28°. The drained cohesion is assumed to be equal to 0. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

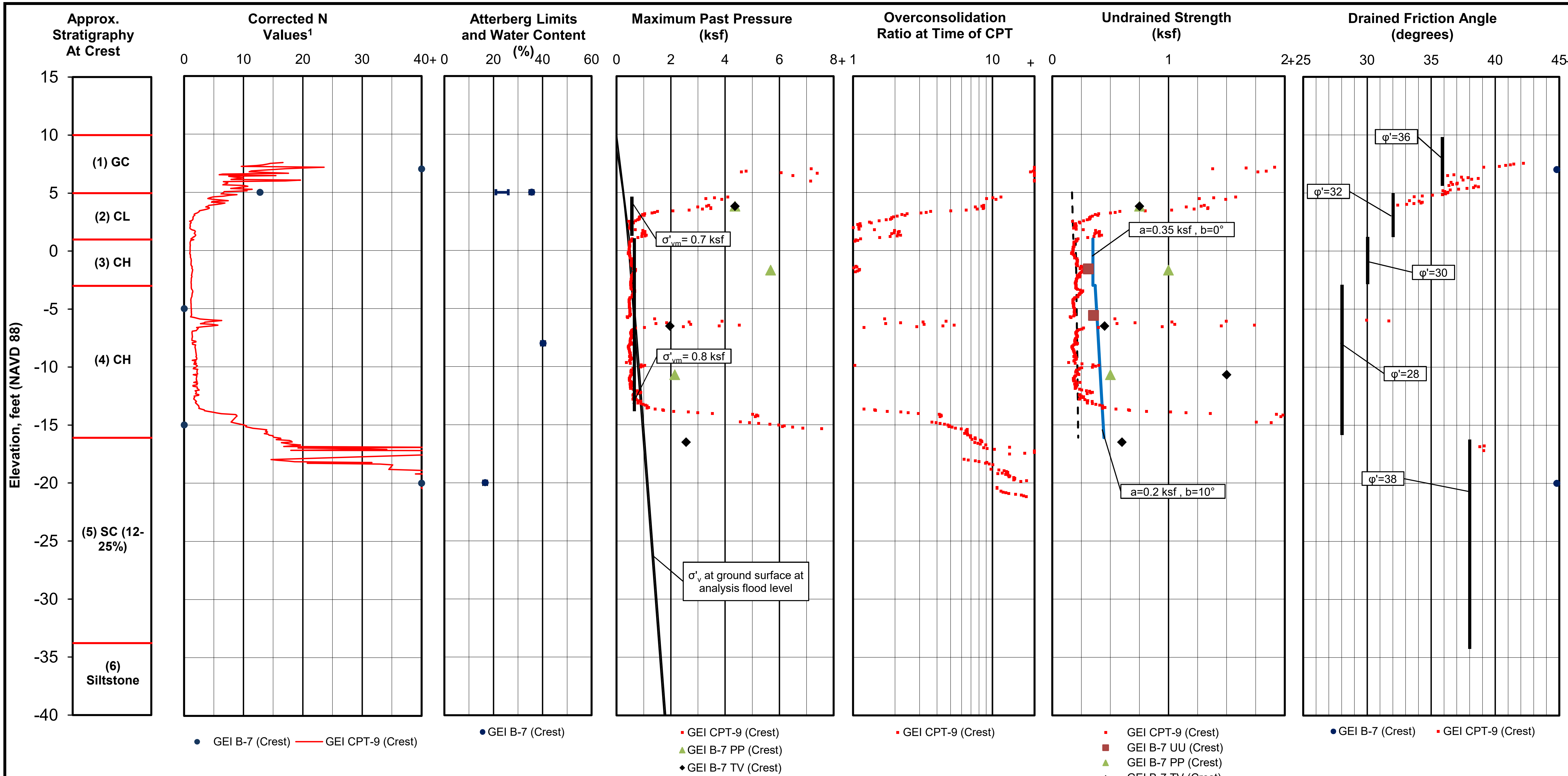
The maximum past pressure was estimated from nearby GEI CPT-9 to be approximately 0.8 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.2 ksf and 10 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 28° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 10° and a = 200 psf, which for analysis purposes convert to: ϕ = 8° and c = 170 psf.</u>

Layer 6 (Siltstone)

Layer 6 is bedrock and analyzed as the bottom of model (BOM).



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5	6
Drained Parameters	c' (psf)	0	50	0	0	0	NA
	ϕ' (deg)	36	32	30	28	38	NA
Undrained Parameters	c (psf)	NA	200	350	170	NA	NA
	ϕ (deg)	NA	3	0	8	NA	NA

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. LL=Liquid Limit, w=As Received Water Content, PL=Plastic Limit.

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and
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SOIL PROPERTIES VS. ELEVATION
STA. 7+00 NC-L (REACH 10)

Project 140457-0 | June 2015 | Fig. 9

Reach 12, Sta. 9+00 BM-L

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) SC (35-49%)	120	0	30	NA	NA
(2) GC	130	0	32	NA	NA
(3) CH	95	0	30	350	0
(4) CH	95	0	28	170	8
(5) Sandy CH	95	50	32	210	9
(6) Siltstone	NA	NA	NA	NA	NA

Layer 1 and 2 are considered coarse-grained soils. Layers 3 and 4 are considered fine-grained soils.

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

Exploration	**Station**	**Offset (ft)**	**Location**	**Ground Surface Elevation (ft, NAVD 88)**	**Drilled Depth (ft)**
GEI_B-6	4+85	0	High ground	12.4	80.5
GEI_CPT-8	9+50	0	High ground	9.5	60.6

Note: Offset is reported as the perpendicular distance landward (positive) or waterward (negative) of the levee centerline

Fig. 10 is a summary of the data from these explorations.

Coarse-Grained Soils:

Layer 1 (SC 35-49%)

Layer 1 is an embankment layer generally consisting of clayey sand. SPT $N_{1(60)}$ values in this layer ranged from 3 to larger than 40 bpf. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) ranges from 27° to larger than 45°. Based on the correlation to tip resistance provided in DWR (2013), the friction angle estimated from CPT soundings ranges from 30° to 32°. Based on the measured blow counts and CPT, the drained friction angle was determined to be 30°.

We used drained strength parameters of ϕ' = 30° and c' = 0 psf.

Layer 2 (GC)

Layer 2 is a foundation layer generally consisting of clayey gravel. The SPT $N_{1(60)}$ value in this layer was 9 bpf. The drained friction angle estimated from the correlation to $N_{1(60)}$ presented in DWR (2013) is larger than 32°. Based on the correlation to tip resistance provided in DWR (2013), the friction angle estimated from CPT soundings is approximately 30° to 39°. Based on the measured blow counts and CPT, the drained friction angle was determined to be 32°.

<u>We used drained strength parameters of ϕ' = 32° and c' = 0 psf.</u>

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

Layer 3 (CH)
Layer 3 is a foundation layer generally consisting of high plasticity clay (bay mud). No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle is 30° based on conclusions drawn by Bonaparte and Mitchell (1979) for bay mud and higher torvane and pocket penetrometer values. The drained cohesion is assumed to be 0.

The maximum past pressure was estimated from nearby GEI CPT-8 to be approximately 1.0 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.35 ksf and 0 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

<u>We used drained strength parameters of ϕ' = 30° and c' = 0 psf.</u>
<u>We used total strength parameters of b = 0° and a = 350 psf, which for analysis purposes convert to: ϕ = 0° and c = 350 psf.</u>

Layer 4 (CH)
Layer 4 is a foundation layer generally consisting of high plasticity clay (bay mud). The SPT $N_{1(60)}$ values in this layer were 0 bpf. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general PI value of 40 and LI value of 1 for bay mud. The estimated drained friction angle based on correlations to plasticity index is 28°. The drained cohesion is assumed to be equal to 0. This is in agreement with conclusions drawn by Bonaparte and Mitchell (1979) for bay mud.

The maximum past pressure was estimated from nearby GEI CPT-8 to range from approximately 1.0 to 1.1 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 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. The values of a and b were determined to be 0.2 ksf and 10 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

We used drained strength parameters of ϕ' = 28° and c' = 0 psf.
We used total strength parameters of b = 10° and a = 200 psf, which for analysis purposes convert to: ϕ = 8° and c = 170 psf.

Layer 5 (Sandy CH)

Layer 5 is a foundation layer generally consisting of sandy CH (older bay mud). No SPT $N_{1(60)}$ values existed in this layer. No Atterberg limits tests were perform in this layer, but existing literature (Bonaparte and Mitchell, 1979) indicate a general plasticity index (PI) value of 40 and liquidity index (LI) value of 1 for bay mud. Note that this material is stiffer than the younger bay mud above and may have a lower PI and LI value. Also, based on CPT data, the soil layer is highly overconsolidated. The estimated drained friction angle based on correlations to plasticity index and due to being highly overconsolidated is 32°. The drained cohesion is assumed to be equal to 50.

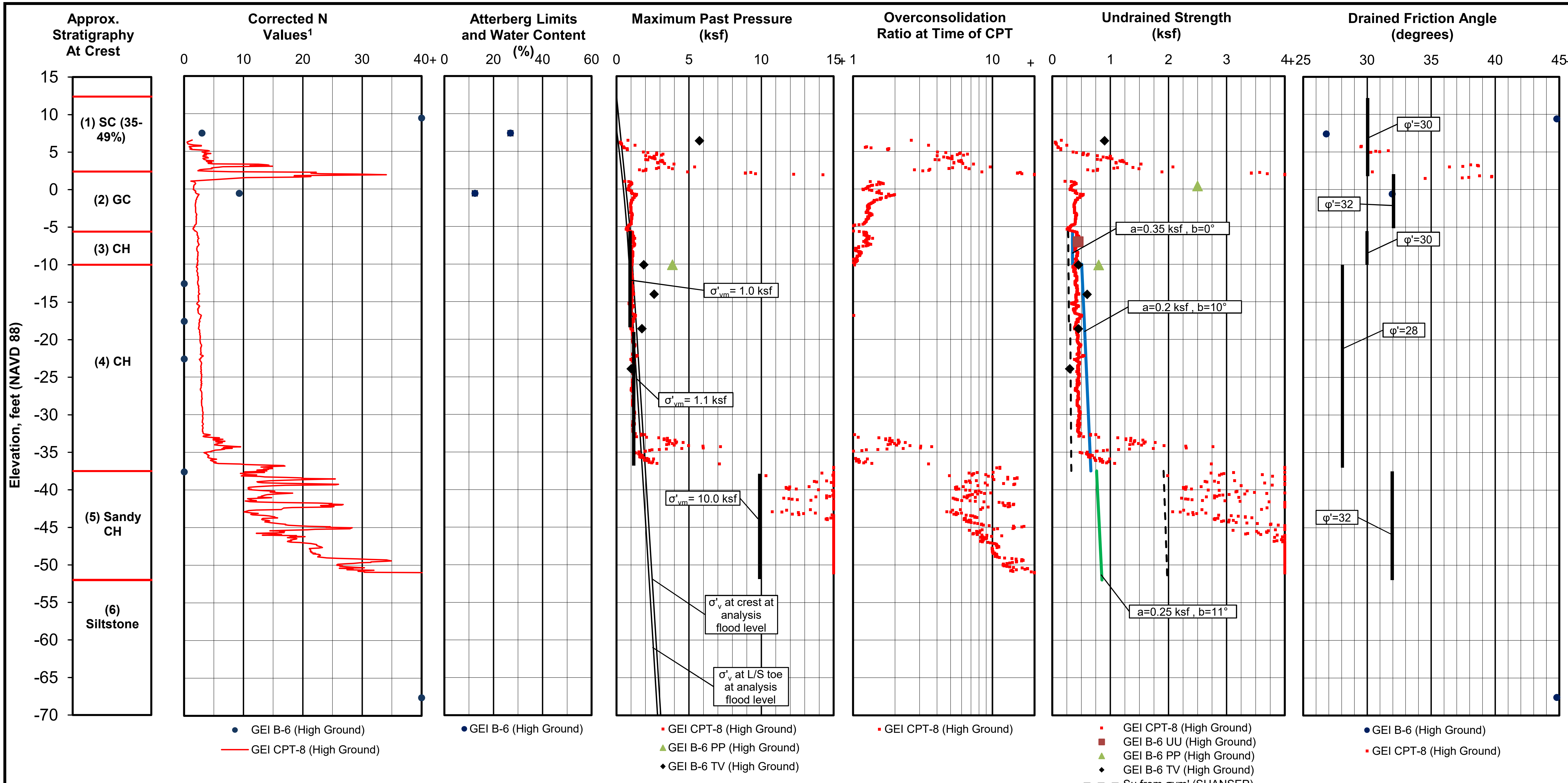
The maximum past pressure was estimated from nearby GEI CPT-8 to be approximately 10 ksf.

The undrained strength values for a and b were determined by fitting the parameters to all available undrained strength data plotted versus vertical effective stress (see Figure 2). This plot provides a project-specific fit of undrained strength versus effective stress for older bay mud that is applied consistently to every Reach. The values of a and b were determined to be 0.25 ksf and 11 degrees respectively. The undrained strength parameters were also validated by comparing results with the calculated undrained strength using the maximum past pressure determined above and the SHANSEP relationship with values of S=0.25 and m=0.8.

We used drained strength parameters of ϕ' = 32° and c' = 50 psf.
We used total strength parameters of b = 11° and a = 250 psf, which for analysis purposes convert to: ϕ = 9° and c = 210 psf.

Layer 6 (Siltstone)

Layer 6 is bedrock and analyzed as the bottom of model (BOM).



Strength Parameters Used for Analysis:

	Layer	1	2	3	4	5	6
Drained Parameters	c' (psf)	0	0	0	0	50	NA
	ϕ' (deg)	30	32	30	28	32	NA
Undrained Parameters	c (psf)	NA	NA	350	170	210	NA
	ϕ (deg)	NA	NA	0	8	9	NA

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. LL=Liquid Limit, w=As Received Water Content, PL=Plastic Limit.

Coyote Creek Levee Evaluation Project
Marin County, California

Marin County Flood Control and
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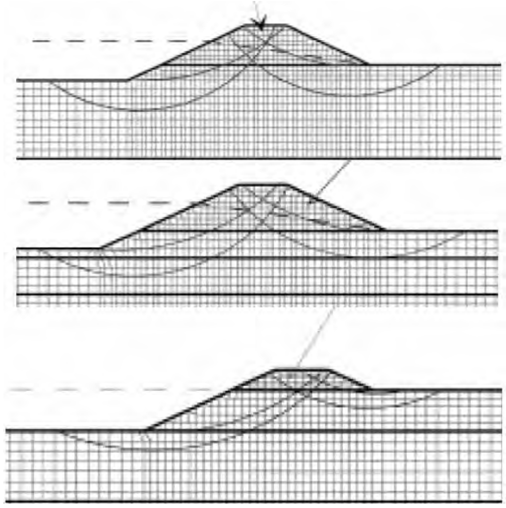
SOIL PROPERTIES VS. ELEVATION
STA. 9+00 BM-L (REACH 12)

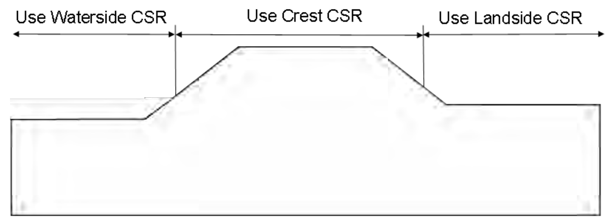
Project 140457-0 | June 2015 | Fig. 10

Appendix C

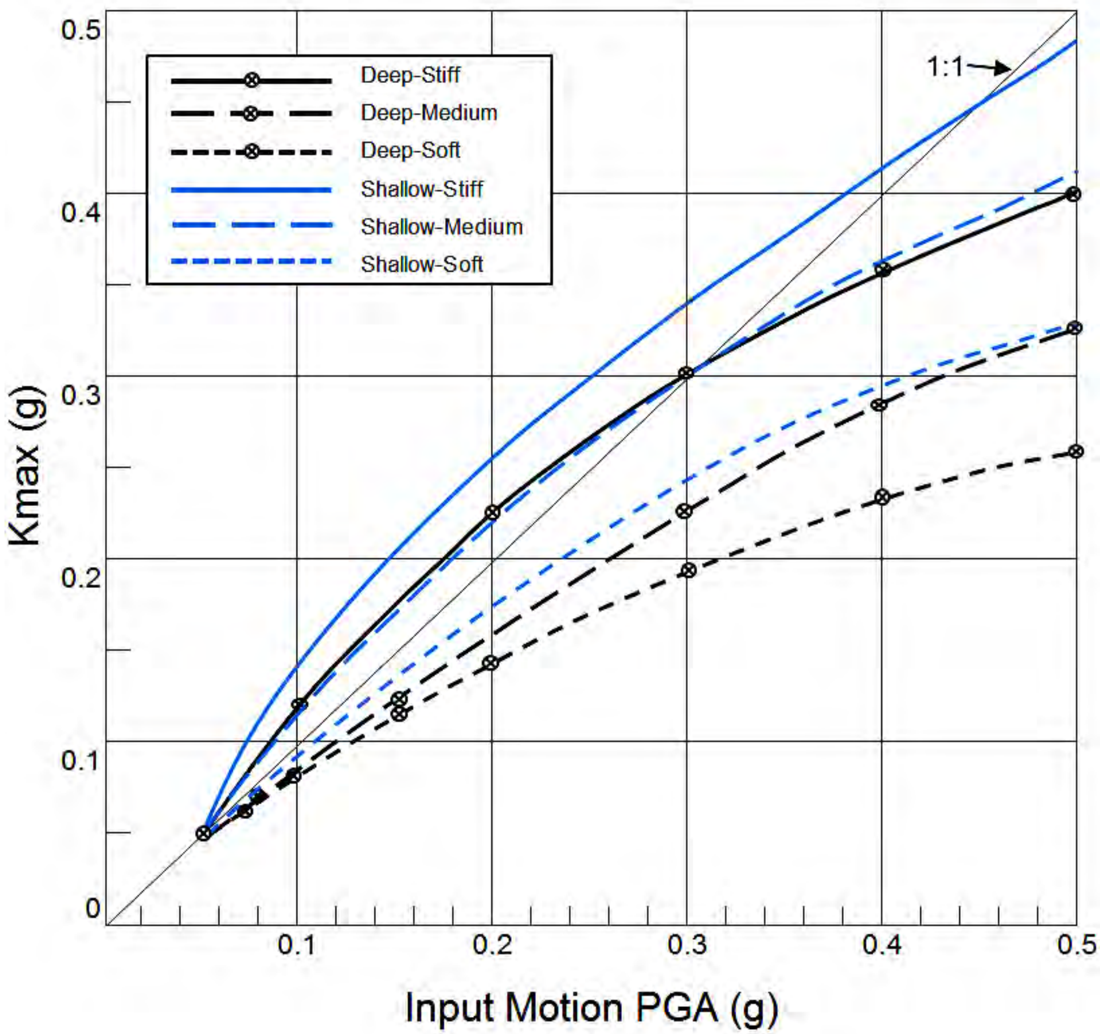
Seismic Deformation Back-up Information

Levee Shape	Resulting Response
Symmetric	"Softer"
Less Symmetric	"Medium"
Asymmetric	"Stiffer"

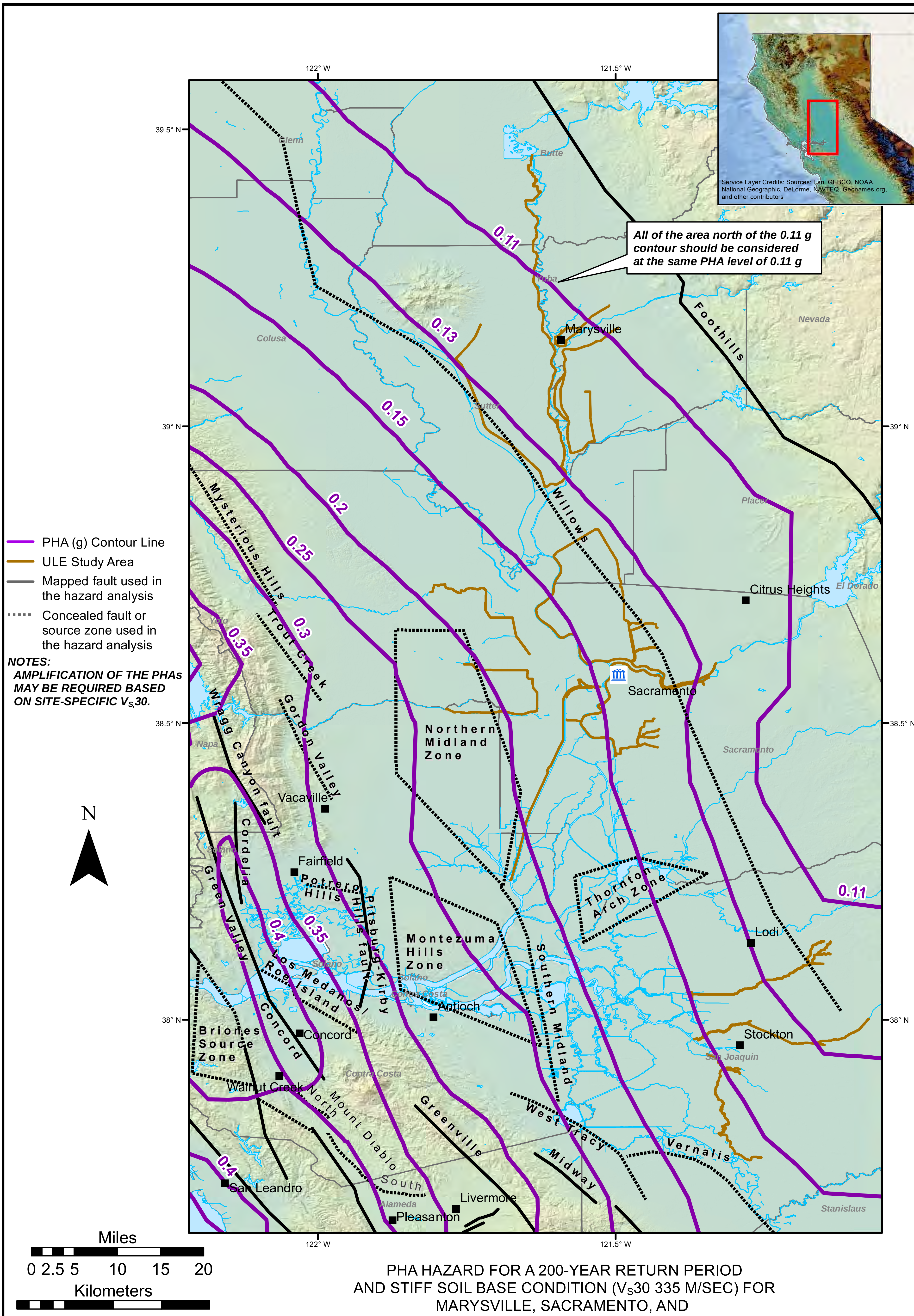




Note: CSR change occurs at $^{2}/_{3}$ the respective levee height (from crest).

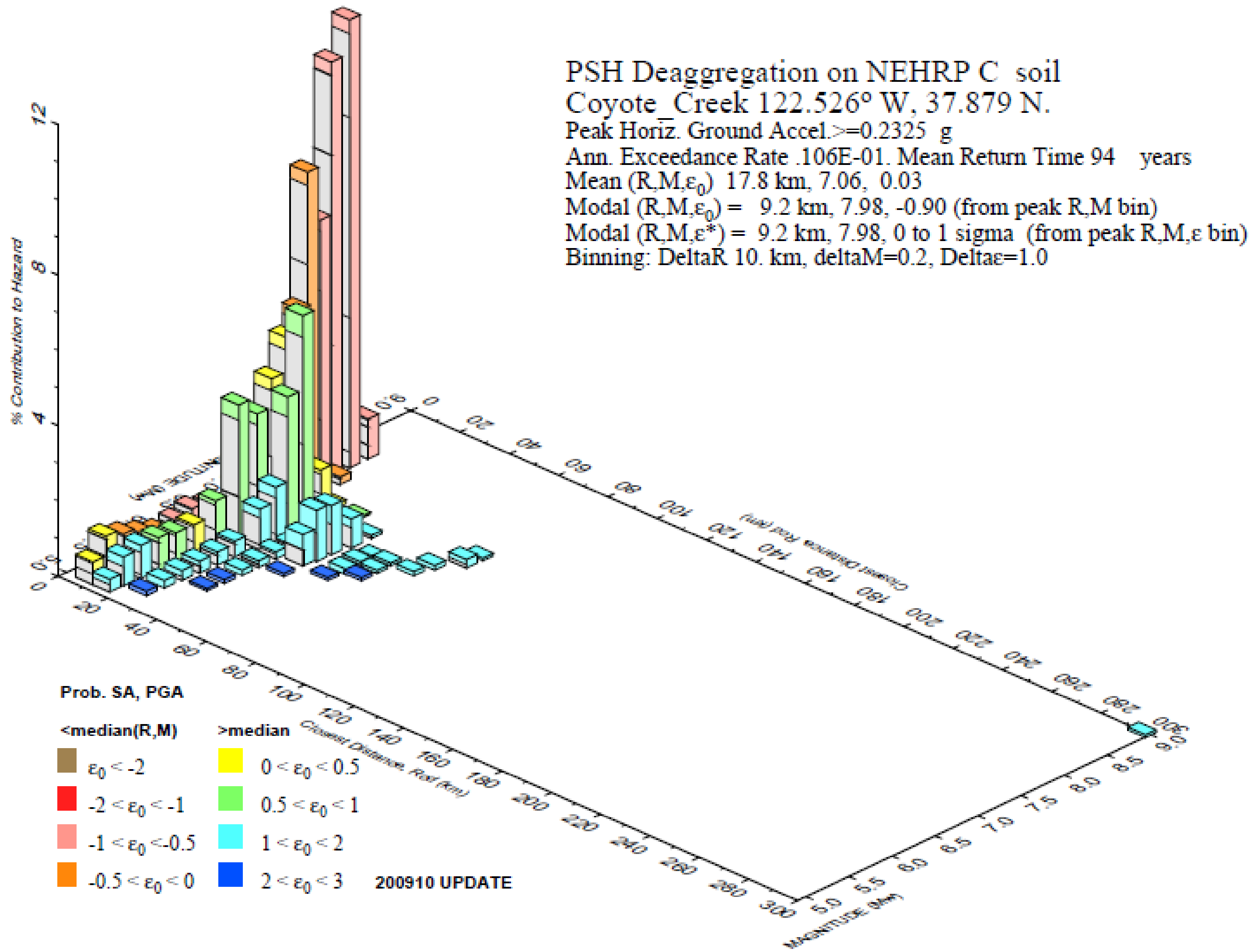


Coyote Creek Levee Evaluation Marin County	GEI Consultants	Figure 6-4 of ULE Guidance Document (Recommended K_{max} vs Input Motion PHA)
Marin County FCWCD		August 2015 Figure C-1

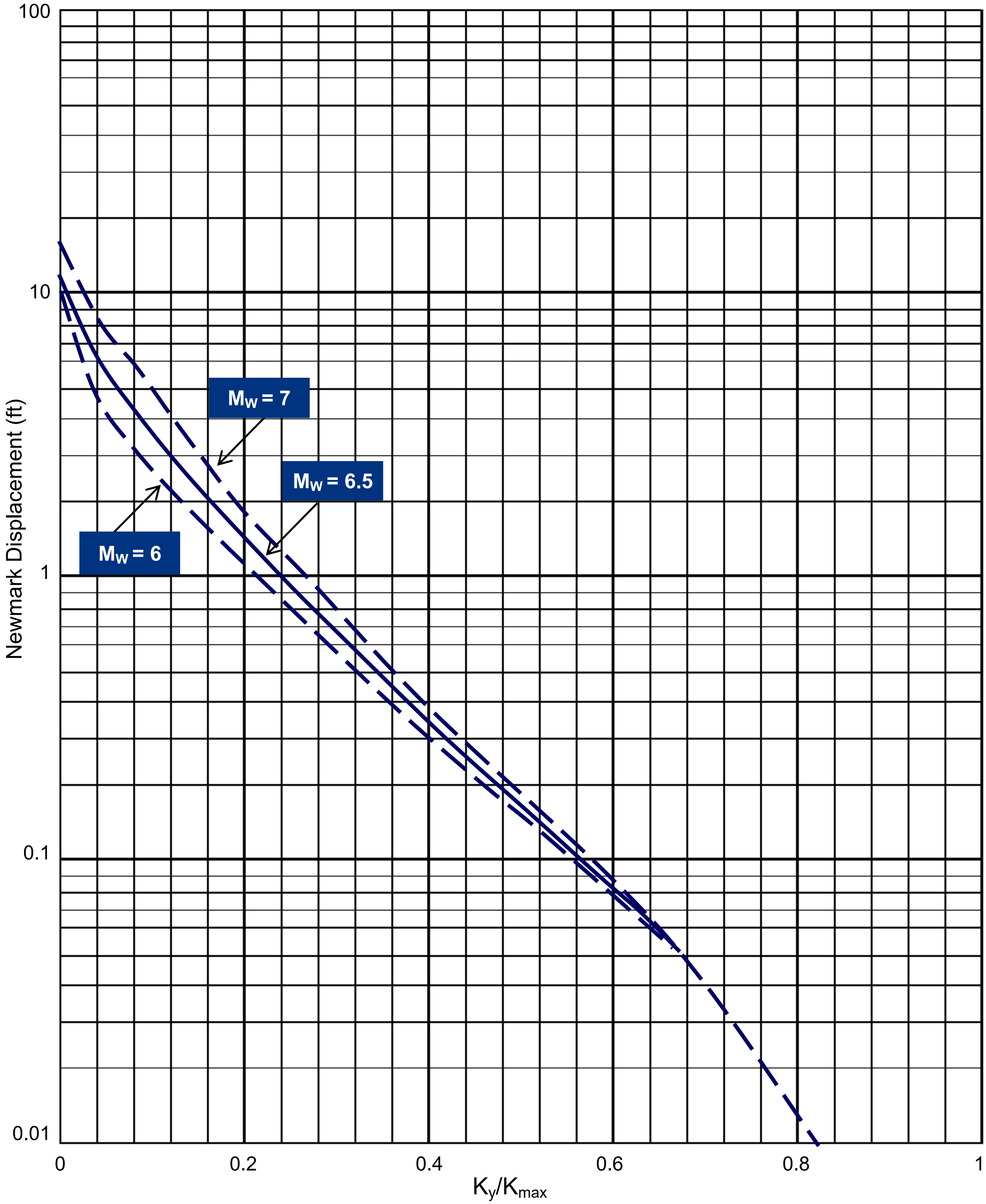


PHA HAZARD FOR A 200-YEAR RETURN PERIOD
AND STIFF SOIL BASE CONDITION (V_S30 335 M/SEC) FOR
MARYSVILLE, SACRAMENTO, AND
STOCKTON REGIONS

Coyote Creek Levee Evaluation Marin County	GEI Consultants	Figure 6-2 of ULE Guidance Document (Seismic Hazard Map)
Marin County FCWCD		August 2015 Figure C-2



Coyote Creek Levee Evaluation Marin County	GEI Consultants	PSH Deaggregation for Seismic Displacement Evaluation
Marin County FCWCD		August 2015 — Figure C-3



Coyote Creek Levee Evaluation Marin County	GEI Consultants	Figure 6-5 of ULE Guidance Document (Recommended Newmark Displacement vs K_y/K_{max})
Marin County FCWCD		August 2015 Figure C-4