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SAFER Bay East Palo Alto

East Palo Alto, San Mateo County, California

BASIS OF DESIGN REPORT

June 18, 2026

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1.0 INTRODUCTION

Purpose and Scope

The San Francisquito Creek Joint Powers Authority (SFCJPA) is a regional government agency founded by the cities of East Palo Alto, Menlo Park, and Palo Alto, the San Mateo County Flood Control District, and Santa Clara Valley Water District in 1999. SFCJPA is implementing the Strategy to Advance Flood Protection, Ecosystems and Recreation along the San Francisco Bay (SAFER Bay) Project which addresses sea level rise and inundation, ecosystem restoration, and recreational improvements along the southwest San Francisco Bay shoreline.

The existing Federal Emergency Management Agency (FEMA) flood study mapping places East Palo Alto's and Menlo Park's entire Bay shoreline within the Special Flood Hazard Area (SFHA) for the 100-year coastal flood event (also referred to as the 1% annual chance of exceedance event or the base flood). This designation indicates that these communities are at risk of coastal flooding. Sea level rise will further exacerbate these coastal flood hazards.

To address these flood hazards, SAFER Bay comprises construction of new or improved flood risk reduction features along the Bay shoreline from the Menlo Park and Redwood City border south to San Francisquito Creek, where another SFCJPA flood protection ecosystem restoration project was recently completed along the East Palo Alto and Palo Alto border. A design team led by HDR advanced the design of these flood protection features to the 10% or 30% design level. The Design Criteria and Considerations Technical Memorandum dated September 6, 2024, prepared by HDR for all reaches of the SAFER Bay project serves as the basis of this report.

The City of East Palo Alto (City or EPA) is further advancing the design of the flood protection system from San Francisquito Creek to Bay Road to a 100% design level (the Project). Led by Sherwood Design Engineers (Sherwood) and its subconsultants (Design Team), an updated Basis of Design Report (BODR) is provided herein to define the basic design basis and guidance that will be utilized during design. It includes basic operations requirements, Project performance requirements, and other stakeholders' design requirements as identified by the City. The BODR expands upon and references the Design Criteria Report (Sherwood 2026) and will document geotechnical, civil, structural, hydrologic and hydraulic standards to be used in the analyses and design. Pertinent codes and references will be cited.

Project Description

The SAFER Bay East Palo Alto Project subject to this report extends approximately 5,250 feet along an existing embankment from the O'Connor Pump Station at San Francisquito Creek to Bay Road in East Palo Alto, see Figure 1.

The Project utilizes a sheet pile floodwall with embankment. The design life of the Project is 50 years. The specific project features are discussed within the report.

Existing Site Information

Survey
A topographic and boundary survey has been conducted by Design Team member Mark Thomas. The survey covers the planned extents of the Project with a buffer sufficient to define edge conditions which may impact design. The field work was conducted November 2025 to May 2026 and the final deliverable is dated May 14, 2026. The survey, conducted by a Professional Licensed Surveyor (PLS) in the State of California, is the primary basis for the project's understanding of existing conditions.

Horizontal and Vertical Datums
The survey was conducted on officially recognized horizontal and vertical datums to provide consistency in documentation and construction. The North American Datum of 1983 (NAD 83) is the primary geometric coordinate system used in the United States. California further divides this horizontal datum into State Plan Zones 1 through 6 to support the state's elongated shape. The Project survey was conducted in NAD 83 State Plane Zone 3.

The Project's vertical datum is the North American Vertical Datum of 1988 (NAVD88). Horizontal and vertical control were established via six (6) point numbers (mag nails and/or rebar set in pavement) which are described on the survey plans and notes.

Existing Site Features
The existing site is marked by an earthen berm/levee with pedestrian trail that fronts bay marshland. The levee elevation is too low to provide significant protection to the land behind it, and as such, is often referred to as a "berm" for the purposes of this report. The terrain is generally flat with the bermed Bay Trail providing a linear ridge that bears approximately north-south. Urban developed land consisting of roadways and residences exist on the western side of the Bay Trail and open water, sloughs, and tidal marshland exist on the eastern side. The site extends from Bay Road at the northern terminus to the O'Connor Street Pump Station at the south, and includes what is generally public Right-of-Way between the private residences and the dedicated marsh. A drainage channel fronts the western side of the Bay Trail between Runnymede and O'Connor Streets, conveying City interior storm drainage to the O'Connor Pump Station.

Also bearing north-south through the site area are PG&E high voltage transmission towers and overhead lines. The transmission towers fall generally within the marsh and adjacent to the existing bermed trail.

Paved public rights-of-way terminate or interface with the trail system. Some of these access points include gravel or dirt trails over formal culvert crossings of a drainage

channel, and others reflect more ad-hoc access methods such as wooden planks which appear to have been placed over the channel by residents.

Existing Site Utilities
Supporting the major transmission towers noted above, additional electric and communications utilities include wooden power poles with guy anchors and underground distribution lines feeding between various junction boxes, control panels, and streetlights. Subsurface electrical vaults are located throughout the site.

In addition to the major pump station and conveyance channel site features, the project area also includes subsurface stormwater conveyance infrastructure. There are multiple large diameter culverts which cross the existing berm/levee system. Notable features include corrugate metal pipes and concrete box culverts, as well as tide gates at outfalls to the Bay. Interior surface drainage is generally collected and conveyed via drain inlets, curb inlets, and maintenance holes.

A sanitary sewer collection main runs parallel to the storm drainage channel which collects sewerage from the City's homes and businesses. Infrastructure includes sanitary sewer cleanouts, grease traps, manholes, and dedicated easements where necessary.

Pressurized distribution systems for both water and gas also exist throughout the site vicinity. Potable water infrastructure includes distribution lines, meters, backflow preventers, and valves. Fire protection is provided via visible hire hydrants and fire department connections adjacent to roadways.

2.0 GEOTECHNICAL BASIS OF DESIGN

Geotechnical Design Criteria

The geotechnical basis of design, including stability, seepage, seismic hazard, and seismic performance criteria are provided in the Design Criteria Report (Sherwood 2026). This report refers to the criteria established in the Design Criteria Report and provides additional details and design assumptions for evaluation of the proposed levee and floodwall system.

Site Conditions

Geotechnical Site Investigations
Previous subsurface investigations at the site are described in the *Preliminary Geotechnical Report for South of Bay Road* (HDR 2024) and the *Geotechnical Data Report (GDR), SAFER Bay Project (STA 00+00 to 69+00)* (Geosyntec 2026). The most recent subsurface exploration, performed by Geosyntec in January 2026, included 27

cone penetration tests, 6 soil borings, vane shear testing, selected seismic CPTs, and laboratory testing to characterize subsurface stratigraphy, strength, compressibility, corrosion potential, and material performance relevant to levee and floodwall design. The locations of these explorations are shown in Figure 2 and the results from this investigation are provided in the GDR (Geosyntec 2026). This exploration was used to supplement the historical explorations and laboratory results, and the resulting dataset provides the geotechnical basis for subsequent evaluation of settlement, seepage, stability, and seismic hazard, as described below.

The historical subsurface exploration program includes sampling within each geologic unit and at a maximum average spacing of 500 feet along the levee/floodwall alignment. This spacing is consistent with the guidance for a site classification of “ordinary” based on definitions provided in EM 1110-2-2502 (USACE 2022).

Subsurface Conditions

The subsurface soil conditions were evaluated based on field observation and soil samples collected during the historical site investigation. Unified Soil Classification System (USCS) designations were provided in accordance with ASTM D2487 for samples that have been subjected to laboratory testing. These USCS designations are provided below and shown on the boring logs included in the GDR (Geosyntec 2026).

The subsurface soils can be idealized and categorized into the soil units described below. Idealized subsurface stratigraphy along the SAFER Bay control line has been subdivided into the unit descriptions below and shown in the geological long section along the SAFER Bay control line and at two cross sections along the alignment, provided as Figures 3 and 4. Due to the complex depositional environment at the project site, there is variability in stratigraphy that can be seen in the different borings, and there is potential for significant variability along the alignment.

Unit 1: Fill – Sandy CLAY to Silty SAND

This soil unit consists of existing fill that had been previously placed during historical development and levee construction. The fill unit was encountered in the upper 3-7 feet of the explorations completed on the existing levee and typically consists of sandy clay to silty sand (USCS of CL/CH to SC/SM). The sandy clay is generally soft to medium stiff while the silty sand is typically loose to medium dense.

Unit 2: Young Bay Mud (YBM)

This soil unit is located beneath the fill in all explorations and consists of medium to high plasticity clay to sandy clay (USCS: CL to CH). This unit contains between 0 and 40 percent fine sand and is described as very soft to medium stiff.

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Unit 3: Old Bay Clay (OBC)

This soil unit was generally encountered beneath the YBM (Unit 2) and consists of lean clay (USCS: CL). This unit is generally medium stiff to very stiff.

Unit 4: Coarse-Grained Soil 1

This soil unit consists of loose to medium dense silty to clayey sand with gravel (USCS: SM and SC)

Unit 5: Coarse-Grained Soil 2

This soil unit consists of medium dense to very dense sand and gravel with some silt and clay (UCSC: SM, SC, and GP).

Ground Model and Material Parameter Selection
Recommended geotechnical parameters for design are provided in Table 1. These design parameters have been developed based on review of laboratory and in-situ tests on each soil unit, and comparison with published literature such as API (200), FHWA (2016), USACE (1991), and NAVFAC (2025).

Groundwater
Groundwater levels encountered during the January 2026 subsurface exploration are summarized in the GDR (Geosyntec 2026). The groundwater levels encountered during the investigation ranged from 1.5 feet to 14 feet below the top of the CPT /soil boring exploration. These groundwater levels were used to estimate existing stresses in the soil. The groundwater elevations encountered during the site investigation are shown on the geotechnical long section and cross sections in Figures 3 and 4.

For the purpose of design evaluation, the groundwater levels are assumed to rise in proportion to SLR. In particular, groundwater levels are expected to increase above the elevations shown in Figures 3 and 4, and there is expected to be standing water above the waterside toe during future high tides. For settlement and liquefaction susceptibility evaluation, water levels are assumed to be 10.8 ft NAVD88, consistent with the “Usual” water level with 3.5 ft of SLR as described in the Design Criteria Report (Sherwood 2026). For the purpose of seepage and stability evaluation, the water levels were assumed to be at the ground surface on the landside, and the water levels on the bay side were assumed to be at the Usual, Unusual, and Extreme water surface elevations, following the hydraulic loading criteria described in the Design Criteria Report. Groundwater levels in the subsurface was then calculated for each hydraulic loading scenario using the program SEEP/W.

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Geotechnical Analyses

Reach Selection

The flood protection system was evaluated and grouped into reaches. Each reach is intended to represent a bounded length of the levee/floodwall that can be grouped together as a single unit for the purpose of geotechnical assessment. The selection process was in general accordance with DWR's Urban Levee Evaluation (ULE) guidance for reach selection (DWR 2015).

The reach selection process considered changes in topography, subsurface stratigraphy, geotechnical/geological conditions, site history, and potential loading conditions. For example, sections of the flood protection system that will be constructed by degrading and reconstructing a new embankment along the same alignment as the existing levee were considered as separate reaches compared to sections of the flood protection system that will be constructed to the waterside of the existing levee within the marsh.

Cross-Section Development

Representative cross sections were selected and developed for analysis for South of Bay Road. Each cross section was selected to represent typical conditions within reach. The stratigraphy and geotechnical conditions within each reach was reviewed based on the available geotechnical boring and CPT data. An idealized ground model was then developed for each reach. In general, the ground profile beneath the flood protection system embankment is based on review and idealization of all borings and CPTs within a given reach, however, additional emphasis was provided to borings and CPTs close to the selected cross section. The full set of borings and CPTs were also reviewed to help identify anomalous areas or sub reaches.

In total, five cross sections were developed in Geo-Studio for analysis of the performance of the sheet pile wall/embankment system, including evaluation of seepage, slope stability, and sheet pile wall structural design. These analyses are discussed in further detail in Section 3 and Appendix A.4. Five cross sections were also developed for evaluation of the embankment settlement, as discussed further below.[1]

Settlement Analyses

The embankment is anticipated to settle over time due to the consolidation of fine-grained foundation soils. The majority of the consolidation settlement is expected to occur within the near-surface YBM (Unit 2) deposits, although the deeper OBC (Unit 3) is also anticipated to contribute to settlement.

[1] The locations of the cross sections do not necessarily coincide between the separate levee engineering analyses, as the section locations were selected separately based on ground conditions and project geometry to model the behavior of interest.

Foundation settlement was evaluated for the proposed levee/floodwall system in accordance with EM 1110-1-1904 (USACE 1990). Settlement analysis was evaluated using the commercial software Settle3 (Rocscience, Inc. 2025). The assumptions and findings from these analyses are described in Appendix A.1.

As described in Appendix A.1, embankment settlement was estimated between the end of earthwork construction and the end of an assumed 50-year project design life. Total settlement estimates ranged between approximately 7 and 20 inches, with approximately 2 to 6 inches of this settlement occurring during earthwork construction, and the remainder of the settlement occurring after earthwork completion.

To accommodate this anticipated settlement, the embankment crest elevation will be increased above the long-term design elevation of 13.0 ft NAVD88 to accommodate expected settlement. The overbuild height was selected so that the anticipated embankment crest elevation after 50 years of primary and secondary consolidation is expected to remain above the design elevation of 13.0 ft. To achieve these overbuild grades, the embankment crest width was kept constant at 20 ft while steeping the landside slope to accommodate the overbuild thickness, following the concept illustrated in Figure 1-2 of EM 1110-2-1913 (USACE, 2022). Over time, settlement will result in gradual lowering of the crest elevation and flattening of the slopes.

The following overbuild thicknesses are included along the levee alignment to maintain the target crest elevation and slopes during the design life:

- STA 0+00 to STA 12+00: 6 inches of overbuild
- STA 12+00 to 12+50: Transition from 6 inches to 12 inches of overbuild
- STA 12+50 to 27+50: 12 inches of overbuild
- STA 27+50 to 28+00: Transition from 12 inches to 18 inches of overbuild
- STA 28+00 to STA 52+72 (Bay Road): 18 inches of overbuild

Although the long-term settlement is anticipated to remain above the design elevation of 13.0 ft NAVD88, the embankment is expected to experience differential settlement. This differential settlement may lead to uneven crest elevations and damage to the paved surface of the Bay Trail. Ongoing maintenance of the embankment crest will be required to address damage resulting from the differential settlement.

Seepage and Stability Analyses

Seepage and stability analyses were performed at selected cross sections along the proposed alignment using the Geostudio software package, including SIGMA/W for stress distribution, SEEP/W for seepage evaluation, and SLOPE/W for slope stability evaluation (Geostudio, 2025). These evaluations were performed as a coupled analysis which included the soil embankment and floodwall system as detailed in Section 3.0.

The results of the slope stability analysis for geotechnical design considered the stability of deep failure surfaces passing beneath the sheet pile wall, as well as localized failure

surfaces to the waterside and landside of the floodwall. The results of the seepage and stability evaluations are described further in Section 3.0 and Appendix A.4, and indicate that the proposed flood protection system achieves the design criteria described in EM-1110-1913 (USACE 2022a), EM-1110-2502 (USACE 2022b), and the Design Criteria Report, for each of the hydraulic loading scenarios considered.

Seismic Hazard

According to ER-1110-2-1806 USACE (2024), the seismic design criteria depend on the project consequence-based feature classification. Since the usual water surface elevation (WSE) with 3.5 ft of sea level rise (SLR) is higher than the landside toe of the levee, the levee is considered "frequently" loaded and therefore a breach of the levee/floodwall system is likely to lead to flooding. The breach could inundate a large area with significant population and property development. Based on the frequency of loading, along with the potential consequences of a breach, the project is considered "critical" when subject to 3.5 ft of SLR, according to definitions provided in Table 1 of ER 1110-2-1806.

ER 1110-2-1806 requires consideration of two seismic loading scenarios:

- Operating Basis Earthquake (OBE). The OBE is a ground motion that can reasonably be expected to occur within the service life of the flood protection system. According to ER 1110-2-1806, the OBE ground motion for a critical structure should have a minimum return period of 475-years.

- Maximum Design Earthquake (MDE). The MDE is a rare but possible earthquake scenario and represents the maximum level of seismic ground motion considered for design of the project. According to ER 1110-2-1806, the MDE is taken as the ground motion with a minimum return period of 2,475-years.[2]

Appendix A.2 presents the results of a simplified probabilistic seismic hazard analysis (PSHA) and simplified deterministic seismic hazard analysis (DSHA) for the site. Based on this review, the OBE is considered to have a peak ground acceleration (PGA) of 0.51 g and a modal magnitude (Mw) of 7.5, while the DBE is considered to have a PGA=0.77g and Mw=7.9.

According to ER 1110-2-1806, critical structures in high seismic hazard regions (such as the San Francisco Bay Area) require considerations for near source effects (e.g. directionality and maximum component factors), directivity, and aftershocks. Final design of critical structures in high seismic hazard regions also require advanced analysis methods such as ground motion time history development, seismic site response

[2] According to ER-1110-1806, the MDE must be taken as the larger of the following two seismic scenarios: (i) the ground motion with a return period of 2,475-years, or (ii) the 84th percentile Maximum Credible Earthquake (MCE) ground motion, where the MCE is the largest earthquake that can reasonably be expected for a given fault or seismic source. For this project site, the 2,475-year return period ground motion was evaluated to be larger than the 84th percentile MCE. See Appendix A.2 for further discussion.

analyses, and/or fully nonlinear deformation seismic analyses methods. These detailed seismic hazard considerations and advance analysis methods are in progress and will be included as part of final design.

Liquefaction Evaluation

Loose, cohesionless soils were encountered beneath the groundwater table during the site investigation. These soils may be susceptible to liquefaction during a seismic event. To evaluate the liquefaction susceptibility and potential consequences of liquefaction, a liquefaction evaluation was performed and has been included as Appendix A.3.

CPT-based and SPT-based liquefaction triggering assessment was performed as part of the liquefaction evaluation, using methods described in Idriss and Boulanger (2008). The liquefaction susceptibility of fine-grained soils was also reviewed as part of this evaluation, but site cohesive soils were generally considered to not be liquefiable based on criteria presented in Bray and Sancio (2006).
Based on this review, many of the coarse-grained soil layers at the site are susceptible to liquefaction under an OBE or MDE seismic event. The lateral and vertical extent of soil layers subject to liquefaction are described in Appendix A.3.

Following a seismic event, the site is expected to experience between approximately 0 and 6 inches of post-liquefaction vertical reconsolidation settlement, up to 10 inches of lateral displacement, and ground surface manifestation (e.g., sand boils) along parts of the alignment. The anticipated extents and consequences of liquefaction have been incorporated into the structural analysis presented in Section 3.0 and Appendix A.4. The anticipated post-liquefaction residual shear strength has also been considered as part of pseudo-static slope stability evaluations and seismic displacement estimates.

3.0 STRUCTURAL BASIS OF DESIGN

Background

Sheet Pile Flood Wall

A cantilever sheet pile floodwall is included in the Project from STA 0+07.5 to 52+27.8. The top of the sheet pile wall will have a reinforced concrete pile cap. See Appendix A.4 for design calculations for the sheet pile flood wall.

Concrete Retaining Wall

A concrete retaining wall will be included on the land side of the trail for portions of the alignment where there are physical constraints limiting the placement of the earthen fill. A retaining wall will be needed near the PG&E transmission tower. See Appendix A.5 for design calculations for the concrete retaining walls.

Design Loading

Dead Load (D)
Below is a list of unit weights that will be used in the design:

- Structure self-weight
  - Concrete: 150 pcf.
  - Structural Steel: 490 pcf.
- Water: 62.4 pcf for fresh water and 64 pcf for seawater.

Live Load (L)

- Vehicle Live Load on Trail = HS-20 Truck Load

Earth Pressure (EH / EV)
The horizontal earth pressure (EH) and vertical earth pressure (EV) on the floodwall and retaining wall are the lateral and vertical forces exerted by the retained soil mass on the structure.

Hydrostatic Loading (Hs)
Hydrostatic loading is the lateral pressure exerted by groundwater or flood waters.

For the sheet pile floodwall the controlling hydrostatic loading is from the water elevations on the waterside of the wall. Still water levels of 10.8 ft NAVD88, 14.5 ft NAVD88, and 16.5 ft NAVD88 are adopted for the Usual, Unusual, and Extreme load cases, respectively, with 16.5 ft NAVD88 corresponding to the HTOL at the top of floodwall elevation.

For the retaining walls the hydrostatic loading is limited by weep holes and perforated drainage pipe placed at the base of the wall.

Seismic Loading (EQ)
Seismic loading has been determined using parameters specified by Geosyntec. See Section 2 for detailed explanation of seismic parameters.

Load Combinations
The following load combinations used are taken from USACE EM 1110-2-2502.

Table 2: Sheet Pile Load Combinations

LC	Load description	Load category	Load combination
LC-1A	Infrequent surge	Unusual	D + EH + EV + Hs
LC-1B	Maximum hydrostatic	Extreme	D + EH + EV + Hs

LC-2	Normal operating	Usual	D + EH + EV + Hs
LC-3A	Earthquake – OBE	Unusual	D + EH + EV + EQ + Hs
LC-3B	Earthquake – MDE	Extreme	D + EH + EV + EQ + Hs

Notes: Groundwater level used in LC-3A and LC-3B is at 3.3 ft NAVD88 for both landside and water side.

Table 3: Retaining Wall Load Combinations

LC	Load description	Load category	Load combination
LC-1	Normal Operating	Unusual	D + EH + EV + Hs
LC-2	Normal operating +Surcharge	Usual	D + EH + EV + Hs + ES
LC-3A	Earthquake – OBE	Unusual	D + EH + EV + EQ + Hs
LC-3B	Earthquake – MDE	Extreme	D + EH + EV + EQ + Hs

Additional Design Considerations

Steel Piling Material

The following materials will be considered for steel piling

- Steel sheet pile: ASTM A328; ASTM A572 grades 50 or 60; or ASTM A690
- Steel pipe pile: ASTM A252 Gr.50

Concrete Materials

The following materials will be considered for concrete elements:

- Concrete strength: f'c = 5,000 psi
- Concrete cover: 3"
- Reinforcing steel: ASTM A615 Gr.60, black bar

Appurtenant Structures

For any additional structural components, or when supplemental design criteria are appropriate, the following documents shall be referenced:

- ASCE/SEI 7-22, Minimum Design Loads for Buildings and Other Structures, (American Society of Civil Engineers 2022).
- ACI 318-19, Building Code Requirements for Structural concrete and Commentary, (American Concrete Institute, 2022).
- California Building Standards Code (Title 24), 2025

Corrosion Considerations

The following corrosion rates will be applied to the design, and are based on recommendations found in California Department of Transportation Corrosion Guidelines Version 3.2 and historic marine structural design practice:

Table 4: Corrosion Rates

Nominal Zone	Steel Corrosion Rate (in. / year)
Splash Zone	0.006
Atmospheric Zone	0.004
Immersed Zone	0.002
Fill or Disturbed Natural Soils	0.0015
Soil Embedded Zone	0.001

Steel sheet piles will not be coated. The upper portion of the sheet pile above the ground elevation will be encased in a concrete pile cap. Below the pile cap the sheet pile design will include a sacrificial thickness to meet a minimum 50-year design life.

4.0 CIVIL BASIS OF DESIGN

Alignment

The project alignment was designed to avoid impacts to the existing high quality tidal marsh within the Faber Tract and Laumeister Marshes to the extent feasible. Therefore, the alignment generally is located outside of the marsh. From Station 0+00 to 23+50 the alignment follows that of the existing levee and Bay Trail such that the proposed waterside toe is at approximately the same location as the existing levee toe, minimizing impact to the marsh while providing as much room as possible for existing utilities and infrastructure to remain along the landside toe. From Station 23+50 to 28+00 the alignment extends into the marsh in order to avoid the existing PG&E transmission tower at Station 25+40, providing 15 feet of clearance from the landside toe to the tower. The alignment remains within the marsh to avoid the Runnymede Stormwater Channel from Station 25+40 to 28+00. The extent to which the alignment deviates into the marsh is reduced through the use of a retaining wall on the landside of the embankment. The existing levee and Bay Trail have a sharp S-curve at Runnymede Street. The proposed alignment also has an S-curve at this location (Station 28+00 to 30+00); however, the transition is more gradual in order to meet Bay Trail horizontal curve design guidelines (discussed below). From Station 30+00 to 33+50 the alignment is located partially over the existing levee and Bay Trail but extending an additional 30

to 40 feet into the marsh. This alignment avoids the existing PG&E transmission tower at Station 33+40 while minimizing the impact to the properties along the alignment at this location which are known to have contaminated soil. At Station 33+50 the alignment transitions to the east to be wholly located within the marsh until Bay Road at Station 52+00. This alignment was chosen to avoid impacting the properties adjacent to the existing levee and Bay Trail along this stretch which are known to have contaminated soils and groundwater, as well as multiple PG&E transmission towers. Further discussion of the known encroachments and utilities that informed the alignment are provided below.

Cross Section

The flood protection system utilizes a sheet pile floodwall within a soil embankment. The hybrid approach was selected to reduce the overall width of the project footprint to minimize impact to the marsh and existing utilities, properties, and site features.

The project features the following cross section:

- Embankment
  - 20 ft embankment crown width
  - 3:1 (H:V) waterside slope
  - 2:1 landside slope stabilized with geogrid
  - 13.0 ft (NAVD 88) top of embankment elevation (long term, post settlement condition)
- Sheet Pile Floodwall
  - Concrete Capped
  - 16.5 ft (NAVD 88) to of wall elevation (long term, post settlement condition)
  - -25 to –40 ft (NAVD 88) tip of wall elevation

The project is expected to experience post-construction settlement. The elevations of the embankment and wall tops will be overbuilt to accommodate the expected settlement. See Sections 2 and 3 for more information regarding the embankment and floodwall design.

At select locations along the alignment, the landside slope is replaced with retaining walls. This occurs at Martin Luther King Park, Cypress Street, and Runnymede Street public access points where the access ramps are located within the embankment prism. This also occurs at the PG&E transmission tower at Station 25+40 where a retaining wall is used to reduce the embankment width.

Encroachment Mitigation

Table 2 below provides a summary of known encroachments, utilities, and penetrations along the flood protection alignment. This list is based on survey information provided by Mark Thomas in the "SDE_SAFER BAY SOBR" Survey dated 05/14/26, site observations, and as-built reviews. Mitigation measures are discussed below.

Table 5: Encroachments

No.	Encroachment	Station	Description	Action
1	O'Connor Pump Station and Outfall to San Francisquito Creek	South of 0+00	Pump station which outfalls into San Francisquito Creek through the existing levee	Protect in Place
2	Detention Pond	0+00 – 5+75	Detention pond for pump station whose eastern bank is approximately 25' west of landside toe	Protect in Place
3	Detention Pond Fencing	0+00 – 5+75	Fencing for detention pond approximately 15' west of landside toe	Relocate STA 0+00 – 5+40 Protect in Place STA 5+40 – 5+75
4	Sewer Main and Manholes	0+25 – 34+50	East Palo Alto Sewer District sewer main running parallel to flood protection alignment.	Relocate STA 0+25 – 24+00 Protect in Place STA 24+00 – 34+50
5	Tidal Channels	~3+00 – ~26+00 and ~28+00 – ~39+00	Tidal channels located along and parallel to the waterside toe	Relocate STA TBD Protect in Place STA TBD
6	Runnymede Stormwater Channel	5+75 – 28+00	Stormwater channel from storm drain culvert in Runnymede Street to O'Connor Detention Pond	Protect in Place STA 5+75 – 17+50 and 19+00 – 21+00 Relocate STA 17+50 – 19+00 and 21+00 – 28+00
7	Runnymede Stormwater Channel Fence	5+75 – 28+00	Fencing for Runnymede Stormwater Channel ranging from approximately 45' west of landside toe to within proposed flood protection prism	Protect in Place STA 5+75 – 15+50 Relocate STA 15+50 – 28+00
8	Martin Luther King Park Access Path	11+60	Access path from Martin Luther King Park to existing levee and Bay Trail	Replace
9	Cypress Street Access Path	19+10	Access Path from Cypress Street to existing levee and Bay Trail	Replace

10	Stormwater Channel Culverts	11+60 and 19+10	(4) Corrugated Metal Pipe Culverts at each location of the access path crossing the stormwater channel	Protect In Place STA 11+60 Replace STA 19+10
11	Garden Street Storm Drain Outfall	21+80	Storm drain outfall into Runnymede Stormwater Channel at Garden Street.	Relocate into new stormwater channel
12	Garden Street Foot Path	22+10	Foot path from Garden Street to existing levee and Bay Trail	Remove
13	PG&E Transmission Tower	25+40	PG&E transmission tower at landside toe	Protect in Place
14	Overhead PG&E Transmission Lines	22+50 – 42+00 and 48+00 – 52+00	Overhead PG&E transmission lines adjacent to and crossing the alignment	Protect in Place
15	Runnymede Stormwater Culvert	28+00 – 29+00	Stormwater culvert directing high flows from Runnymede Street storm main outfall to Runnymede Stormwater Channel	Protect in Place
16	Runnymede Street Storm Main Outfall	29+00	Outfall structure for 72” storm main in Runnymede Street with tide gate valves.	Cap outfall structure to direct flows to culvert and abandon in place.
17	Runnymede Street Access Path	28+00	Access Path from Runnymede Street to existing levee and Bay Trail	Replace
18	Faber Laumeister Trail Access	27+90	Access to Faber Laumeister Trail via existing Bay Trail	Replace
19	Fence	28+50 – 34+25	Fence adjacent to landside toe	Protect in Place STA 28+50 – 31+60 Relocate STA 31+60 – 34+25
20	Monitoring Wells	29+30, 31+50, 34+00, 38+40, 39+70, 39+80, 40+80 41+00, 42+70, 43+50	Monitoring wells for contaminated soils and groundwater on properties adjacent to alignment	Protect in Place STA 29+30, 39+70, 40+80, 42+70 Relocate STA 31+50, 34+00, 38+40, 39+80, 41+00, 43+50
21	Survey Monuments	31+60 and 34+50	Survey monuments adjacent to alignment	Replace
22	PG&E Transmission Tower	33+40	PG&E transmission tower at waterside toe	Protect in Place
23	PG&E Transmission Tower Access Ramp	33+40	Access ramp from existing Bay Trail to PG&E transmission tower	Replace
24	Weeks Street Access Path	34+40	Access path from Weeks Street to existing levee and Bay Trail	Replace

25	Contaminated Soils	28+50 – 34+40	Known contaminated soils within property adjacent to alignment	Protect in Place to extent feasible
26	Capped Containment Cell	34+70 – 40+00	Capped containment cell for contaminated soil located within property approximately 25’ - 40’ from landside toe	Protect in Place
27	Slurry Wall	40+00 – 41+75	Slurry wall for contamination located within property adjacent to project approximately 35’ - 50’ from landside toe	Protect in Place
28	Fence	34+70 – 51+50	Fence along west side of existing levee crest and Bay Trail approximately 25’ - 50’ from landside toe	Protect in Place
29	PG&E Transmission Tower	42+25	PG&E transmission tower approximately 25’ from landside toe	Protect in Place
30	PG&E Transmission Tower	47+75	PG&E transmission tower approximately 35’ from landside toe	Protect in Place

Runnymede Stormwater Channel Relocation

An existing stormwater channel connecting the Runnymede Stormwater Culvert to the O’Connor Pump Station runs roughly parallel to the flood protection alignment from Runnymede Street to the south end of the project. The stormwater channel will be relocated to the west from approximately Station 21+00 to 28+00 to accommodate the proposed flood protection footprint in lieu of pushing the flood protection alignment further into the Faber Tract Marsh. Additionally, from station 17+50 to 19+50 the eastern bank of the stormwater channel will be realigned to accommodate the flood protection footprint.

The relocated channel from Station 25+00 to 28+00 is typically 10’ wide (from toe to toe) with 2:1 maximum side slopes along the west and east banks and provides a minimum of 15 feet from the landside toe to the top of channel bank. From Station 21+00 to 25+00 the channel width is 10’ and the eastern bank slopes at 2:1 while the western bank is supported by a retaining wall so that the channel fits within the existing property line on the west while still providing a minimum of 15 feet from the landside toe to the top of channel bank. The wall is protected from scour with Caltrans Class VI riprap. The relocated channel’s cross section and profile is informed by the 2014 Phase 2 Runnymede Storm Drain Improvements record drawings and the existing conditions. As the width and side slopes of the existing channel varies greatly, the relocated portion matches the 10’ wide, 2:1 side slope channel shown in the record drawings. Due to the organic nature of the earthen channel where sediment and soil migrate over time, the existing depth also varies between approximately 2’ to 3’ along the alignment. The relocated channel depth is designed at a constant 3’2”, which reflects the average channel depth found in the 2014 record drawings. Much like the existing depth of the

channel, the existing gradeline also varies considerably along the existing alignment. The elevations along the existing flowline (west and east banks) differ from 1' to 2' along the various peaks and valleys between Stations 21+00 and 28+00. The relocated channel alignment maintains a 0.08% gradeline which exceeds the 0.05% designed slope found in the 2014 record drawings.

Runnymede Stormwater Channel Culverts

The culverts for the existing public access path at Martin Luther King Park are to be protected in place; however, the culverts at Cypress Street will be replaced to accommodate the flood protection footprint in lieu of pushing the flood protection alignment further into Faber Tract Marsh. To maximize available space, the (4) 54" corrugated metal pipe culverts will be replaced by a single 12' wide and 5' high precast concrete box culvert which yields a similar capacity. Caltrans Class VI Rock Outlet Protection will be placed on the downstream side of the box culvert to help prevent erosion and incising the adjacent earthen channel.

Runnymede Street Storm Main Outfall and Culvert

Based on the review of as-built information provided by the City and confirmation by Kevin Lewis of East Palo Alto Public works, the 72" storm drain main in Runnymede Street currently outfalls into the marsh through the existing levee. An overflow culvert connects this 72" storm drain main to the Runnymede stormwater channel. It appears that the 72" storm drain invert is below the elevation of the marsh. According to as-builts dated 1962 a lift station located at the existing Bay Trail levee pumped stormwater through the levee into the Bay. It appears this lift station has been removed by the City and two tide check valves were installed in the former lift station wet well wall. Based on the meeting with Kevin Lewis, the 72" storm main currently relies solely on gravity flow and the 72" storm main typically has standing water as the outfall elevation is above the pipe. The 2010 Phase 1 Runnymede Storm Drain Improvements project was intended to provide high flow and/or high tide bypass of the outfall to the Runnymede stormwater channel and ultimately the O'Connor Pump Station. The outfall to the stormwater channel is 5.3 feet lower in elevation than the outfall directly to the Bay. The proposed SAFER Bay levee will cut off access to the Bay for the Runnymede outfall; therefore, the existing outfall to the Bay needs to be capped, directing all stormwater to the Runnymede stormwater channel. Because the Runnymede system drains to the stormwater channel in the existing condition, this approach does not detrimentally impact the City drainage system.

PG&E Infrastructure

PG&E Infrastructure is to be protected in place and access to existing transmission towers is to be replaced by the project. The flood protection alignment diverts to the east and a landside retaining wall will be used to avoid a PG&E transmission tower at Station 25+40 and to provide 15 feet clear from the landside toe to the tower (see Vehicular Access section), while minimizing impacts to the marsh. The flood protection

alignment diverts to the west to avoid a PG&E transmission tower at Station 33+40. The waterside toe is located at the transmission tower. The existing access ramp to the tower will be replaced. Transmission towers at Stations 42+25 and 47+75 are to be protected in place. Overhead transmission lines cross or run adjacent to the flood protection alignment at Stations 22+50 – 42+00 and 48+00 – 52+00. Overhead transmission lines will be protected in place during construction. Further coordination is needed with PG&E to understand and document the clearance and access requirements around the transmission towers and overhead transmission lines.

East Palo Alto Sewer District Sewer Main

An existing East Palo Alto Sewer District sewer main runs parallel to the project alignment from Station 0+25 to 34+50. The sewer main will be relocated outside of the footprint of the embankment, but within the 15-foot clear zone from the landside toe from Station 0+25 to 24+00. The remaining sewer main will be protected in place. The sewer relocation alignment is documented in the project plans, however the design and construction of the sewer relocation will be under a separate contract administered by the City prior to or concurrent with the SAFER Bay project.

Tidal Channels

The project alignment was designed to avoid impacts to the existing tidal marsh to the extent feasible. However, the levee footprint or the waterside limits of work will impact some tidal channels that currently provide hydraulic connectivity to marsh areas otherwise unaffected by the levee. To minimize the impacts of filling these channels, replacement tidal channels will be provided to allow for alternative hydraulic connectivity pathways. The exact location and extent of relocated channels will be determined in future submittals and deliverables.

Access Paths

In coordination with the City and community members, access paths to the existing levee and Bay Trail at Martin Luther King Park, Cypress Street, and Runnymede Street will be replaced with ADA compliant accessible paths to the landside toe. ADA compliant ramps will be provided on the landside slope to the new Bay Trail. Curb ramps will be provided at Cypress Street and Runnymede Street. The existing informal footpath at Garden Street will be removed. Vehicular access to the new Bay Trail will be maintained at the O'Connor Pump Station and the existing path at Weeks Street will be replaced with a non-ADA compliant vehicular ramp. It is expected that this vehicular access will be replaced with ADA-compliant access by private developers in the future.

Contaminated Soil and Remedial Infrastructure

Soil and water contamination is known to exist in properties adjacent to the project. This contamination is documented in the project Environmental Impact Report, and in the "Remediation Activity Review Report" dated May 11, 2026, which summarizes

additional documentation of known contamination provided by the City. The following properties are known to have contamination South of Bay Road:

- 1990 Bay Road
- 1250 Weeks Street
- 1275 Runnymede Street

These properties are subject to Land Use Covenants (LUCs) issued by the Regional Water Quality Control Board (RWQCB). These LUC restrict the use of these properties to control exposure to contamination. These properties are managed and operated according to a Comprehensive Site Management Plan (SMP) (S.S. Papdopulos & Associates [SSP&A] 2014) and Construction Risk Management Plan (RMP) (SSP&A 2020) approved by the RWQCB. The SMP describes risk management measures implemented at the site and specifies the individual tasks and responsibilities required to monitor and maintain the components of the remedy for the site while the Construction RMP specifies soil and groundwater management for construction on the properties. The SMP also includes copies of the LUCs (SSP&A 2014).

In general, the remedial strategies in place are avoided by the project footprint, however, portions of the project footprint are within areas of mapped soil and groundwater contamination. Construction on these properties and in areas of mapped soil and groundwater contamination will need to follow the SMP (SSP&A 2014), the RMP (SSP&A 2020), applicable LUCs, and all other applicable project-specific permits and approvals. Major construction work will also require notification and an initial scoping meeting with the RWQCB at least 90 days in advance of the work, along with written approval by the RWQCB in order to proceed.

Details of the subject properties affected by this work are summarized below:

- At 1990 Bay Road (adjacent to Station 34+70 to 52+00) a capped containment cell and slurry wall have been put in place for arsenic contamination in the soil and groundwater. The flood protection alignment is located within the marsh at this location to avoid these remedial strategies. However, the project intersects an area of mapped arsenic contamination in soil and groundwater in the marsh, referred to as the Wetland Operable Unit (Wetland OU) in the SMP (SSP&A 2014). In addition, ground water monitoring wells are located within the marsh in this area. Any wells within the flood protection footprint will be relocated by a qualified environmental engineer with approval from the RWQCB and in coordination with the City.

- At 1250 Weeks Street (adjacent to Station 31+60 to 34+40) arsenic contamination in the soil is known to remain on-site in the vicinity of the existing sanitary sewer main. A cement-bentonite barrier was placed around the sanitary sewer main and groundwater monitoring wells are located on the property. Due to the existing PG&E transmission tower at Station 33+40 the flood protection alignment is located within this property. Any contaminated soil encountered within the property will be disposed of in accordance with the SMP (SSP&A 2014), Construction RMP (SSP&A 2020), and any project-specific permits and approvals for the property. Should the barrier be damaged, it will be replaced in kind in accordance with the Site Management Plan for the property. The groundwater wells within the flood protection footprint will be relocated by a qualified environmental engineer with approval from the RWQCB and in coordination with the City.

- At 1275 Runnymede Street and the Runnymede Street right-of-way (adjacent to Station 28+50 to 31+60) arsenic contamination in the soil is known to exist in areas untreated by previous remedial activities and around the sanitary sewer main. Groundwater monitoring wells are located on the property and in the right-of-way. Due to the existing PG&E transmission tower at Station 33+40 the flood protection alignment is located within this property and right-of-way. Any contaminated soil encountered within the property will be disposed of in accordance with the SMP (SSP&A 2014), Construction RMP (SSP&A 2020), and any project-specific permits and approvals for the property. Any groundwater wells within the flood protection footprint will be relocated by a qualified environmental engineer with approval from the RWQCB and in coordination with the City.

Fences

Existing fences located within the flood protection footprint and the proposed 15-foot clear zone will be relocated outside of the clear zone. All other fences will remain protected in place, unless relocated for access paths and ramps.

Survey Monuments

Existing survey monuments located within the flood protection footprint will be replaced by a surveyor licensed in the State of California.

Access and Trail Design

Bay Trail

The Bay Trail will be relocated on top of the embankment. The Bay Trail design is based on the 2025 San Francisco Bay Trail Design Guidelines and Toolkit 2.0 (Bay Trail Guidelines).

The total available width at the embankment crown is 20 feet. The Bay Trail Guidelines recommend that trails adjacent to 2:1 slopes with vertical drops of 4 feet or more provide either a 5-foot clearance from the edge of pavement to the slope or provide a barrier. In order to reduce project cost, the 5-foot clearance was selected as the preferred alternative for the majority of the alignment.

Where the 5-foot clearance was not achievable at the retaining walls for the PG&E transmission tower and access ramps at Martin Luther King Park, Cypress Street, and Runnymede Street, guardrails are proposed atop the walls to provide fall protection from the trail.

The 5-foot-wide clear zone is reserved to include 2 feet of landscaping along the edge of the slope and a 3-foot-wide shoulder, the minimum recommended shoulder width from the Bay Trail Guidelines. The clear zone will also include space for benches and signage a minimum distance of 2 feet clear from the edge of pavement.

As a result of the clear zone, the available width for the paved trail is 15 feet. In speaking with City of East Palo Alto staff and community members, it is understood that the existing Bay Trail is used extensively by community members walking and biking for recreation and commuting. When user level is high, and in suburban contexts, the Bay Trail Guidelines recommend using mode separation to separate pedestrians from wheeled trail users (bikers, skateboarders, etc.). The National Association of City Transportation Officials (NACTO) recommends bidirectional bike lanes are at least 5 feet wide in each direction. The bike lanes are located on the landside of the trail. The remaining 5 feet of pavement adjacent to the floodwall is reserved for pedestrians. This will allow pedestrians to pause at the wall and look out into the marshes adjacent to the project. Pavement striping and markings per the California Manual on Uniform Traffic Control Devices (CA MUTCD) will be used to facilitate the separation of bike traffic and pedestrian traffic.

The cross-slope of the trail is designed to be 1.5% and the longitudinal slope is designed to not exceed 4.5%.

Pedestrian Access Paths

Access Paths at Martin Luther King Park, Cypress Street, and Runnymede Street have been designed to be compliant with the Americans with Disabilities Act (ADA). The cross-slope of the paths are designed to be 1.5% and the longitudinal slope is designed to not exceed 4.5%. Where the longitudinal slope exceeds 5% handrails and landings are provided and no ramps are designed to exceed 7.5% longitudinal slope in accordance with ADA requirements. Guardrails are provided on all ramp walls for fall protection.

Vehicular Access

Vehicular access to the top of the embankment/Bay Trail is provided at the O'Connor Pump Station where the project connects to the existing Bay Trail and a paved connection to O'Connor Street already exists. Additionally, a vehicular ramp to the Bay Trail is provided at Weeks Street. Additional vehicular access will be possible at Bay Road and will be included in future design.

A 15-foot wide, unpaved Operations and Maintenance (O&M) corridor and clear zone is provided at the landside toe along the length of the alignment. The O&M corridor is provided for access to the landside toe of the embankment for inspection, maintenance, and emergency repairs. Vehicular access to the O&M corridor is provided at the O'Connor Pump Station and Weeks Street. Additional vehicular access will be possible at Bay Road and will be included in future design.

Vehicular access to and along the Bay Trail and O&M corridor is designed to have a maximum cross-slope of 5% and a maximum longitudinal slope of 10%, however in some locations a longitudinal slope of up to 15% is necessary to conform with existing grading.

Pavement Section

The pavement section for the new Bay Trail was developed using the Caltrans R-value method. The Traffic Index (TI) was calculated assuming a singular maintenance or emergency vehicle equivalent to an HS-20 vehicle load accesses the Bay Trail bi-weekly over a 20-year service life, resulting in a TI of 4.24 which is rounded up to a minimum TI of 5.

The compacted earthfill used for the earthen embankment will be the sub-base for the pavement and have an R-value of 20 which results in a total pavement section Gravel Equivalent (GE) requirement of 1.28 feet.

$$GE_{Total} = 0.0032(TI)(100 - R_{SB})$$

The minimum hot mix asphalt (HMA) GE is calculated to be 0.55 assuming the R-value of the aggregate base is 78.

$$GE_{HMA} = 0.0032(TI)(100 - R_{AB})$$

The Gravel Factor for HMA is calculated to be 2.54.

$$G_f = \frac{5.67}{\sqrt{TI}} \text{ when } t \leq 0.5\ ft$$

Which results in a minimum HMA thickness of 0.22 feet.

$$t = \frac{GE}{G_f}$$

The HMA thickness is rounded up to 0.25 feet (3") and as a result the HMA GE is updated to 0.63 feet.

$$GE = t_{HMA} \cdot G_f$$

To meet the total GE requirement, the aggregate base (AB) GE must be at least 0.66 feet.

$$GE_{total} = GE_{HMA} + GE_{AB}$$

The Gravel Factor for AB is a constant 1.1 regardless of TI or thickness resulting in a minimum required thickness of 0.60 feet which is rounded up to 0.67 feet (8").

The resulting pavement section is 3" HMA over 8" aggregate base for vehicular asphalt pavement.

Due to the large, expected settlement of the embankment, the Bay Trail will likely need to be repaved prior to the 20-year service life to maintain a smooth surface for trail users as differential settlement may result in uneven pavement. This will be especially important at the access paths and ramps where differential settlement may result in the ramps falling out of ADA compliance.

Pedestrian asphalt pavement section, pedestrian concrete pavement section, and concrete curb, gutter and curb ramps are provided in accordance with the 2022 San Mateo County Standard Drawings for Public Improvements.

Stormwater Treatment Compliance

This project is subject to the Municipal Regional Stormwater Permit Order No. R2-2022-0018, but is exempt due to specific exclusions included in the order. Per section C.3.b.ii(4)(d)(iii), "Impervious trails that direct stormwater runoff to adjacent vegetated areas, or other non-erodible permeable areas, preferably away from creeks or towards the outboard side of levees, where those areas are at least half as large as the contributing impervious surface area." Since the project's pathways/trails are all graded to direct runoff to nearby vegetated areas, the project is excluded from the provisions for regulated projects.

Stormwater Conveyance

<u>Drainage Swales between Weeks Street and Bay Road</u>

The portion of flood protection between Weeks St. and Bay Rd. is located on the waterside of the existing levee/Bay Trail within Laumeister Marsh which creates a depression swale along the west side of the levee. In order to drain this area, the project proposes to fill the depression with levee earthfill and grade in a swale to convey runoff originating from the top and landside slope of the proposed levee. To adequately convey this runoff, the swale is sized using the Rational Method for a 10 year / 5-minute duration storm event (2.44 in/hr per NOAA Atlas 14 precipitation intensity). The vegetated swale is 2' wide with 2:1 side slopes along with a longitudinal slope between 0.2% to 0.3% sloping away from the high point located near Station 42+00. This yields a flow rate of 2.3 CFS and max depth of 0.9' at both ends of the swale. This flow rate is more than adequately conveyed by the drainage swale as the runoff gets intercepted by two separate drop inlets located near Weeks St. and Bay Rd. Downstream of the drop inlets runoff is conveyed by 12" storm drain lines sloped at 1% which yield pipes just over half full. Downstream, these pipes connect to storm drain maintenance holes and eventually will connect to future storm drain mains located in Weeks St. and Bay Rd. to be designed and built by EPA Public Works.

Utilities

Sewer Relocation

As discussed in the encroachments section, an existing East Palo Alto Sewer District sewer main runs parallel to the project alignment from Station 0+25 to 34+50. The sewer main will be relocated outside of the footprint of the embankment where it encroaches on the embankment prism. The remaining sewer main will be protected in place. The sewer relocation alignment is documented in the project plans, however the design and construction of the sewer relocation will be under a separate contract administered by the City prior to or concurrent with the SAFER Bay project.

Irrigation Points of Connection

Connections to the City of East Palo Alto's water system are proposed at Runnymede Street, Weeks Street, and Bay Road to supply water for the proposed irrigation system. Irrigation points of connection will be made in accordance with the 2022 San Mateo County Standard Drawings for Public Improvements. A water meter and back flow preventer will be provided at each point of connection. Additional points of connection may be added as part of future design.

Erosion Control

Because the construction work for this project will include more than one (1) acre of earthwork, the project will need a National Pollutant Discharge Elimination System (NPDES) permit. Compliance with the NPDES permit includes the development and implementation of a Stormwater Pollution Prevention Plan (SWPPP). The project SWPPP details the specific erosion and sediment control measures that will be used to prevent exposed soil from eroding from the site during rain events, which can clog storm

drains, destroy aquatic habitats, and lead to heavy regulatory fines. Erosion and sediment control measures may be needed to prevent soil migration due to stormwater, wind, contact with equipment, or other variables. Sherwood will develop the project SWPPP to be implemented by the Contractor.

Erosion and sediment control measures will be detailed and implemented on a temporary basis during construction and on a permanent basis for the final conditions of the project site. Temporary erosion and sediment control measures will stabilize disturbed soil using temporary seeding, mulching, or erosion control blankets to stop erosion at the source. Simultaneously, perimeter barriers like silt fences, straw wattles, and storm drain inlet protection will filter sediment from runoff before it leaves the site. Additionally, stabilized construction entrances with equipment wheel washes will be established to prevent vehicles from tracking mud and/or dust onto public roads. Finally, the Contractor will inspect these measures regularly and after any major rainfall to verify they remain fully functional throughout the construction of the project.

Permanent erosion and sediment control measures for the site include (but are not limited to) geosynthetic reinforcement, biodegradable woven mesh, and establishment of vegetation. The Owner will regularly inspect and maintain the final conditions of the project to verify the continued functionality of the erosion and sediment control measures.

Interior Drainage

The interior drainage analysis is being conducted concurrently with the development of the levee construction documents. This analysis defines the future SFHA that will exist within East Palo Alto following levee construction, fulfilling FEMA's requirements for levee certification. This certification is a prerequisite for formal flood map revisions, which will subsequently update the number of buildings requiring flood insurance.

The City of East Palo Alto provided digital data from the hydrologic and hydraulic modeling conducted during the 2023 Ravenswood Business District Specific Plan. This model, built in MIKE+, was converted into an InfoWorks ICM (Version 2026.3) model, which is consistent with the modeling platform used for interior drainage in Menlo Park. Draft and final Interior Drainage Reports will be produced during this levee design project to document the data sources, assumptions, modeling methodologies, and FEMA-required residual flood maps for East Palo Alto.

5.0 CONSTRUCTION CONSIDERATIONS

Temporary Flood Protection

During construction it will be necessary at certain locations to degrade the existing levee prior to constructing the proposed levee/floodwall. It will be the contractor’s

responsibility to maintain flood protection during construction. Methods to provide temporary flood protection during construction are at the sole discretion of the contractor however the following criteria will be provided to the contractor:

- The contractor shall prepare a Temporary Flood Protection plan for review by the engineer and City.
- During construction, where the elevation of existing levee has been reduced below its current elevation of approximately 11 ft NAVD88, temporary flood protection must be provided to either the Mean Higher High Water plus 10-year coastal event elevation, or 11ft NAVD88, whichever is less.
- The means and methods by which the contractor achieves temporary flood protection must be contained within the defined Limits of Work and meet the requirements of the project's Environmental Impact Report.
- Any marsh which is disturbed through the temporary flood protection measures shall be restored to its pre-construction condition at the contractor's expense.
- The contractor shall not degrade more than 500 linear feet of existing levee at any given time.
- Contractor shall monitor weather and tidal forecasts and prepare the site accordingly.
- The contractor shall conform to flood barrier manufacturer's instructions when deploying commercial systems.
- Flood barriers shall not be deployed within environmentally sensitive areas identified on the Plans or fenced in the field.
- Flood barrier locations are subject to review and approval by the engineer.
- All facilities constructed to protect excavations and/or provide a temporary flood barrier during construction are temporary, the property of the contractor, and shall be removed from the project site upon completion of all work necessitating the temporary flood protection.

Sheet Pile Installation with Overhead Transmission Lines

At locations where the alignment crosses beneath overhead transmission lines, the clearance between the transmission lines and the top of levee will be evaluated to verify sufficient space is available for sheet pile installation. Where insufficient clearance exists, the steel sheet piles will be field-spliced during installation. All field splice welds will be subject to 100% ultrasonic testing (UT) inspection.

6.0 REFERENCES

Structural Design Codes

- American Concrete Institute (ACI) 318, Building Code Requirements for Structural Concrete

- American Society of Civil Engineers (ASCE) 7-22, Minimum Design Loads for Buildings and Other Structures
- ACI 350, Code for Environmental Engineering Concrete Structures and Commentary
- American Institute of Steel Construction (AISC) Steel Construction Manual

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- HDR. (2024). Design Criteria and Considerations Technical Memorandum, Strategy to Advance Flood Protection, Ecosystems and Recreation along the Bay (SAFER Bay Project), East Palo Alto and Menlo Park, San Francisquito Creek Joint Powers Authority, 10% and 30% Designs. September 6.
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- Mark Thomas. (2026). Topographic Map, SDE-SAFER Bay SOBR, May 14.
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Figure 1:
Project Map

Figure 1 - Project Map

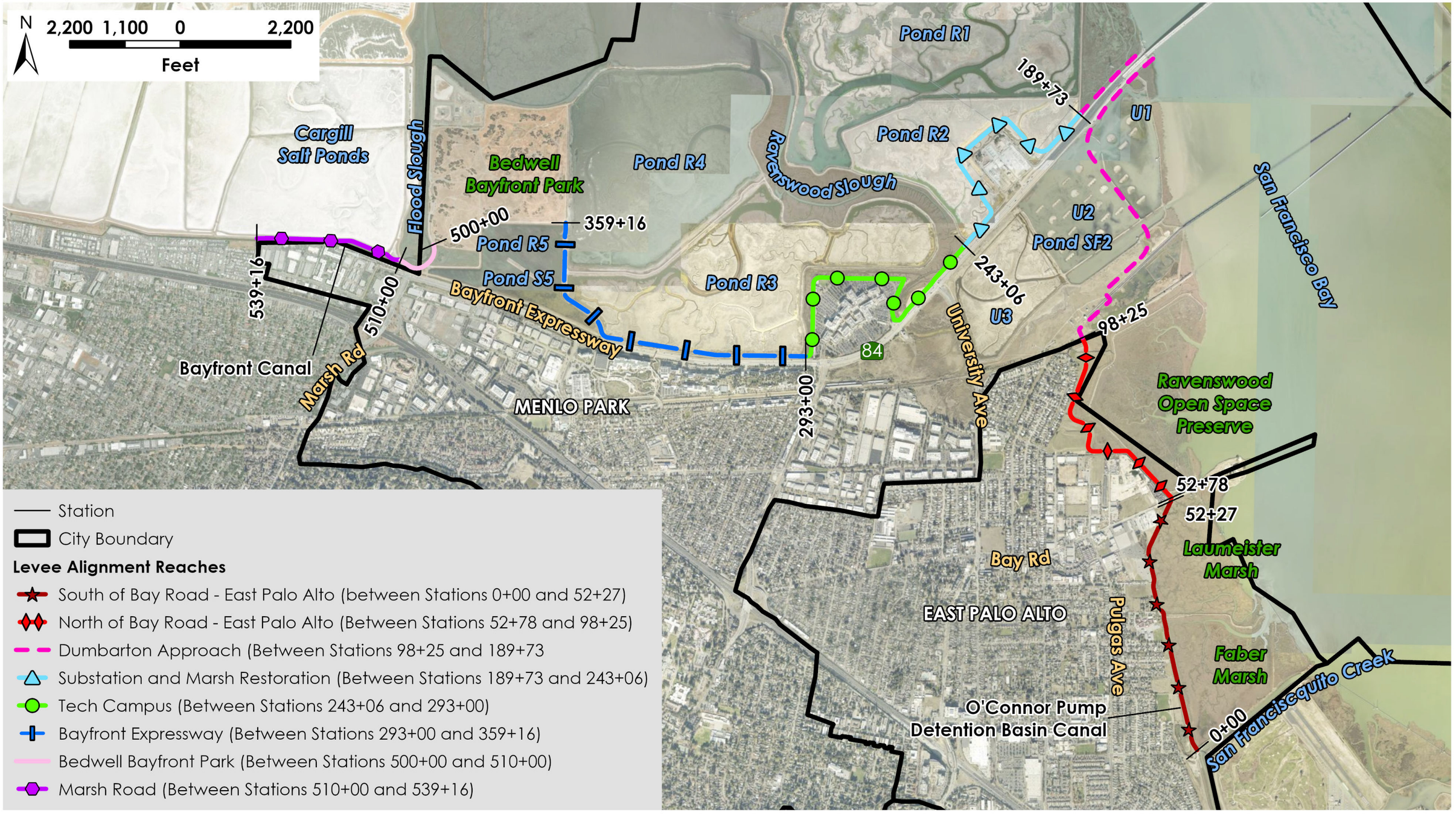

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Figure 2:
Investigation Locations

LEGEND

- INVESTIGATION LOCATION
- LEVEE DESIGN 5' CONTOUR
- EXISTING GROUND ELEVATION 5' CONTOUR
- EXISTING GROUND ELEVATION 1' CONTOUR
- SAFER LEVEE CONTROL LINE

NOTES

CPT = CONE PENETRATION TEST
B = BORING
GS26- = GEOSYNTEC EXPLORATORY BORINGS (GEOSYNTEC 2026)
A- = ARUP EXPLORATORY BORINGS (ARUP 2021)
C-/B- = HDR EXPLORATORY BORINGS (HDR 2024)

1. TOPOGRAPHIC MAP BY MARK THOMAS, DATED MAY 2026 USING NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND NATIONAL GEODETIC VERTICAL DATUM OF 1988 (NAVD88).
2. EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS.
3. HISTORICAL EXPLORATION LOCATIONS ARE BASED ON THE LOCATIONS SHOWN ON THE PRELIMINARY GEOTECHNICAL REPORT FOR SOUTH OF BAY ROAD (HDR 2024) AND THE GEOTECHNICAL DATA REPORT SFPUC REVANSWOOD FACILITY TO BEDWELL BAYFRONT PARK (HDR 2025).

INVESTIGATION LOCATIONS
SAFER EAST PALO ALTO
STA 0+00 TO 52+72
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | MAY 2026

FIGURE 2

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WG3607 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

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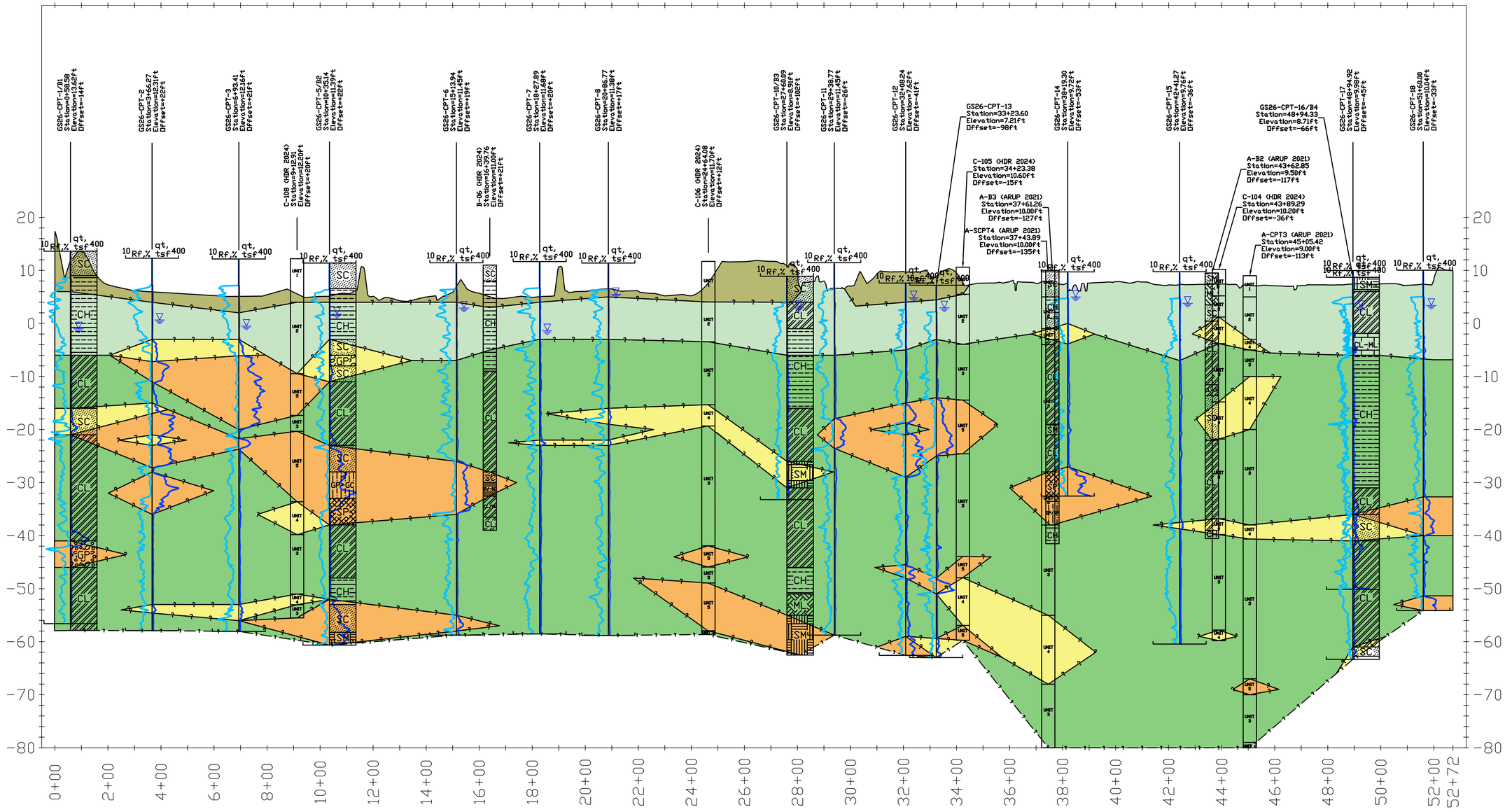
Figure 3:
Geologic Cross Section



GEOLOGIC CROSS SECTION

VERTICAL EXAGGERATION: 20

LEGEND

- UNIT 1 (FILL) - SANDY CLAY (CL/CH) TO SILTY SAND (SC/SM), SOFT TO MEDIUM STIFF
- UNIT 2 (YOUNG BAY MUD [YBM]) - VERY SOFT TO MEDIUM STIFF
- UNIT 3 (OLD BAY CLAY [OBC]) - MEDIUM STIFF TO VERY STIFF
- UNIT 4 (COARSE-GRAINED SOIL 1) - LOOSE TO MEDIUM DENSE
- UNIT 5 (COARSE-GRAINED SOIL 2) - MEDIUM DENSE TO VERY DENSE
- GROUNDWATER LEVEL ENCOUNTERED DURING INVESTIGATION (NOTE 5)

NOTES

CPT = CONE PENETRATION TEST
B = BORING
OFFSET = DISTANCE FROM SAFER BAY CONTROL LINE TO INVESTIGATION LOCATION (BAY SIDE IS POSITIVE AND LAND SIDE IS NEGATIVE).

1. STATIONING BASED ON ALIGNMENT OF SAFER BAY PROJECT CONTROL LINE PROVIDED BY SHERWOOD ENGINEERS ON MAY 2026.
2. EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS.
3. TOPOGRAPHIC SURVEY BY MARK THOMAS, DATED MAY 2026 WAS USED FOR THIS CROSS SECTION. HORIZONTAL DATUM USED WAS NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND THE VERTICAL DATUM USED WAS THE NATIONAL GEODETIC VERTICAL DATUM OF 1988 (NAVD88).
4. GROUNDWATER ELEVATIONS ARE BASED ON THE ESTIMATED HYDROSTATIC PORE PRESSURE AT THE END OF CPT PORE PRESSURE DISSIPATION TESTS TAKEN IN GRANULAR MATERIAL.
5. CPT DATA FROM HISTORICAL INVESTIGATIONS WERE INTERPRETED CONSISTENT WITH THE GEOSYNTEC CPTS AND USED TO EVALUATE THE DEPTHS AND THICKNESSES OF UNITS 1 THROUGH 5.

GEOLOGIC CROSS SECTION
SAFER EAST PALO ALTO
STA 0+00 TO 52+72
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | MAY 2026 | FIGURE 3

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WO3007 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

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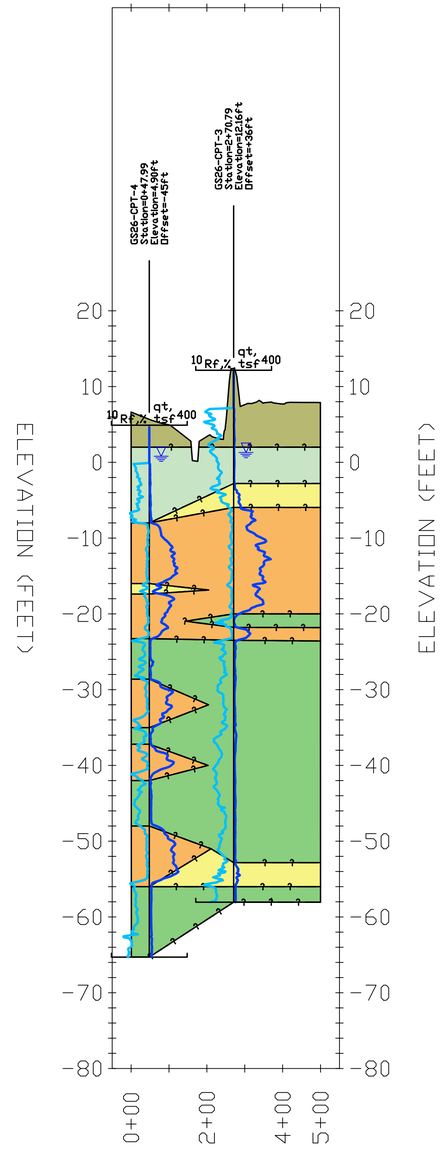
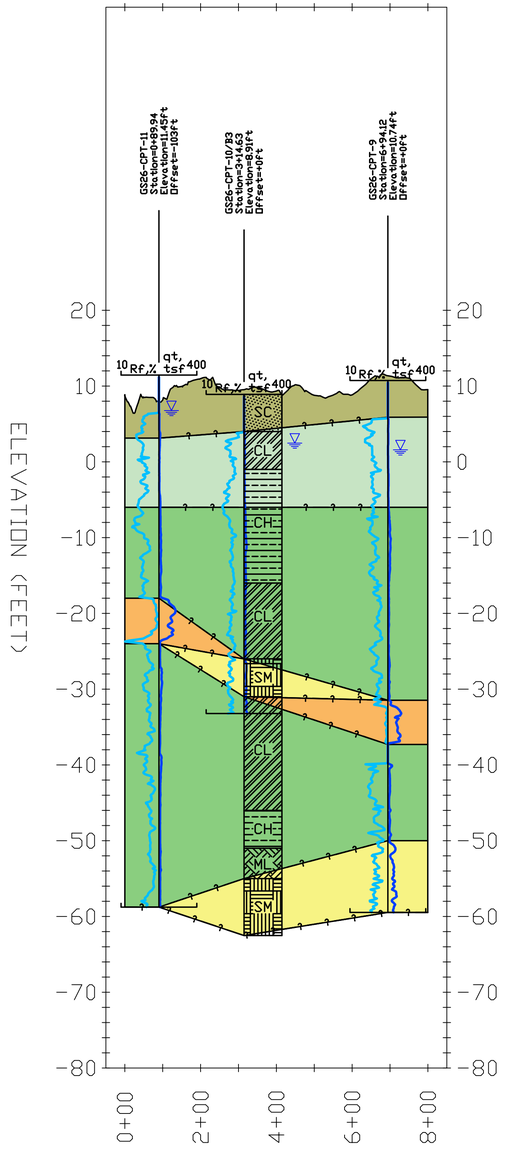
Figure 4:
Perpendicular Cross Section



1/1 STA 7+10 CROSS SECTION

VERTICAL EXAGGERATION: 20



2/1 STA 27+80 CROSS SECTION

VERTICAL EXAGGERATION: 20

LEGEND

- UNIT 1 (FILL) - SANDY CLAY (CL/CH) TO SILTY SAND (SC/SM), SOFT TO MEDIUM STIFF
- UNIT 2 (YOUNG BAY MUD (YBM)) - VERY SOFT TO MEDIUM STIFF
- UNIT 3 (OLD BAY CLAY (OBC)) - MEDIUM STIFF TO VERY STIFF
- UNIT 4 (COARSE-GRAINED SOIL 1) - LOOSE TO MEDIUM DENSE
- UNIT 5 (COARSE-GRAINED SOIL 2) - MEDIUM DENSE TO VERY DENSE
- GROUNDWATER LEVEL ENCOUNTERED DURING INVESTIGATION (NOTE 5)

NOTES

CPT = CONE PENETRATION TEST
B = BORING
OFFSET = DISTANCE FROM CROSS SECTION TO INVESTIGATION LOCATION (LEFT FACING THE BAY IS NEGATIVE AND RIGHT FACING THE BAY IS POSITIVE).

1. STATIONING BASED ON ALIGNMENT OF SAFER BAY PROJECT CONTROL LINE PROVIDED BY SHERWOOD ENGINEERS ON MAY 2026.
2. EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS.
3. TOPOGRAPHIC SURVEY BY MARK THOMAS, DATED MAY 2026 WAS USED FOR THIS CROSS SECTION. HORIZONTAL DATUM USED WAS NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND THE VERTICAL DATUM USED WAS THE NATIONAL GEODETIC VERTICAL DATUM OF 1988 (NAVD88).
4. GROUNDWATER ELEVATIONS ARE BASED ON THE ESTIMATED HYDROSTATIC PORE PRESSURE AT THE END OF CPT PORE PRESSURE DISSIPATION TESTS TAKEN IN GRANULAR MATERIAL.
5. CPT DATA FROM HISTORICAL INVESTIGATIONS WERE INTERPRETED CONSISTENT WITH THE GEOSYNTEC CPTS AND USED TO EVALUATE THE DEPTHS AND THICKNESSES OF UNITS 1 THROUGH 5.

PERPENDICULAR CROSS SECTION
SAFER EAST PALO ALTO
STA 7+10 AND STA 27+80
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | MAY 2026

FIGURE 4

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WO3807 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

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Table 1:
Material Properties

Table 1: Summary of Material Properties
SAFER Bay Project (South of Bay Road)
East Palo Alto, California

Soil Unit	Material	USCS	Total Unit Weight[1]	Strength					Settlement										Permeability		Pile Capacity		
				Undrained/Total Stress Parameters			Drained/Effective Stress Parameters					Modified Compression Indices			Coefficient of Consolidation	Oedometric Modulus	Elastic Modulus	Coefficient of Volume Compressibility	Hydraulic Conductivity	Anisotropy Ratio	Shaft Friction Factor[5][6][8]	Shaft Adhesion Factor [7]	Limiting Shaft Friction[8]
				Undrained Shear Strength[2]	Total Cohesion	Total Friction Angle	Effective Cohesion	Effective Friction Angle	Void Ratio	Porosity	Initial Overconsolidation Ratio	Modified Compression Index	Modified Recompression Index	Modified Secondary Compression Index									
			γ_t (pcf)	S_u (psf)	c_R (psf)	φ_R (degrees)	c' (psf)	φ' (degrees)	e -	n -	OCR -	C_{cc} -	C_{rc} -	C_{ac} -	c_v (ft^2/yr)	E_{oed} (ksf)	E' (ksf)	m_v (1/ksf)	K_h (cm/s)	K_h/K_v -	β -	α -	q_s (psf)
Unit 1	Existing Fill	CL,CH,SC,SM	120	--	--	--	0	30	--	--	--	--	--	--	--	--	200	--	2.0 X 10^{-6}	4	0.29	--	1400
Unit 2	Young Bay Mud [YBM]	CL,CH	115	400	400	0	200	30	1.1	0.52	2.0	0.20	0.03	0.01	15	50	--	2.0 X 10^{-2}	1.5 X 10^{-6}	4	0.20	1.0	400
	Young Bay Mud [YBM] (Marsh)			200	200	0			1.3	0.57	1.0					50		2.0 X 10^{-2}	1.5 X 10^{-6}		0.20	1.0	200
Unit 3	Old Bay Clay [OBC]	CL,CH,ML	120	500 + 21z	500 + 21z	0	0	30	0.9	0.47	2.2	0.20	0.02	0.01	20	150	--	6.5 X 10^{-3}	4.0 X 10^{-6}	4	0.23	0.9	450 + 19z
Unit 4	Coarse-Grained Soil 1	SC,SW,SM,GC,GP	125	-- $(S_r/\sigma'_{vc} = 0.17)^{[3][4]}$	--	--	0 --	30 --	--	--	-- --	-- --	-- --	-- --	-- --	-- --	500 --	-- --	3.0 X 10^{-4} --	4	0.29 --	-- --	1400 --
Unit 5	Coarse-Grained Soil 2	SC,SP,SM,GP,GP-GC	125	-- $(S_r/\sigma'_{vc} = 0.40)^{[3][4]}$	--	--	0 --	38 --	--	--	-- --	-- --	-- --	-- --	-- --	-- --	1000 --	-- --	1.0 X 10^{-2} --	4	0.37 --	-- --	1700 --

Abbreviations:
-- : Not applicable based on soil type
σ'_v: Vertical effective stress at the depth of interest
ksi: kips per square inch
pcf: pounds per cubic foot
pci: pounds per cubic inch
psf: Pounds per square foot
z: depth below top of OBC
S_r: Residual shear strength
σ'_{vc}: effective stress

Notes:
1. Total unit weight refers to the moist density. For soil beneath the groundwater table, the effective unit weight can be estimated by subtracting the unit weight of water (62.4 pcf).
2. Undrained shear strength can be used to represent total stress cohesion (c) of cohesive soil layers at 5% strain, for sheetpile analysis in LPILE.
3. Values in parentheses are provided to model conditions after soil liquefaction due to seismic loading, and assume use of a soft clay p-y model to represent liquefied soil.
4. Post liquefaction residual shear strength ratio (S_r/σ'_{vc}) for Unit 4 and 5 is based on the relationship provided in Figure 89 of: Idriss, I.M., and Boulanger, R.W.. 2008. "Soil Liquefaction During Earthquakes". Earthquake Engineering Research Institute (EERI) Monograph MNO-12.
5. Shaft friction factor for Unit 1, 4, & 5 is based on API (2002): American Petroleum Institute. 2002. *API Recommended Practice 2A-WSD-Planning, Designing, and Constructing Fixed Offshore Platforms - Working Stress Design*. 21st ed.
6. Shaft friction factor for Unit 2 & 3 is based on FHWA (2016): *Design and Construction of Driven Pile Foundations - Volume I* (Publication No. FHWA-NHI-16-009) and USACE (1991): *Design of Pile Foundations* (EM-1110-2-2906).
7. Shaft Adhesion Factor is based on DM7.2 (Foundation and Earth Structures) UFC 3-220-20 (16 January 2025).
8. Limiting Shaft Friction values represent ultimate, not allowable loads. To obtain allowable loads, the structural engineer should apply appropriate factors of safety (FS) when using allowable stress design, or by applying load and resistance factors when using load and resistance factor design.
9. All parameters shown represent conditions for static loading below the groundwater table unless noted otherwise.

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Appendix A:
Levee/Floodwall Engineering Analysis

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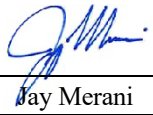
Appendix A.1:
Levee/Floodwall Engineering Analysis
Settlement Evaluation

Geosyntec consultants

COMPUTATION COVER SHEET

Client: **Sherwood Design Engineers** Project: **SAFER Bay Project (South of Bay Road)** Project No.: **WG3607**

Title of Computations **South of Bay Road Levee Settlement Evaluation**

Computations by:	Signature		5/28/2026
	Printed Name	Armaan Khamenian	Date
	Title	Senior Staff Professional	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		6/1/2026
	Printed Name	Jay Merani	Date
	Title	Senior Engineer	
Computations Checked by:	Signature		6/1/2026 & 6/2/2026
	Printed Name	Jay Merani/David Umberg	Date
	Title	Senior Engineer/Senior Engineer	
Computations backchecked by: (originator)	Signature		6/2/2026
	Printed Name	Armaan Khamenian	Date
	Title	Senior Staff Professional	
Approved by: (pm or designate)	Signature		6/2/2026
	Printed Name	David Umberg	Date
	Title	Senior Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	A. Khamenian	Date:	5/28/2026	Reviewed by:	D. Umberg	Date:	6/2/2026
Client:	Sherwood Design Engineers	Project:	SAFER Bay East Palo Alto (South of Bay Road)	Project No.:	WG3607	Task No.:	07

1. PURPOSE

This package presents an analysis of settlement for the proposed flood control system embankment and underlying subsurface soils along the SAFER (Strategy to Advance Flood Protection, Ecosystems and Recreation) Bay Project control line (alignment), from STA 0+00 to STA 52+72, located in East Palo Alto, California, and referred to here as "South of Bay Road". The estimated settlements will be used to refine embankment overbuild heights along the alignment so that the long-term settled profile of the embankment meets or exceeds the long-term design elevation for flood protection during the project's 50-year design life. Additionally, settlement profiles will be incorporated in the sheet pile design calculations being performed separately by COWI, Inc.

2. DESIGN CRITERIA

Geometry of the flood control system embankment requires a minimum embankment crest elevation of +13.0 feet NAVD88 to be maintained over the duration of an assumed 50-year project design life. The design post-settlement embankment geometry is intended to have a landside slope of 2 Horizontal (H):1 Vertical (V) and a waterside slope of 3H:1V and a crown width of 20 feet. (Sherwood 2026).

3. METHODOLOGY

Five cross sections along the levee were selected for settlement analysis based on the subsurface profile shown in Figure 2.

The settlement analysis assumed the following:

- Immediate settlement, primary consolidation, and secondary compression will occur;
- The top two (2) feet of the foundation subgrade soil will be excavated and replaced with earthfill prior to levee earthfill construction. This subgrade excavation and replacement is assumed to occur over a minimum duration of 1 month;
- Levee sections will be constructed over one season (i.e., 3 months);
- Levee height, geometry, and width may vary to meet long term (i.e., 50 years) elevation and geometry;
- The levee embankment will be overbuilt above the long-term embankment crest design elevation to compensate for the anticipated settlement that is expected to occur in the 50 years following earthwork construction. The overbuild heights are intended to achieve a

minimum long-term design elevation of 13.0 ft NAVD88 or greater following long-term settlement.

- Overbuilding of the levee will be performed with additional earthfill material (i.e., additional loading) to reach long-term design elevation and geometry. Settlement that occurs during the 4-month earthwork construction period is assumed to be compensated by adding additional fill as needed to achieve the target overbuild elevations.

One-dimensional consolidation settlement was evaluated using the commercial software Settle3 (Rocscience, Inc., 2025b). Further discussions on settlement calculations are presented in Section 5.

4. SETTLEMENT ANALYSIS

The commercial software Settle3 (Rocscience, Inc., 2025b) was used to estimate settlement of the proposed embankment. The program estimates loading stress in three dimensions; however, settlement is only estimated in one dimension as vertical displacements using the Terzaghi 1D consolidation theory (Rocscience, Inc., 2025a). Stress distributions due to the levee load were estimated with the Boussinesq method, which assumes the underlying material is elastic, homogeneous, and isotropic.

4.1 Subsurface Conditions and Properties

The general subsurface conditions found below the levee alignment consist of existing levee fill where the proposed levee alignment follows the existing levee (Fill), an upper layer of weak compressible marine clay (Young Bay Mud [YBM]), and a lower layer of older and stiffer marine clay (Old Bay Clay [OBC]). Interbedded in the clay layers are a series of discontinuous, loose to medium dense coarse-grained soils (Coarse-Grained Soil 1) and medium dense to very dense coarse-grained soils (Coarse-Grained Soil 2). The subsurface conditions were modeled to a minimum depth of 70 feet, consistent with the geological profile in Figure 2. Drainage layers were assumed to be the Coarse-Grained Soil 1, Coarse-Grained Soil 2, and ground surface. Drainage was also assumed at the base of the OBC if it was the bottommost layer in the soil profile. An idealized geologic cross-section of the levee alignment can be found in Figure 2. The subsurface soil layers used for each analysis section are shown on the figure. Soil parameters for each of the soil units are shown in Table 2.

To account for the fluctuating tides, anticipated sea level rise (SLR), and the dynamic groundwater table within the levee, the groundwater level was set to the waterside toe of the proposed levee. This allows for the model to account for settlement of soils that are currently above the existing groundwater level but may become submerged with SLR. Groundwater levels

within the proposed levee earthfill above the base of the embankment were not modeled because, while possible, these would be unconservative by reducing the current embankment loading.

4.2 Levee Geometry and Material Properties

The levee crest elevation and side slopes grade were increased above the long-term design elevation of 13.0 ft NAVD88 to accommodate expected settlement (i.e., over time, settlement will result in gradual lowering of the crest elevation and flattening of the slopes). This was modeled using the concept illustrated in Figure 1-2 of EM 1110-2-1913 (USACE, 2022). The overbuild height and geometry for each analysis section was chosen so that the minimum crest elevation of the levee was met or exceeded after 50 years of estimated settlement. The geometry iterations are shown on the schematic provided in Figure 3.

As seen in Figure 1, the proposed alignment generally follows the existing levee alignment except for sections approximately between STA 28+00 to STA 29+50 and STA 34+00 to STA 52+72, where the proposed flood control system will be constructed in the marsh to the waterside of the existing levee. For modeled cross sections of the proposed flood control along the existing levee alignment, ground elevation was assumed to be the average of the elevations of the waterside and landside toes. This elevation assumed the existing levee would be removed before construction of the new levee. For modeled cross sections where the proposed flood control system would be constructed in the marsh, the ground elevation was taken as the existing elevation. Table 1 summarizes the station number and ground elevation used for each settlement model.

For the proposed embankment fill material, similar parameters to the existing levee fill (Table 2) were used but with a higher total unit weight of 125 pounds per cubic foot (pcf) based on recommended presumptive material properties of replacement levee fill found in Table 5-5 of the Urban Levee Geotechnical Evaluations Project *Guidance Document for Geotechnical Analyses* (URS, 2015).

4.3 Embankment Construction Sequencing

Embankment construction is assumed to be completed within approximately 3 months with each month including one-third of the height being constructed. This was represented as three stages in the Settle3 software, with each stage corresponding to one month of construction.

Prior to construction in the marsh area (i.e., approximately between STA 28+00 to STA 29+50 and STA 34+00 to STA 52+72), the top two (2) feet of marsh material (i.e., YBM) will be removed and replaced with embankment fill. This excavation and fill placement was assumed to take one additional month prior to levee construction. In the software, the initial stage (Stage 0) was defined as a 2-foot-deep excavation below the entire footprint and backfill with imported

levee earthfill. The next three stages would be the levee embankment construction as described above.

For each settlement model, total settlement was defined as settlement at the end of embankment construction. Settlement during construction (before time = zero) is assumed to be accounted for with extra fill placement during the earthwork construction to reach design overbuild elevations.

5. SETTLE3 MODEL VALIDATION

Settlement results of the Settle3 model were compared to the results of a simplified one-dimensional consolidation analysis using a spreadsheet with similar input soil parameters, groundwater conditions, and levee geometry. Total consolidation settlement (i.e., primary consolidation) was considered in the validation as it accounts for the majority of the settlement in the Settle3 model.

For the spreadsheet validation model, STA 49+00 was used with the same material parameters as the Settle3 model. Total immediate settlement in the Settle3 model for STA 49+00 was less than a tenth of an inch. Therefore, immediate settlement of coarse-grained layers (i.e., Fill, Coarse-Grained Soil 1, and Coarse-Grained Soil 2) was not estimated for as part of this validation model. Additionally, instead of using the Boussinesq load distribution, a simplified 2:1 method was used where the load of the levee on the subsurface soil was assumed to spread outwards at a slope of 2V:1H with depth (Rocscience, Inc., 2025a). Lastly, since the spreadsheet calculation did not consider time-rate of consolidation, the consolidation settlement totals from the Settle3 model were taken from the start of the levee fill construction (i.e., after removal and replacement of the top 2 feet of YBM).

The results of the spreadsheet model indicated a total consolidation settlement of 21.3 inches which was similar to Settle3's total estimate of 19.3 inches at STA 49+00 (including settlement that occurs during earthwork construction). The higher settlement prediction from the spreadsheet method may be due to the 2:1 load distribution method. See Attachment A for the spreadsheet validation results.

6. RESULTS

Results of the settlement analysis and recommended overbuild heights are summarized in Table 3 for the levee post-construction settlement. Settle3 outputs for settlement estimates following the completion of earthwork construction can be found in Attachment B. These plots provide estimates of settlement versus time at the top of the YBM layer (i.e., at the base of the proposed levee embankment), settlement versus depth profiles beneath the embankment centerline, and horizontal settlement profiles for the top of YBM for cross sections perpendicular to the SAFER Bay control line.

Settle3 outputs for total settlement profiles for the top of the YBM along cross sections perpendicular to the SAFER Bay control line can be found in Attachment C. In this attachment, negative settlement refers to settlement that is expected to occur during earthwork construction and must be compensated by adding additional fill to reach the target overbuild elevations. Positive settlement represents settlement estimates from the end of earthwork construction through a design life of 50-year. As shown, total settlement estimates ranged between approximately 7 and 20 inches, with approximately 2 to 6 inches of this settlement occurring during earthwork construction, and the remainder of the settlement occurring after earthwork completion. Overbuild thicknesses listed below and in the figures were selected based on the settlement that occurs after construction because settlement during construction is expected to be accounted for with additional fill added during the construction.

The following overbuild thicknesses are recommended along the levee alignment to maintain the target crest elevation and slopes during the design life:

- STA 0+00 to STA 12+00: 6 inches of overbuild
- STA 12+00 to 12+50: Transition from 6 inches to 12 inches of overbuild
- STA 12+50 to 27+50: 12 inches of overbuild
- STA 27+50 to 28+00: Transition from 12 inches to 18 inches of overbuild
- STA 28+00 to STA 52+72 (Bay Road): 18 inches of overbuild

7. REFERENCES

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URS. 2015. *Urban Levee Evaluations Project, Guidance Document for Geotechnical Analyses.* Contract 4600008101. Prepared for California Department of Water Resources. Dated April 2015.

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FIGURES

LEGEND

- INVESTIGATION LOCATION
- EXISTING GROUND ELEVATION 5' CONTOUR
- EXISTING GROUND ELEVATION 1' CONTOUR
- SAFER BAY CONTROL LINE
- 6 IN. OF OVERBUILT
- TRANSITION
- 12 IN. OF OVERBUILD
- 18 IN. OF OVERBUILD

NOTES

CPT = CONE PENETRATION TEST
B = BORING
GS26- = GEOSYNTEC EXPLORATORY BORINGS (GEOSYNTEC 2026)
A- = ARUP EXPLORATORY BORINGS (ARUP 2021)
C-/B- = HDR EXPLORATORY BORINGS (HDR 2024)

1. TOPOGRAPHIC MAP BY MARK THOMAS, DATED MAY 2026 USING NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
2. GEOSYNTEC EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS, AS DESCRIBED IN THE GEOTECHNICAL DATA REPORT (GDR) (GEOSYNTEC, 2026).
3. HISTORICAL EXPLORATION LOCATIONS ARE BASED ON THE LOCATIONS SHOWN ON THE PRELIMINARY GEOTECHNICAL REPORT FOR SOUTH OF BAY ROAD (HDR 2024) AND THE GEOTECHNICAL DATA REPORT SFPUC REVANSWOOD FACILITY TO BEDWELL BAYFRONT PARK (HDR 2025).

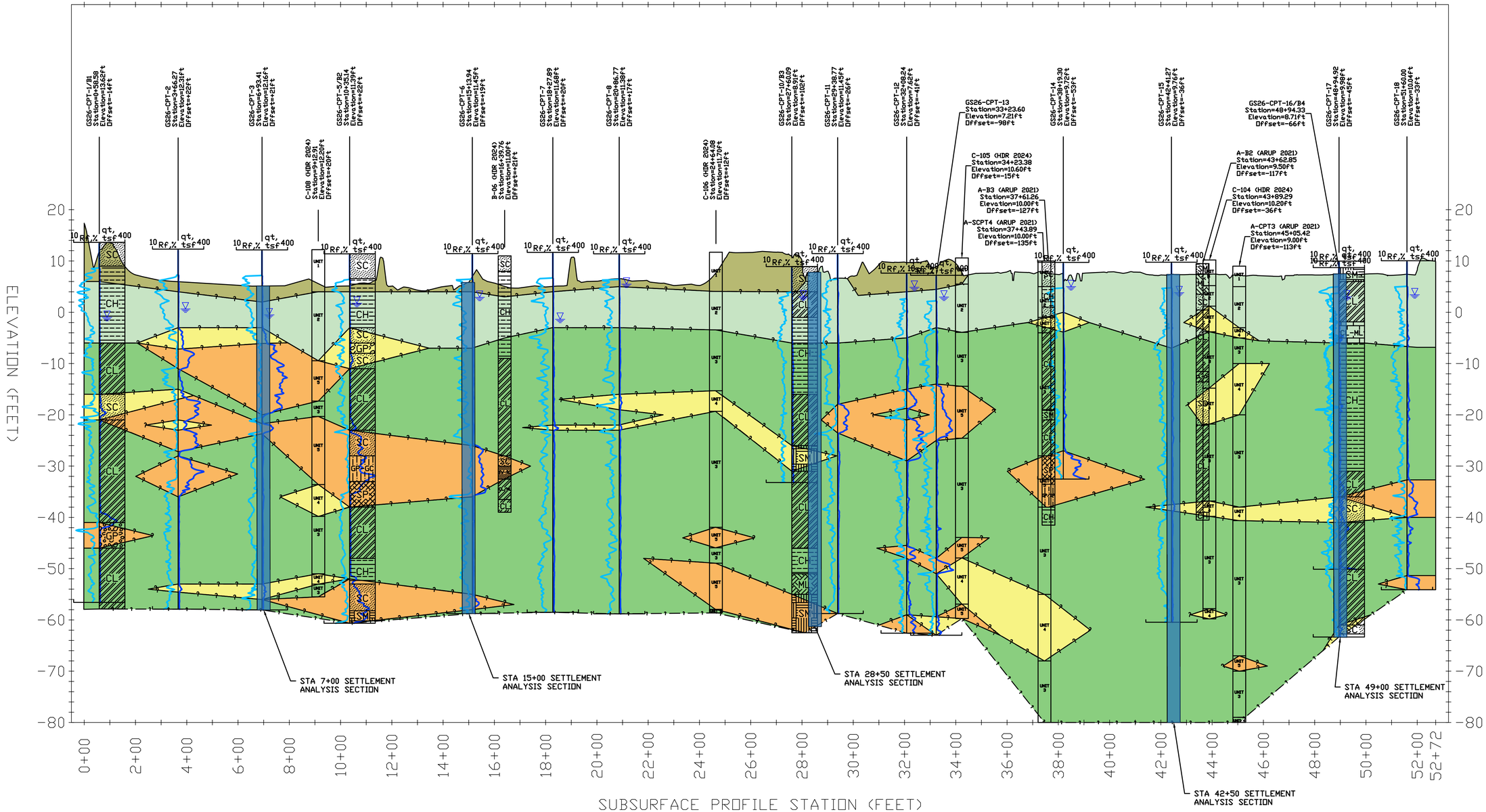
PLAN VIEW
SAFER BAY PROJECT
SOUTH OF BAY ROAD
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | JUNE 2026

FIGURE 1

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WG3807 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS



LEGEND

- UNIT 1 (FILL) - SANDY CLAY (CL/CH) TO SILTY SAND (SC/SM), SOFT TO MEDIUM STIFF
- UNIT 2 (YOUNG BAY MUD (YBM)) - VERY SOFT TO MEDIUM STIFF
- UNIT 3 (OLD BAY CLAY (OBC)) - MEDIUM STIFF TO VERY STIFF
- UNIT 4 (COARSE-GRAINED SOIL 1) - LOOSE TO MEDIUM DENSE
- UNIT 5 (COARSE-GRAINED SOIL 2) - MEDIUM DENSE TO VERY DENSE
- GROUNDWATER LEVEL ENCOUNTERED DURING GEOSYNTEC INVESTIGATION (NOTE 4)

NOTES

CPT = CONE PENETRATION TEST
B = BORING
OFFSET = DISTANCE FROM SAFER BAY CONTROL LINE TO INVESTIGATION LOCATION (BAY SIDE IS POSITIVE AND LAND SIDE IS NEGATIVE).

1. STATIONING BASED ON ALIGNMENT OF SAFER BAY PROJECT CONTROL LINE PROVIDED BY SHERWOOD ENGINEERS ON MAY 2026.
2. GEOSYNTEC EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS, AS DESCRIBED IN THE GEOTECHNICAL DATA REPORT (GDR) (GEOSYNTEC, 2026).
3. TOPOGRAPHIC SURVEY BY MARK THOMAS, DATED MAY 2026 WAS USED FOR THIS CROSS SECTION. HORIZONTAL DATUM USED WAS NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND THE VERTICAL DATUM USED WAS THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
4. GROUNDWATER ELEVATIONS ARE BASED ON THE ESTIMATED HYDROSTATIC PORE PRESSURE AT THE END OF CPT PORE PRESSURE DISSIPATION TESTS TAKEN IN GRANULAR MATERIAL.
5. CPT DATA FROM HISTORICAL INVESTIGATIONS WERE INTERPRETED CONSISTENT WITH THE GEOSYNTEC CPTS AND USED TO EVALUATE THE DEPTHS AND THICKNESSES OF UNITS 1 THROUGH 5.

GEOLOGIC CROSS SECTION
SAFER BAY PROJECT
SOUTH OF BAY ROAD
EAST PALO ALTO, CALIFORNIA

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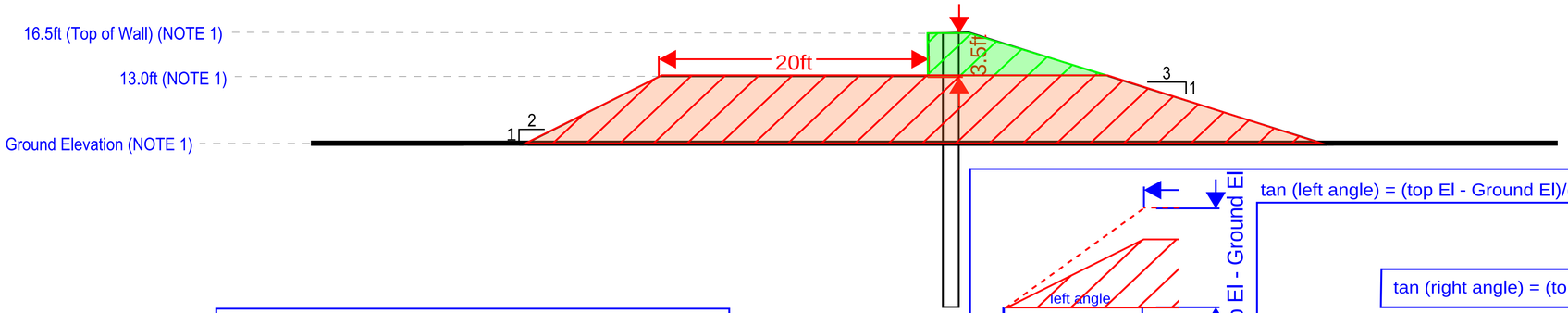
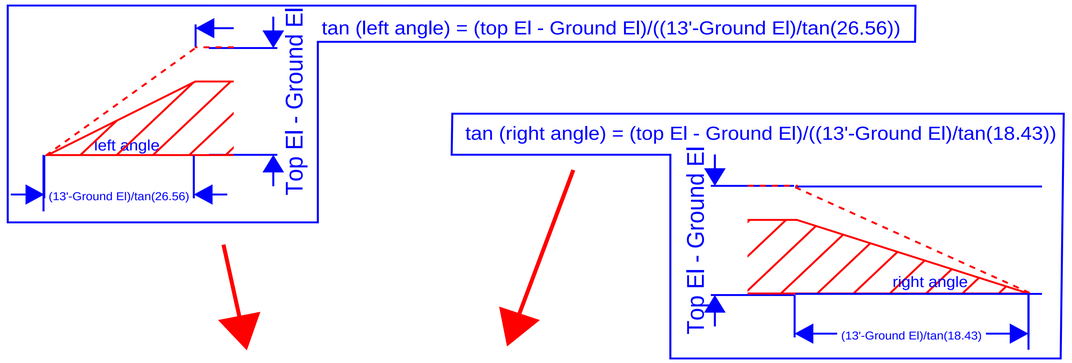
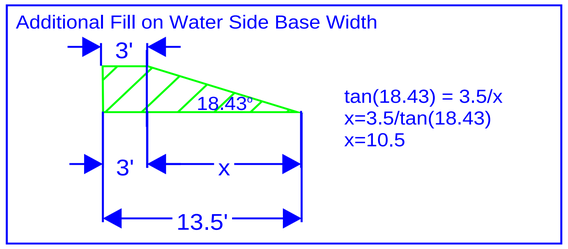
PROJECT NO: 26007427 | JUNE 2026 | FIGURE 2

P:\CAD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WG3607 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

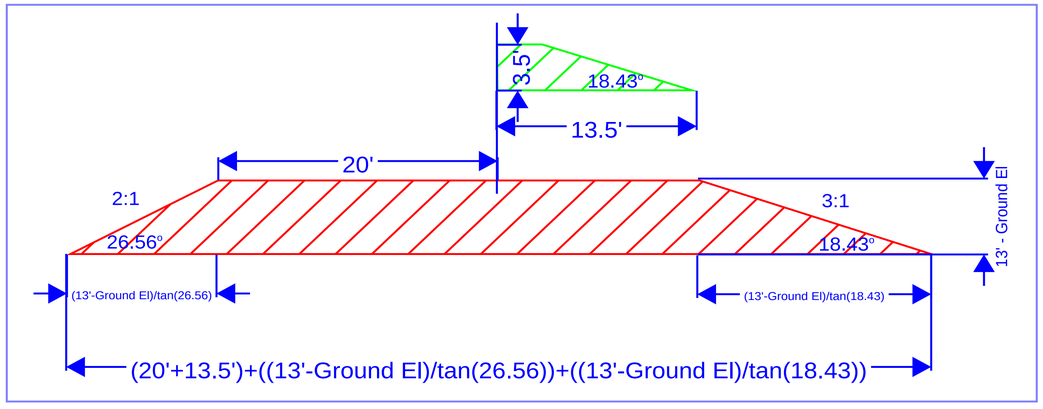
Levee Geometry Used for Settlement Calculations



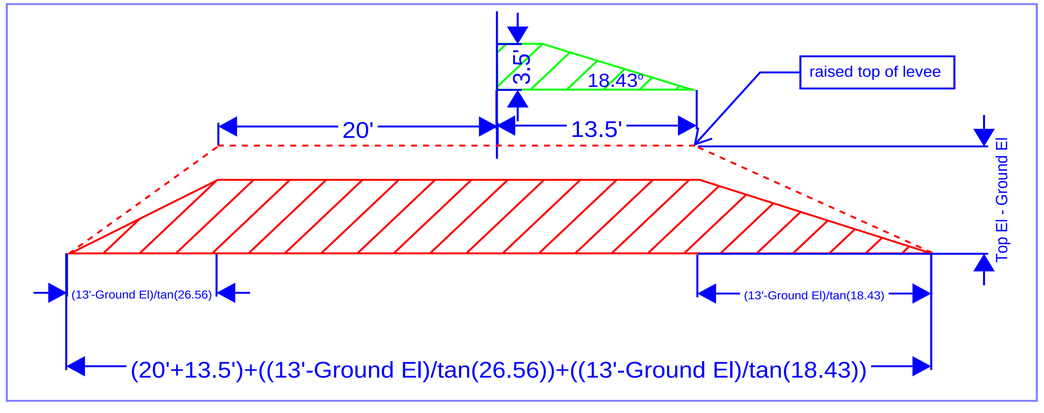




Levee Geometry with no Overbuild



Levee Geometry with Overbuild (steepen slopes to allow for settlement)



NOTES

1. TOP OF WALL ELEVATION AND GROUND SURFACE ELEVATIONS REPRESENT MINIMUM DESIGN ELEVATIONS AFTER LONG-TERN SETTLEMENT OVER THE PROJECT'S ASSUMED 50-YEAR DESIGN LIFE.
2. ELEVATIONS ARE IN TERMS OF THE NAVD88 VERTICAL DATUM.
3. LONG-TERM WALL SETTLEMENT HAS BEEN ESTIMATED SEPARATELY BY COWI. SOIL-STRUCTURE INTERACTION HAS NOT BEEN CONSIDERED AS PART OF THIS CALCULATION PACKAGE.

LEVEE GEOMETRY CALCULATION
SAFER BAY PROJECT
SOUTH OF BAY ROAD
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

FIGURE 3

PROJECT NO: 26007427 | JUNE 2026

P:\CAD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WS36\07 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

TABLES

Table 1: Modeled Ground Elevations at Each Cross Section

Cross Section Station	Modeled Ground Elevation (ft NAVD88)
7+00	5.4
15+00	6.5
28+50	7.8
42+50	7.5
49+00	7.5

Table 2: Summary of Settlement Material Properties
SAFER Bay Project (South of Bay Road)
East Palo Alto, California

Soil Unit	Material	USCS	Total Unit Weight[(1)]	Settlement							
				Void Ratio	Porosity	Initial Overconsolidation Ratio	Modified Compression Indices			Coefficient of Consolidation	Elastic Modulus
							Modified Compression Index	Modified Recompression Index	Modified Secondary Compression Index Ratio		
			γ_t (pcf)	e -	n -	OCR -	$C_{c\varepsilon}$ -	$C_{r\varepsilon}$ -	$C_{\alpha\varepsilon}/C_{c\varepsilon}$ -	c_v (ft^2/yr)	E' (ksf)
Unit 1	Existing Fill	CL,CH,SC,SM	120	--	--	--	--	--	--	--	200
Unit 2	Young Bay Mud [YBM]	CL,CH	115	1.1	0.52	2.0	0.20	0.03	0.05	15	--
	Young Bay Mud [YBM] (Marsh)			1.3	0.57	1.0					
Unit 3	Old Bay Clay [OBC]	CL,CH,ML	120	0.9	0.47	2.2	0.20	0.02	0.05	20	--
Unit 4	Coarse-Grained Soil 1	SC,SW,SM,GC,GP	125	--	--	-- --	-- --	-- --	-- --	-- --	500
Unit 5	Coarse-Grained Soil 2	SC,SP,SM,GP,GP-GC	125	--	--	-- --	-- --	-- --	-- --	-- --	1000

Abbreviations:
-- : Not applicable based on soil type
pcf: pounds per cubic foot
ft^2/yr: square feet per year
ksf: kips per square foot

Notes:
1. Total unit weight refers to the moist density. For soil beneath the groundwater table, the effective unit weight can be estimated by subtracting the unit weight of water (62.4 pcf).
2. All parameters shown represent conditions for static loading below the groundwater table unless noted otherwise.
3. A Modified Secondary Compression Index of $C_{\alpha\varepsilon}/C_{r\varepsilon}$ = 0.05 was also used to represent secondary compression of overconsolidated soil.

Table 3: Settlement Results
SAFER Bay Project (South of Bay Road)
East Palo Alto, California

Station	Ground Surface Elevation[1][2] (ft NAVD88)	Overbuild Height (ft)	Post-Construction Levee Top Elevation (ft NAVD88)	Approximate Embankment Height (ft)	Stage Height (Embankment Height/3)[6] (ft)	Levee Base Width (ft)	Left Slope[11] (°)	Right Slope[11] (°)	50 Year Post-Construction Settlement[7] (in)	Total 50 Year Settlement[13][14] (in)	Long-Term Embankment Top Elevation[3] (ft NAVD88)
7+00	5.4	**0.5**	13.5	8.1	2.70	71.5	28.1	19.6	**5.3**	7.8	**13.1**
15+00	6.5	**1.0**	14.0	7.5	2.50	66.0	30.0	21.0	**9.9**	11.8	**13.2**
28+50	7.8	**1.5**	14.5	6.7	2.23	59.5	32.8	23.2	**15.9**	20.8	**13.2**
42+50	7.5	**1.5**	14.5	7.0	2.33	61.0	32.5	23.0	**16.6**	21.5	**13.1**
49+00	7.5	**1.5**	14.5	7.0	2.33	61.0	32.5	23.0	**16.3**	21.4	**13.1**

Notes

1. For sections of the levee alignment outside of the existing levee footprint, the existing ground elevation was taken on the SAFER Bay control line based on the May 2026 topographic survey by Mark Thomas.
2. For sections of the levee alignment within the existing levee footprint, the existing ground elevation was obtained by averaging the elevations of the existing levee landside and waterside toe based on the May 2026 topographic survey by Mark Thomas.
3. Target long-term (50 Years) top elevation of the levee is 13.0 feet.
4. Water table elevation set to the ground surface elevation at the toe of the levee.
5. Levee construction duration assumed to be three months for STA 7+00 and 15+00 with a third of the total levee height built at the end of each month/stage of construction.
6. For sections of the levee alignment outside of the existing levee footprint (i.e., for STA 28+50, 42+50, and 49+50, construction is four months), an extra month of construction was assumed prior to levee construction to excavate two feet of Young Bay Mud and replace it with two feet of new fill.
7. Settlement has been estimated between the end of earthwork construction and the end of the assumed 50-year project design life. Settlement that occurs during construction is assumed to be compensated during earthwork placement to reach the target post-construction levee top elevation.
8. Soil properties obtained from Table 2.
9. Soil layers obtained from Figure 2.
10. Fill unit weight taken from ULE Guidance Document for Geotechnical Analyses (April 2015): Section 5.8.6 Table 5-5
11. Overbuild slopes steepened in accordance with Figure 1-2 of EM 1110-2-1913 (USACE, 2022) to achieve design long-term slopes of 2H:1V landside and 3H:1V waterside. Levee height, base width, and slope angles represent assumptions adopted in Settle3. Actual earthwork geometry may vary slightly. Refer to the project design drawings for reference.
12. Height of pile cap fill set to 3.5ft.
13. Settlement during and after construction.
14. For sections of the levee alignment outside of the existing levee footprint, total settlement was calculated after the top two feet of Young Bay Mud was excavated and replaced with new fill during the first month of construction.

ATTACHMENT A
Model Validation Results

STA 49+00

Levee Fill Height (ft)	6.8
Levee base Width (ft)	61.0
Levee Fill Additional Load (psf)	658.4
Groundwater Table Elevation (ft)	7.5
Total Primary Consolidation (in)	21.3

Soil Unit	Unit Weight (pcf)	OCR	C_{rc}	C_{rc}
New Fill	125			
Existing Fill	120			
YBM (outside of existing levee footprint)	115	1	0.2	0.03
YBM (below existing levee footprint)	115	2.0	0.2	0.03
OBC	120	2.2	0.2	0.02
Coarse-Grained Soil 1	125			
Coarse-Grained Soil 2	125			

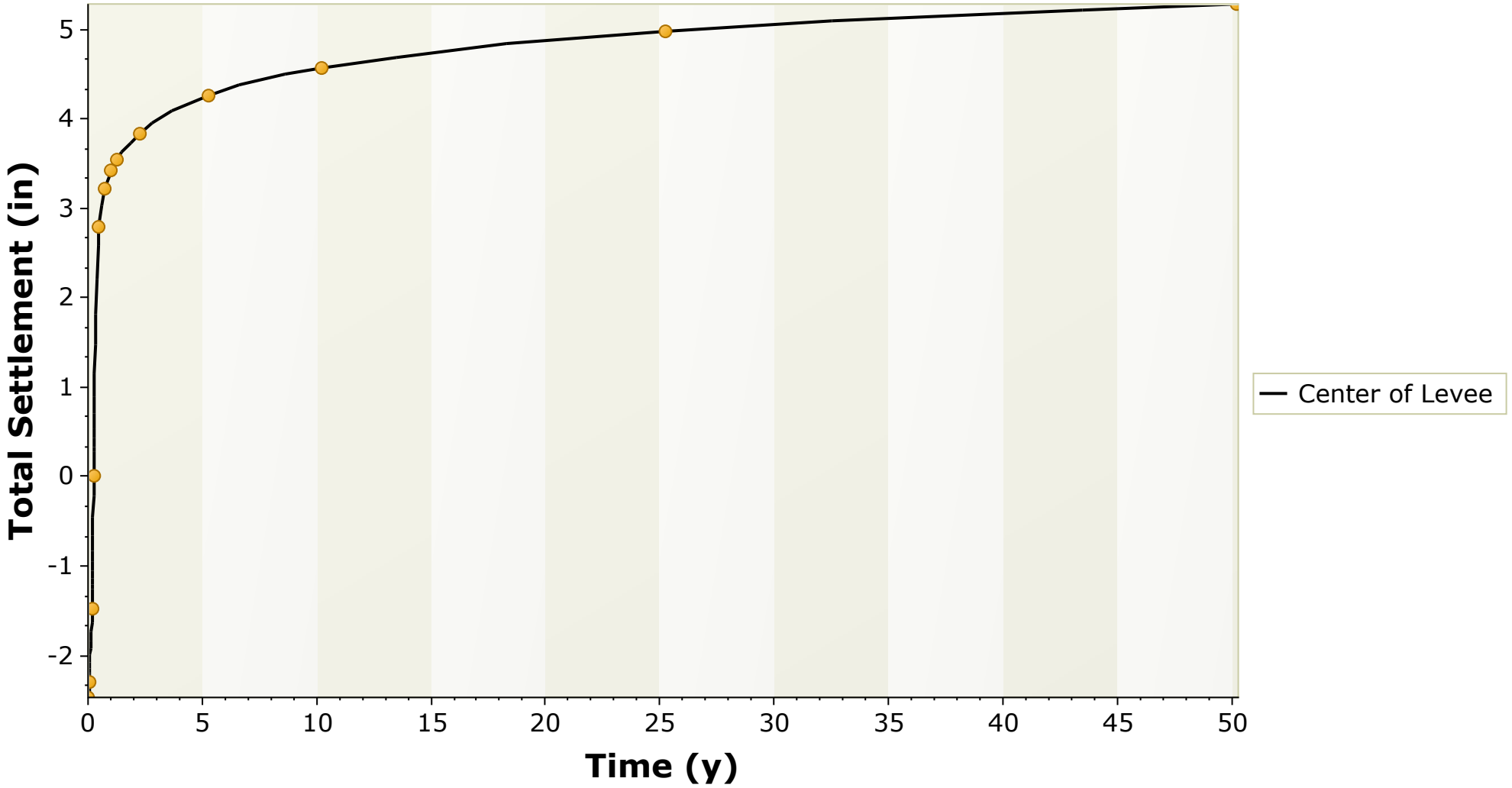
2:1 for continuous footing

Top Elevation (ft)	Bottom Elevation (ft)	Mid-Elevation (ft)	Soil Type	Layer Thickness (ft)	σ_0 (psf)	u (psf)	σ'_0 (psf)	OCR	P_p (psf)	Δq (psf)	σ'_f (psf)	C_{cc}	C_{rc}	C_{ac}	ΔH_{layer} (ft)
7.5	7.0	7.25	New Fill	0.5	62.5	15.6	46.9	0.0	0.0	655.7	702.6	0	0	0	0.0000
7.0	6.5	6.75	New Fill	0.5	125.0	46.8	78.2	0.0	0.0	650.4	728.6	0	0	0	0.0000
6.5	6.0	6.25	New Fill	0.5	187.5	78.0	109.5	0.0	0.0	645.2	754.7	0	0	0	0.0000
6.0	5.5	5.75	New Fill	0.5	250.0	109.2	140.8	0.0	0.0	640.0	780.8	0	0	0	0.0000
5.5	5.0	5.25	YBM (outside of existing levee footprint)	0.5	307.5	140.4	167.1	1.0	167.1	635.0	802.1	0.2	0.03	0.01	0.0681
5.0	4.5	4.75	YBM (outside of existing levee footprint)	0.5	365.0	171.6	193.4	1.0	193.4	630.0	823.4	0.2	0.03	0.01	0.0629
4.5	4.0	4.25	YBM (outside of existing levee footprint)	0.5	422.5	202.8	219.7	1.0	219.7	625.1	844.8	0.2	0.03	0.01	0.0585
4.0	3.5	3.75	YBM (outside of existing levee footprint)	0.5	480.0	234.0	246.0	1.0	246.0	620.3	866.3	0.2	0.03	0.01	0.0547
3.5	3.0	3.25	YBM (outside of existing levee footprint)	0.5	537.5	265.2	272.3	1.0	272.3	615.5	887.8	0.2	0.03	0.01	0.0513
3.0	2.5	2.75	YBM (outside of existing levee footprint)	0.5	595.0	296.4	298.6	1.0	298.6	610.8	909.4	0.2	0.03	0.01	0.0484
2.5	2.0	2.25	YBM (outside of existing levee footprint)	0.5	652.5	327.6	324.9	1.0	324.9	606.2	931.1	0.2	0.03	0.01	0.0457
2.0	1.5	1.75	YBM (outside of existing levee footprint)	0.5	710.0	358.8	351.2	1.0	351.2	601.7	952.9	0.2	0.03	0.01	0.0433
1.5	1.0	1.25	YBM (outside of existing levee footprint)	0.5	767.5	390.0	377.5	1.0	377.5	597.2	974.7	0.2	0.03	0.01	0.0412
1.0	0.5	0.75	YBM (outside of existing levee footprint)	0.5	825.0	421.2	403.8	1.0	403.8	592.8	996.6	0.2	0.03	0.01	0.0392
0.5	0.0	0.25	YBM (outside of existing levee footprint)	0.5	882.5	452.4	430.1	1.0	430.1	588.5	1018.6	0.2	0.03	0.01	0.0374
0.0	-0.5	-0.25	YBM (outside of existing levee footprint)	0.5	940.0	483.6	456.4	1.0	456.4	584.2	1040.6	0.2	0.03	0.01	0.0358
-0.5	-1.0	-0.75	YBM (outside of existing levee footprint)	0.5	997.5	514.8	482.7	1.0	482.7	580.0	1062.7	0.2	0.03	0.01	0.0343
-1.0	-1.5	-1.25	YBM (outside of existing levee footprint)	0.5	1055.0	546.0	509.0	1.0	509.0	575.8	1084.8	0.2	0.03	0.01	0.0329
-1.5	-2.0	-1.75	YBM (outside of existing levee footprint)	0.5	1112.5	577.2	535.3	1.0	535.3	571.7	1107.0	0.2	0.03	0.01	0.0316
-2.0	-2.5	-2.25	YBM (outside of existing levee footprint)	0.5	1170.0	608.4	561.6	1.0	561.6	567.7	1129.3	0.2	0.03	0.01	0.0303
-2.5	-3.0	-2.75	YBM (outside of existing levee footprint)	0.5	1227.5	639.6	587.9	1.0	587.9	563.7	1151.6	0.2	0.03	0.01	0.0292
-3.0	-3.5	-3.25	YBM (outside of existing levee footprint)	0.5	1285.0	670.8	614.2	1.0	614.2	559.8	1174.0	0.2	0.03	0.01	0.0281
-3.5	-4.0	-3.75	YBM (outside of existing levee footprint)	0.5	1342.5	702.0	640.5	1.0	640.5	555.9	1196.4	0.2	0.03	0.01	0.0271
-4.0	-4.5	-4.25	YBM (outside of existing levee footprint)	0.5	1400.0	733.2	666.8	1.0	666.8	552.1	1218.9	0.2	0.03	0.01	0.0262
-4.5	-5.0	-4.75	YBM (outside of existing levee footprint)	0.5	1457.5	764.4	693.1	1.0	693.1	548.3	1241.4	0.2	0.03	0.01	0.0253
-5.0	-5.5	-5.25	YBM (outside of existing levee footprint)	0.5	1515.0	795.6	719.4	1.0	719.4	544.6	1264.0	0.2	0.03	0.01	0.0245
-5.5	-6.0	-5.75	YBM (outside of existing levee footprint)	0.5	1572.5	826.8	745.7	1.0	745.7	540.9	1286.6	0.2	0.03	0.01	0.0237
-6.0	-6.5	-6.25	OBC	0.5	1632.5	858.0	774.5	2.2	1703.9	537.3	1311.8	0.2	0.02	0.01	0.0229
-6.5	-7.0	-6.75	OBC	0.5	1692.5	889.2	803.3	2.2	1767.3	533.7	1337.0	0.2	0.02	0.01	0.0221
-7.0	-7.5	-7.25	OBC	0.5	1752.5	920.4	832.1	2.2	1830.6	530.2	1362.3	0.2	0.02	0.01	0.0214
-7.5	-8.0	-7.75	OBC	0.5	1812.5	951.6	860.9	2.2	1894.0	526.7	1387.6	0.2	0.02	0.01	0.0207
-8.0	-8.5	-8.25	OBC	0.5	1872.5	982.8	889.7	2.2	1957.3	523.3	1413.0	0.2	0.02	0.01	0.0201
-8.5	-9.0	-8.75	OBC	0.5	1932.5	1014.0	918.5	2.2	2020.7	519.9	1438.4	0.2	0.02	0.01	0.0195
-9.0	-9.5	-9.25	OBC	0.5	1992.5	1045.2	947.3	2.2	2084.1	516.6	1463.9	0.2	0.02	0.01	0.0189
-9.5	-10.0	-9.75	OBC	0.5	2052.5	1076.4	976.1	2.2	2147.4	513.3	1489.4	0.2	0.02	0.01	0.0184
-10.0	-10.5	-10.25	OBC	0.5	2112.5	1107.6	1004.9	2.2	2210.8	510.0	1514.9	0.2	0.02	0.01	0.0178
-10.5	-11.0	-10.75	OBC	0.5	2172.5	1138.8	1033.7	2.2	2274.1	506.8	1540.5	0.2	0.02	0.01	0.0173
-11.0	-11.5	-11.25	OBC	0.5	2232.5	1170.0	1062.5	2.2	2337.5	503.6	1566.1	0.2	0.02	0.01	0.0168
-11.5	-12.0	-11.75	OBC	0.5	2292.5	1201.2	1091.3	2.2	2400.9	500.5	1591.8	0.2	0.02	0.01	0.0164
-12.0	-12.5	-12.25	OBC	0.5	2352.5	1232.4	1120.1	2.2	2464.2	497.4	1617.5	0.2	0.02	0.01	0.0160
-12.5	-13.0	-12.75	OBC	0.5	2412.5	1263.6	1148.9	2.2	2527.6	494.3	1643.2	0.2	0.02	0.01	0.0155
-13.0	-13.5	-13.25	OBC	0.5	2472.5	1294.8	1177.7	2.2	2590.9	491.3	1669.0	0.2	0.02	0.01	0.0151
-13.5	-14.0	-13.75	OBC	0.5	2532.5	1326.0	1206.5	2.2	2654.3	488.3	1694.8	0.2	0.02	0.01	0.0148
-14.0	-14.5	-14.25	OBC	0.5	2592.5	1357.2	1235.3	2.2	2717.7	485.3	1720.6	0.2	0.02	0.01	0.0144
-14.5	-15.0	-14.75	OBC	0.5	2652.5	1388.4	1264.1	2.2	2781.0	482.4	1746.5	0.2	0.02	0.01	0.0140
-15.0	-15.5	-15.25	OBC	0.5	2712.5	1419.6	1292.9	2.2	2844.4	479.6	1772.5	0.2	0.02	0.01	0.0137
-15.5	-16.0	-15.75	OBC	0.5	2772.5	1450.8	1321.7	2.2	2907.7	476.7	1798.4	0.2	0.02	0.01	0.0134
-16.0	-16.5	-16.25	OBC	0.5	2832.5	1482.0	1350.5	2.2	2971.1	473.9	1824.4	0.2	0.02	0.01	0.0131
-16.5	-17.0	-16.75	OBC	0.5	2892.5	1513.2	1379.3	2.2	3034.5	471.1	1850.4	0.2	0.02	0.01	0.0128
-17.0	-17.5	-17.25	OBC	0.5	2952.5	1544.4	1408.1	2.2	3097.8	468.4	1876.5	0.2	0.02	0.01	0.0125
-17.5	-18.0	-17.75	OBC	0.5	3012.5	1575.6	1436.9	2.2	3161.2	465.7	1902.6	0.2	0.02	0.01	0.0122
-18.0	-18.5	-18.25	OBC	0.5	3072.5	1606.8	1465.7	2.2	3224.5	463.0	1928.7	0.2	0.02	0.01	0.0119
-18.5	-19.0	-18.75	OBC	0.5	3132.5	1638.0	1494.5	2.2	3287.9	460.3	1954.8	0.2	0.02	0.01	0.0117
-19.0	-19.5	-19.25	OBC	0.5	3192.5	1669.2	1523.3	2.2	3351.3	457.7	1981.0	0.2	0.02	0.01	0.0114
-19.5	-20.0	-19.75	OBC	0.5	3252.5	1700.4	1552.1	2.2	3414.6	455.1	2007.2	0.2	0.02	0.01	0.0112
-20.0	-20.5	-20.25	OBC	0.5	3312.5	1731.6	1580.9	2.2	3478.0	452.5	2033.4	0.2	0.02	0.01	0.0109
-20.5	-21.0	-20.75	OBC	0.5	3372.5	1762.8	1609.7	2.2	3541.3	450.0	2059.7	0.2	0.02	0.01	0.0107
-21.0	-21.5	-21.25	OBC	0.5	3432.5	1794.0	1638.5	2.2	3604.7	447.5	2086.0	0.2	0.02	0.01	0.0105
-21.5	-22.0	-21.75	OBC	0.5	3492.5	1825.2	1667.3	2.2	3668.1	445.0	2112.3	0.2	0.02	0.01	0.0103
-22.0	-22.5	-22.25	OBC	0.5	3552.5	1856.4	1696.1	2.2	3731.4	442.6	2138.7	0.2	0.02	0.01	0.0101
-22.5	-23.0	-22.75	OBC	0.5	3612.5	1887.6	1724.9	2.2	3794.8	440.1	2165.0	0.2	0.02	0.01	0.0099
-23.0	-23.5	-23.25	OBC	0.5	3672.5	1918.8	1753.7	2.2	3858.1	437.7	2191.4	0.2	0.02	0.01	0.0097
-23.5	-24.0	-23.75	OBC	0.5	3732.5	1950.0	1782.5	2.2	3921.5	435.4	2217.9	0.2	0.02	0.01	0.0095
-24.0	-24.5	-24.25	OBC	0.5	3792.5	1981.2	1811.3	2.2	3984.9	433.0	2244.3	0.2	0.02	0.01	0.0093
-24.5	-25.0	-24.75	OBC	0.5	3852.5	2012.4	1840.1	2.2	4048.2	430.7	2270.8	0.2	0.02	0.01	0.0091
-25.0	-25.5	-25.25	OBC	0.5	3912.5	2043.6	1868.9	2.2	4111.6	428.4	2297.3	0.2	0.02	0.01	0.0090
-25.5	-26.0	-25.75	OBC	0.5	3972.5	2074.8	1897.7	2.2	4174.9	426.1	2323.8	0.2	0.02	0.01	0.0088
-26.0	-26.5	-26.25	OBC	0.5	4032.5	2106.0	1926.5	2.2	4238.3	423.9	2350.4	0.2	0.02	0.01	0.0086
-26.5	-27.0	-26.75	OBC	0.5	4092.5	2137.2	1955.3	2.2	4301.7	421.7	2377.0	0.2	0.02	0.01	0.0085
-27.0	-27.5	-27.25	OBC	0.5	4152.5	2168.4	1984.1	2.2	4365.0	419.5	2403.6	0.2	0.02	0.01	0.0083
-27.5	-28.0	-27.75	OBC	0.5	4212.5	2199.6	2012.9	2.2	4428.4	417.3	2430.2	0.2	0.02	0.01	0.0082
-28.0	-28.5	-28.25	OBC	0.5	4272.5	2230.8	2041.7	2.2	4491.7	415.1	2456.8	0.2	0.02	0.01	0.0080
-28.5	-29.0	-28.75	OBC	0.5	4332.5	2262.0	2070.5	2.2	4555.1	413.0	2483.5	0.2	0.02	0.01	0.0079
-29.0	-29.5	-29.25	OBC	0.5	4392.5	2293.2	2099.3	2.2	4618.5	410.9	2510.2	0.2	0.02	0.01	0.0078
-29.5	-30.0	-29.75	OBC	0.5	4452.5	2324.4	2128.1	2.2	4681.8	408.8	2536.9	0.2	0.02	0.01	0.0076
-30.0	-30.5	-30.25	OBC	0.5	4512.5	2355.6	2156.9	2.2	4745.2	406.7	2563.6	0.2	0.02	0.01	0.0075
-30.5	-31.0	-30.75	OBC	0.5	4572.5	2386.8	2185.7	2.2	4808.5	404.7	2590.4	0.2	0.02	0.01	0.0074
-31.0	-31.5	-31.25	OBC	0.5	4632.5	2418.0	2214.5	2.2	4871.9	402.6	2617.1	0.2	0.02	0.01	0.0073
-31.5	-32.0	-31.75	OBC	0.5	4692.5	2449.2	2243.3	2.2	4935.3	400.6	2643.9	0.2	0.02	0.01	0.0071
-32.0	-32.5	-32.25	OBC	0.5	4752.5	2480.4	2272.1	2.2	4998.6	398.6	2670.7	0.2	0.02	0.01	0.0070
-32.5	-33.0	-32.75	OBC	0.5	4812.5	2511.6	2300.9	2.2	5062.0	396.7	2697.6	0.2	0.02	0.01	0.0069
-33.0	-33.5	-33.25	OBC	0.5	4872.5	2542.8	2329.7	2.2	5125.3	394.7	2724.4	0.2	0.02	0.01	0.0068
-33.5	-34.0	-33.75	OBC	0.5	4932.5	2574.0	2358.5	2.2	5188.7	392.8	2751.3	0.2	0.02	0.01	0.0067
-34.0	-34.5	-34.25	OBC	0.5	4992.5	2605.2	2387.3	2.2	5252.1	390.9	2778.2	0.2	0.02	0.01	0.0066
-34.5	-35.0	-34.75	OBC	0.5	5052.5	2636.4	2416.1	2.2	5315.4	389.0	2805.1	0.2	0.02	0.01	0.0065
-35.0	-35.5	-35.25	OBC	0.5	5112.5	2667.6	2444.9	2.2	5378.8	387.1	2832.0	0.2	0.02	0.01	0.0064
-35.5	-36.0	-35.75	OBC	0.5	5172.5	2698.8	2473.7	2.2	5442.1	385.3	2859.0	0.2	0.02	0.01	0.0063
-36.0	-36.5	-36.25	Coarse-Grained Soil 1	0.5	5235.0	2730.0	2505.0	0.0	0.0	383.4	2888.4	0	0	0	0.0000
-36.5	-37.0	-36.75	Coarse-Grained Soil 1	0.5	5297.5	2761.2	2536.3	0.0	0.0	381.6	2917.9	0	0	0	0.0000
-37.0	-37.5	-37.25	Coarse-Grained Soil 1	0.5	5360.0	2792.4	2567.6	0.0	0.0	379.8	2947.4	0	0	0	0.0000
-37.5	-38.0	-37.75	Coarse-Grained Soil 1	0.5	5422.5	2823.6	2598.9	0.0	0.0	378.0	2976.9	0	0	0	0.0000
-38.0	-38.5	-38.25	Coarse-Grained Soil 1	0.5	5485.0	2854.8	2630.2	0.0	0.0	376.2	3006.4	0	0	0	0.0000
-38.5	-39.0	-38.75	Coarse-Grained Soil 1	0.5	5547.5	2886.0	2661.5	0.0	0.0	374.5	3036.0	0	0	0	0.0000
-39.0	-39.5	-39.25	Coarse-Grained Soil 1	0.5	5610.0	2917.2	2692.8	0.0	0.0	372.7	3065.5	0	0	0	0.0000
-39.5	-40.0	-39.75	Coarse-Grained Soil 1	0.5	5672.5	2948.4	2724.1	0.0	0.0	371.0	3095.1	0	0	0	0.0000
-40.0	-40.5	-40.25	Coarse-Grained Soil 1	0.5	5735.0	2979.6	2755.4	0.0	0.0	369.3	3124.7	0	0	0	0.0000
-40.5	-41.0	-40.75	Coarse-Grained Soil 1	0.5	5797.5	3010.8	2786.7	0.0	0.0	367.6	3154.3	0	0	0	0.0000
-41.0	-41.5	-41.25	OBC	0.5	5857.5	3042.0	2815.5	2.2	6194.1	365.9	3181.4	0.2	0.02	0.01	0.0053
-41.5	-42.0	-41.75	OBC	0.5	5917.5	3073.2	2844.3	2.2	6257.5	364.3	3208.6	0.2	0.02	0.01	0.0052
-42.0	-42.5	-42.25	OBC	0.5	5977.5	3104.4	2873.1	2.2	6320.8	362.6	3235.7	0.2	0.02	0.01	0.0052
-42.5	-43.0	-42.75	OBC	0.5	6037.5	3135.6	2901.9	2.2	6384.2	361.0	3262.9	0.2	0.02	0.01	0.0051
-43.0	-43.5	-43.25	OBC	0.5	6097.5	3166.8	2930.7	2.2	6447.5	359.4	3290.1	0.2	0.02	0.01	0.0050
-43.5	-44.0	-43.75	OBC	0.5	6157.5	3198.0	2959.5	2.2	6510.9	357.8	3317.3	0.2	0.02	0.01	0.0050
-44.0	-44.5	-44.25	OBC	0.5	6217.5	3229.2	2988.3	2.2	6574.3	356.2	3344.5	0.2	0.02	0.01	0.0049
-44.5	-45.0	-44.75	OBC	0.5	6277.5	3260.4	3017.1	2.2	6637.6	354.6	3371.7	0.2	0.02	0.01	0.0048

-45.0	-45.5	-45.25	OBC	0.5	6337.5	3291.6	3045.9	2.2	6701.0	353.1	3399.0	0.2	0.02	0.01	0.0048
-45.5	-46.0	-45.75	OBC	0.5	6397.5	3322.8	3074.7	2.2	6764.3	351.5	3426.2	0.2	0.02	0.01	0.0047
-46.0	-46.5	-46.25	OBC	0.5	6457.5	3354.0	3103.5	2.2	6827.7	350.0	3453.5	0.2	0.02	0.01	0.0046
-46.5	-47.0	-46.75	OBC	0.5	6517.5	3385.2	3132.3	2.2	6891.1	348.5	3480.8	0.2	0.02	0.01	0.0046
-47.0	-47.5	-47.25	OBC	0.5	6577.5	3416.4	3161.1	2.2	6954.4	347.0	3508.1	0.2	0.02	0.01	0.0045
-47.5	-48.0	-47.75	OBC	0.5	6637.5	3447.6	3189.9	2.2	7017.8	345.5	3535.4	0.2	0.02	0.01	0.0045
-48.0	-48.5	-48.25	OBC	0.5	6697.5	3478.8	3218.7	2.2	7081.1	344.0	3562.7	0.2	0.02	0.01	0.0044
-48.5	-49.0	-48.75	OBC	0.5	6757.5	3510.0	3247.5	2.2	7144.5	342.5	3590.0	0.2	0.02	0.01	0.0044
-49.0	-49.5	-49.25	OBC	0.5	6817.5	3541.2	3276.3	2.2	7207.9	341.1	3617.4	0.2	0.02	0.01	0.0043
-49.5	-50.0	-49.75	OBC	0.5	6877.5	3572.4	3305.1	2.2	7271.2	339.6	3644.7	0.2	0.02	0.01	0.0042
-50.0	-50.5	-50.25	OBC	0.5	6937.5	3603.6	3333.9	2.2	7334.6	338.2	3672.1	0.2	0.02	0.01	0.0042
-50.5	-51.0	-50.75	OBC	0.5	6997.5	3634.8	3362.7	2.2	7397.9	336.8	3699.5	0.2	0.02	0.01	0.0041
-51.0	-51.5	-51.25	OBC	0.5	7057.5	3666.0	3391.5	2.2	7461.3	335.4	3726.9	0.2	0.02	0.01	0.0041
-51.5	-52.0	-51.75	OBC	0.5	7117.5	3697.2	3420.3	2.2	7524.7	334.0	3754.3	0.2	0.02	0.01	0.0040
-52.0	-52.5	-52.25	OBC	0.5	7177.5	3728.4	3449.1	2.2	7588.0	332.6	3781.7	0.2	0.02	0.01	0.0040
-52.5	-53.0	-52.75	OBC	0.5	7237.5	3759.6	3477.9	2.2	7651.4	331.2	3809.1	0.2	0.02	0.01	0.0040
-53.0	-53.5	-53.25	OBC	0.5	7297.5	3790.8	3506.7	2.2	7714.7	329.9	3836.6	0.2	0.02	0.01	0.0039
-53.5	-54.0	-53.75	OBC	0.5	7357.5	3822.0	3535.5	2.2	7778.1	328.5	3864.0	0.2	0.02	0.01	0.0039
-54.0	-54.5	-54.25	OBC	0.5	7417.5	3853.2	3564.3	2.2	7841.5	327.2	3891.5	0.2	0.02	0.01	0.0038
-54.5	-55.0	-54.75	OBC	0.5	7477.5	3884.4	3593.1	2.2	7904.8	325.9	3919.0	0.2	0.02	0.01	0.0038
-55.0	-55.5	-55.25	OBC	0.5	7537.5	3915.6	3621.9	2.2	7968.2	324.5	3946.4	0.2	0.02	0.01	0.0037
-55.5	-56.0	-55.75	OBC	0.5	7597.5	3946.8	3650.7	2.2	8031.5	323.2	3973.9	0.2	0.02	0.01	0.0037
-56.0	-56.5	-56.25	OBC	0.5	7657.5	3978.0	3679.5	2.2	8094.9	321.9	4001.4	0.2	0.02	0.01	0.0036
-56.5	-57.0	-56.75	OBC	0.5	7717.5	4009.2	3708.3	2.2	8158.3	320.7	4029.0	0.2	0.02	0.01	0.0036
-57.0	-57.5	-57.25	OBC	0.5	7777.5	4040.4	3737.1	2.2	8221.6	319.4	4056.5	0.2	0.02	0.01	0.0036
-57.5	-58.0	-57.75	OBC	0.5	7837.5	4071.6	3765.9	2.2	8285.0	318.1	4084.0	0.2	0.02	0.01	0.0035
-58.0	-58.5	-58.25	OBC	0.5	7897.5	4102.8	3794.7	2.2	8348.3	316.9	4111.6	0.2	0.02	0.01	0.0035
-58.5	-59.0	-58.75	OBC	0.5	7957.5	4134.0	3823.5	2.2	8411.7	315.6	4139.1	0.2	0.02	0.01	0.0034
-59.0	-59.5	-59.25	OBC	0.5	8017.5	4165.2	3852.3	2.2	8475.1	314.4	4166.7	0.2	0.02	0.01	0.0034
-59.5	-60.0	-59.75	OBC	0.5	8077.5	4196.4	3881.1	2.2	8538.4	313.2	4194.3	0.2	0.02	0.01	0.0034
-60.0	-60.5	-60.25	OBC	0.5	8137.5	4227.6	3909.9	2.2	8601.8	311.9	4221.8	0.2	0.02	0.01	0.0033
-60.5	-61.0	-60.75	OBC	0.5	8197.5	4258.8	3938.7	2.2	8665.1	310.7	4249.4	0.2	0.02	0.01	0.0033
-61.0	-61.5	-61.25	Coarse-Grained Soil 1	0.5	8260.0	4290.0	3970.0	0.0	0.0	309.5	4279.5	0	0	0	0.0000
-61.5	-62.0	-61.75	Coarse-Grained Soil 1	0.5	8322.5	4321.2	4001.3	0.0	0.0	308.3	4309.6	0	0	0	0.0000
-62.0	-62.5	-62.25	Coarse-Grained Soil 1	0.5	8385.0	4352.4	4032.6	0.0	0.0	307.2	4339.8	0	0	0	0.0000
-62.5	-63.0	-62.75	Coarse-Grained Soil 1	0.5	8447.5	4383.6	4063.9	0.0	0.0	306.0	4369.9	0	0	0	0.0000
-63.0	-63.5	-63.25	Coarse-Grained Soil 1	0.5	8510.0	4414.8	4095.2	0.0	0.0	304.8	4400.0	0	0	0	0.0000
-63.5	-64.0	-63.75	Coarse-Grained Soil 1	0.5	8572.5	4446.0	4126.5	0.0	0.0	303.7	4430.2	0	0	0	0.0000
-64.0	-64.5	-64.25	Coarse-Grained Soil 1	0.5	8635.0	4477.2	4157.8	0.0	0.0	302.5	4460.3	0	0	0	0.0000

ATTACHMENT B
Settle3 Outputs (Post Earthwork Construction)

Time vs. Total Settlement

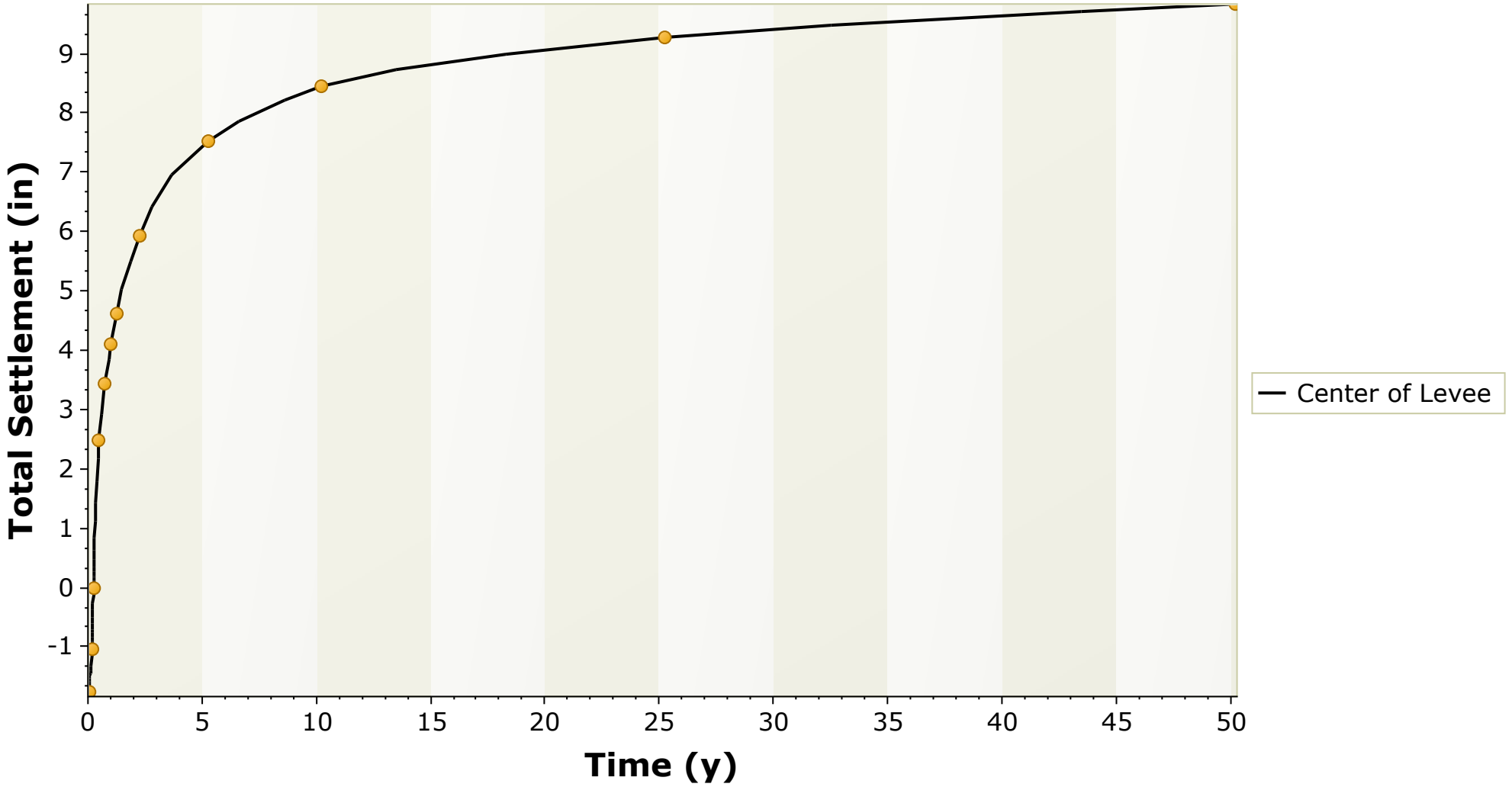


Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y
Total Settlement at Elevation = 5.4 ft

ROCSCIENCE	*Project*	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis of STA 7+00	
	Drawn By	Armaan Khamenian	*Company* Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	*File Name* SAFER (South of Bay Road) - STA 7+00 (Overbuild).s3z

SETTLE3 5.027

Time vs. Total Settlement

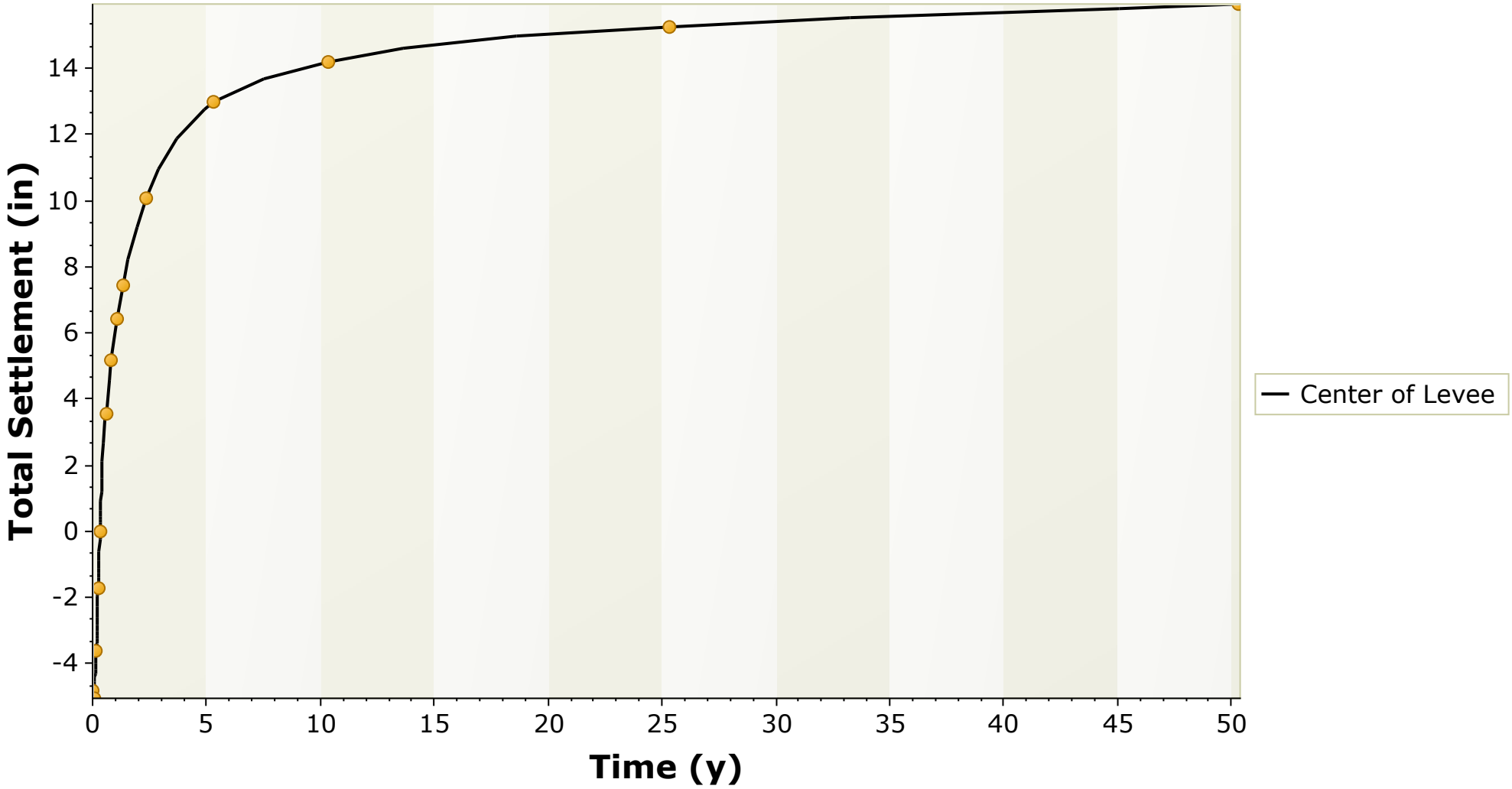


Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y
Total Settlement at Elevation = 6.5 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis of STA 15+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 15+00 (Overbuild).s3z

SETTLE3 5.027

Time vs. Total Settlement

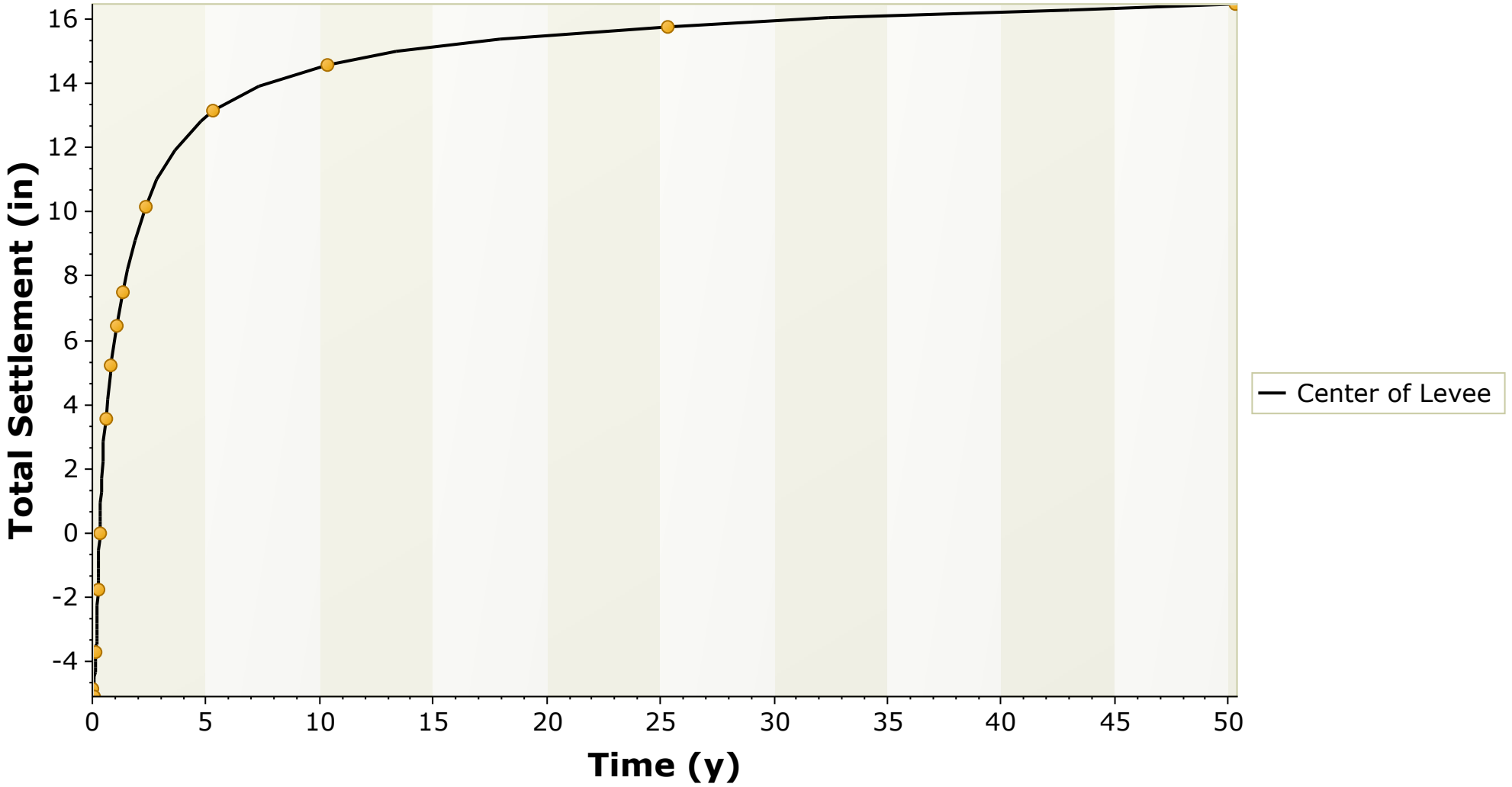


Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y
Total Settlement at Elevation = 5.8 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 28+50	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 28+50 (Overbuild).s3z

SETTLE3 5.027

Time vs. Total Settlement

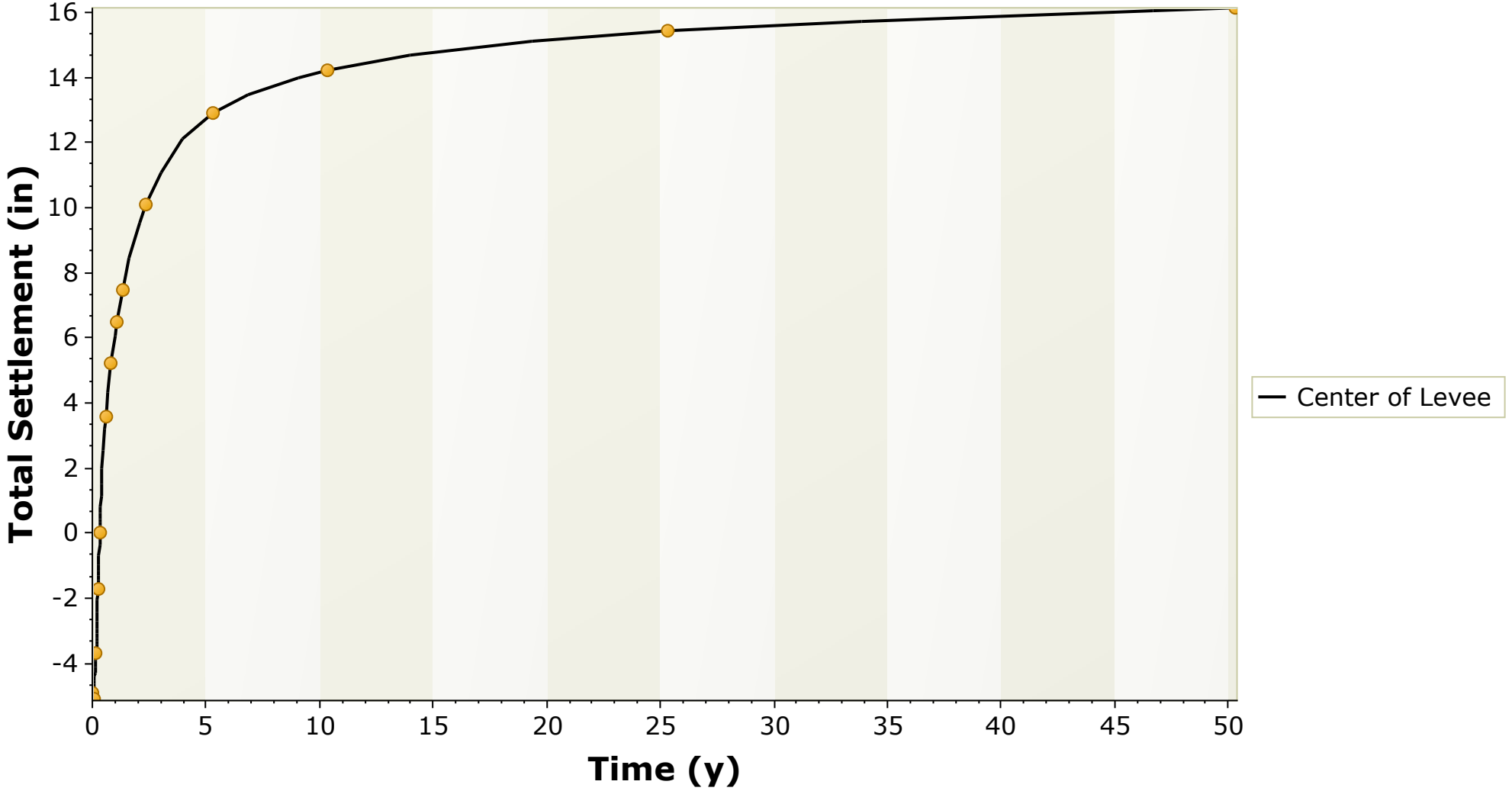


Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y
Total Settlement at Elevation = 5.5 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 42+50	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 42+50 (Overbuild).s3z

SETTLE3 5.027

Time vs. Total Settlement

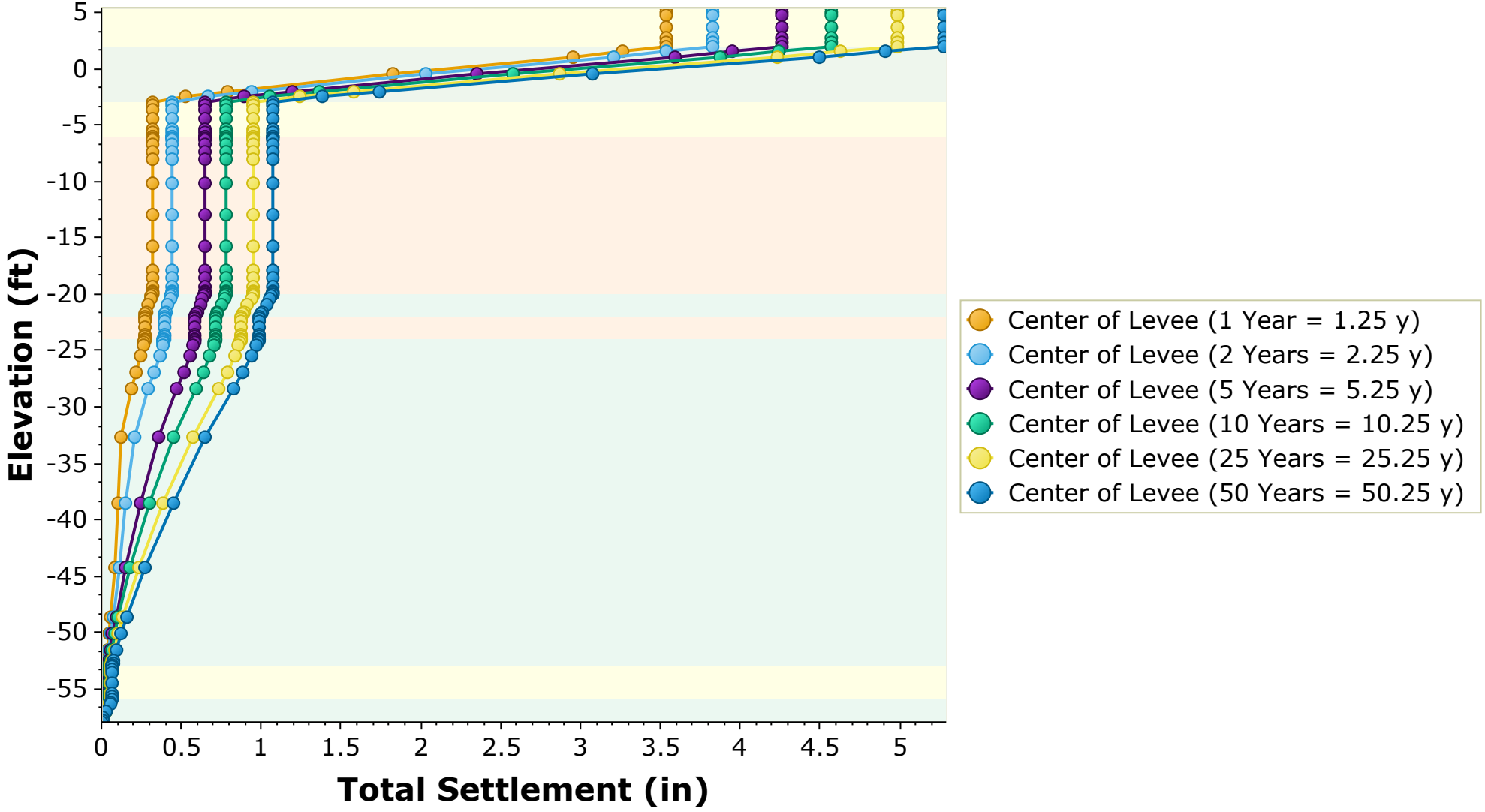


Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y
Total Settlement at Elevation = 5.5 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 49+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 49+00 (Overbuild).s3z

SETTLE3 5.027

Total Settlement vs. Elevation

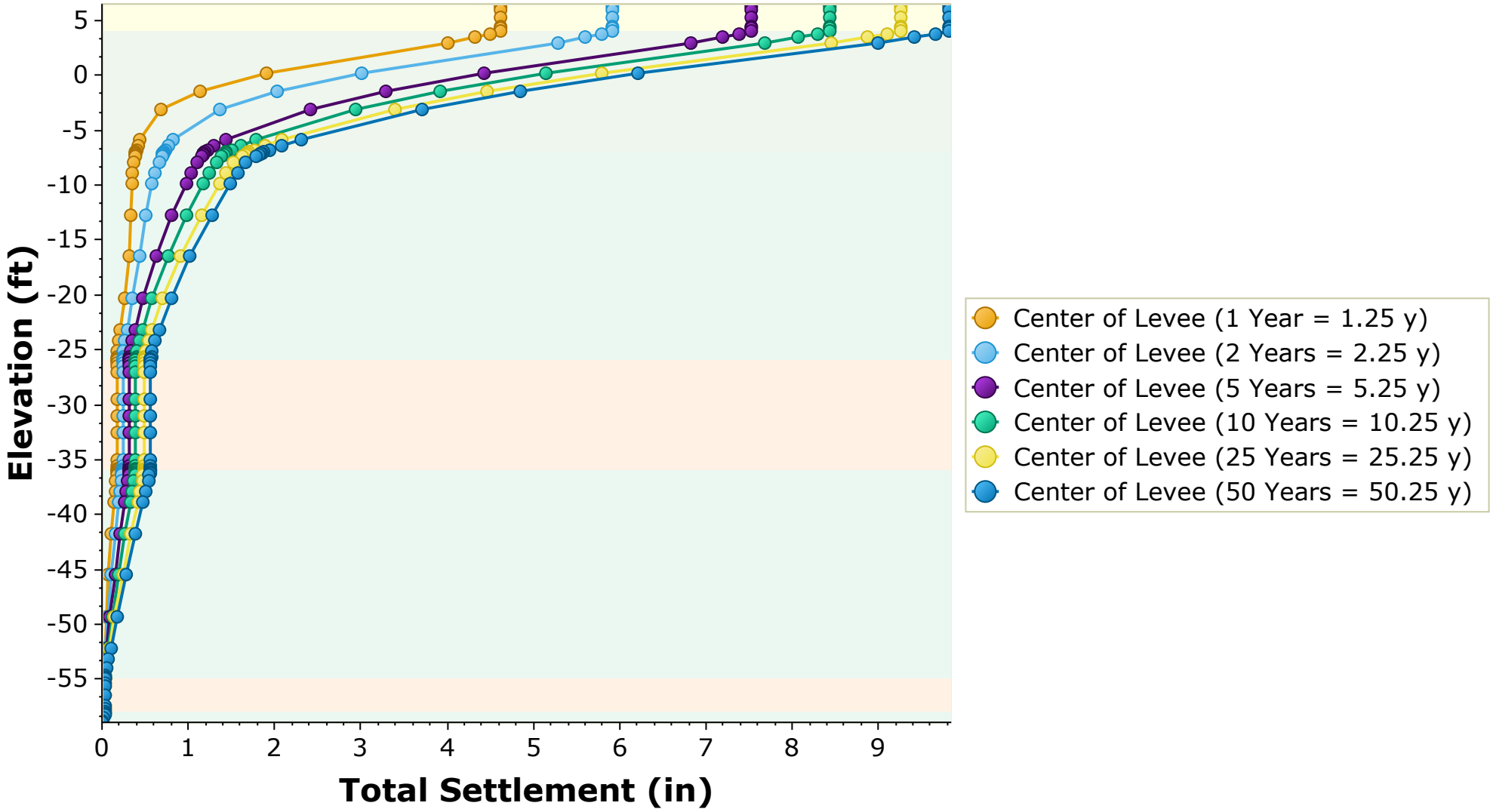


Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y

ROCSCIENCE	Project: SAFER (South of Bay Road)	
	Analysis Description: Settlement Analysis of STA 7+00	
	Drawn By: Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date: 5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 7+00 (Overbuild).s3z

SETTLE3 5.027

Total Settlement vs. Elevation



Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y

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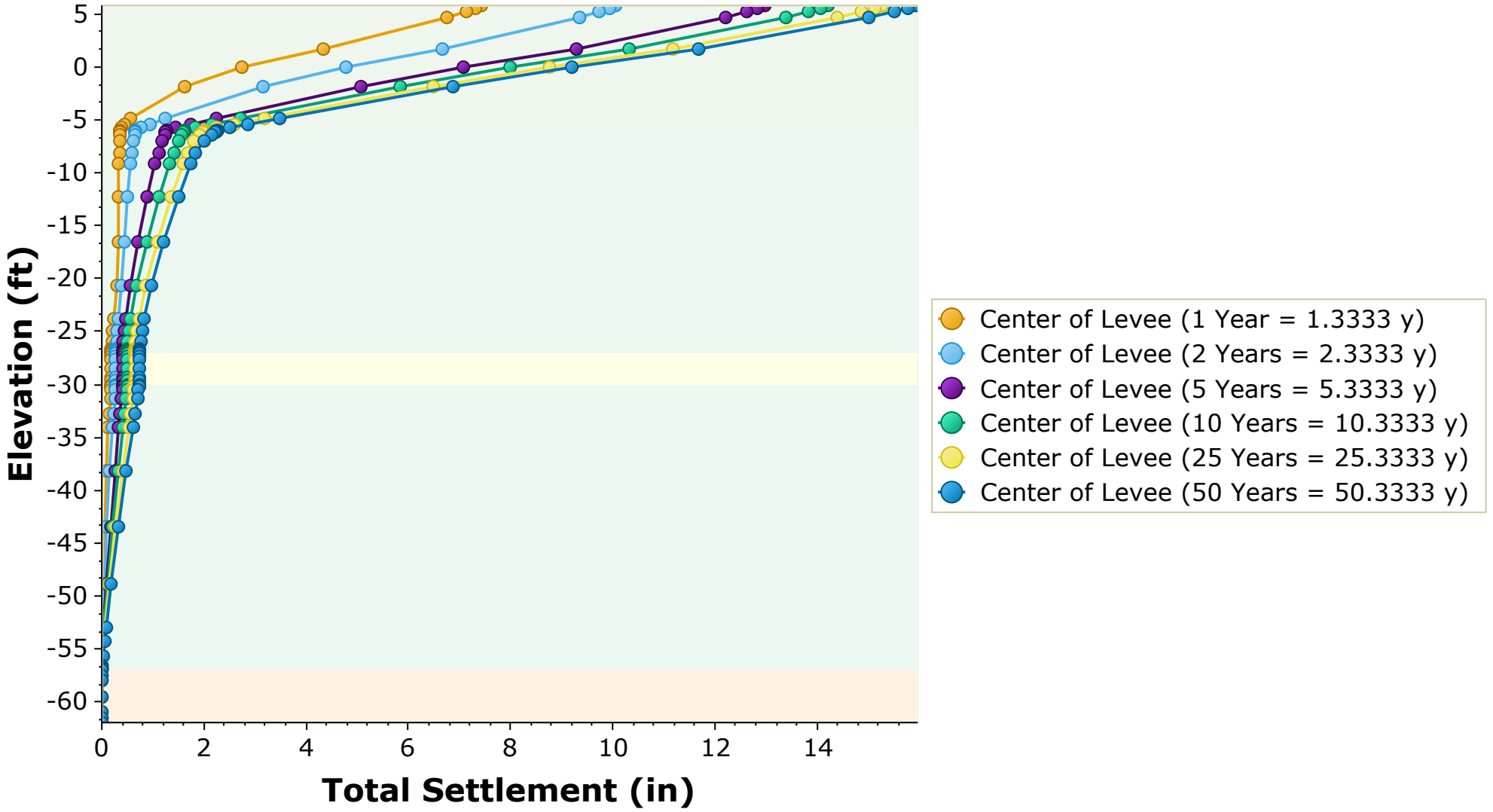
	Analysis Description Settlement Analysis of STA 15+00	
	Drawn By Armaan Khamenian	*Company* Geosyntec Consultants, Inc.
SETTLE3 5.027	*Date* 5/6/2026, 2:09:25 PM	*File Name* SAFER (South of Bay Road) - STA 15+00 (Overbuild).s3z

Total Settlement vs. Elevation



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ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 28+50	
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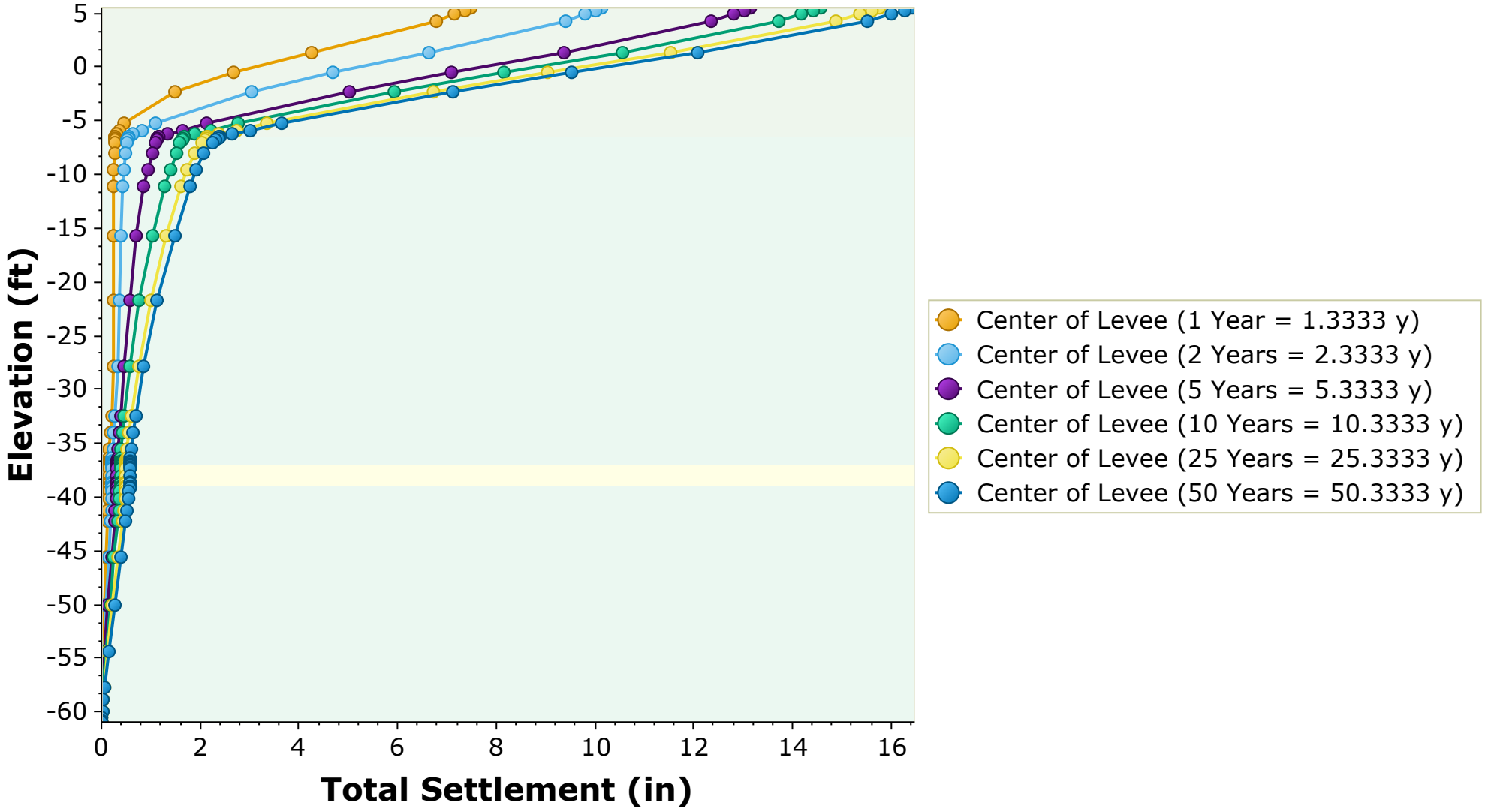
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 28+50 (Overbuild).s3z

SETTLE3 5.027

Total Settlement vs. Elevation

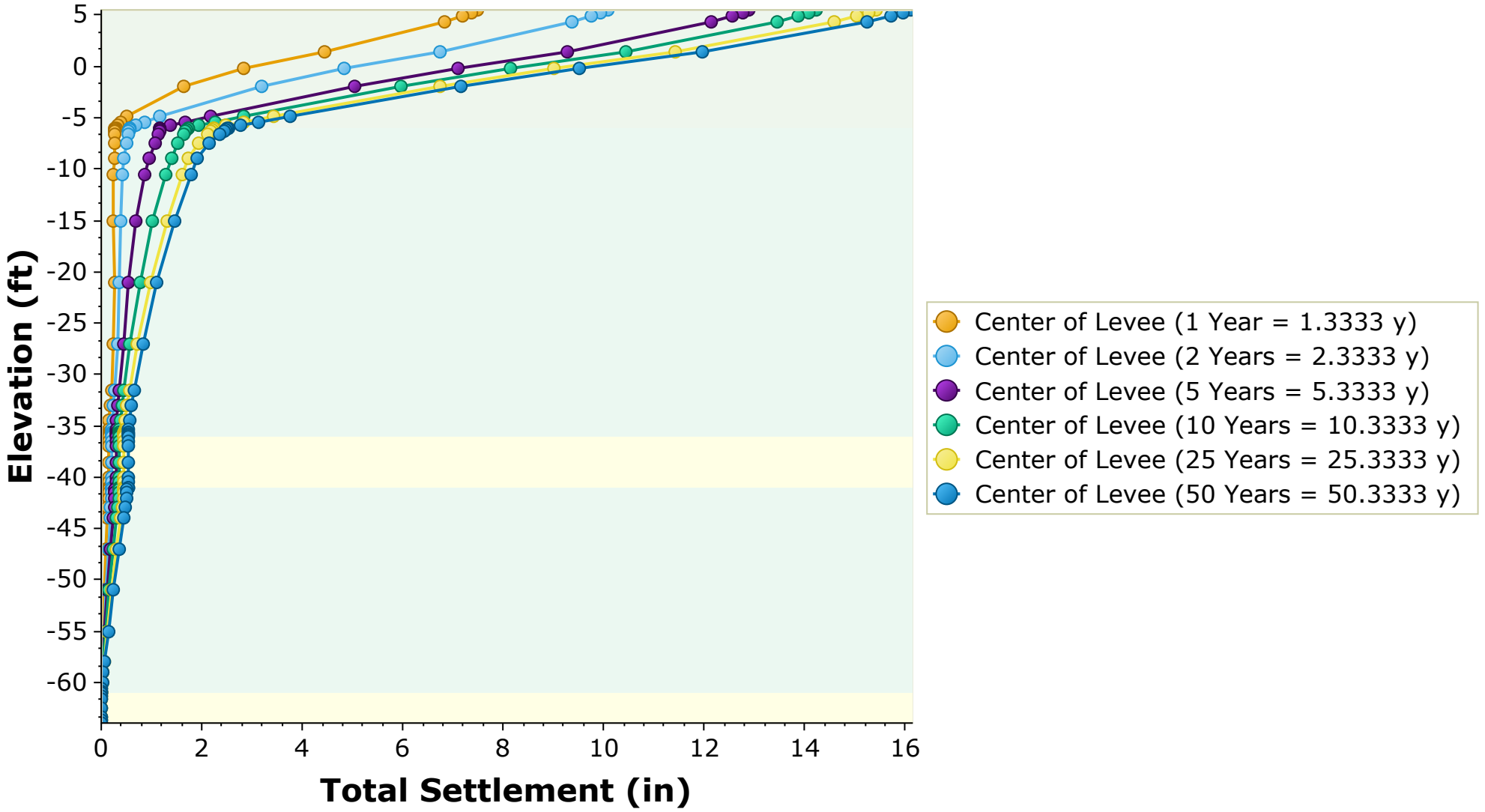


Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 42+50	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 42+50 (Overbuild).s3z

SETTLE3 5.027

Total Settlement vs. Elevation

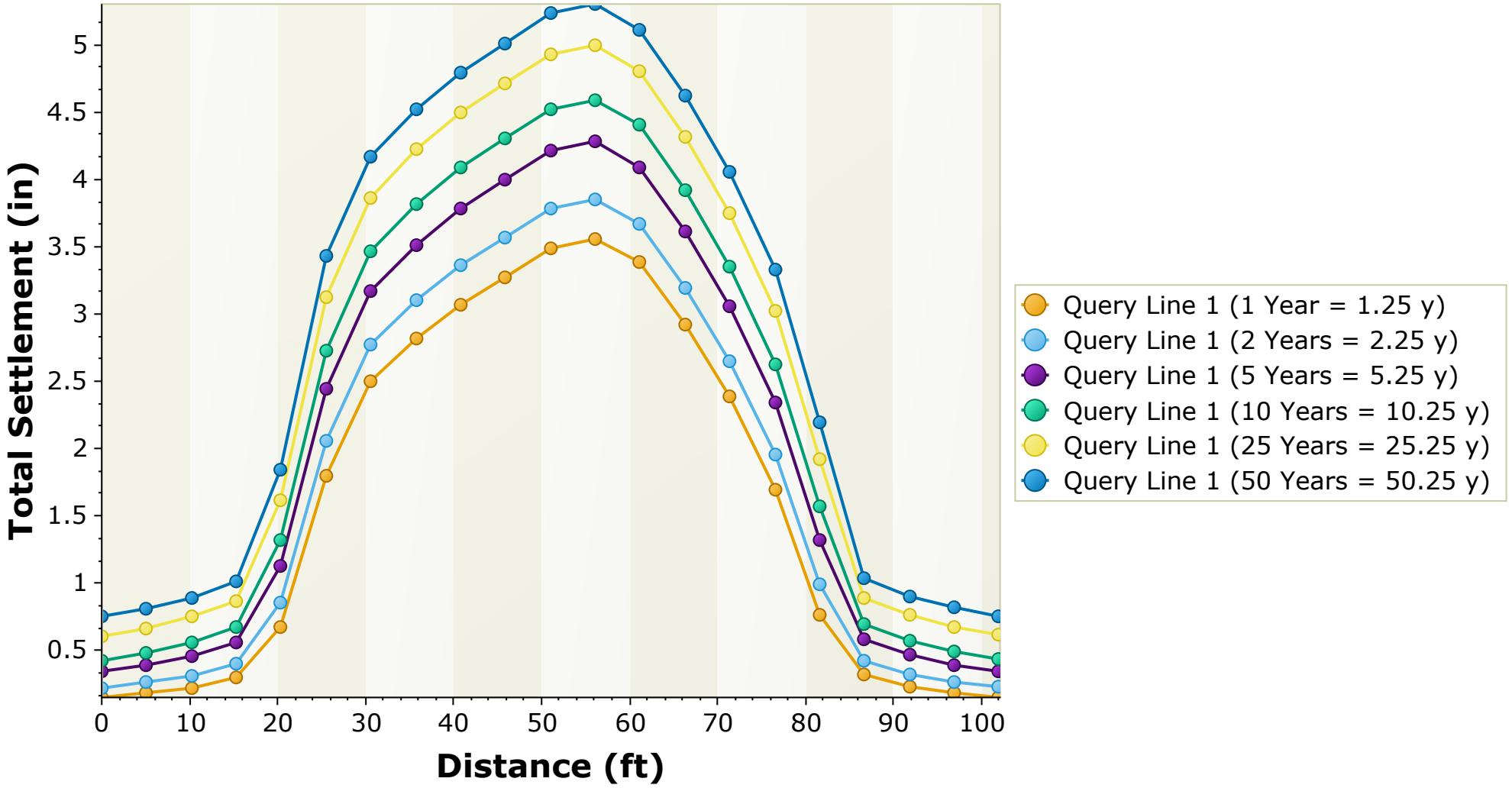


Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 49+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 49+00 (Overbuild).s3z

SETTLE3 5.027

Distance vs. Total Settlement

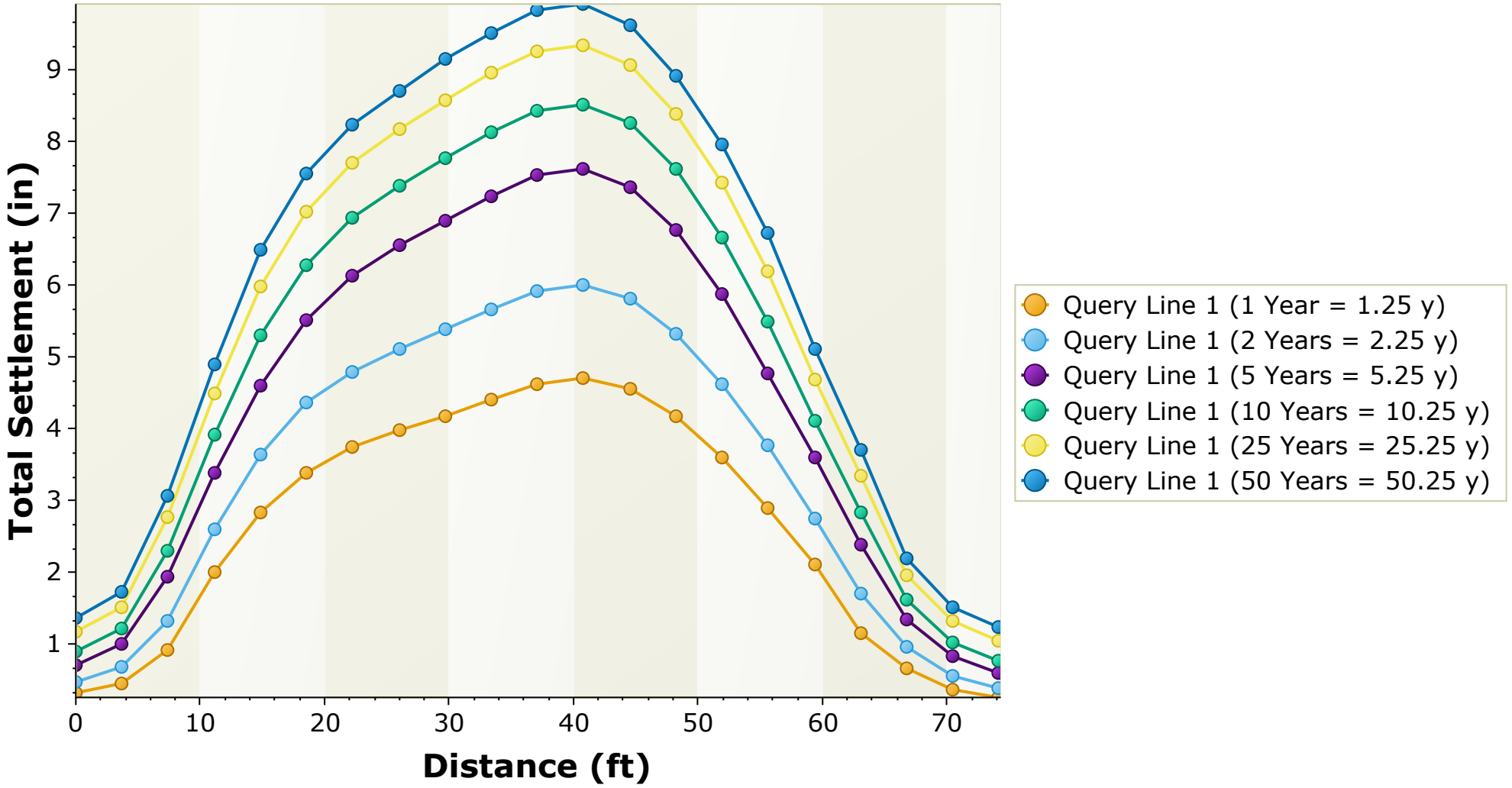


Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y
Total Settlement at Elevation = 5.4 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis of STA 7+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 7+00 (Overbuild).s3z

SETTLE3 5.027

Distance vs. Total Settlement

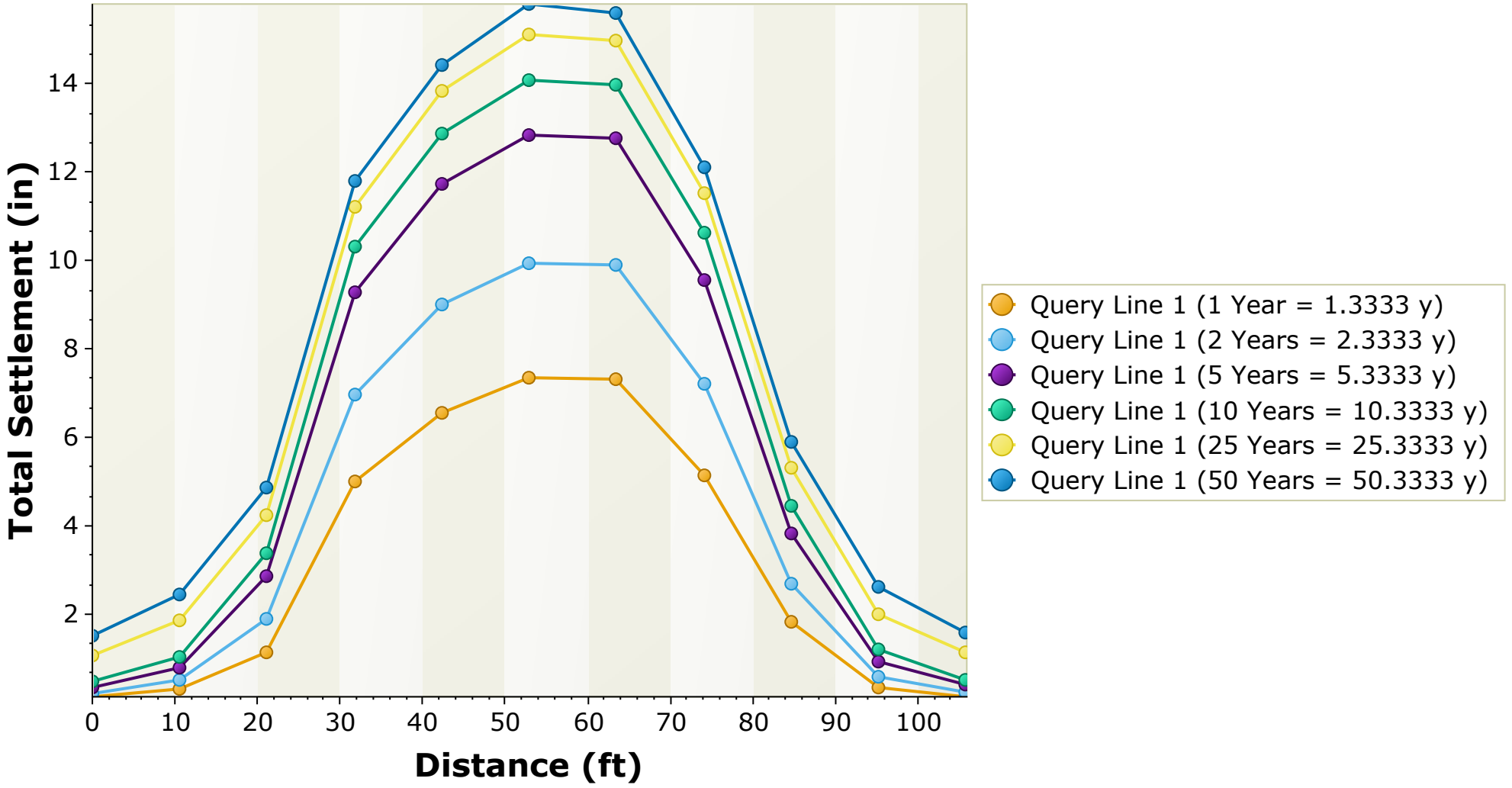


Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y
Total Settlement at Elevation = 6.5 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis of STA 15+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 15+00 (Overbuild).s3z

SETTLE3 5.027

Distance vs. Total Settlement

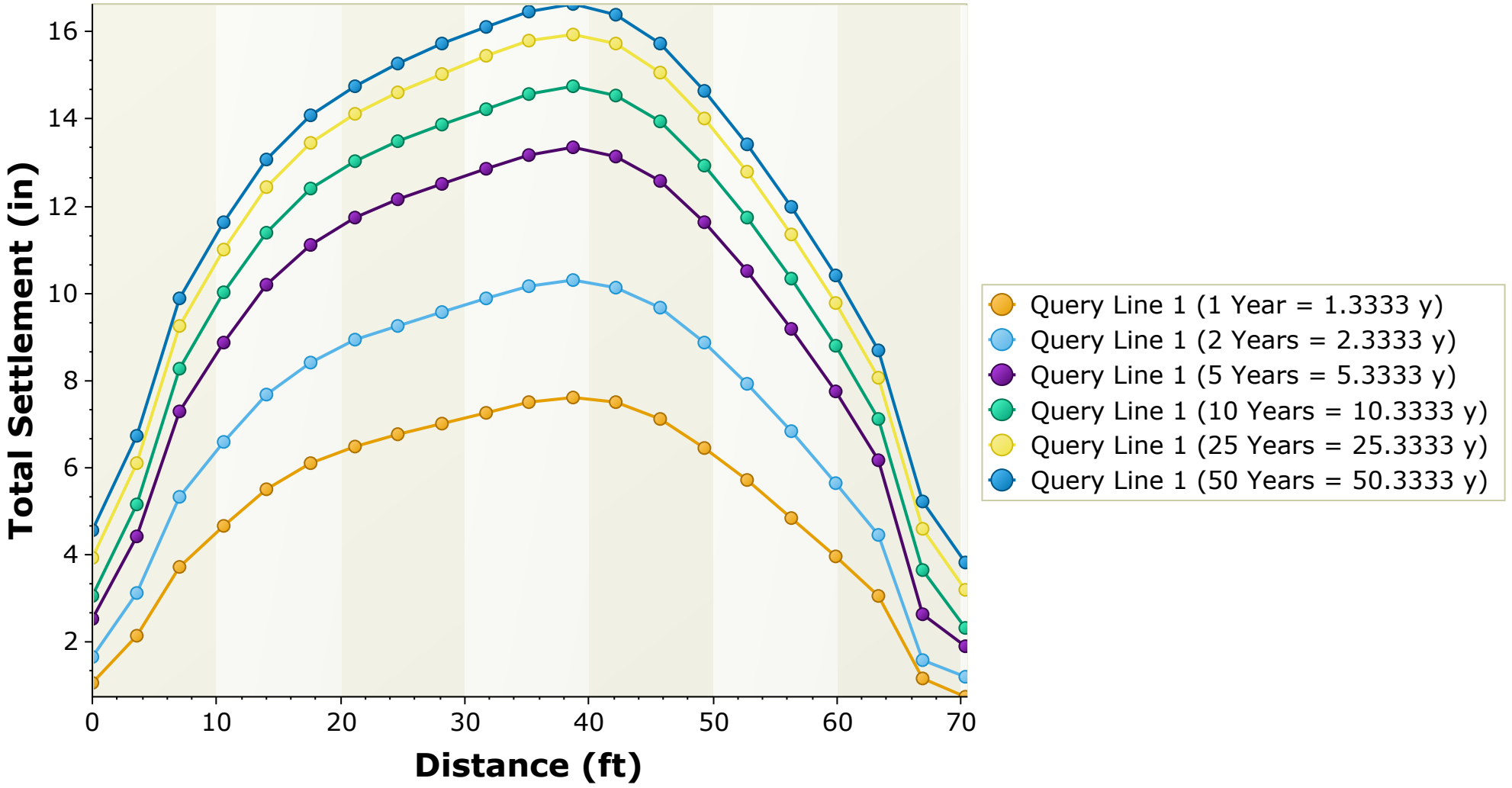


Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y
Total Settlement at Elevation = 5.8 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 28+50	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 28+50 (Overbuild).s3z

SETTLE3 5.027

Distance vs. Total Settlement

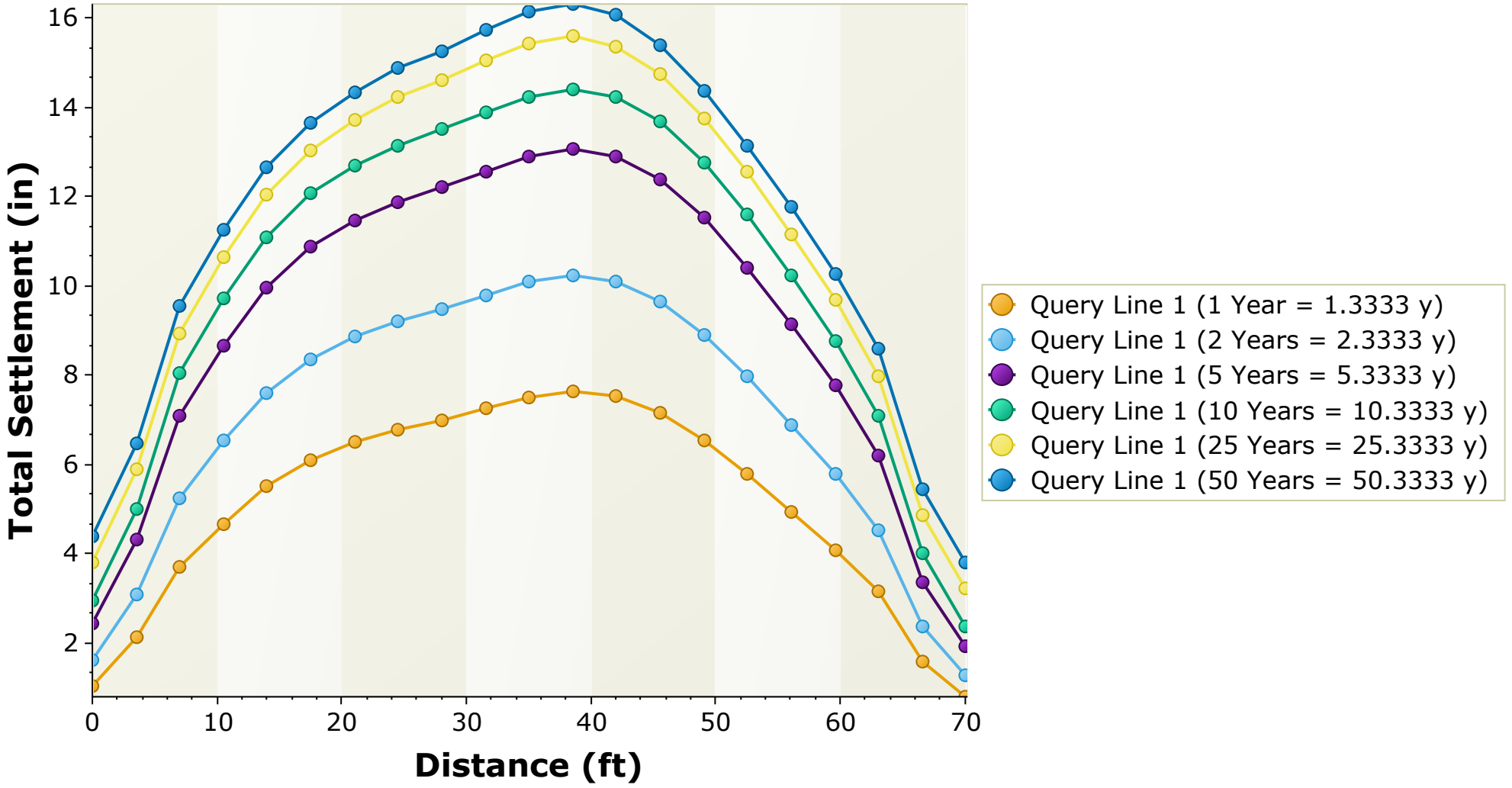


Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y
Total Settlement at Elevation = 5.5 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 42+50	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 42+50 (Overbuild).s3z

SETTLE3 5.027

Distance vs. Total Settlement



Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y
Total Settlement at Elevation = 5.5 ft

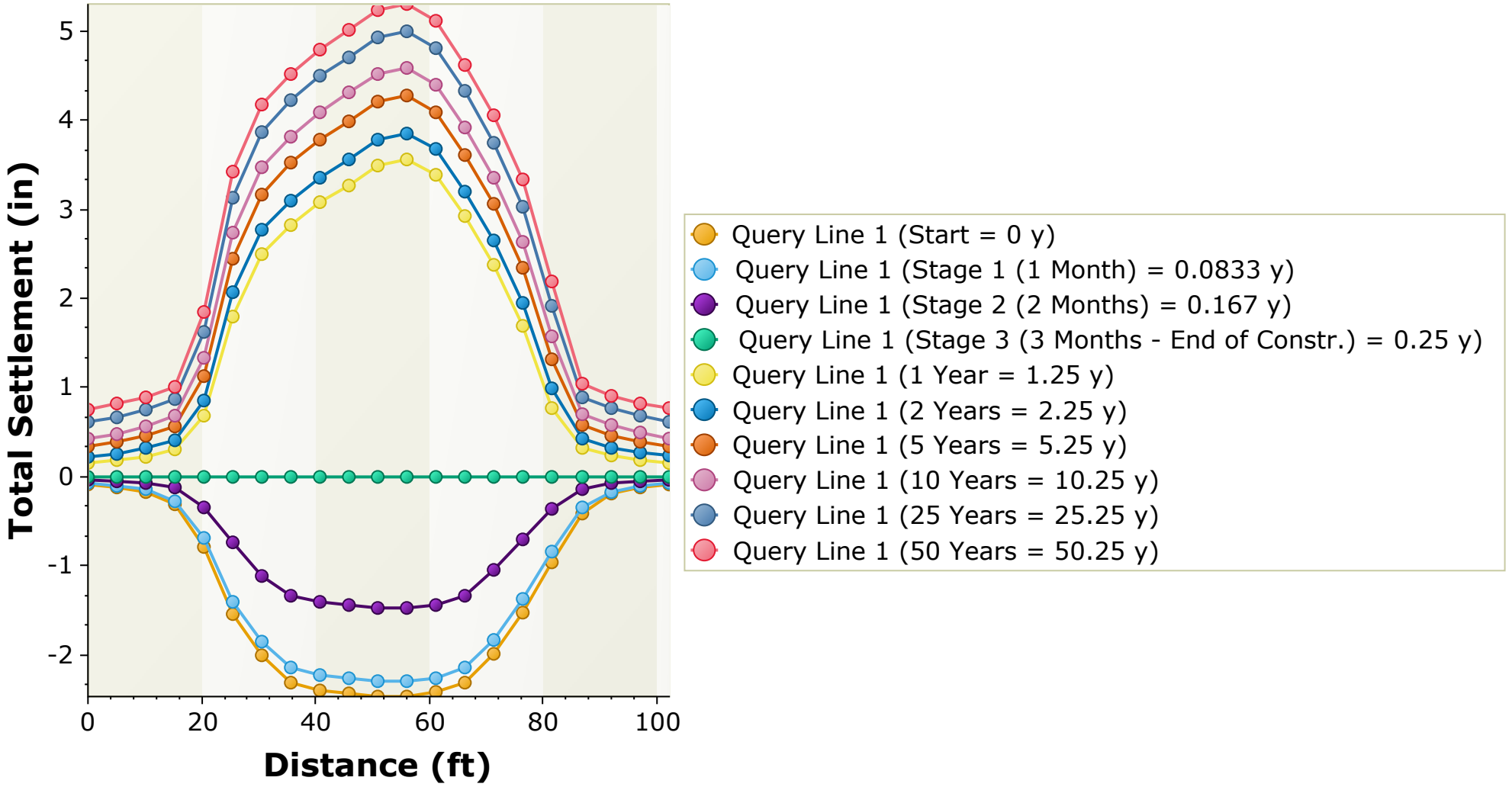
ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 49+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 49+00 (Overbuild).s3z

SETTLE3 5.027

ATTACHMENT C

Settle3 Outputs (Pre and Post Earthwork Construction)

Distance vs. Total Settlement

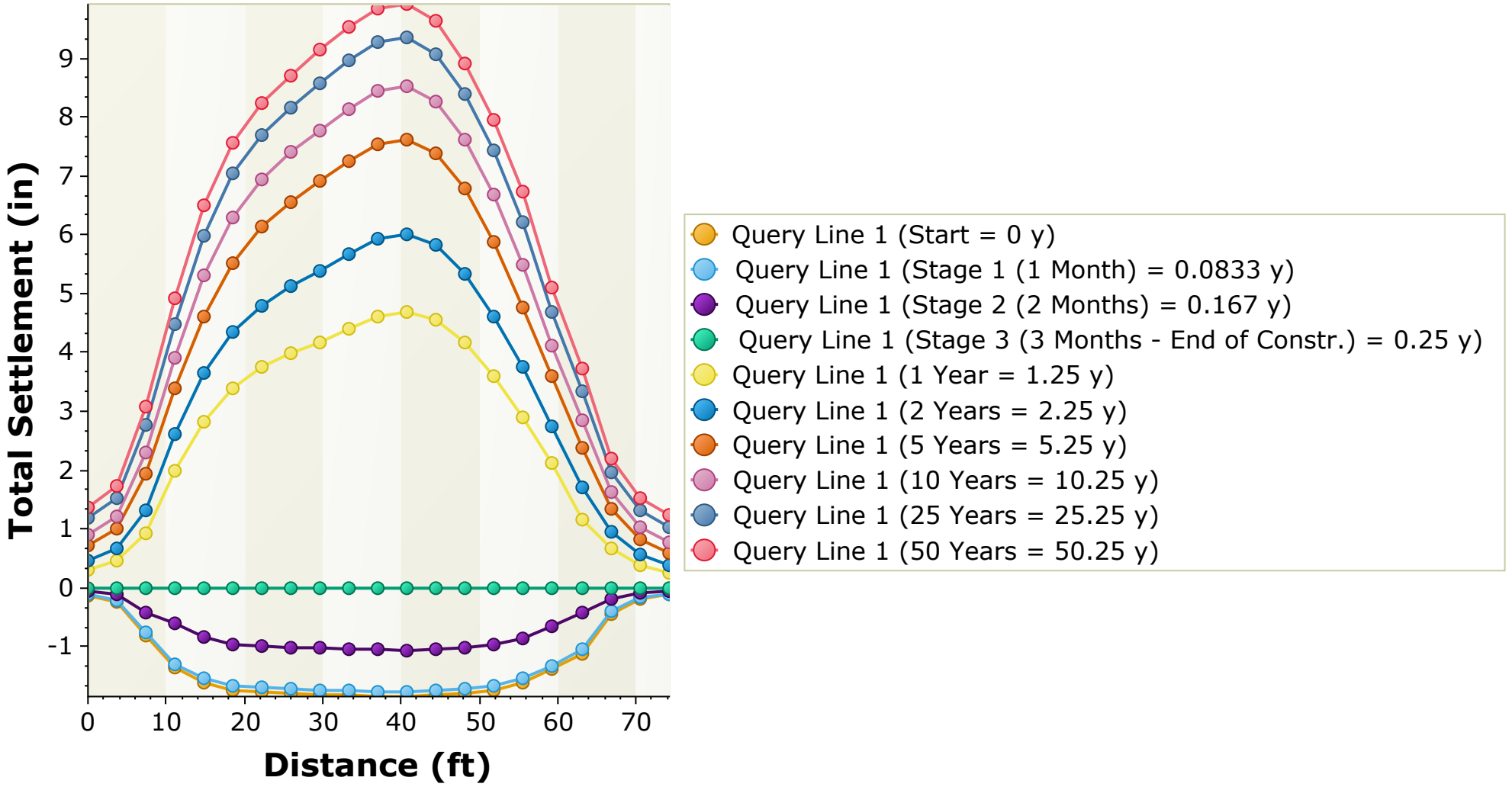


Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y
Total Settlement at Elevation = 5.4 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis of STA 7+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 7+00 (Overbuild).s3z

SETTLE3 5.027

Distance vs. Total Settlement

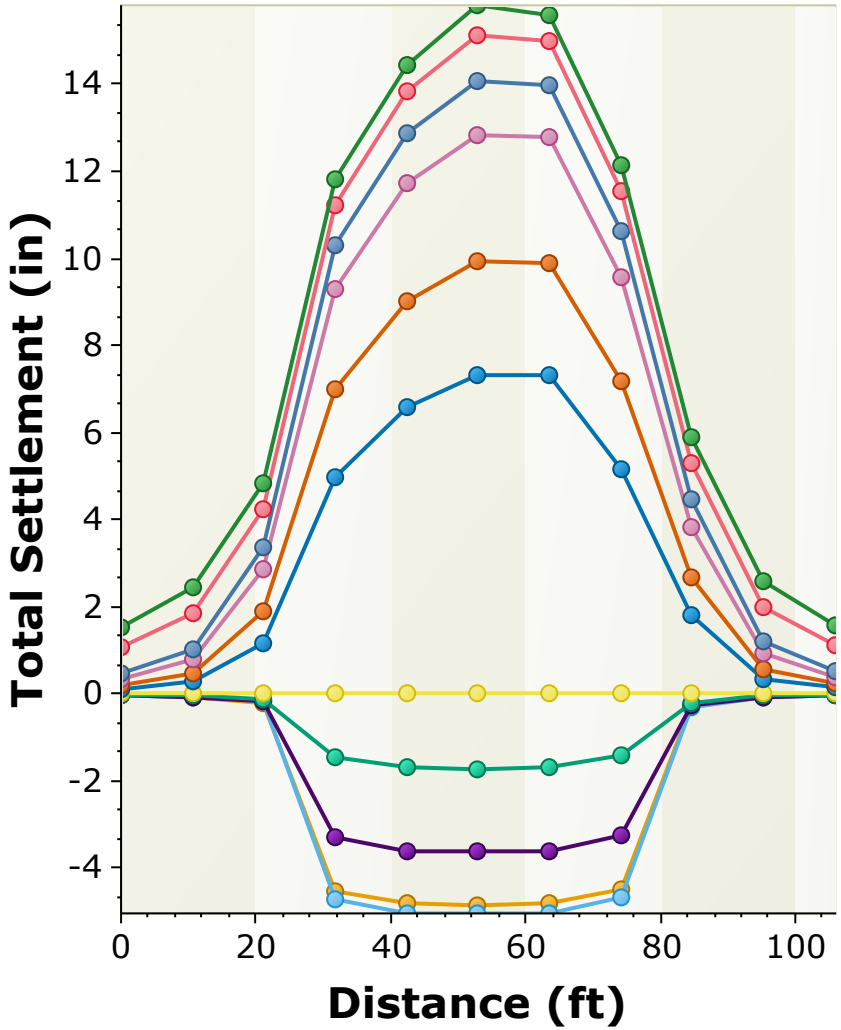


Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y
Total Settlement at Elevation = 6.5 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis of STA 15+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 15+00 (Overbuild).s3z

SETTLE3 5.027

Distance vs. Total Settlement



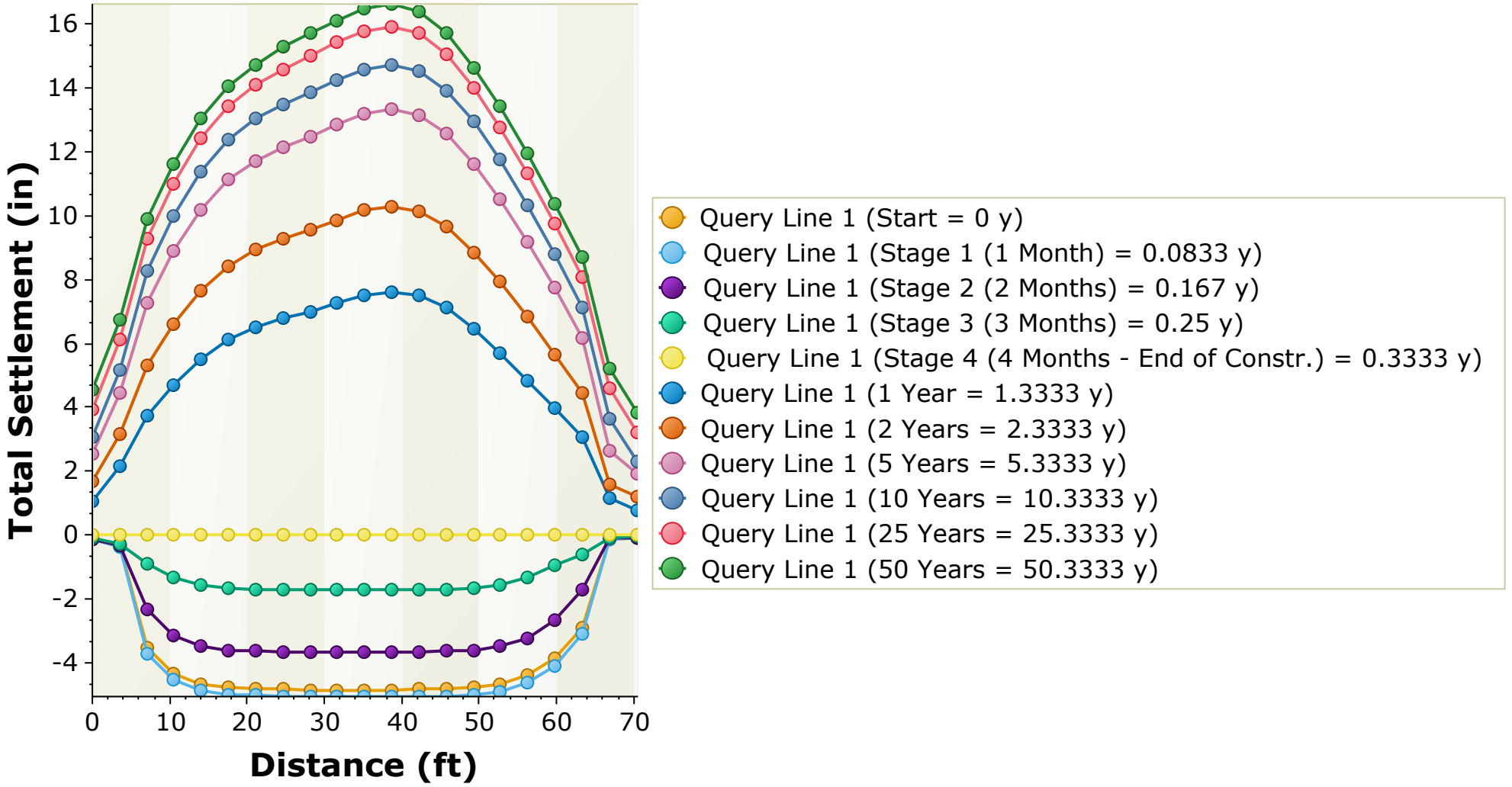
- Query Line 1 (Start = 0 y)
- Query Line 1 (Stage 1 (1 Month) = 0.0833 y)
- Query Line 1 (Stage 2 (2 Months) = 0.167 y)
- Query Line 1 (Stage 3 (3 Months) = 0.25 y)
- Query Line 1 (Stage 4 (4 Months - End of Constr.) = 0.3333 y)
- Query Line 1 (1 Year = 1.3333 y)
- Query Line 1 (2 Years = 2.3333 y)
- Query Line 1 (5 Years = 5.3333 y)
- Query Line 1 (10 Years = 10.3333 y)
- Query Line 1 (25 Years = 25.3333 y)
- Query Line 1 (50 Years = 50.3333 y)

Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y
Total Settlement at Elevation = 5.8 ft

ROCSCIENCE	*Project*	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 28+50	
	Drawn By	Armaan Khamenian	*Company* Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	*File Name* SAFER (South of Bay Road) - STA 28+50 (Overbuild).s3z

SETTLE3 5.027

Distance vs. Total Settlement



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Total Settlement at Elevation = 5.5 ft

ROCSCIENCE	Project: SAFER (South of Bay Road)	
	Analysis Description: Settlement Analysis at STA 42+50	
	Drawn By: Armaan Khamenian	Company: Geosyntec Consultants, Inc.
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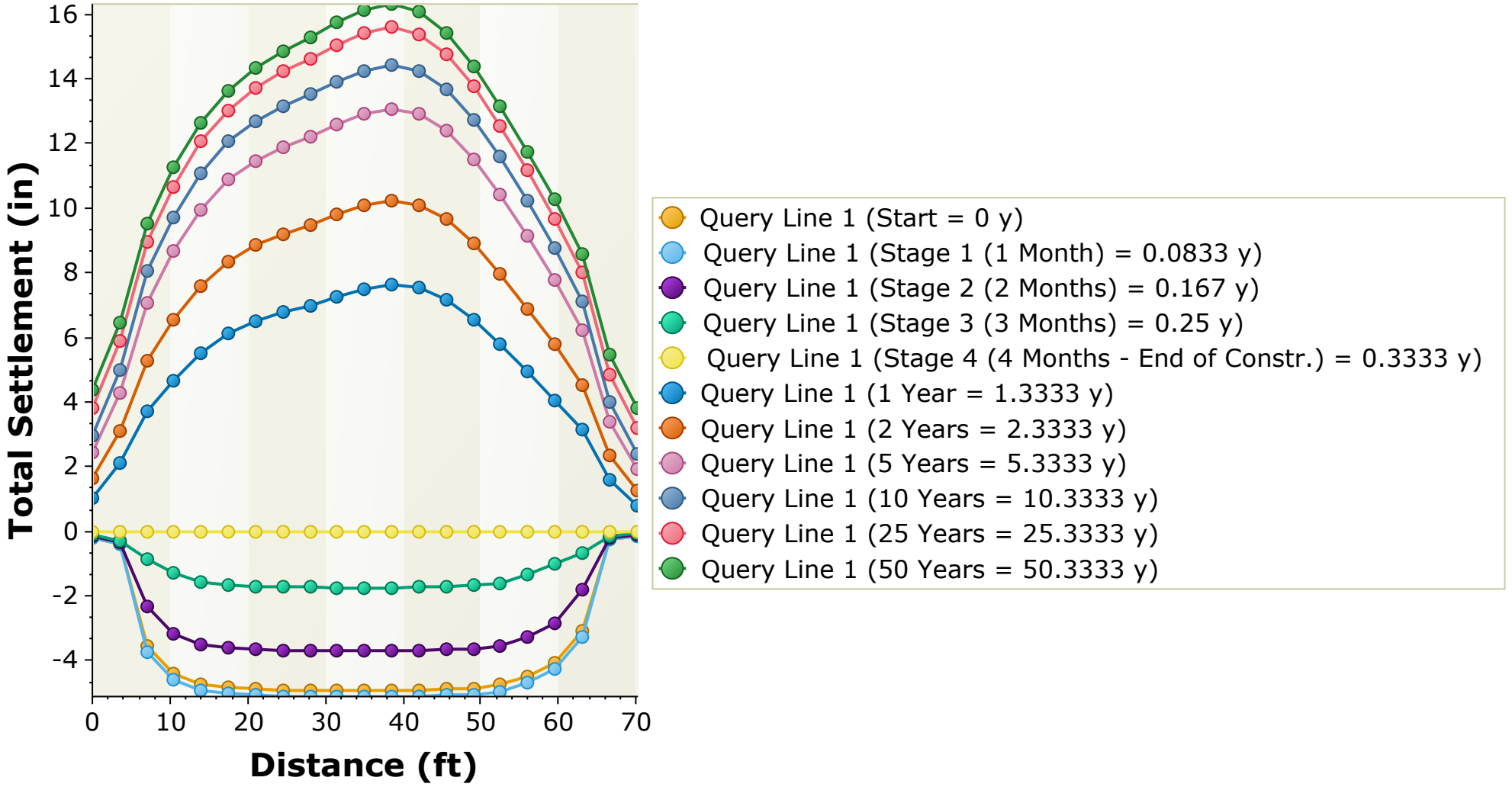
SETTLE3 5.027

Distance vs. Total Settlement



Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y
Total Settlement at Elevation = 5.5 ft

ROCSCIENCE	Project	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis at STA 49+00	
	Drawn By	Armaan Khamenian	Company: Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	File Name: SAFER (South of Bay Road) - STA 49+00 (Overbuild).s3z

SETTLE3 5.027

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DESIGN ENGINEERS

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+1 (415) 677-7300 | WWW.SHERWOODENGINEERS.COM

Appendix A.2:
Levee/Floodwall Engineering Analysis
Seismic Hazard

Geosyntec consultants

COMPUTATION COVER SHEET

Client:	**Sherwood Design Engineers**	Project:	**SAFER East Palo Alto, STA 0+00 to 69+00**	Project/ Proposal No.:	**WG3607**
				Phase/ Task No.	**007**

Title of Computations **Seismic Hazard Analysis**

Computations by:	Signature	Elizabeth Eckes	5/11/2026
	Printed Name	Elizabeth Eckes	Date
	Title	Staff Professional	
Assumptions and Procedures Checked by: (peer reviewer)	Signature	Christie N Hale	5/20/2026
	Printed Name	Christie Hale	Date
	Title	Senior Professional	
Computations Checked by:	Signature	Christie N Hale	5/20/2026
	Printed Name	Christie Hale	Date
	Title	Senior Professional	
Computations backchecked by: (originator)	Signature	Elizabeth Eckes	5/22/2026
	Printed Name	Elizabeth Eckes	Date
	Title	Staff Professional	
Approved by: (pm or designate)	Signature		5/26/2026
	Printed Name	David Umberg	Date
	Title	Senior Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	**Elizabeth Eckes**	Date:	**5/11/2026**	Reviewed by:	**Christie Hale**	Date:	**5/20/2026**
Client:	**SDE**	Project:	**SAFER Bay East Palo Alto**	Project No.	**WG3607**	Task No.:	**007**

1. INTRODUCTION

This calculation package presents the results of a simplified probabilistic seismic hazard analysis (PSHA) and simplified deterministic seismic hazard analysis (DSHA) for the site surrounding the South of Bay Road Reach of the project known as the Strategy to Advance Flood Protection, Ecosystems, and Recreation along San Francisco Bay Project (SAFER Bay Project or Project) which is located along the eastern edge of East Palo Alto, California city boundaries.

The following USACE publication is used to evaluate seismic hazard and seismic performance criteria for the project levees, embankments, and floodwalls:

- ER 1110-2-1806, Engineering and Design: Earthquake Analysis, Evaluation, and Design for Civil Works Projects; Effective 29 June 2024 (USACE 2024).

For preliminary design, the simplified seismic analysis methods discussed in this calculation package have been used. According to ER 1110-2-1806, critical structures in high seismic hazard regions (such as the San Francisco Bay Area) require advanced analysis methods such as ground motion time history development, seismic site response analyses, and/or fully nonlinear deformation seismic analyses methods. These analyses also do not include considerations for near source effects (e.g. directionality and maximum component factors), directivity, or aftershocks. These advanced analyses methods and additional considerations are not included as part of the current scope of work but are recommended for more accurate characterization of the seismic hazard and to meet the requirements of ER-1110-2-1806.

According to ER-1110-2-1806 USACE (2024), the seismic design criteria depends on the project consequence-based feature classification. Since the usual water surface elevation (WSE) with 3.5 ft of sea level rise (SLR) is higher than the landside toe of the levee, the levee is considered "frequently" loaded and therefore a breach of the levee/floodwall system is likely to lead to flooding. The breach could inundate a large area with significant population and property development. Based on the frequency of loading, along with the potential consequences of a breach, the project is considered "critical" when subject to 3.5 ft of SLR, according to definitions provided in Table 1 of ER 1110-2-1806.

ER 1110-2-1806 requires consideration of two seismic loading scenarios:

- Operating Basis Earthquake (OBE). The OBE is a ground motion that can reasonably be expected to occur within the service life of the flood protection system. According to ER 1110-2-1806, the OBE ground motion for a critical structure should have a minimum return period of 475-years.
- Maximum Design Earthquake (MDE). The MDE is a rare but possible earthquake scenario and represents the maximum level of seismic ground motion considered for design of the

project. According to ER 1110-2-1806, the MDE must be taken as the larger of the following two seismic scenarios:

- The ground motion with a return period of 2,475-years.
- The Maximum Credible Earthquake (MCE) ground motion. The MCE is the largest earthquake that can reasonably be expected for a given fault or seismic source. The MCE is required to be evaluated and/or interpolated between the median and 84th percentile ground motion, depending on the seismic source slip rate.

Section 2 below presents the parameters, assumptions, and models used in the simplified PSHA and DSHA completed for the Project. Section 3 presents the resulting seismic design parameters and response spectra from the PSHA and DSHA, and outlines the methods used to develop the two design seismic loading scenarios (OBE and MDE).

2. SEISMIC HAZARD ANALYSES

The Project is located in a seismically active region in Northern California where numerous capable faults together accommodate the relative motion between the Pacific and North American plates. **Figure 1** shows the site location along with faults from the most recent Western United States (WUS) fault model developed by the United States Geological Survey (USGS, Hatem et al. 2023) and used in the 2023 edition of the National Seismic Hazard Model (NSHM). Faults within 30 kilometers (km) of the Project are listed in **Table 1**.

Table 1. Summary of faults near SAFER Bay East Palo Alto, STA 0+00 to 69+00

Fault	Map Distance (km)
Monte Vista - Shannon	9.0
San Andreas (Peninsula)	13.0
Pilarcitos	14.5
Silver Creek	15.0
Hayward (South)	16.5
Mission (Connected)	19.0
Calaveras (North)	25.5
San Gregorio (North)	27.8

Table 2 includes the Project site characterization parameters used in the seismic hazard analyses.

Table 2. Project site characterization parameters

Latitude	37.4686°
Longitude	-122.1262°

V_{s30}	185 m/s
$Z_{1.0}$	0.55 km
$Z_{2.5}$	1.15 km

Notes:

1. A V_{S30} of 185 m/s was estimated based on the average of calculated V_{s30} from seismic cone penetration test (SCPTu) profiles near the project control line (i.e., GS26-CPT-5, -10, -16, -21, and -25) (Geosyntec, 2026).
2. $Z_{1.0}$ and $Z_{2.5}$ are the depths to a VS of 1 and 2.5 kilometers per second (km/s) respectively and were estimated from the USGS NCM Geophysical Profiles tool in the USGS Earthquake Hazard Toolbox (USGS, 2025).

2.1 Probabilistic Seismic Hazard Analysis

Probabilistic ground motions were extracted for the Site from the USGS 2023 NSHM via the Earthquake Hazard Toolbox (USGS 2025). The 2023 NSHM is based on an earthquake rupture forecast (ERF) that describes the probabilistic locations, sizes, and rates of potential earthquakes and ground motion models (GMMs) that define probabilistic levels of ground shaking conditioned on an earthquake rupture and site characteristics.

The seismic sources included in the USGS ERF and relevant to the seismic hazard at the Site include explicitly modeled active crustal faults and a gridded seismicity source. Active crustal faults included in the ERF are presented in **Figure 1** and those within about 30 km of the Site contribute most to the probabilistic hazard. The WUS gridded seismicity source covers the entire region and accounts for future earthquake scenarios that are not localized to a known or mapped fault. The gridded seismicity models used in the 2023 NSHM are based on an updated earthquake catalog that includes earthquakes that occurred through 2022 (Peterson et al. 2023).

The 2023 NSHM uses a set of GMMs for shallow crustal earthquakes in active tectonic regions, applicable to the explicitly modeled faults and gridded seismicity source model in the WUS. The following NGA-West2 (Next Generation Attenuation West 2) models are implemented in the 2023 NSHM: Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014). The USGS applies additional epistemic uncertainty to the NGA-West2 GMMs using the model of Rezaeian et al. (2014) to account for the limited recordings of large-magnitude earthquakes, limited recordings from many of the regions for which the GMMs are applied, and the high degree of interaction in the NGA-West2 Project.

2.2 Deterministic Seismic Hazard Analysis

Based on distance and slip rate, the San Andreas (Peninsula) and Hayward (South) faults are considered likely to govern ground motions at the Project. The DSHA was performed considering the effects of these two faults. The DSHA for the other faults listed in Table 1 are not included as part of the current scope of work but are recommended for more accurate characterization of the seismic hazard and to meet the requirements of ER-1110-2-1806.

The characterization of faults for the DSHA was based on the WUS fault model (Hatem et al. 2023). The WUS fault model provides the three-dimensional (3D) geometry of fault sources, which are defined by a fault trace, style of faulting, dip, and upper and lower seismogenic depths. We recognize the inherent subjectivity of defining a single multi-segment rupture scenario and associated magnitude and considered multiple references in making our decision including: magnitudes of significant historical earthquakes on each fault, UCERF2 connected rupture scenarios (Field et al. 2009), and the disaggregated magnitude by fault source from the USGS 2023 NSHM at peak ground acceleration (PGA) and a return period of 2,475-yrs. For the DSHA, moment magnitudes (**M**) of 7.8 for the San Andreas (Peninsula) fault and 7.2 for the Hayward (south) fault were used.

The same four NGA-West2 GMMs used by the USGS in the PSHA were used in the DSHA for consistency. Table 3 shows the inputs into the NGA-West2 GMM tool (Ancheta et al. 2015) for the deterministic scenarios considered. **Figure 3** illustrates the input parameters diagrammatically.

Table 3. Inputs for HGA-West2 GMMs in DSHA

Fault	M	R_{rup}	R_{jb}	R_x	R_{y0}	Dip	Z_{hyp}	Z_{tor}	W
	-	km	km	km	km	deg	km	km	km
San Andreas (Peninsula)	7.8	12.9	12.9	-13.3	0	90	6.5	0.0	13.0
Hayward (South)	7.2	16.7	16.7	-16.2	0	76	6.7	0.0	13.4

3. SEISMIC DESIGN PARAMETERS

Uniform hazard spectra (UHS) from the PSHA spanning a range of return periods are presented in **Figure 4** and provided in Table 4. The UHS represent response spectra with an equal probability of exceedance at each spectral period and the response spectra describe the 5%-damped pseudo-spectral acceleration (PSA) for the average horizontal component. A disaggregation analysis breaks down the total mean hazard into contributions from different earthquake scenarios. **Figure 5** presents the disaggregation results in terms of the percent contributions for different magnitude-distance pairs at PGA and for return periods of 475 and 2,475 years (USGS 2025).

Figure 6 and Table 5 provide the 84th percentile response spectra for the Hayward and San Andreas earthquake scenarios resulting from the DSHA. The response spectrum for the San Andreas earthquake scenario is greater than the response spectrum for the Hayward earthquake scenario, and therefore, the MCE is considered equivalent to the response spectra for the San Andreas scenario. **Figure 7** provides a comparison between the 2,475-yr UHS and the MCE deterministic spectrum, as required by ER 1110-2-1806. As shown in Figure 7, the 2,475 year return period UHS is larger than the MCE, and therefore the 2,475 UHS was selected to represent the MDE-GM.

Figure 8 and Table 6 provide the preliminary recommended OBE-GM and MDE-GM based on the simplified PSHA and DSHA. As discussed in the introduction of this calculation package according to ER 1110-2-1806, critical structures in high seismic hazard regions (such as the San Francisco Bay Area) require advanced analysis methods such as ground motion time history development, seismic site response analyses, and/or fully nonlinear deformation seismic analyses methods. Seismic design for critical structurers should also include considerations for near source effects (e.g. directionality and maximum component factors), directivity, and aftershocks. These advanced analyses methods and considerations are not included as part of the current scope of work but are recommended for final design and to meet the requirements of ER-1110-2-1806.

Table 4. Probabilistic UHS

Period T [s]	Horizontal pseudo-spectral acceleration (PSA) [g]					
	100-yr	225-yr	475-yr	975-yr	2,475-yr	10,000-yr
PGA	0.296	0.397	0.503	0.614	0.765	1.025
0.01	0.298	0.400	0.507	0.619	0.771	1.034
0.02	0.292	0.398	0.505	0.615	0.773	1.023
0.03	0.294	0.394	0.497	0.608	0.758	1.021
0.05	0.319	0.433	0.541	0.667	0.837	1.142
0.075	0.399	0.533	0.678	0.829	1.062	1.491
0.1	0.484	0.650	0.809	0.996	1.261	1.753
0.15	0.616	0.802	1.003	1.198	1.501	2.013
0.2	0.697	0.915	1.121	1.344	1.637	2.134
0.25	0.746	0.995	1.219	1.476	1.776	2.299
0.3	0.778	1.046	1.306	1.574	1.932	2.486
0.4	0.769	1.060	1.355	1.651	2.082	2.721
0.5	0.734	1.030	1.330	1.641	2.095	2.779
0.75	0.574	0.832	1.117	1.404	1.830	2.550
1	0.469	0.702	0.953	1.232	1.656	2.354
1.5	0.329	0.513	0.729	0.963	1.321	1.958
2	0.244	0.395	0.575	0.786	1.111	1.693
3	0.148	0.252	0.380	0.528	0.763	1.198
4	0.099	0.172	0.265	0.377	0.551	0.879
5	0.069	0.124	0.195	0.280	0.417	0.673
7	0.035	0.064	0.102	0.150	0.226	0.370
10	0.021	0.038	0.061	0.089	0.136	0.224

Table 5. Deterministic Response Spectra

Period T [s]	Horizontal PSA [g]	
	Hayward (South) 84th %	San Andreas (Peninsula) 84th %
0.01	0.388	0.488
0.02	0.384	0.482
0.03	0.380	0.473
0.05	0.406	0.494
0.075	0.492	0.584
0.1	0.583	0.677
0.15	0.728	0.824
0.2	0.835	0.945
0.25	0.917	1.053
0.3	0.980	1.153
0.4	1.024	1.262
0.5	1.015	1.298
0.75	0.855	1.163
1	0.754	1.085
1.5	0.586	0.933
2	0.473	0.810
3	0.314	0.586
4	0.217	0.430
5	0.155	0.321
7.5	0.073	0.162
10	0.039	0.089

Notes:

1. Site Lat: 37.4686°, Long: -122.1262°
2. V_{S30} = 185 m/s
3. Damping = 5%

Table 6. Design Response Spectra

Period, T [s]	Horizontal PSA [g]	
	MDE-GM	OBE-GM
0.01	0.771	0.507
0.02	0.773	0.505
0.03	0.758	0.497
0.05	0.837	0.541
0.075	1.062	0.678
0.1	1.261	0.809
0.15	1.501	1.003
0.2	1.637	1.121
0.25	1.776	1.219
0.3	1.932	1.306
0.4	2.082	1.355
0.5	2.095	1.330
0.75	1.830	1.117
1	1.656	0.953
1.5	1.321	0.729
2	1.111	0.575
3	0.763	0.380
4	0.551	0.265
5	0.417	0.195
7.5	0.226	0.102
10	0.136	0.061

Notes:

1. MDE-GM and OBE-GM are the Maximum Design Earthquake ground motion and Operating Basis Earthquake ground motion per ER 1110-2-1806 (USACE 2024), respectively.
2. The MDE-GM is consistent with the 2,475-yr UHS from the USGS 2023 NSHM.
3. The OBE-GM is consistent with the 475-yr UHS from the USGS 2023 NSHM.
4. Site Lat: 37.4686°, Long: -122.1262°
5. V_{S30} = 185 m/s
6. Damping = 5%

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A.E., Haynie, K.L., Hearn, E.H., Johnson, K.M., Kortum, Z.A., Kwong, N.S., Makdisi, A.J., Mason, H.B., McNamara, D.E., McPhillips, D.F., Okubo, P.G., Page, M.T., Pollitz, F.F., Rubinstein, J.L., Shaw, B.E., Shen, Z.K., Shiro, B.R., Smith, J.A., Stephenson, W.J., Thompson, E.M., Thompson Jobe, J.A., Wirth, E.A., and Witter, R.C. 2023. *Data Release for the 2023 U.S. 50-State National Seismic Hazard Model - Overview: U.S. Geological Survey data release*, https://doi.org/10.5066/P9GNPCOD.

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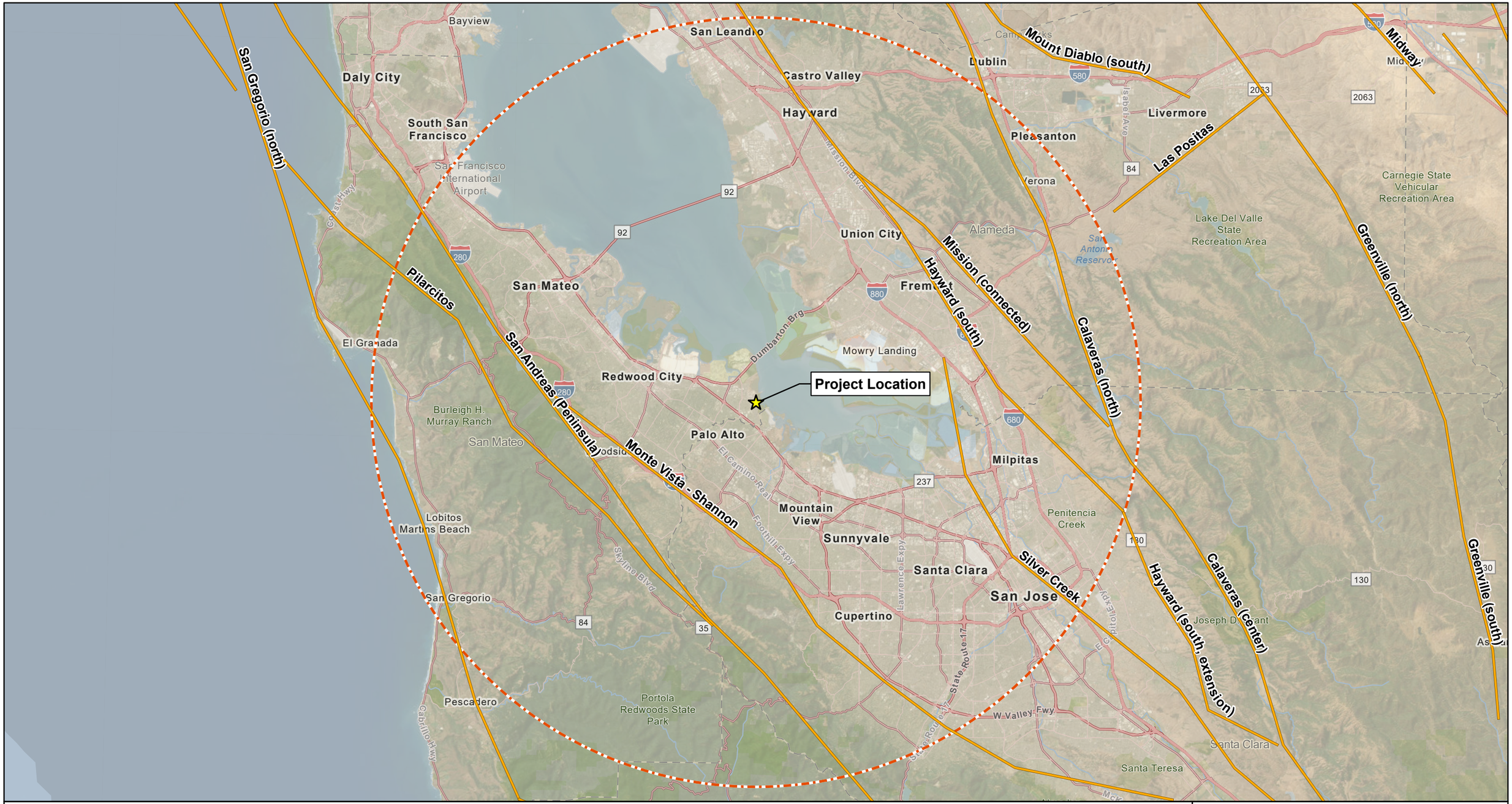
FIGURES



Legend

- Project Location
- 30-Kilometer Radius
- 2023 National Seismic Hazard Model Fault Line

Notes:

- Fault locations are from Hatem et al. (2021) and represent the fault sections used in the USGS 2023 National Seismic Hazard Model.

N

0 5 Miles

Site Location Map

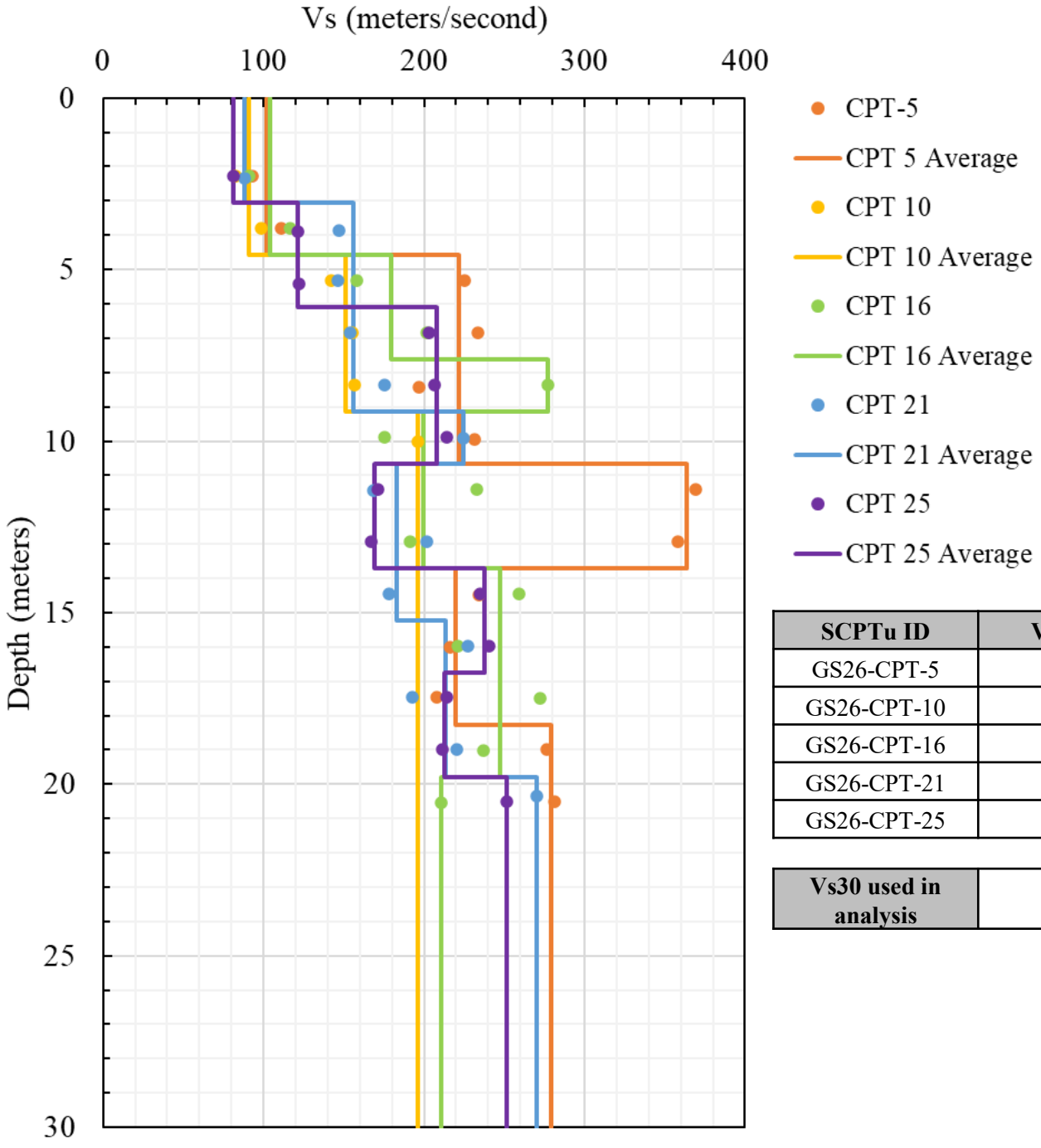
SAFER East Palo Alto
STA 0+00 to 69+00
East Palo Alto, California

Geosyntec consultants

Figure 1

WG3607 | May 2026

\\oakland-01\data\GIS\SAFERBayProject\Project\2026-05_FaultMap\Figures.aprx\Fig01_Faults 5/8/2026 1:39 PM (kwalton)



SCPTu ID	V_{s30} [m/s]
GS26-CPT-5	210
GS26-CPT-10	161
GS26-CPT-16	185
GS26-CPT-21	183
GS26-CPT-25	177

Vs30 used in analysis	185

Notes:
1. Shear wave velocity measurements are from Geosyntec's 2025 Geotechnical Investigation. See the project Geotechnical Data Report (GDR) (Geosyntec 2026) for further information.
2. The shear wave velocity used in seismic hazard analyses is a rounded average of the shear wave velocity from each SCPTu profile collected by Geosyntec for CPTs near the project control line.

Shear Wave Velocity Measurements Compared to V_{s30}

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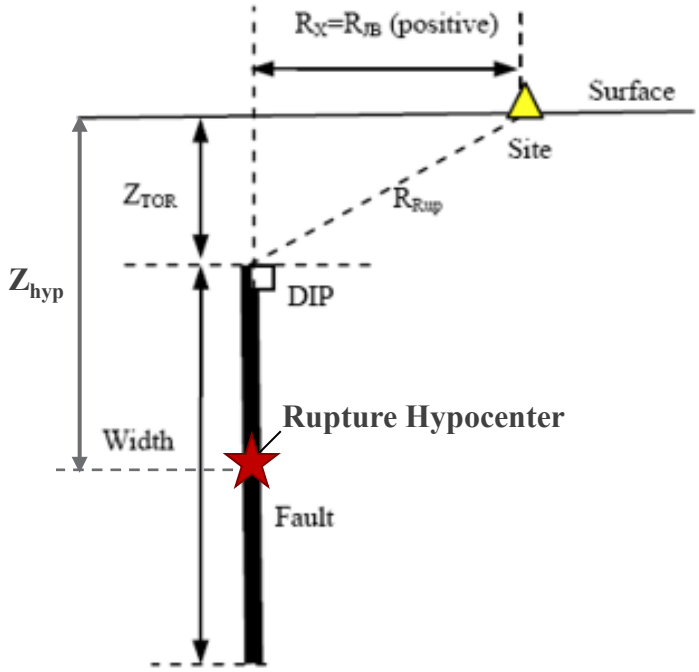
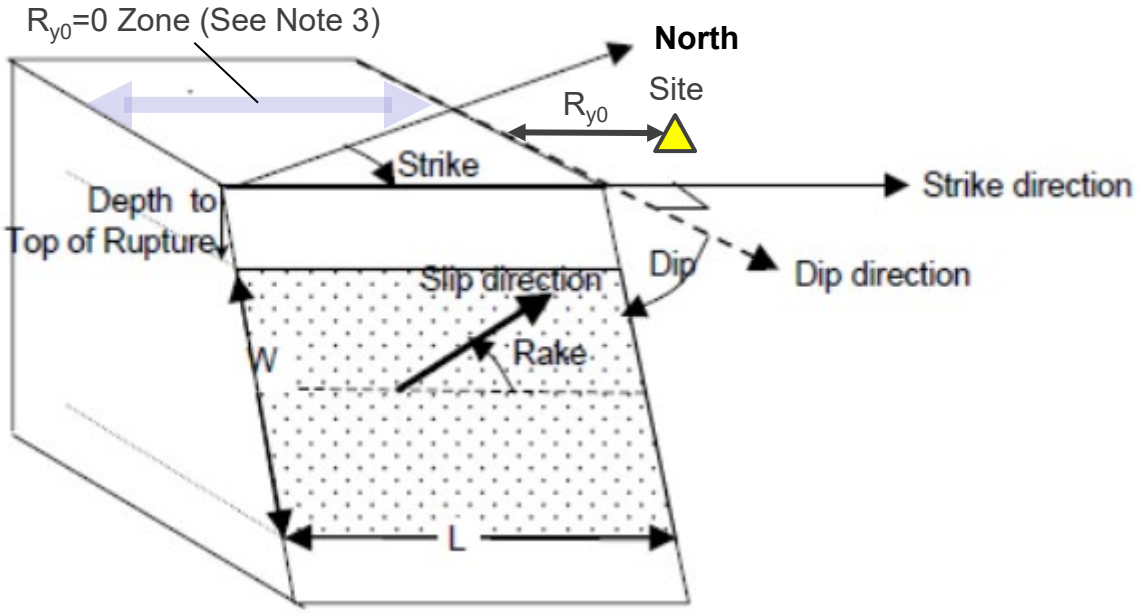
Figure 2

\\FS-US-CA-OAK\data\PRJ2003Geo\Sherwood Design Engineers\SAFER_EastPaloAlto\04. Technical\Seismic Hazard

(a) Distances (R) and Depths (Z) relative to ruptures of strike slip faults



(b) Fault rupture Width (W) and Dip angle measurement



Notes
1. Figures originally included in NGA West2 GMM spreadsheet tool (Seyhan 2014) for strike-slip faults.
2. Geosyntec added label for the horizontal distance off the end of the rupture measured parallel to the strike (R_{y0}) and the depth to the hypocenter (Z_{hyp})
3. If the rupture passes the site (so that the distance from the end of the rupture measured parallel to the strike overlaps the same length along the strike as L) then R_{y0} is equal to zero.

Geometric Definitions of Input Parameters for NGA-West2 GMM

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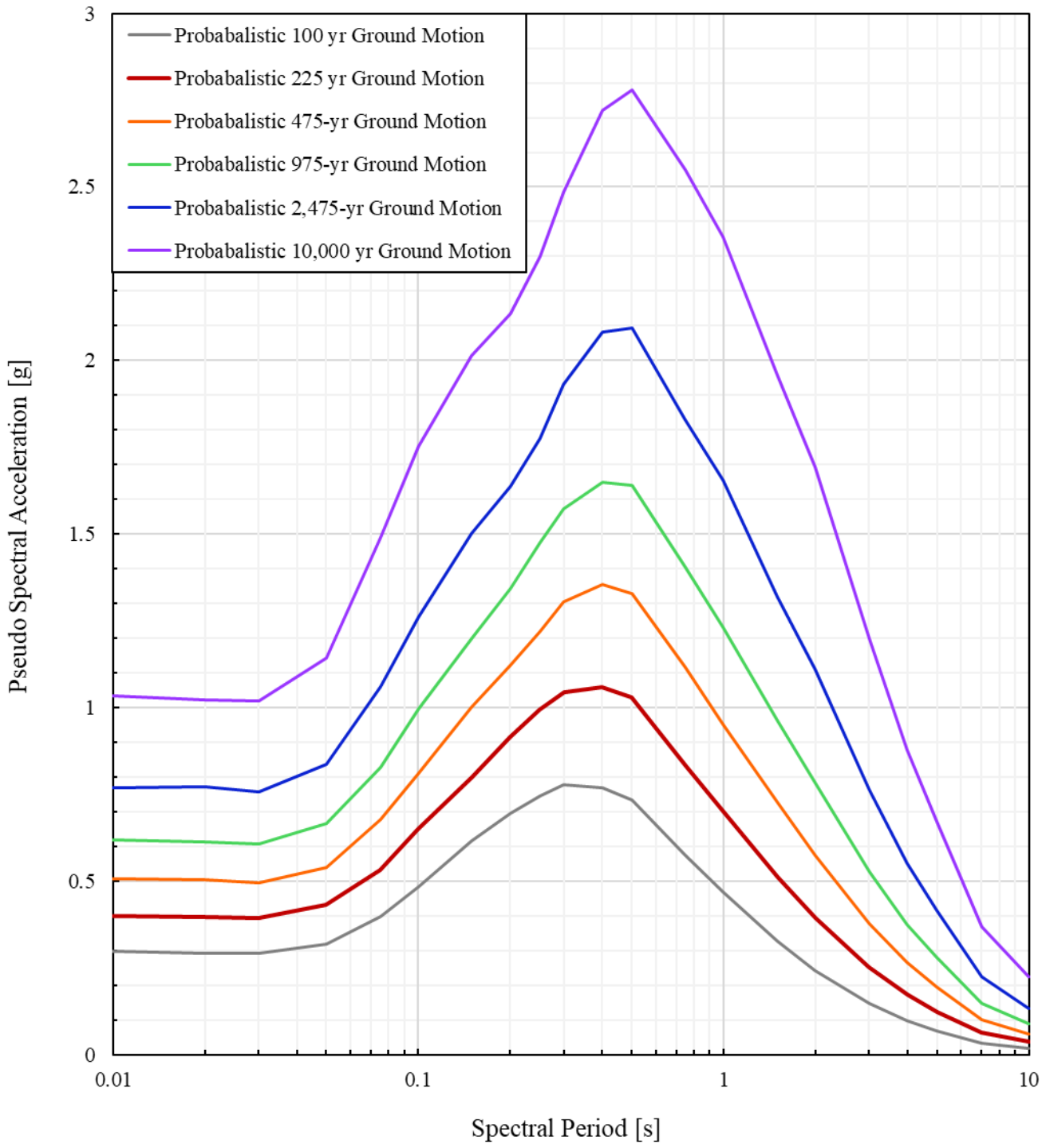
Figure 3



Notes
1. Site Lat: 37.4686°, Long: -122.1262°
2. V_{S30} = 185 m/s
3. Damping = 5%

Development of Design Response Spectra
Uniform Hazard Curves as a Result of PSHA

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Figure 4

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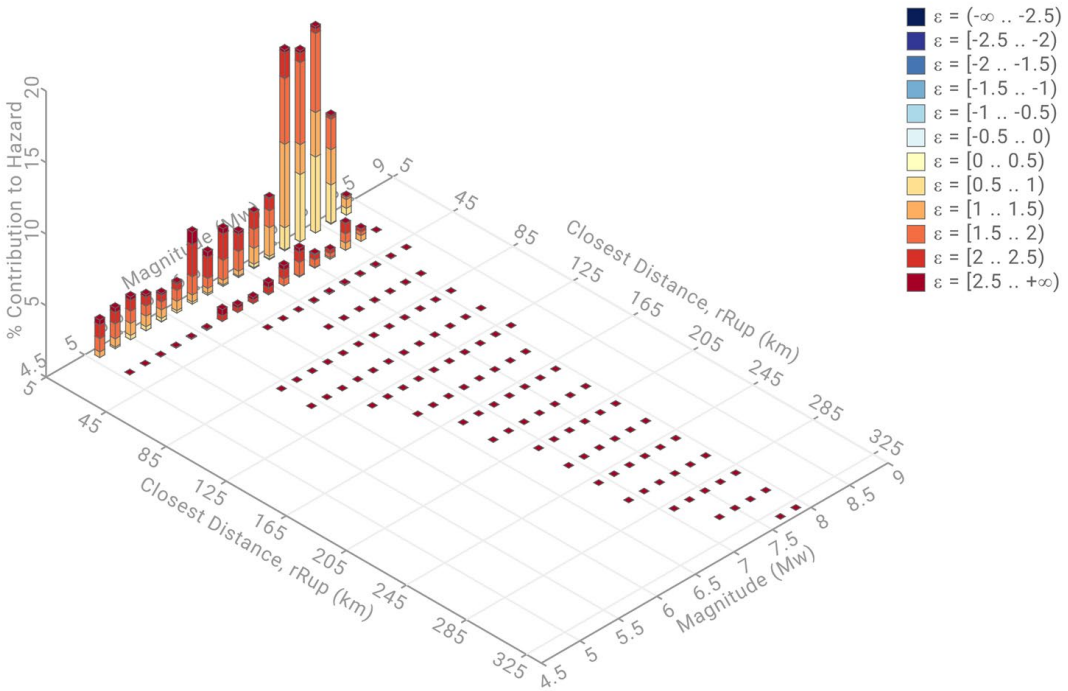
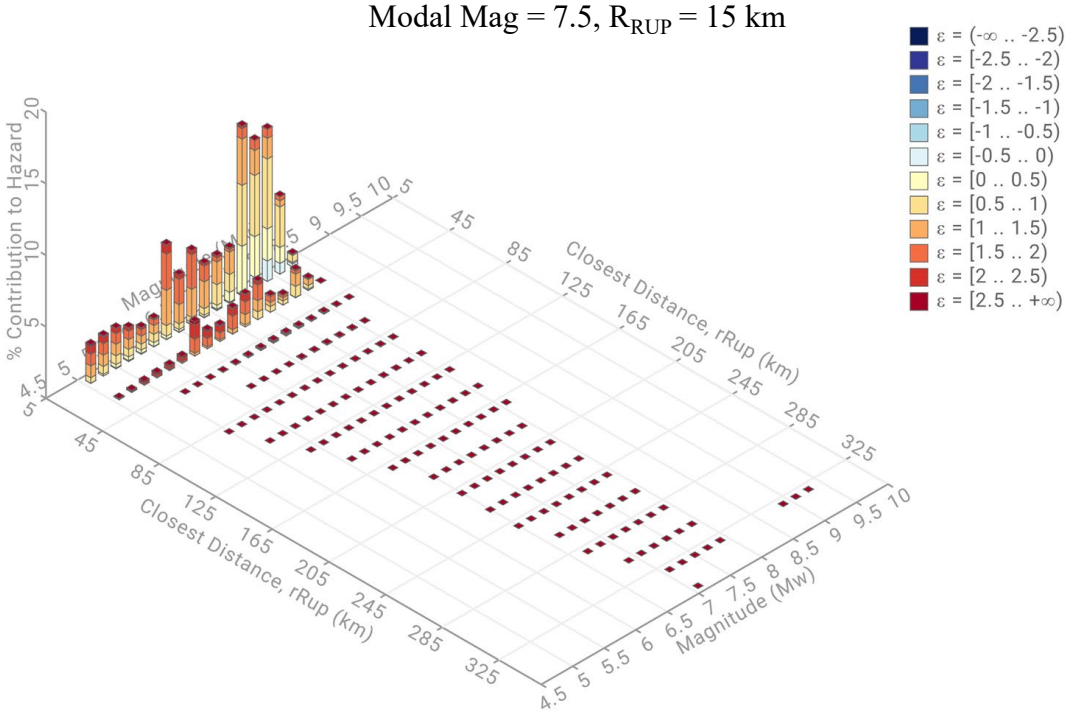
\\FS-US-CA-OAK\data\PRJ2003Geo\Sherwood Design Engineers\SAFER_EastPaloAlto\04. Technical\Seismic Hazard

(a) PGA, 2,475-yr: Mean Mag = 7.2, R_{RUP} = 15 km
Modal Mag = 7.9, R_{RUP} = 13 km



(b) PGA, 475-yr: Mean Mag = 7.0, R_{RUP} = 17 km
Modal Mag = 7.5, R_{RUP} = 15 km



Notes:

1. Disaggregation results are from the USGS 2023 NSHM via the Earthquake Hazard Toolbox (USGS 2025).
2. Site Lat: 37.4686°, Long: -122.1262°
3. V_{S30} = 185 m/s

Disaggregation Results

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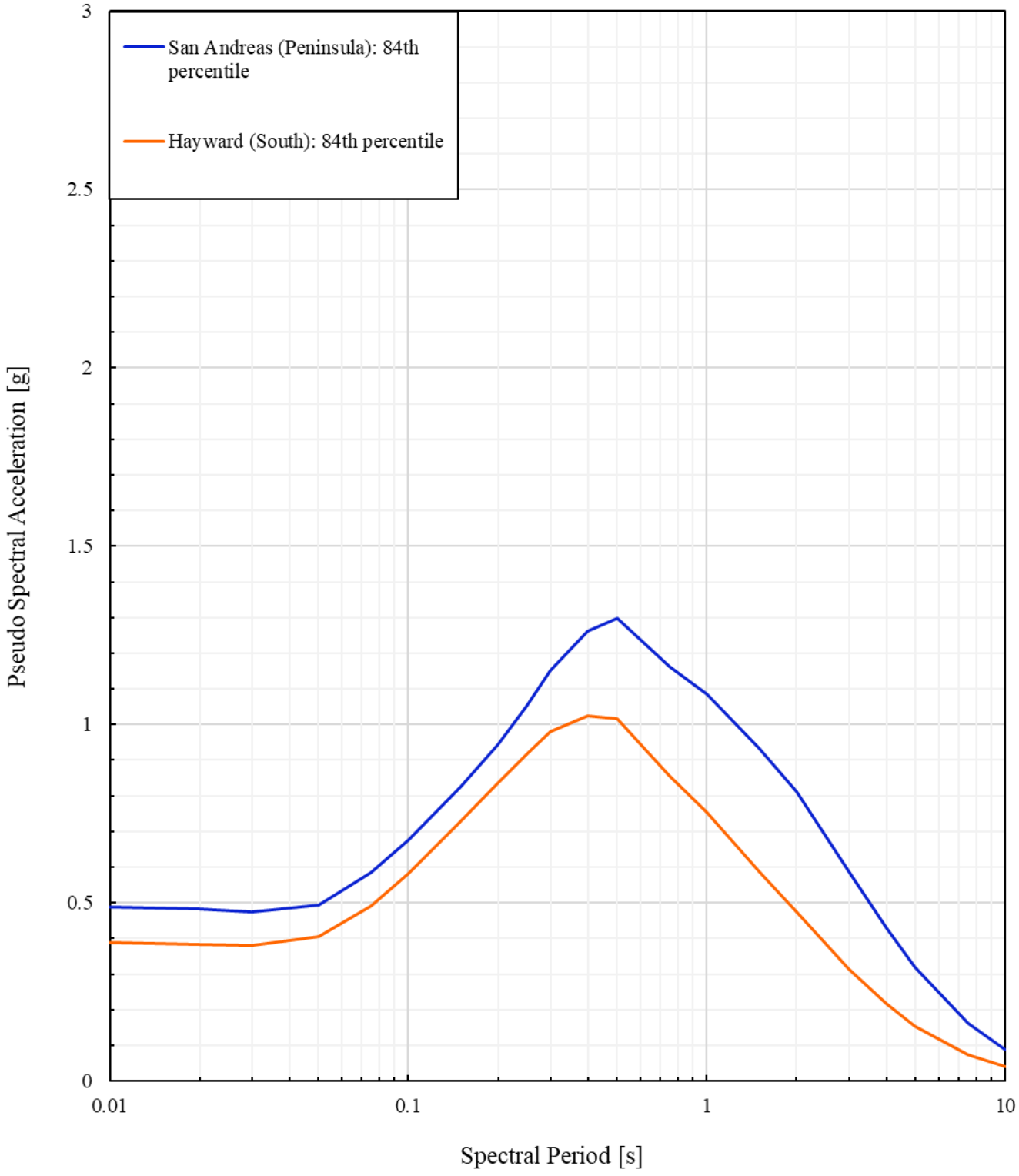
Figure 5



Notes:
1. Site Lat: 37.4686°, Long: -122.1262°
2. V_{S30} = 185 m/s
3. Damping = 5%

Development of Design Response Spectra
Uniform Hazard Curves as a Result of DSHA

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East Palo Alto, California

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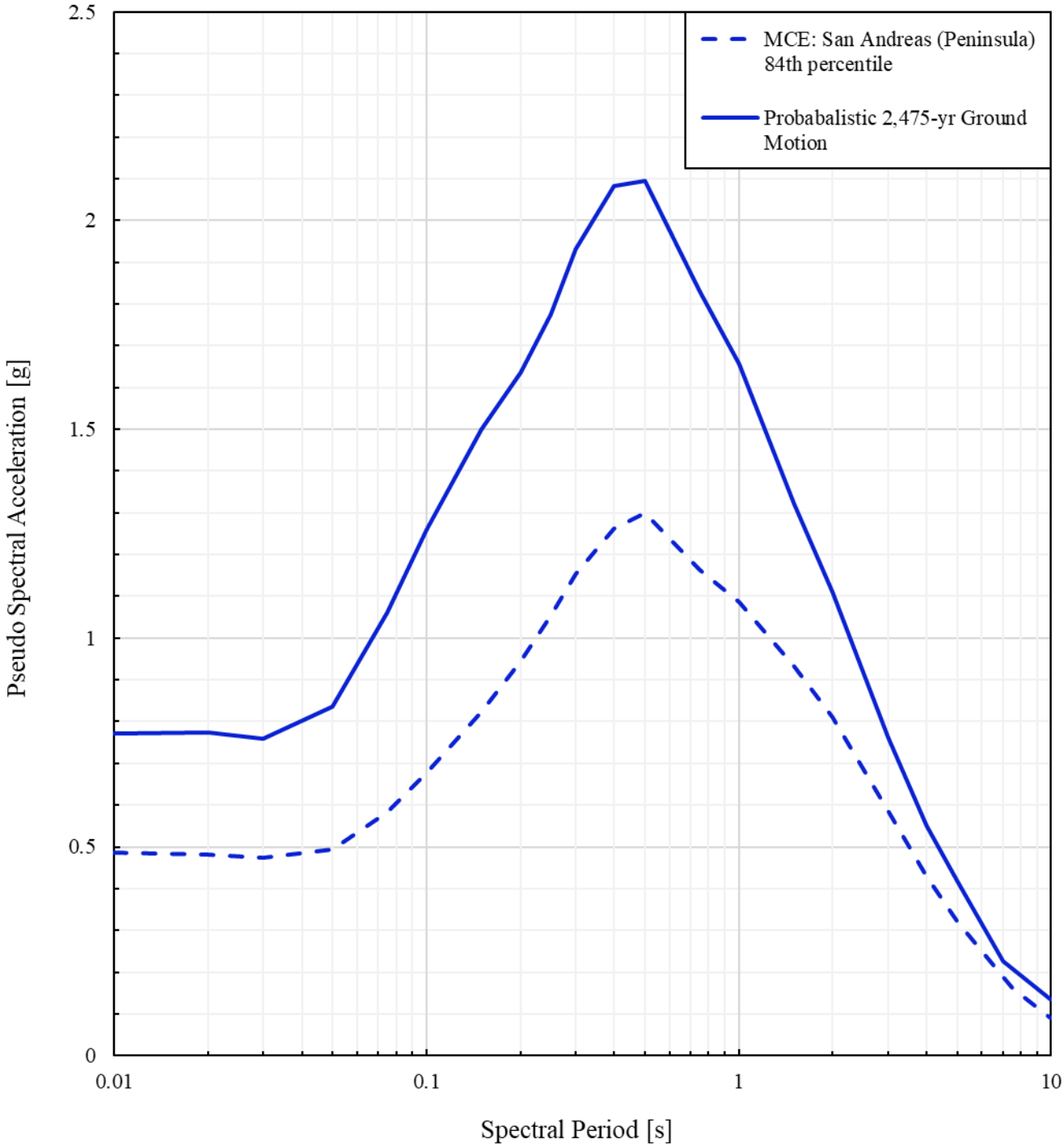
Figure 6

Project No. WG3607 | May 2026

\\FS-US-CA-OAK\data\PRJ2003Geo\Sherwood Design Engineers\SAFER_EastPaloAlto\04. Technical\Seismic Hazard



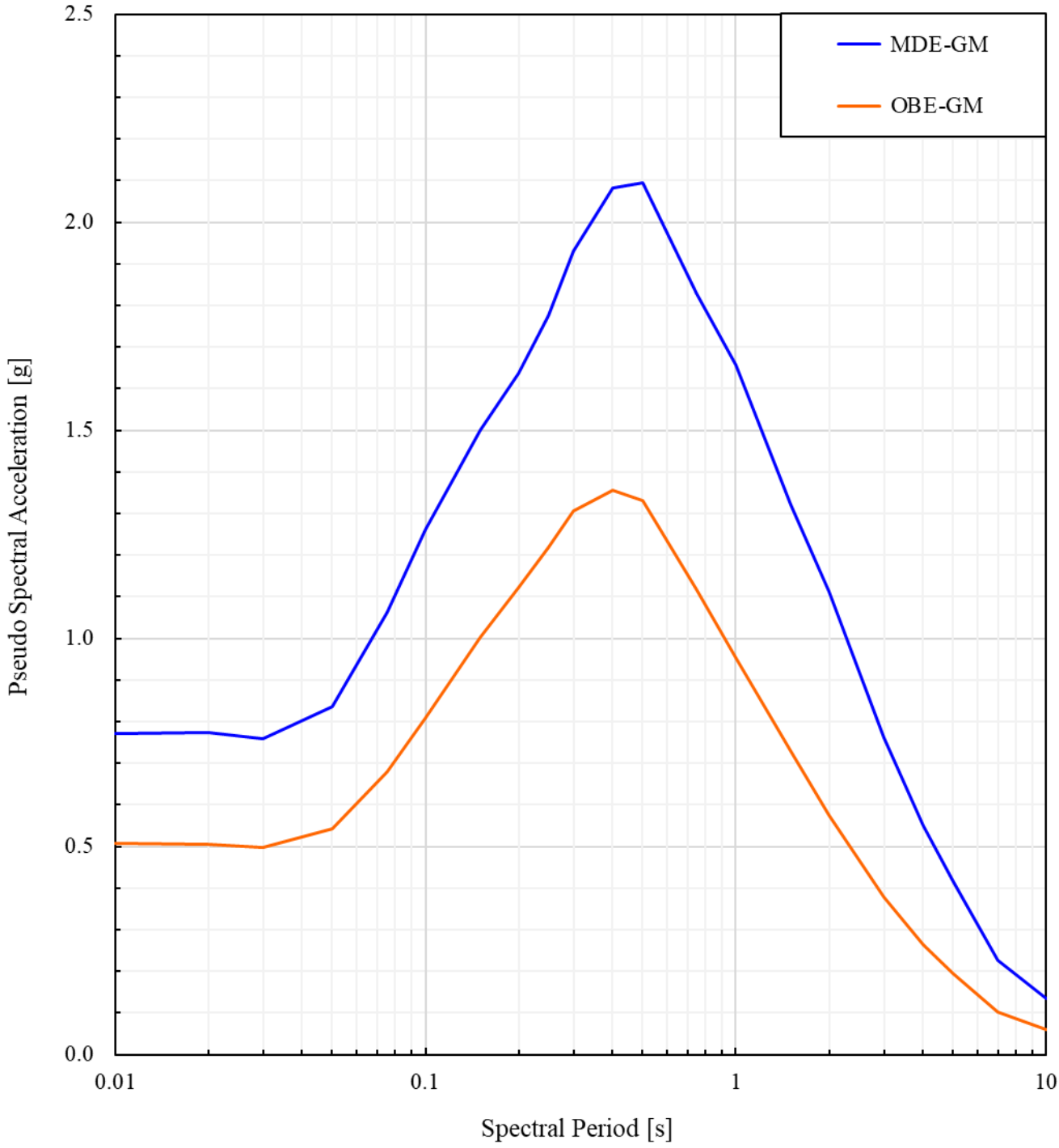
Notes:
1. Site Lat: 37.4686°, Long: -122.1262°
2. V_{S30} = 185 m/s
3. Damping = 5%

Development of Design Response Spectra
Comparison between MCE from DSHA and 2,475-yr Motion from PSHA
SAFER Bay East Palo Alto, STA 0+00 to 69+00
East Palo Alto, California

Geosyntec consultants

Figure 7

Project No. WG3607 | May 2026



Notes:

1. MDE-GM and OBE-GM are the Maximum Design Earthquake ground motion and Operating Basis Earthquake ground motion per USACE ER 1110-2-1806 (USACE 2024), respectively.
2. The MDE-GM is consistent with the 2,475-yr UHS from the USGS 2023 NSHM.
3. The OBE-GM is consistent with the 475-yr UHS from the USGS 2023 NSHM.
4. Site Lat: 37.4686°, Long: -122.1262°
5. V_{S30} = 185 m/s
6. Damping = 5%

Design Response Spectra

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STA 0+00 to 69+00
East Palo Alto, California

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Project No. WG3607	May 2026

Figure 8

\\FS-US-CA-OAK\data\PRJ2003Geo\Sherwood Design Engineers\SAFER_EastPaloAlto\04. Technical\Seismic Hazard

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Appendix A.3:
Levee/Floodwall Engineering Analysis
Liquefaction Evaluation

Geosyntec consultants

COMPUTATION COVER SHEET

Client: **Sherwood Design Engineers** Project: **SAFER Bay Project (South of Bay Road)** Project No.: **WG3607**

Title of Computations **South of Bay Road Liquefaction Susceptibility and Consequences Evaluation**

Computations by:	Signature		5/29/2026
	Printed Name	Armaan Khamenian	Date
	Title	Senior Staff Professional	
Assumptions and Procedures Checked by: (peer reviewer)	Signature		6/1/2026
	Printed Name	Jay Merani	Date
	Title	Senior Engineer	
Computations Checked by:	Signature		6/1/2026 & 6/2/2026
	Printed Name	Jay Merani/David Umberg	Date
	Title	Senior Engineer/Senior Engineer	
Computations backchecked by: (originator)	Signature		6/2/2026
	Printed Name	Armaan Khamenian	Date
	Title	Senior Staff Professional	
Approved by: (pm or designate)	Signature		6/2/2026
	Printed Name	David Umberg	Date
	Title	Senior Engineer	

Approval notes:

Revisions (number and initial all revisions)

No.	Sheet	Date	By	Checked by	Approval

Written by:	**A. Khamenian**	Date:	**5/29/2026**	Reviewed by:	**D. Umberg**	Date:	**6/2/2026**
Client:	**Sherwood Design Engineers**	Project:	**SAFER Bay East Palo Alto (South of Bay Road)**	Project No.:	**WG3607**	Task No.:	**07**

1. PURPOSE

This package evaluates the liquefaction susceptibility of soil deposits underlying the future flood protection system to be implemented as part of the SAFER (Strategy to Advance Flood Protection, Ecosystems and Recreation) Bay Project from STA 0+00 to STA 52+72, located in East Palo Alto, California, and referred to here as "South of Bay Road". This package also includes evaluation of potential consequences of liquefaction including post-liquefaction reconsolidation settlement, surface manifestation (i.e., sand boils), and lateral spread displacement. These evaluations will be incorporated into the flood protection system design components such as the sheet pile wall and embankment.

2. INTRODUCTION

Liquefaction is a phenomenon where cohesionless soils (i.e., gravel, sand, and cohesionless silt) below the water table lose strength as a result of shaking under earthquake loading. According to Marcuson (1978) and Youd et al (2001), "liquefaction is defined as the transformation of granular material from a solid to a liquefied state as a consequence of increased pore water pressure and reduced effective stress." According to Idriss and Boulanger (2008), the term liquefaction is used in reference to strength loss and deformation in saturated sands and other cohesionless soils. Based on reviews of field case histories of liquefied cohesionless soils, the liquefaction resistance of such soils is dependent on in-situ strengths and fines contents.

Idriss and Boulanger (2008) and Boulanger and Idriss (2014) present the procedures to evaluate liquefaction triggering and the potential consequences of liquefaction. Cone Penetration Test (CPT)-based soil liquefaction software CLiq v.3.3.2.9 developed by Geologismiki (2018) using the Boulanger and Idriss (2014) method was used to perform the evaluation based on the CPT data presented in Geosyntec (2026a). Additionally, a standard penetration test (SPT)-based soil liquefaction evaluation using the Idriss and Boulanger (2008) method was used to perform the evaluation based on the SPT and laboratory data presented in Geosyntec (2026a). The location of each investigation point can be found in Figure 1.

3. FIELD INVESTIGATIONS AND LABORATORY TESTING

The liquefaction evaluation used the information collected in sixteen CPT borings and four mud-rotary (MR) borings performed by Geosyntec in January 2026. This data, along with information from historical site explorations, is presented as part of the Geotechnical Data Report (GDR) (Geosyntec, 2026a). The CPTs used in the liquefaction evaluation are GS26-CPT-1 through

GS26-CPT-18 generally near or on the alignment of the proposed embankment. The MR borings along with their laboratory tests of samples used in the SPT-based liquefaction evaluation are GS26-B1 through GS26-B4. In addition to the current investigation data, historical investigation data was also considered when selecting the depths and extents of the liquefiable soil layers along the entire levee profile. Historical investigation points can be seen in Figures 1 and 2.

4. GROUNDWATER LEVEL

Both CPT and SPT-based liquefaction methods allow the user to enter in-situ (i.e., existing) groundwater conditions as well as separate groundwater conditions assumed during the time of the earthquake.

For in-situ groundwater conditions, groundwater was based on estimated water levels at each CPT based on pore pressures measured during the CPT explorations along with pore pressure dissipation tests performed during the investigation. MR boring in-situ groundwater levels were based on water level estimates from their paired CPT exploration, as MR does not allow for accurate groundwater estimation due to use of drilling mud to advance the boring along with the continuous pumping of drilling fluid into the boring during the investigation. Inputting in-situ groundwater conditions allows both methods to estimate the in-situ soil properties.

As described, seismic liquefaction may occur in saturated soil. To account for the higher groundwater table associated with sea level rise (SLR) during a seismic event, the groundwater level during an earthquake was set to the Mean Higher High Water (MHHW) plus 3.5 feet of SLR which is taken to be +10.8 feet NAVD88. This water level corresponds with the “Usual” water level with SLR, as described in the *Design Criteria Report* (Sherwood, 2026).

The water levels assumed for the interpretation of CPT/SPT results and for liquefaction susceptibility during the design earthquake are described in Geosyntec (2026a) and are presented below in Table 1.

Table 1: Groundwater Levels

CPT	Approximate Depth of water below existing ground surface at the time of drilling (ft)
GS26-CPT-1	14.6
GS26-CPT-2	11.7
GS26-CPT-3	12.7

GS26-CPT-4	4.2
GS26-CPT-5	9.7
GS26-CPT-6	8.8
GS26-CPT-7	13.0
GS26-CPT-8	6.0
GS26-CPT-9	8.9
GS26-CPT-10	6.0
GS26-CPT-11	5.1
GS26-CPT-12	3.0
GS26-CPT-13	4.5
GS26-CPT-14	4.7
GS26-CPT-15	6.0
GS26-CPT-16	1.7
GS26-CPT-17	7.0
GS26-CPT-18	6.6

5. SEISMIC HAZARD

The preliminary seismic design basis is described in a separate calculation package titled *Seismic Hazard Analysis* (Geosyntec, 2026b). As described in Geosyntec (2026b), the levee/floodwall system is considered a "critical" project feature and two seismic loading scenarios are considered based on ER-1110-1806 (USACE, 2024): an Operating Basis Earthquake (OBE) which considers a return period of 475-years and Maximum Design Earthquake (MDE) which considers a return period of 2,475-years. Table 2 summarizes the seismic parameters used in the liquefaction evaluation which includes the peak ground acceleration (PGA) and modal magnitude (M_w) used in both analysis methods.

Table 2: Seismic Design Parameters

Seismic Loading Scenario	Earthquake Return Period	Probability of Exceedance	PGA (g)	M_w
OBE	475-years	10% in 50 years	0.51	7.5
MDE	2,475-years	2% in 50 years	0.77	7.9

6. LIQUEFACTION EVALUATION AND RESULTS

For the CPT soil liquefaction evaluation, liquefaction analysis software CLiq v.3.3.2.9 (Geologismiki, 2018) was used as described in Section 2. Data from GS26-CPT-1 through GS26-CPT-3, GS26-CPT-5 through GS26-CPT-8, and GS26-CPT-10 through GS26-CPT-18 were imported into the software along with the seismic design parameters and groundwater levels described above. Soil unit weights were calculated automatically by the software based on the CPT outputs and were found to be similar to unit weights estimated through laboratory tests for each soil unit in the SPT-based evaluation. To model post-construction site conditions, the additional embankment fill of the proposed flood protection system was modeled to account for the future in-situ stresses of the subsurface soils. The height of earthfill was based on the difference between the existing ground elevation of each CPT during the investigation and the long-term design crest elevation of +13.0 feet NAVD88 (Sherwood, 2026). The fill unit weight was assumed to be 125 pounds per cubic foot (pcf) based on the recommended presumptive material properties of replacement levee fill found in *Table 5-5* of the *Urban Levee Geotechnical Evaluations Project, Guidance Document for Geotechnical Analyses* (URS, 2015). For GS26-CPT-1, the existing ground elevation was greater than +10 feet NAVD88, so no additional fill was added.

As an additional check, and to incorporate laboratory fines content into the liquefaction evaluation, an SPT-based liquefaction evaluation was also conducted. Results of the SPT-based method were compared to the CPT-based method to verify reasonableness of the results of the CPT-based method. As part of this evaluation, we considered liquefaction potential of fine-grained soils using the method developed by Bray and Sancio (2006) and generally concluded that the fine-grained material was not susceptible to liquefaction. See Attachment B for the results of the Bray and Sancio (2006) method.

For the SPT-based soil liquefaction evaluation, the Idriss and Boulanger (2008) method was used. Similar to the CPT-based evaluation, the same seismic design parameters and groundwater levels were input along with the additional levee fill. A hammer energy ratio was also applied to estimate an energy-corrected SPT blow count (N_{60}); this hammer energy ratio was based on the equipment-specific calibration of the drilling rig used for the investigation (Geosyntec, 2026a) and other blow count correction factors were based on Idriss and Boulanger (2008). To calculate effective stresses, the soil profile from the investigation boring logs was used along with estimated unit weights of each soil unit based on interpretation of Geosyntec's geotechnical investigation (Geosyntec, 2026a). The assumed soil units and corresponding unit weight can be found in Table 3. Fines content of granular soils was based on the laboratory results (Geosyntec, 2026a) or assumed based on visual classification if both coarse and fine fractions were observed during drilling, but the fines content was not measured. The results of the CLiq and SPT evaluations are provided in Attachment A.

In coarse-grained soil layers where the CPT-based and SPT-based soil liquefaction evaluations differed in defining potentially liquefiable layers, the fines contents in the CPT-based approach were adjusted to match the laboratory results used in the SPT-based approach. Table 4 shows the adjusted fines contents for the CPT and SPT comparison. For each of these sand layers, the SPT-based approach did not estimate the layer to be potentially liquefiable, which the CPT-based approach did. In general, the adjustment of fines contents had minimal effect on the depths of liquefiable layers and the potential consequences of liquefaction which will be discussed in Section 6. The SPT-based approach may have estimated the layers as not potentially liquefiable due to the sample being taken in a denser part of the coarse-grained layer. This is reflected in the fluctuating factors of safety seen within the coarse-grained layers in the CPT-based evaluation. The results of the adjusted fines content in the CPT-based approach are included in Attachment A.

Based on evaluation of the stratigraphy from current and historical investigations, both liquefaction susceptibility methods, and both seismic loading scenarios, the depths and extents of potentially liquefiable soil layers identified are shown on Figure 2. Based on this review, the lateral and vertical extent of soils that are potentially susceptible to liquefaction does not vary significantly between the OBE and MDE events.

Table 3: Soil Unit Weights

Soil Unit	Total Unit Weight (pcf)
Earthfill	125
Existing Fill	120
Young Bay Mud [YBM]/YBM (Marsh)	115
Old Bay Clay [OBC]	120
Coarse-Grained Soil 1/ Coarse-Grained Soil 2	125

Table 4: Cliq Adjusted Fines Contents for Comparison to SPT-Based Evaluation

CPT/Boring Pairing	Layer Depth (ft)	Fines Content (%)
GS26-CPT-1/GS26-B1	30 to 35	29
GS26-CPT-5/GS26-B2	35 to 40	18
GS26-CPT-5/GS26-B2	40 to 45	5

6.1 Post-Liquefaction Reconsolidation Settlement

Post-liquefaction reconsolidation vertical settlement was estimated using CLiq using the Idriss and Boulanger (2008) method which was adapted from Ishihara and Yoshimine (1992). In this method, post-liquefaction volumetric strain is correlated to maximum shear strains [1].

For both loading scenarios, vertical settlements due to post-liquefaction reconsolidation are shown in Attachment A and were found to range between approximately 0 and 6 inches. Table 5 summarizes the estimated post-liquefaction vertical settlements estimated at each CPT and for each loading scenario. As shown in Attachment A and Table 5, the estimated post-liquefaction reconsolidation settlement does not vary significantly between the OBE and MDE events.

Table 5: Estimated Vertical Settlement Due to Post-Liquefaction Reconsolidation

CPT	Seismic Settlement (in)	
	OBE	MDE
GS26-CPT-1	1.1	1.1
GS26-CPT-1 (with adjusted fines content)	1.1	1.1
GS26-CPT-2	4.1	4.3
GS26-CPT-3	2.1	2.2
GS26-CPT-4	4.4	4.7
GS26-CPT-5	2.8	3.1
GS26-CPT-5 (with adjusted fines content)	2.5	2.7
GS26-CPT-6	3.4	3.5
GS26-CPT-7	0.3	0.3
GS26-CPT-8	0.7	0.7
GS26-CPT-9	5.5	5.6

[1] This method estimates vertical settlement due to post-liquefaction reconsolidation settlement only. Shear strains and slope displacement due to seismic loading and post-liquefaction strength loss are considered as part of a separate calculation package.

GS26-CPT-10	0.3	0.3
GS26-CPT-11	1.4	1.5
GS26-CPT-12	5.5	5.6
GS26-CPT-13	3.2	3.3
GS26-CPT-14	1.1	1.1
GS26-CPT-15	0.3	0.3
GS26-CPT-16	5.2	5.2
GS26-CPT-17	1.1	1.1
GS26-CPT-18	4.6	4.6

6.2 Lateral Displacement

Idriss and Boulanger (2008) evaluates lateral spreading displacement "by integrating maximum shear strains versus depth" which is adapted from Zhang et al. (2004). Based on the aerial topographic survey provided by Mark Thomas (Job Number 25-B12676 Dated May 13, 2026) of the levee alignment and of the marsh on the bay side of the proposed flood control structure, the ground surface to the bay side of the proposed control line ranges from flat to a maximum slope of 0.5% towards the San Francsico Bay. The lateral displacement potential was evaluated using CLiq and is shown in Attachment A. Table 6 summarizes the estimates of lateral displacements at each CPT for a sloping ground of 0.5% and for each seismic loading scenario. This evaluation suggests lateral displacements ranging from less than 1 inch to approximately 10 inches for both the OBE and MDE events.

Table 6: Estimated Lateral Displacements

CPT	**Lateral Displacement (in)**	
	OBE	**MDE**
GS26-CPT-1	1.9	2.0
GS26-CPT-1 (with adjusted fines content)	2.0	2.0
GS26-CPT-2	6.1	6.4
GS26-CPT-3	3.3	3.4
GS26-CPT-4	6.2	6.4
GS26-CPT-5	3.7	3.9

GS26-CPT-5 (with adjusted fines content)	3.4	3.7
GS26-CPT-6	5.4	5.5
GS26-CPT-7	0.6	0.6
GS26-CPT-8	1.6	1.6
GS26-CPT-9	8.3	8.3
GS26-CPT-10	0.7	0.7
GS26-CPT-11	2.0	2.1
GS26-CPT-12	9.2	9.4
GS26-CPT-13	5.3	5.4
GS26-CPT-14	2.0	2.1
GS26-CPT-15	0.6	0.6
GS26-CPT-16	10.0	10.0
GS26-CPT-17	2.4	2.4
GS26-CPT-18	8.0	8.0

6.3 Ground Surface Manifestation

Idriss and Boulanger (2008) presents a method to estimate potential manifestation of ground damage due to liquefaction based on the relative thickness of liquefiable and non-liquefiable layers, adapted from Ishihara (1985). The OBE estimated PGA of 0.51g was used in this evaluation, as the maximum PGA used in the Ishihara (1985) method is 0.4g to 0.5g. For an MDE scenario, surface manifestations can be assumed to be similar to or greater than that of the OBE scenario. See Attachment C for results of the Ishihara (1985) evaluation.

The thickness of the non-liquefiable surface layer ranges from 7 to 44 feet and the thickness of the underlying liquefiable layer ranges from 0 to 33 feet. Based on the thickness of the non-liquefiable surface layer and corresponding thickness of liquefiable soil at each investigation location, there is potential for liquefaction-induced ground damage such as sand boils, ejecta, cracking, or other surface manifestations of liquefaction to occur approximately between the following stations:

- STA 3+00 to STA 10+00
- STA 32+00 to STA 35+00
- STA 43+00 to STA 44+00

7. CONCLUSION

Cohesionless soils along the levee alignment have the potential to undergo liquefaction under both the OBE and MDE design earthquake scenarios as shown on Figure 2. This liquefaction may result in seismic settlement ranging from 0 to 6 inches (Table 5), lateral displacement of the embankment of up to 10 inches with larger relative displacements predicted North of Runnymede (Table 6), and ground surface manifestation (e.g., sand boils, cracking, etc.) at the surface (between STA 3+00 - 10+00, STA 32+00 – 35+00, and STA 43+00 – 44+00).

Lateral and vertical slope deformation may also occur due to seismic internal forces and/or due to the reduction in shear strength in liquefiable soil deposits immediately following an earthquake which could result in post-earthquake instability. Estimation of these effects is outside the scope of this calculation package but will be discussed in a separate slope stability and seismic deformation calculation package.

8. REFERENCES

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FIGURES

LEGEND

- INVESTIGATION LOCATION
- LEVEE DESIGN 5' CONTOUR
- EXISTING GROUND ELEVATION 5' CONTOUR
- EXISTING GROUND ELEVATION 1' CONTOUR
- SAFER BAY CONTROL LINE

NOTES

CPT = CONE PENETRATION TEST
B = BORING
GS26- = GEOSYNTEC EXPLORATORY BORINGS (GEOSYNTEC 2026)
A- = ARUP EXPLORATORY BORINGS (ARUP 2021)
C-/B- = HDR EXPLORATORY BORINGS (HDR 2024)

1. TOPOGRAPHIC MAP BY MARK THOMAS, DATED MAY 2026 USING NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
2. GEOSYNTEC EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS, AS DESCRIBED IN THE GEOTECHNICAL DATA REPORT (GDR) (GEOSYNTEC, 2026).
3. HISTORICAL EXPLORATION LOCATIONS ARE BASED ON THE LOCATIONS SHOWN ON THE PRELIMINARY GEOTECHNICAL REPORT FOR SOUTH OF BAY ROAD (HDR 2024) AND THE GEOTECHNICAL DATA REPORT SFPUC REVANSWOOD FACILITY TO BEDWELL BAYFRONT PARK (HDR 2025).

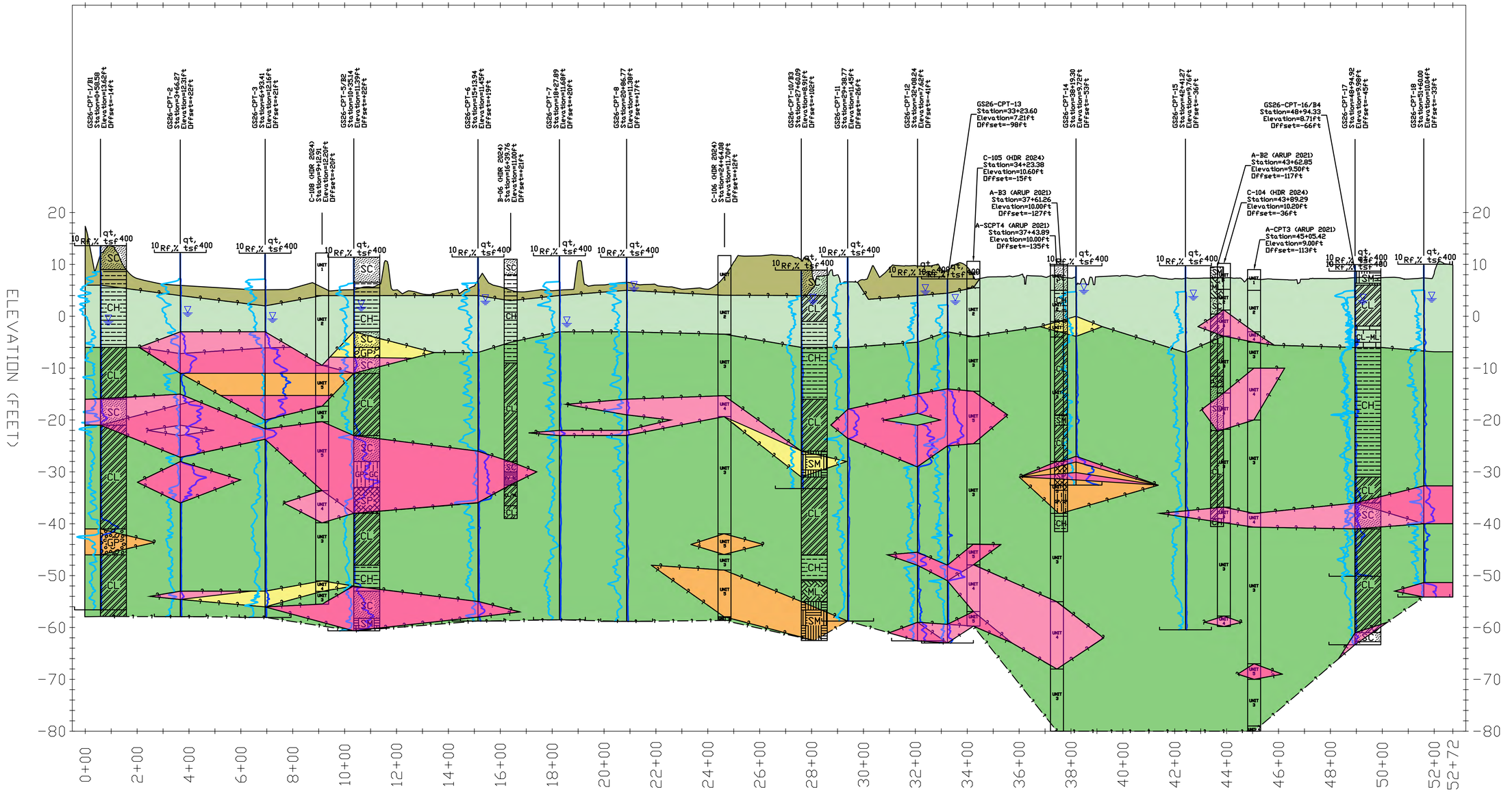
INVESTIGATION LOCATIONS
SAFER BAY PROJECT
SOUTH OF BAY ROAD
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | JUNE 2026

FIGURE 1

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WG3807 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS



GEOLOGIC CROSS SECTION

VERTICAL EXAGGERATION: 20

LEGEND

- UNIT 1 (FILL) - SANDY CLAY (CL/CH) TO SILTY SAND (SC/SM), SOFT TO MEDIUM STIFF
- UNIT 2 (YOUNG BAY MUD [YBM]) - VERY SOFT TO MEDIUM STIFF
- UNIT 3 (OLD BAY CLAY [OBC]) - MEDIUM STIFF TO VERY STIFF
- UNIT 4 (COARSE-GRAINED SOIL 1) - LOOSE TO MEDIUM DENSE
- UNIT 5 (COARSE-GRAINED SOIL 2) - MEDIUM DENSE TO VERY DENSE
- GROUNDWATER LEVEL ENCOUNTERED DURING GEOSYNTEC INVESTIGATION (NOTE 4)
- POTENTIALLY LIQUEFIABLE LAYERS

NOTES

CPT = CONE PENETRATION TEST
B = BORING
OFFSET = DISTANCE FROM SAFER BAY CONTROL LINE TO INVESTIGATION LOCATION (BAY SIDE IS POSITIVE AND LAND SIDE IS NEGATIVE).

1. STATIONING BASED ON ALIGNMENT OF SAFER BAY PROJECT CONTROL LINE PROVIDED BY SHERWOOD ENGINEERS ON MAY 2026.
2. GEOSYNTEC EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS, AS DESCRIBED IN THE GEOTECHNICAL DATA REPORT (GDR) (GEOSYNTEC, 2026).
3. TOPOGRAPHIC SURVEY BY MARK THOMAS, DATED MAY 2026 WAS USED FOR THIS CROSS SECTION. HORIZONTAL DATUM USED WAS NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND THE VERTICAL DATUM USED WAS THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
4. GROUNDWATER ELEVATIONS ARE BASED ON THE ESTIMATED HYDROSTATIC PORE PRESSURE AT THE END OF CPT PORE PRESSURE DISSIPATION TESTS TAKEN IN GRANULAR MATERIAL.
5. CPT DATA FROM HISTORICAL INVESTIGATIONS WERE INTERPRETED CONSISTENT WITH THE GEOSYNTEC CPTS AND USED TO EVALUATE THE DEPTHS AND THICKNESSES OF UNITS 1 THROUGH 5.

GEOLOGIC CROSS SECTION
SAFER BAY PROJECT
SOUTH OF BAY ROAD
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | JUNE 2026 | FIGURE 2

P:\CAD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\W3007 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

ATTACHMENT A
SPT and CPT-Based Liquefaction Analysis Outputs

Operating Basis Earthquake Ground Motion (OBE-GM) SPT and CPT-based Liquefaction Outputs

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: SAFER - East Palo Alto, South of Bay Road
Location: East Palo Alto, California

Project Number: WG3607
Prepared By: AK

Checked by: JM
Date: 5/21/2026

Boring: GS25-B-1
Date: 16-Jan-26
By: Geosyntec Consultants, Inc.

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Mud Rotary
Ground Elevation (ft)[2]: 13.62
Boring Coordinates[2]: N: 1994551.8, E: 6090246.2

$a_{max\ (ground\ surface)}$: 0.51 g[3]
Earthquake Magnitude: 7.5 [3]
MSF: 1.00 Idriss & Boulanger (2008)[4a]

Assumed depth to groundwater at time of earthquake (ft)[24]: 2.80
Assumed depth to groundwater at time of drilling (ft)[24]: 14.6

Potential Liquefaction: $FS_{liq} <$ 1.1

Depth	Elevation	Soil Unit Weight	Soil Unit	USCS Class	Borehole Diameter	Sample Type[26]	ER[5]	N_{field}	σ_v	σ_v', during drilling	σ_v', during EQ[24]	N_{field} Correction Factors					N_{60}	Fines Content	Fines Content Method	[11]	[12]	$(N_1)_{60}$[13]	$\Delta(N_1)_{60}$[14]	$(N_1)_{60cs}$[15]	[4b]	[4c]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]
												C_{rod}[6]	C_{sample}[7]	C_b[8]	C_S[9]	C_{SPT}[10]				m	C_N				MSF_{max}	MSF	α	β	r_d	C_σ	K_σ	$CRR_{M7.5,\sigma'vc}$	$CSR_{M7.5,\sigma'vc}$	NL/L
(ft)	(ft)	(pcf)			(mm)		(%)	(blows/ft)	(psf)	(psf)	(psf)						(blows/ft)	%				(blows/ft)		(blows/ft)										
0	13.6																																	
30.0	-16.4	125	Clayey SAND	SC	150.0	SPT	87	26	3545	2583	1848	0.95	1.45	1.05	1.00	1.00	38	29	Lab	0.298	0.94	35	5.3	40	2.20	1.00	-0.60	0.07	0.91	0.300	1.041	4.13	0.555	NL
55.0	-41.4	125	GRAVEL w/ Sand	GP	150.0	SPT	87	17	6570	4048	3313	1.00	1.45	1.05	1.00	1.00	26	0	Est	0.449	0.75	19	0.0	19	1.45	1.00	-1.32	0.15	0.80	0.128	0.942	0.19	0.556	L

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12"

[2] Horizontal coordinates are based on California State Plan, Zone III. The vertical datum is NAVD88.

[3] a_{max} and earthquake magnitude based on 2008 USGS data. See Seismic Parameter calculation package for details.

[4a] MSF=6.9exp(-M/4)-0.058≤1.8 is magnitude scaling factor (Equation 51-52, I&B 2008)

[4b] MSF_{max}=1.09+($(N_1)_{60cs}$/31.5)^2≤2.2 is maximum magnitude scaling factor (Equation 2.21, I&B 2014)

[4c] MSF=1+(MSF_{max}-1)[8.64exp(-M/4)-1.325] is magnitude scaling factor (Equation 2.19, I&B 2014)

[5] Hammer Energy Ratio based on site-specific calibration of drill rig used for project [Pitcher Drilling; East Palo Alto, California], for SPT and MOD-CAL samples

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] $C_{sampler}$, accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. C_{SPT} is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] m=0.784-0.0768sqrt($(N_1)_{60cs}$)≥0.264 is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] C_N=$(P_a/\sigma'_v)^m$≤1.7 accounts for effective overburden stress (Equation 33, I&B 2008)

[13] $(N_1)_{60}$=N_{60}*C_N is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] $\Delta(N_1)_{60}$=exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2] represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)

[15] $(N_1)_{60cs}$=$(N_1)_{60}$ + $\Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] α(z) = -1.012-1.126sin((z/11.73)+5.133) in which z is depth in meters (Equation 23, I&B 2008)

[17] β(z) = 0.106+0.118sin((z/11.28)+5.142) in which z is depth in meters (Equation 24, I&B 2008)

[18] r_d=exp[α(z)+β(z)M] is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] C_σ=1/(18.9-2.55sqrt($(N_1)_{60cs}$))≤0.3 is the coefficient for K_σ (Equation 56, I&B 2008)

[20] K_σ = 1-$C_\sigma$ln(σ'_{vc}/P_a)≤1.1 is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'vc}$=0.65(a_{max}/g)(σ_v/σ'_v)r_d(1/MSF)(1/K_σ) is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable, defined as factor of safety against the triggering of liquefaction (FS_{liq}) of 1.3 or below

[24] Groundwater is conservatively assumed to be at a depth observed during the field investigation (for blow count correction), and at Mean Higher-High Water Level (MHHW) + 3.5ft Sea-Level Rise (SLR) during an earthquake event.

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project:	SAFER - East Palo Alto, South of Bay Road	Project Number:	WG3607	Checked by:	JM
Location:	East Palo Alto, California	Prepared By:	AK	Date:	5/21/2026

Boring:	GS25-B-2	Hammer Type:	Automatic 140 lb./30-in.	$a_{max\ (ground\ surface)}$:	0.51 g[3]	Assumed depth to groundwater at time of earthquake (ft)[24]:	0.60
Date:	16-Jan-26	Drilling Method:	Mud Rotary	Earthquake Magnitude:	7.5 [3]	Assumed depth to groundwater at time of drilling (ft)[24]:	9.7
By:	Geosyntec Consultants, Inc.	Ground Elevation (ft)[2]:	11.39	MSF:	1.00 Idriss & Boulanger (2008)[4a]		
		Boring Coordinates[2]:	N: 1995502.5, E: 6090021.0				

Potential Liquefaction: FS_{liq} < 1.1

Depth	Elevation	Soil Unit Weight	Soil Unit	USCS Class	Borehole Diameter	Sample Type[20]	ER[5]	N_{field}	σ_v	σ_v', during drilling	σ_v', during EQ[24]	N_{field} Correction Factors					N_{60}	Fines Content	Fines Content Method	[11]	[12]	$(N_1)_{60}$[13]	$\Delta(N_1)_{60}$[14]	$(N_1)_{60cs}$[15]	[4b]	[4c]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]
(ft)	(ft)	(pcf)			(mm)		(%)	(blows/ft)	(psf)	(psf)	(psf)	C_{rod}[6]	C_{energy}[7]	C_b[8]	C_S[9]	C_{SPT}[10]	(blows/ft)	%		m	C_N	(blows/ft)		(blows/ft)	MSF_{max}	MSF	α	β	r_d	C_σ	K_σ	$CRR_{M7.5,\sigma'vc}$	$CSR_{M7.5,\sigma'vc}$	NL/L
0	11.4																																
20.0	-8.6	125	Clayey SAND	SC	150.0	SPT	87	10	2546	1901	1335	0.95	1.45	1.05	1.00	1.00	14	32	Lab	0.441	1.05	15	5.4	20	1.49	1.00	-0.35	0.04	0.95	0.133	1.061	0.21	0.564	**L**
35.0	-23.6	125	Clayey SAND w/ Gravel	SC	150.0	SPT	87	28	4346	2765	2199	1.00	1.45	1.05	1.00	1.00	43	18	Lab	0.275	0.93	40	4.1	44	2.20	1.00	-0.74	0.08	0.89	0.300	0.988	19.43	0.588	**NL**
40.0	-28.6	125	Poorly graded GRAVEL w/ Clay	GP	150.0	SPT	87	45	4986	3093	2527	1.00	1.45	1.05	1.00	1.00	69	5	Lab	0.264	0.90	62	0.0	62	2.20	1.00	-0.89	0.10	0.86	0.300	0.947	50.00	0.597	**NL**
45.0	-33.6	125	SAND with Gravel	SP	150.0	MOD-CAL	87	49	5636	3431	2865	1.00	1.45	1.05	1.00	0.60	45	0	Est	0.304	0.86	39	0.0	39	2.20	1.00	-1.03	0.11	0.84	0.300	0.909	3.02	0.604	**NL**
70.0	-58.6	125	Silty SAND	SM	150.0	SPT	87	54	8746	4981	4415	1.00	1.45	1.05	1.00	1.00	82	12	Est	0.264	0.80	66	2.1	68	2.20	1.00	-1.71	0.19	0.73	0.300	0.779	50.00	0.617	**NL**

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes* , EERI Monograph MNO-12"

[2] Horizontal coordinates are based on California State Plan, Zone III. The vertical datum is NAVD88.

[3] a_{max} and earthquake magnitude based on 2008 USGS data. See Seismic Parameter calculation package for details.

[4a] MSF=6.9exp(-M/4)-0.058≤1.8 is magnitude scaling factor (Equation 51-52, I&B 2008)

[4b] MSF_{max}=1.09+(((N_1)$_{60cs}$/31.5)^2≤2.2 is maximum magnitude scaling factor (Equation 2.21, I&B 2014)

[4c] MSF=1+(MSF_{max}-1)[8.64exp(-M/4)-1.325] is magnitude scaling factor (Equation 2.19, I&B 2014)

[5] Hammer Energy Ratio based on site-specific calibration of drill rig used for project [Pitcher Drilling: East Palo Alto, California], for SPT and MOD-CAL samples

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] C_{energy} accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. C_{SPT} is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] m=0.784-0.0768sqrt((N_1)$_{60cs}$)≥0.264 is iteratively calculated until (N_1)$_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] C_N=(P_a/σ'_v)^m≤1.7 accounts for effective overburden stress (Equation 33, I&B 2008)

[13] (N_1)$_{60}$=N_{60}*C_N is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] Δ(N_1)$_{60}$=exp{1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2} represents the change in (N_1)$_{60}$ with fines content (Equation 76, I&B 2008)

[15] (N_1)$_{60cs}$=(N_1)$_{60}$ + Δ(N_1)$_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] α(z) = -1.012-1.126sin((z/11.73)+5.133) in which z is depth in meters (Equation 23, I&B 2008)

[17] β(z) = 0.106+0.118sin((z/11.28)+5.142) in which z is depth in meters (Equation 24, I&B 2008)

[18] r_d=exp[α(z)+β(z)M] is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] C_σ=1/(18.9-2.55sqrt((N_1)$_{60cs}$))≤0.3 is the coefficient for K_σ (Equation 56, I&B 2008)

[20] K_σ = 1-$C_\sigma$ln(σ'_{vc}/P_a)≤1.1 is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'vc}$=0.65(a_{max}/g)(σ_v/σ'_v)r_d(1/MSF)(1/K_σ) is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable, defined as factor of safety against the triggering of liquefaction (FS_{liq}) of 1.3 or below

[24] Groundwater is conservatively assumed to be at a depth observed during the field investigation (for blow count correction), and at Mean Higher-High Water Level (MHHW) + 3.5ft Sea-Level Rise (SLR) during an earthquake event.

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: SAFER - East Palo Alto, South of Bay Road
Location: East Palo Alto, California

Project Number: WG3607
Prepared By: AK

Checked by: JM
Date: 5/21/2026

Boring: GS25-B-3
Date: 16-Jan-26
By: Geosyntec Consultants, Inc.

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Mud Rotary
Ground Elevation (ft)[2]: 8.90
Boring Coordinates[2]: N: 1997202.5, E: 6089734.2

$a_{max\ (ground\ surface)}$: 0.51 g[3]
Earthquake Magnitude: 7.5 [3]
MSF: 1.00 Idriss & Boulanger (2008)[4a]

Assumed depth to groundwater at time of earthquake (ft)[24]: -1.90
Assumed depth to groundwater at time of drilling (ft)[24]: 6.0

Potential Liquefaction: $FS_{liq} <$ 1.1

Depth	Elevation	Soil Unit Weight	Soil Unit	USCS Class	Borehole Diameter	Sample Type[26]	ER[5]	N_{field}	σ_v	$\sigma_{v,drilling}'$ during drilling	$\sigma_{v,EQ}'$ during EQ[24]	N_{field} Correction Factors					N_{60}	Fines Content	Fines Content Method	[11]	[12]	$(N_1)_{60}$[13]	$\Delta(N_1)_{60}$[14]	$(N_1)_{60cs}$[15]	[4b]	[4c]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]
(ft)	(ft)	(pcf)			(mm)		(%)	(blows/ft)	(psf)	(psf)	(psf)	C_{rod}[6]	C_{enerpr}[7]	C_b[8]	C_S[9]	C_{SPT}[10]	(blows/ft)	%		m	C_N	(blows/ft)		(blows/ft)	MSF_{max}	MSF	α	β	r_d	C_σ	K_σ	$CRR_{M7.5,\sigma'vc}$	$CSR_{M7.5,\sigma'vc}$	NL/L
0	8.9																																
65.0	-56.1	125	Silty SAND	SM	150.0	SPT	87	31	8267	4586	4092	1.00	1.45	1.05	1.00	1.00	47	35	Lab	0.286	0.80	38	5.5	42	2.20	1.00	-1.59	0.17	0.75	0.300	0.802	8.41	0.628	**NL**
70.0	-61.1	125	Silty SAND	SM	150.0	SPT	87	24	8892	4899	4405	1.00	1.45	1.05	1.00	1.00	37	35	Lab	0.350	0.75	27	5.5	32	2.12	1.00	-1.71	0.19	0.73	0.223	0.836	0.64	0.586	**L**

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12"
[2] Horizontal coordinates are based on California State Plan, Zone III. The vertical datum is NAVD88.
[3] a_{max} and earthquake magnitude based on 2008 USGS data. See Seismic Parameter calculation package for details.
[4a] MSF=6.9exp(-M/4)-0.058≤1.8 is magnitude scaling factor (Equation 51-52, I&B 2008)
[4b] MSF_{max}=1.09+($(N_1)_{60cs}$/31.5)^2≤2.2 is maximum magnitude scaling factor (Equation 2.21, I&B 2014)
[4c] MSF=1+(MSF_{max}-1)[8.64exp(-M/4)-1.325] is magnitude scaling factor (Equation 2.19, I&B 2014)
[5] Hammer Energy Ratio based on site-specific calibration of drill rig used for project [Pitcher Drilling; East Palo Alto, California], for SPT and MOD-CAL samples
[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)
[7] C_{enerpr} accounts for rod energy delivered to sampler (Table 3, I&B 2008)
[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)
[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)
[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. C_{SPT} is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)
[11] m=0.784-0.0768sqrt($(N_1)_{60cs}$)≥0.264 is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)
[12] C_N=$(P_a/\sigma'_v)^m$≤1.7 accounts for effective overburden stress (Equation 33, I&B 2008)
[13] $(N_1)_{60}$=N_{60}*C_N is the overburden corrected penetration resistance (Equation 31, I&B 2008)
[14] $\Delta(N_1)_{60}$=exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2] represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)
[15] $(N_1)_{60cs}$=$(N_1)_{60}$ + $\Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)
[16] α(z) = -1.012-1.126sin((z/11.73)+5.133) in which z is depth in meters (Equation 23, I&B 2008)
[17] β(z) = 0.106+0.118sin((z/11.28)+5.142) in which z is depth in meters (Equation 24, I&B 2008)
[18] r_d=exp[α(z)+β(z)M] is shear stress reduction coefficient (Equation 22, I&B 2008)
[19] C_σ=1/(18.9-2.55sqrt($(N_1)_{60cs}$))≤0.3 is the coefficient for K_σ (Equation 56, I&B 2008)
[20] K_σ = 1-$C_\sigma$ln(σ'_{vc}/P_a)≤1.1 is the overburden correction factor (Equation 54, I&B 2008)
[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)
[22] $CSR_{M7.5,\sigma'vc}$=0.65(a_{max}/g)(σ_v/σ_v')r_d(1/MSF)(1/K_σ) is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)
[23] NL = non-liquefiable; L = potentially liquefiable, defined as factor of safety against the triggering of liquefaction (FS_{liq}) of 1.3 or below
[24] Groundwater is conservatively assumed to be at a depth observed during the field investigation (for blow count correction), and at Mean Higher-High Water Level (MHHW) + 3.5ft Sea-Level Rise (SLR) during an earthquake event.

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: SAFER - East Palo Alto, South of Bay Road
Location: East Palo Alto, California

Project Number: WG3607
Prepared By: AK

Checked by: JM
Date: 5/21/2026

Boring: GS25-B-4
Date: 16-Jan-26
By: Geosyntec Consultants, Inc.

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Mud Rotary
Ground Elevation (ft)[2]: 8.71
Boring Coordinates[2]: N: 1999210.6, E: 6089580.6

$a_{max\ (ground\ surface)}$: 0.51 g[3]
Earthquake Magnitude: 7.5 [3]
MSF: 1.00 Idriss & Boulanger (2008)[4a]

Assumed depth to groundwater at time of earthquake (ft)[24]: -2.10
Assumed depth to groundwater at time of drilling (ft)[24]: 1.7

Potential Liquefaction: $FS_{liq} <$ 1.1

Depth	Elevation	Soil Unit Weight	Soil Unit	USCS Class	Borehole Diameter	Sample Type[26]	ER[5]	N_{field}	σ_v	$\sigma_{v,drilling}'$ during drilling	$\sigma_{v,EQ}'$ during EQ[24]	N_{field} Correction Factors					N_{60}	Fines Content	Fines Content Method	[11]	[12]	$(N_1)_{60}$[13]	$\Delta(N_1)_{60}$[14]	$(N_1)_{60cs}$[15]	[4b]	[4c]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]
(ft)	(ft)	(pcf)			(mm)		(%)	(blows/ft)	(psf)	(psf)	(psf)	C_{rod}[6]	C_{energy}[7]	C_b[8]	C_S[9]	C_{SPT}[10]	(blows/ft)	%		m	C_N	(blows/ft)		(blows/ft)	MSF_{max}	MSF	α	β	r_d	C_σ	K_σ	$CRR_{M7.5,\sigma'vc}$	$CSR_{M7.5,\sigma'vc}$	NL/L
0	8.7																																	
45.0	-36.3	125	Clayey SAND w/ Gravel	SC	150.0	MOD-CAL	87	11	5866	3164	2927	1.00	1.45	1.05	1.00	0.60	10	12	Est	0.541	0.80	8	2.1	10	1.19	1.00	-1.03	0.11	0.84	0.092	0.970	0.12	0.576	L
70.0	-61.3	125	Clayey SAND	SC	150.0	SPT	87	15	8876	4614	4377	1.00	1.45	1.05	1.00	1.00	23	12	Est	0.458	0.70	16	2.1	18	1.42	1.00	-1.71	0.19	0.73	0.124	0.910	0.18	0.541	L

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12"
[2] Horizontal coordinates are based on California State Plan, Zone III. The vertical datum is NAVD88.
[3] a_{max} and earthquake magnitude based on 2008 USGS data. See Seismic Parameter calculation package for details.
[4a] MSF=6.9exp(-M/4)-0.058≤1.8 is magnitude scaling factor (Equation 51-52, I&B 2008)
[4b] MSF_{max}=1.09+($(N_1)_{60cs}$/31.5)^2≤2.2 is maximum magnitude scaling factor (Equation 2.21, I&B 2014)
[4c] MSF=1+(MSF_{max}-1)[8.64exp(-M/4)-1.325] is magnitude scaling factor (Equation 2.19, I&B 2014)
[5] Hammer Energy Ratio based on site-specific calibration of drill rig used for project [Pitcher Drilling; East Palo Alto, California], for SPT and MOD-CAL samples
[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)
[7] C_{energy}, accounts for rod energy delivered to sampler (Table 3, I&B 2008)
[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)
[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)
[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. C_{SPT} is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)
[11] m=0.784-0.0768sqrt($(N_1)_{60cs}$)≥0.264 is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)
[12] C_N=(P_a/σ'_v)^m≤1.7 accounts for effective overburden stress (Equation 33, I&B 2008)
[13] $(N_1)_{60}$=N_{60}*C_N is the overburden corrected penetration resistance (Equation 31, I&B 2008)
[14] $\Delta(N_1)_{60}$=exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2] represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)
[15] $(N_1)_{60cs}$=$(N_1)_{60}$ + $\Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)
[16] α(z) = -1.012-1.126sin((z/11.73)+5.133) in which z is depth in meters (Equation 23, I&B 2008)
[17] β(z) = 0.106+0.118sin((z/11.28)+5.142) in which z is depth in meters (Equation 24, I&B 2008)
[18] r_d=exp[α(z)+β(z)M] is shear stress reduction coefficient (Equation 22, I&B 2008)
[19] C_σ=1/(18.9-2.55sqrt[$(N_1)_{60cs}$])≤0.3 is the coefficient for K_σ (Equation 56, I&B 2008)
[20] K_σ = 1-$C_\sigma$ln(σ'_{vc}/P_a)≤1.1 is the overburden correction factor (Equation 54, I&B 2008)
[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)
[22] $CSR_{M7.5,\sigma'vc}$=0.65(a_{max}/g)(σ_v/σ'_v)r_d(1/MSF)(1/K_σ) is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)
[23] NL = non-liquefiable; L = potentially liquefiable, defined as factor of safety against the triggering of liquefaction (FS_{liq}) of 1.3 or below
[24] Groundwater is conservatively assumed to be at a depth observed during the field investigation (for blow count correction), and at Mean Higher-High Water Level (MHHW) + 3.5ft Sea-Level Rise (SLR) during an earthquake event.

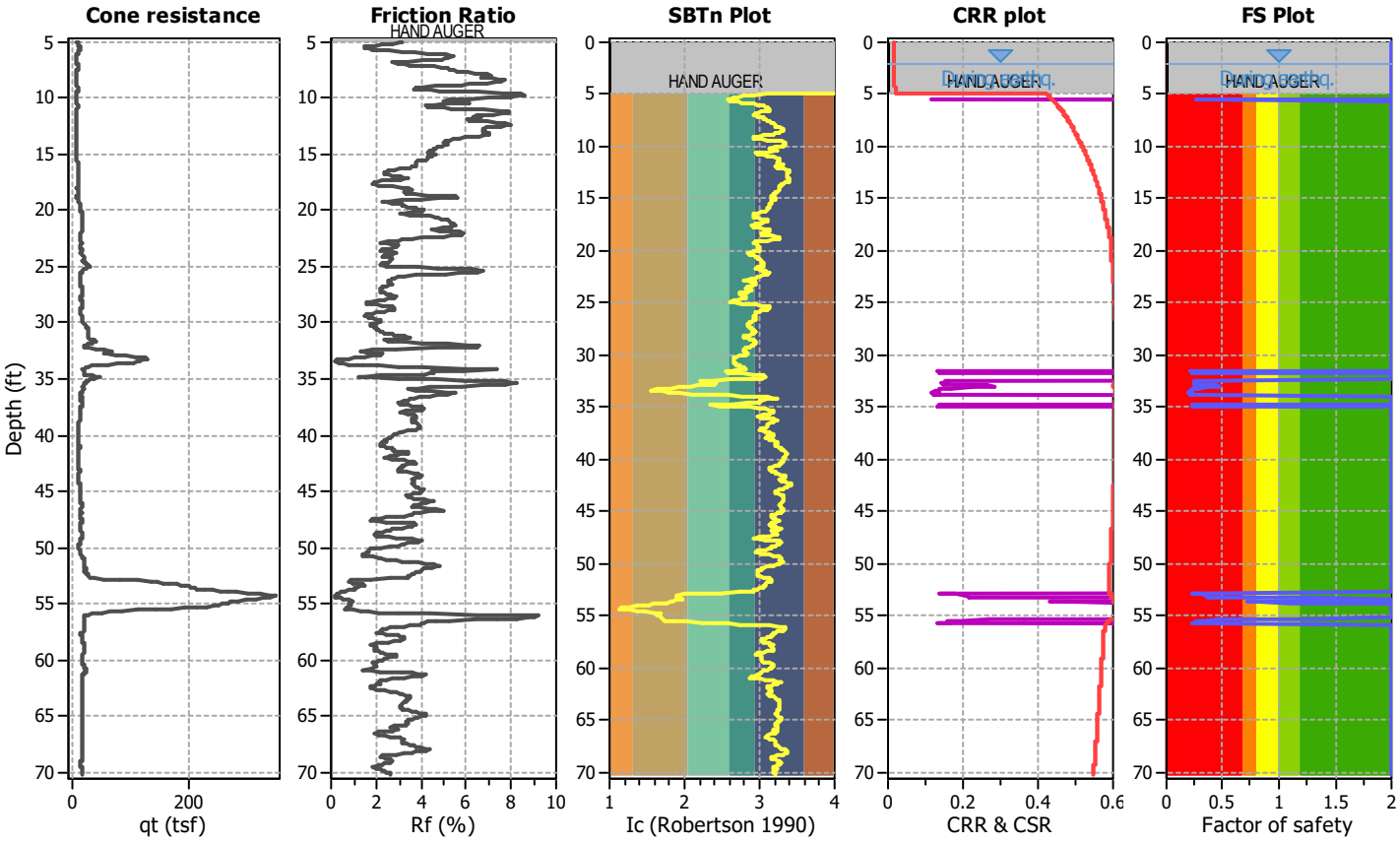
LIQUEFACTION ANALYSIS REPORT

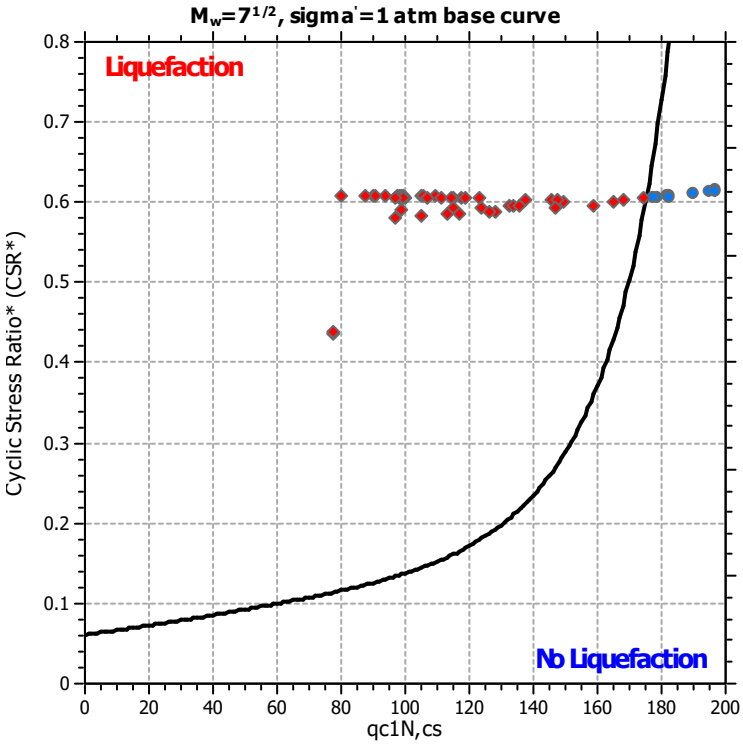
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

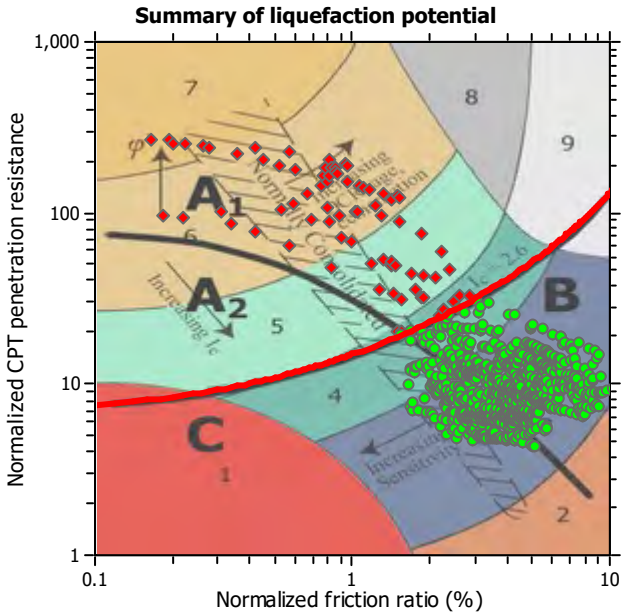
CPT file : GS26-CPT-01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.59 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	0.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

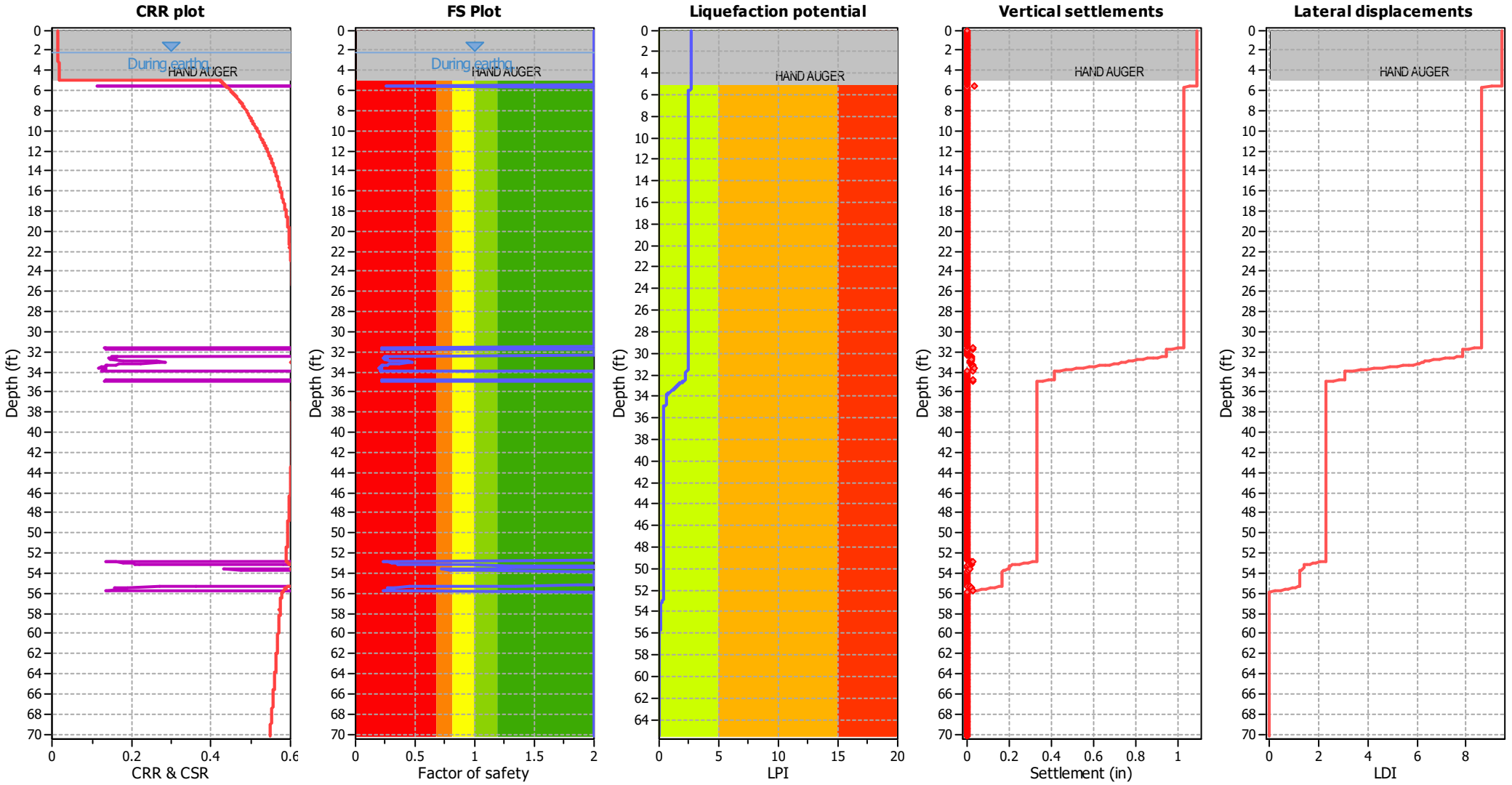






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

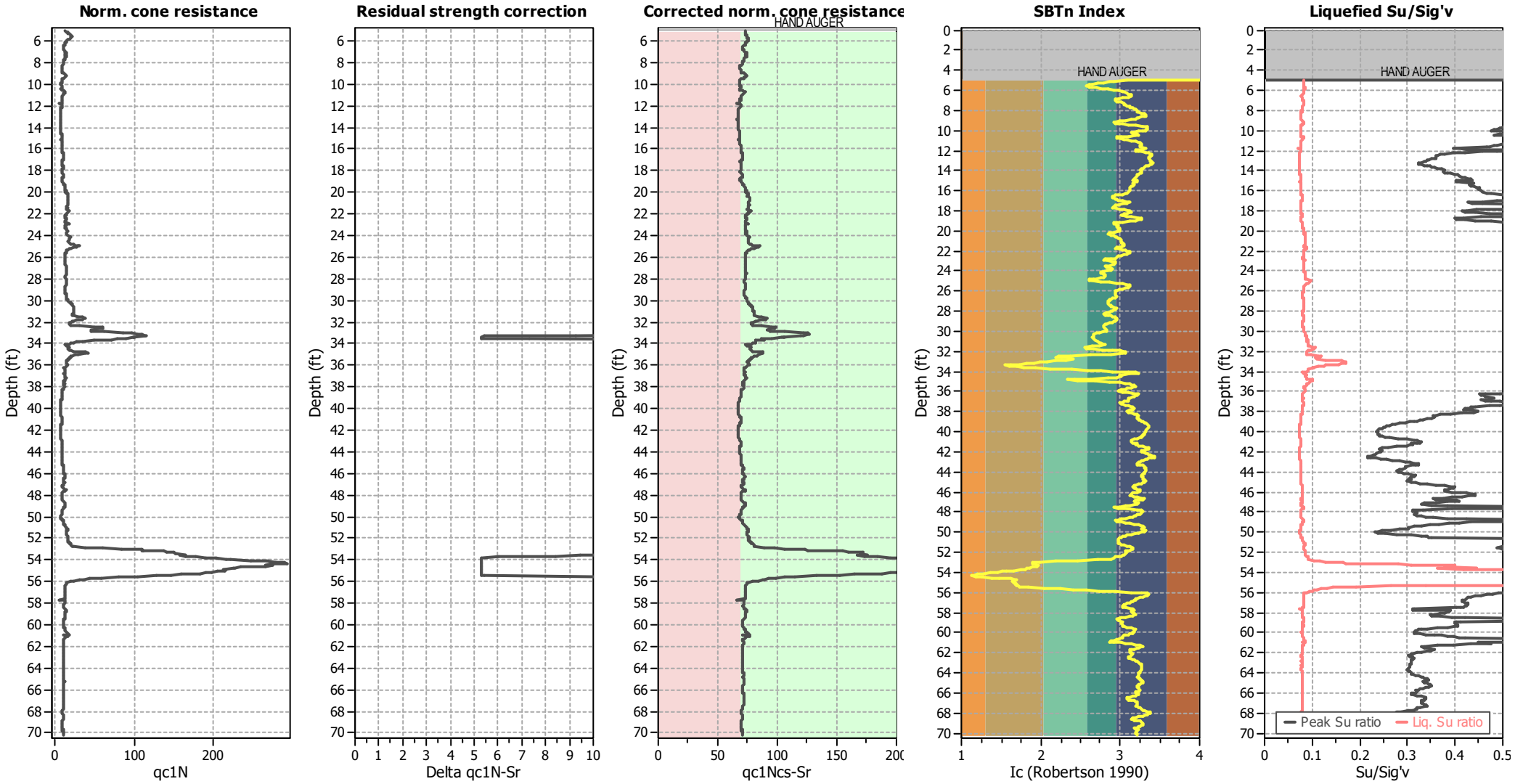
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

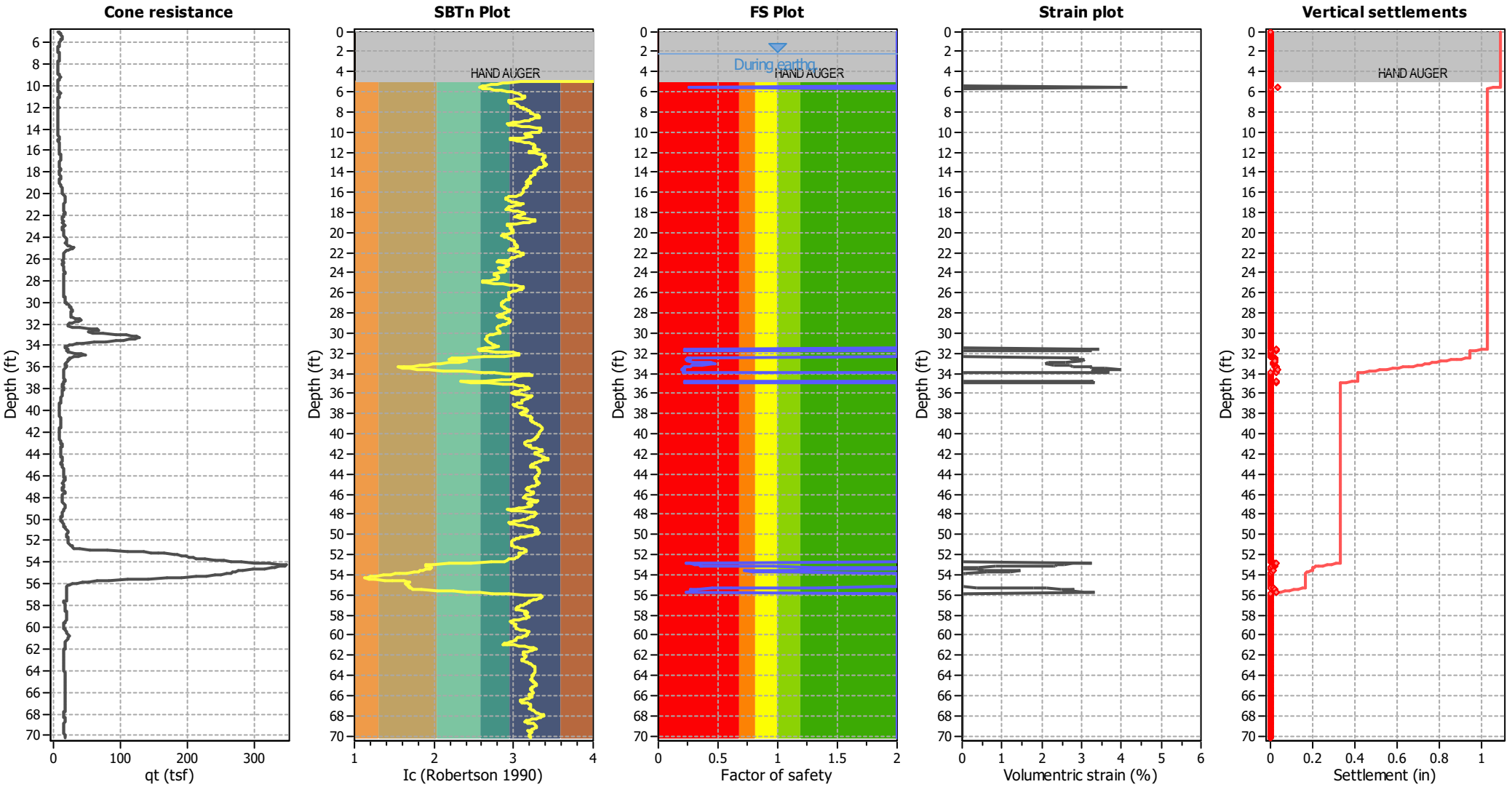
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

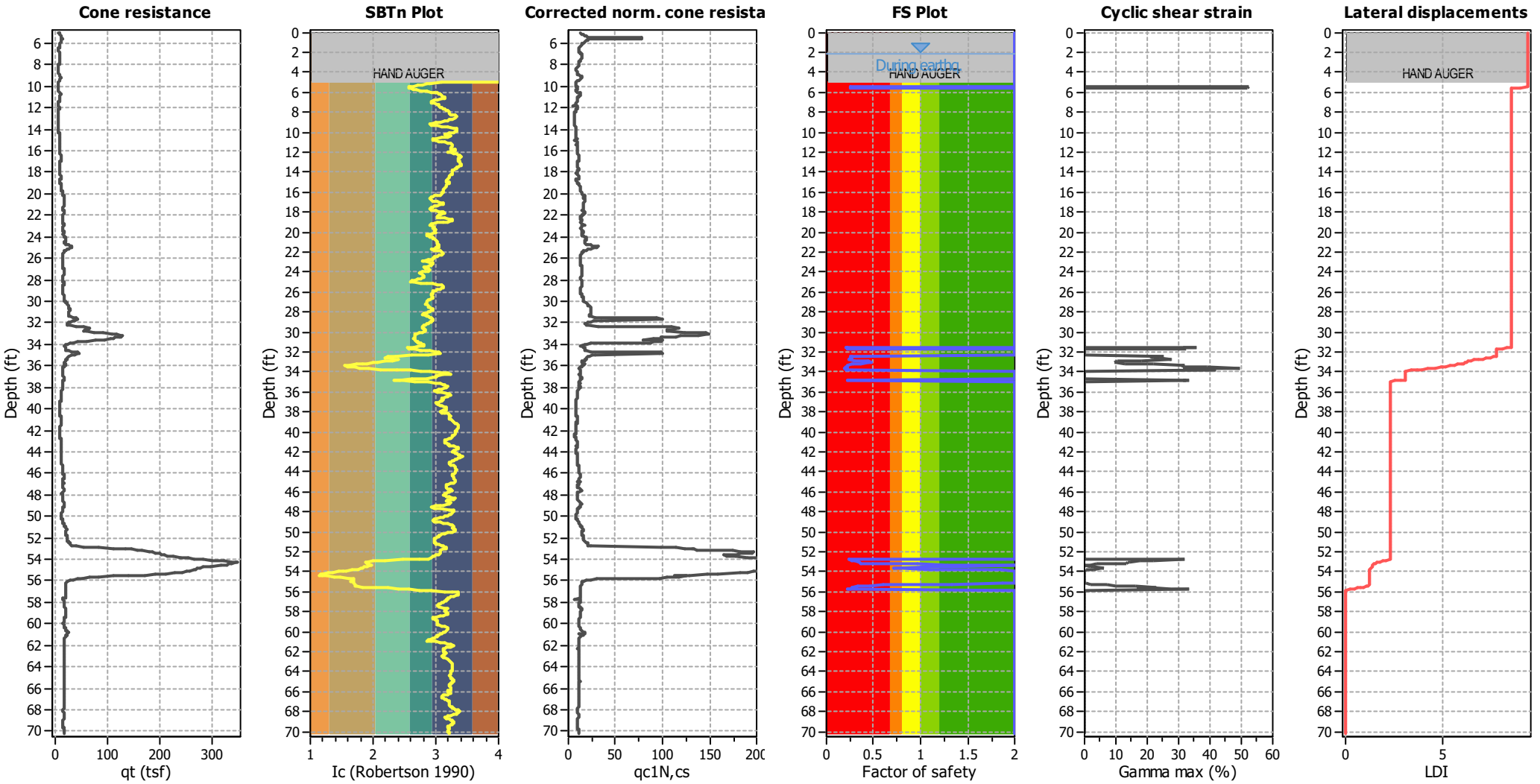
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

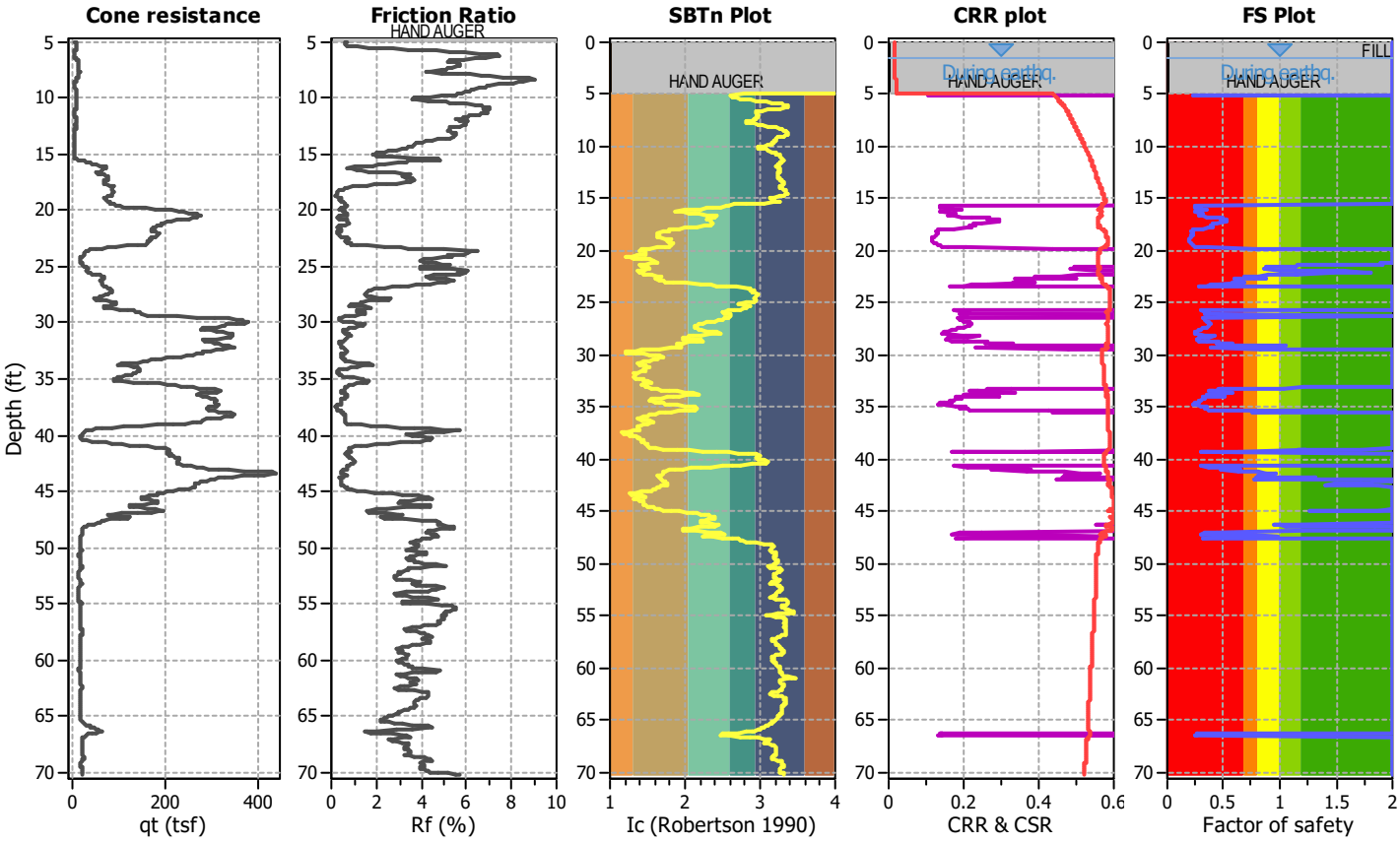
LIQUEFACTION ANALYSIS REPORT

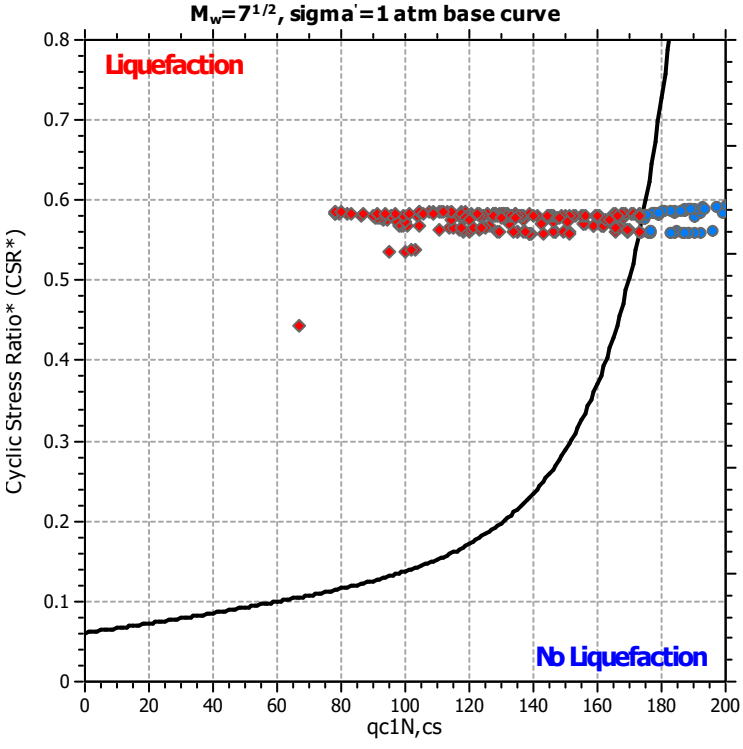
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

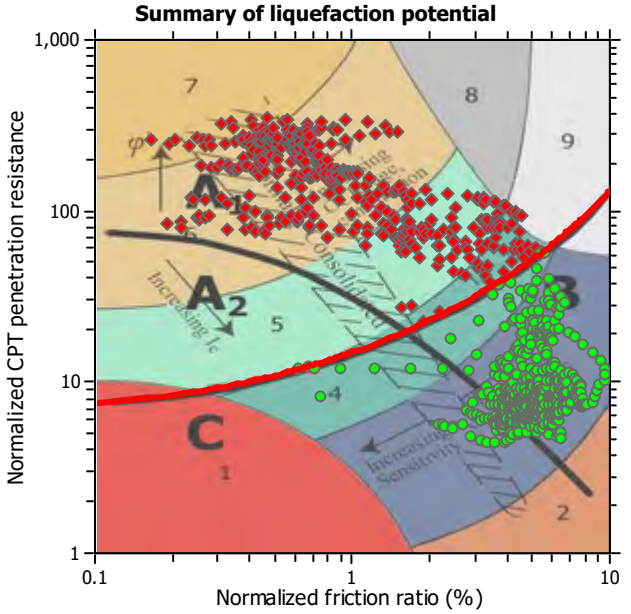
CPT file : GS26-CPT-02

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	11.71 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	0.70 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

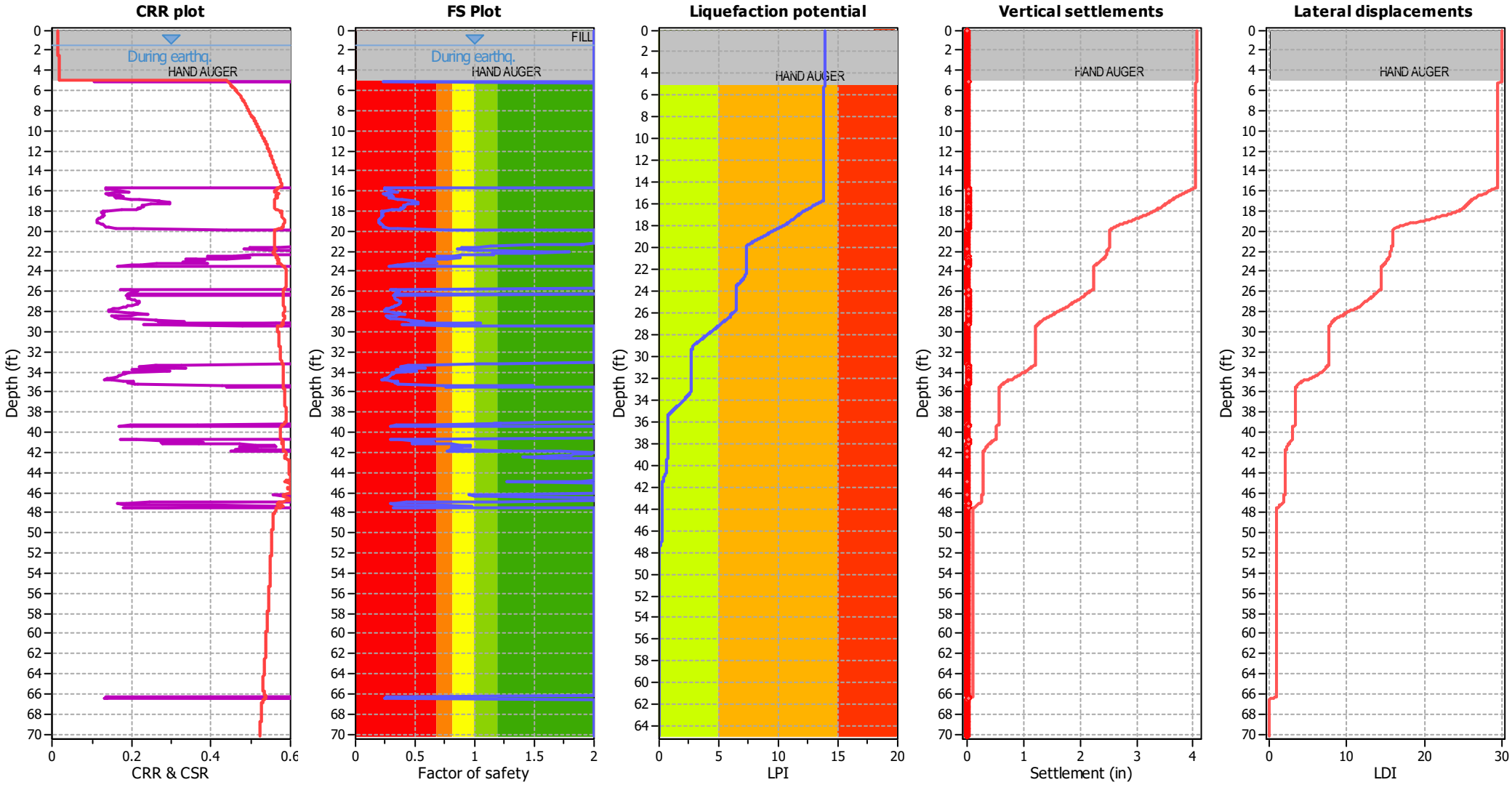






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	11.71 ft	Fill height:	0.70 ft	Limit depth:	N/A

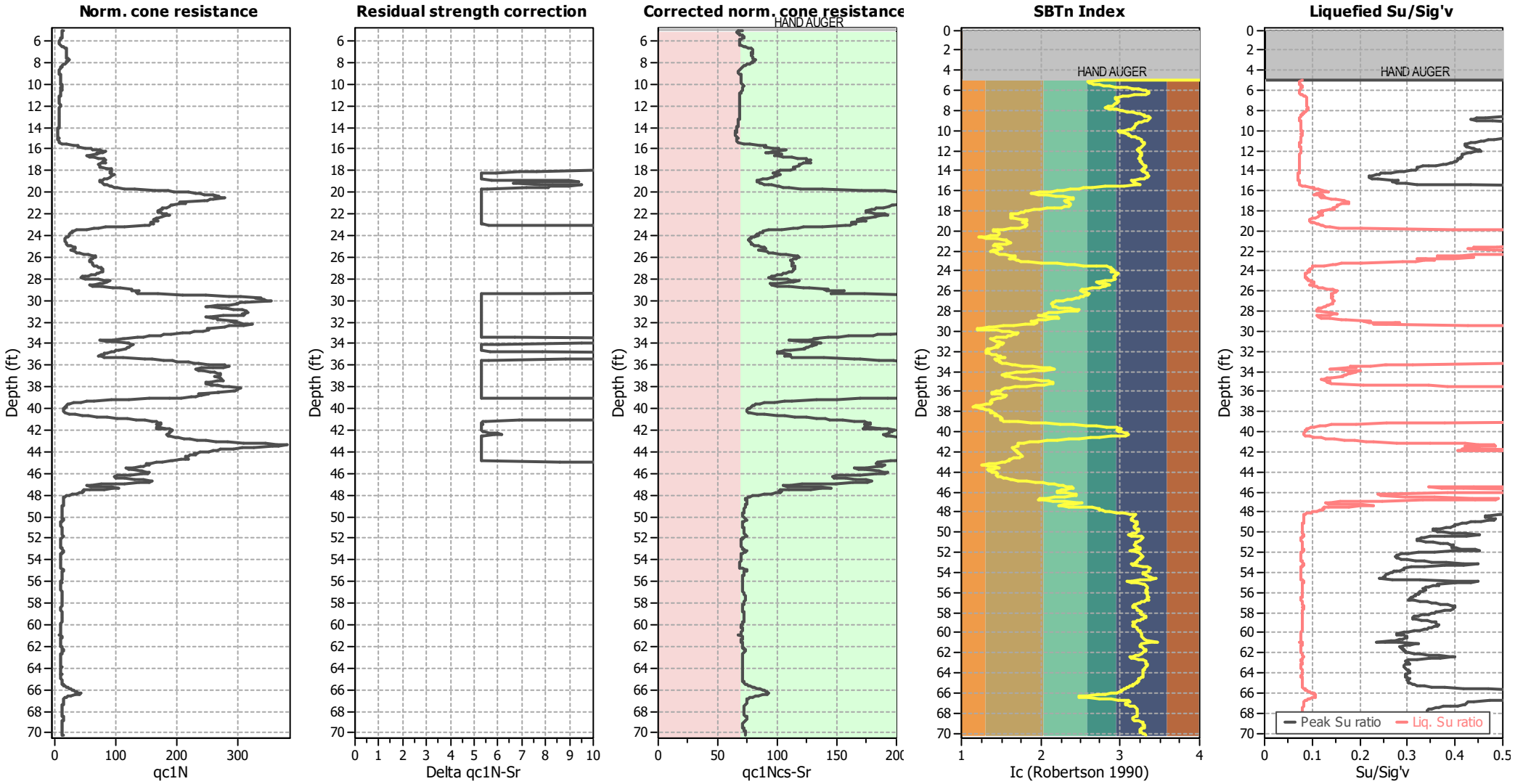
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

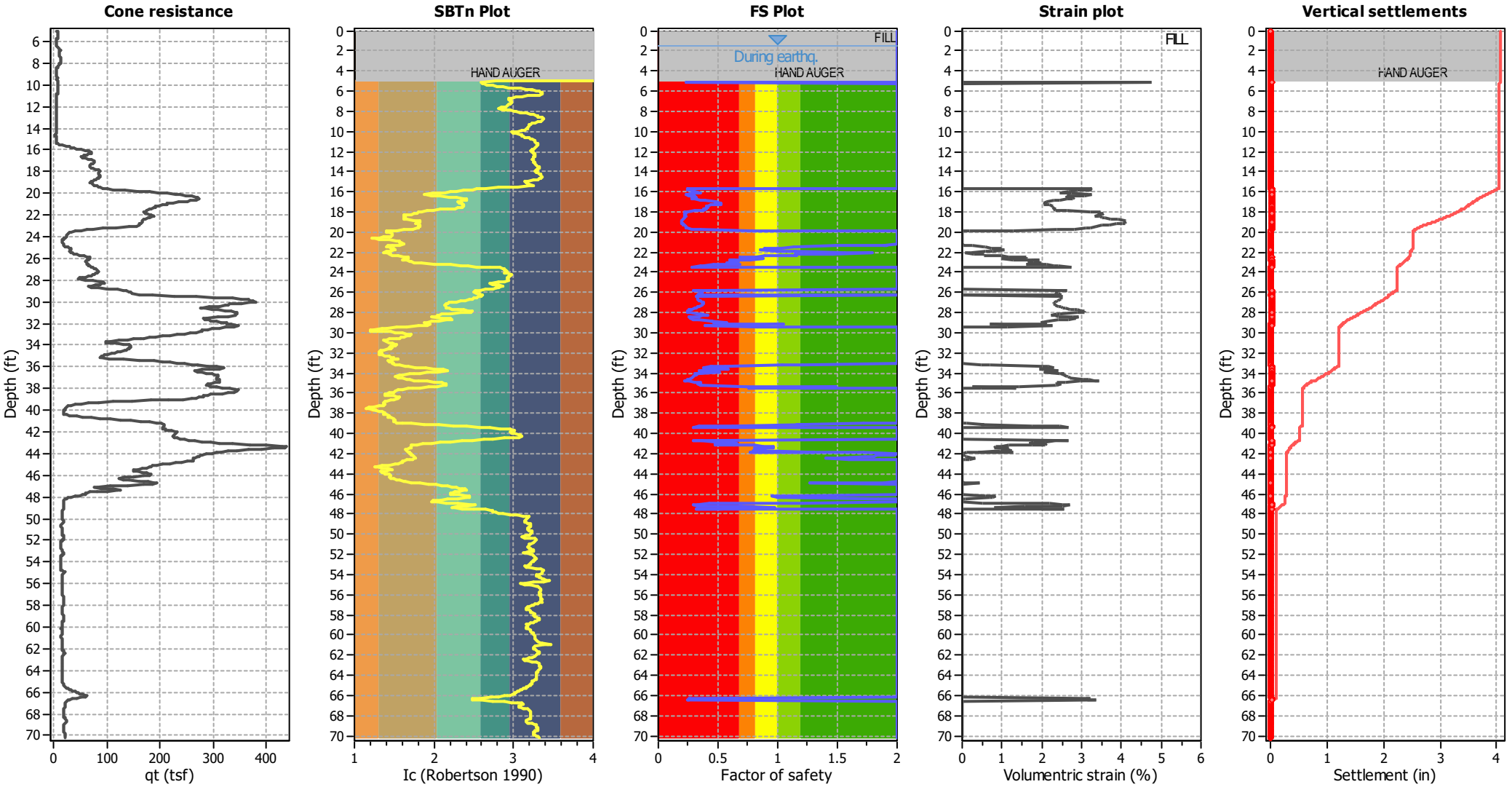
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	11.71 ft	Fill height:	0.70 ft	Limit depth:	N/A

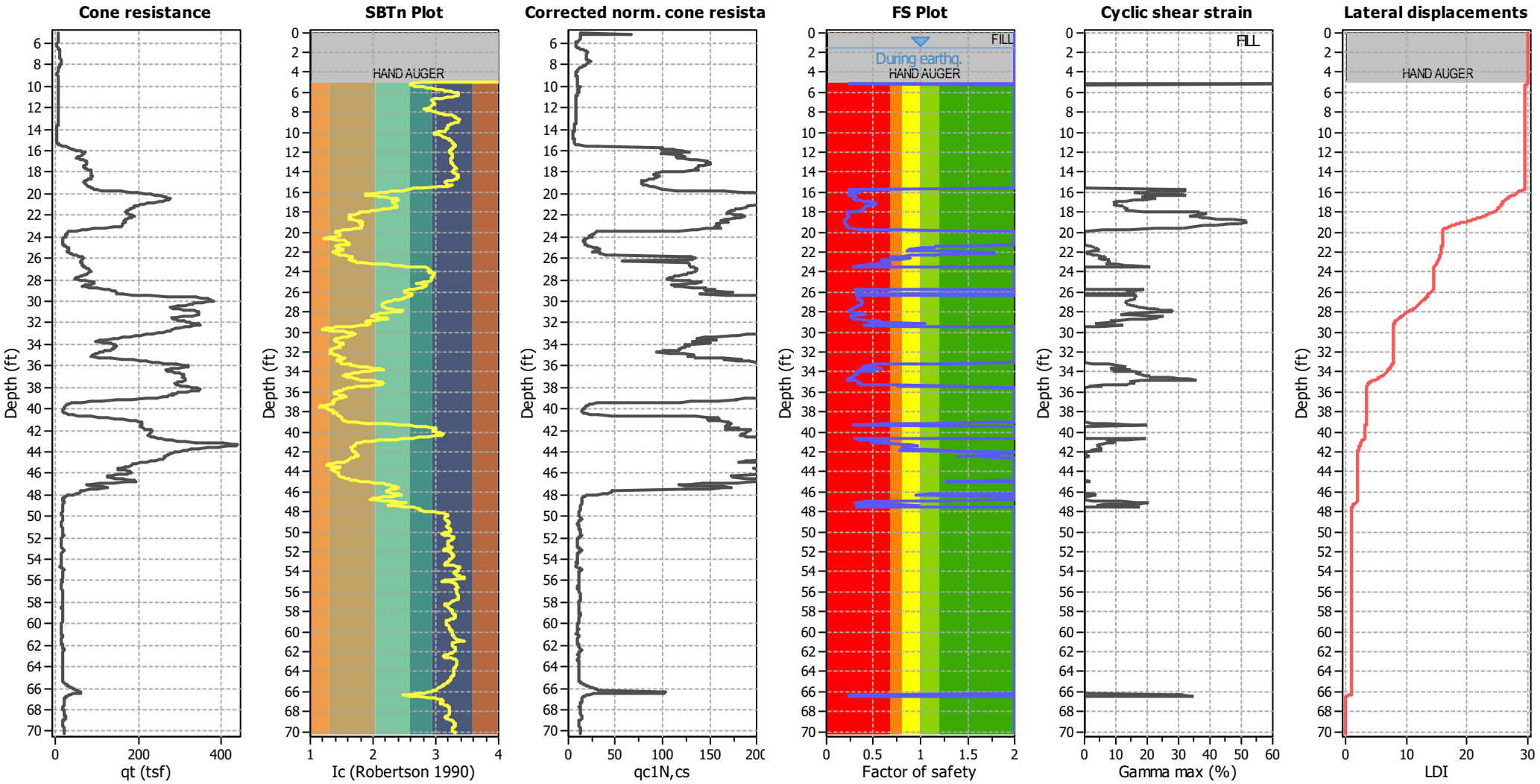
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

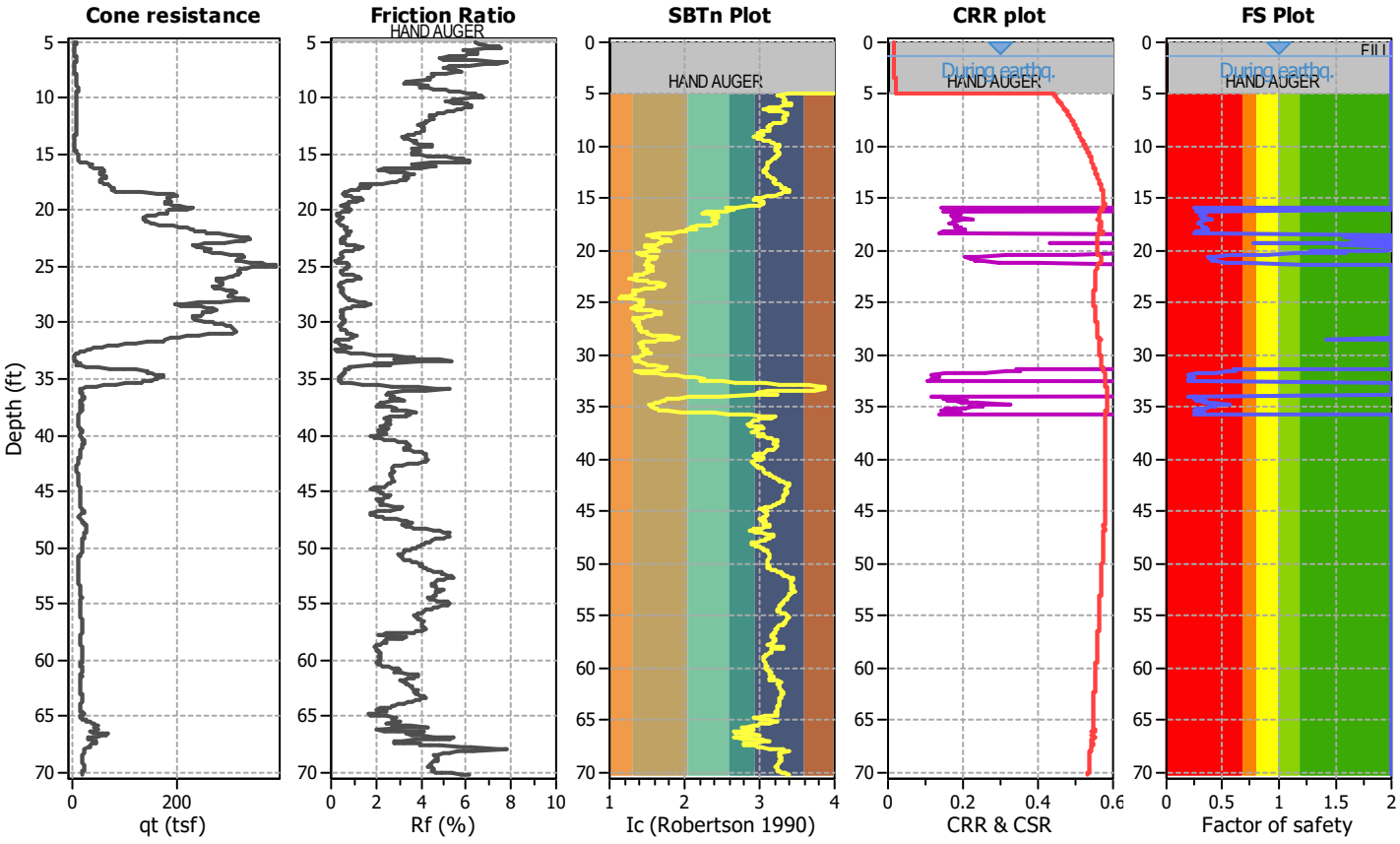
LIQUEFACTION ANALYSIS REPORT

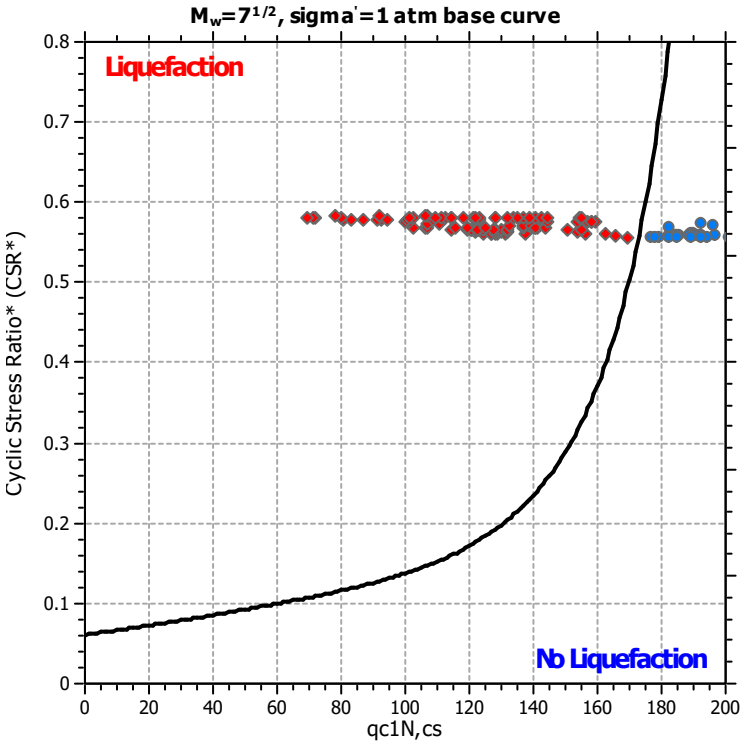
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

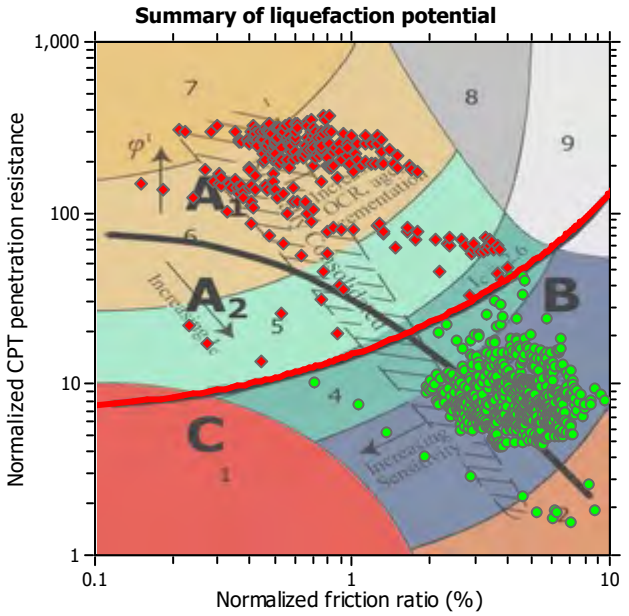
CPT file : GS26-CPT-03

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	12.68 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	0.80 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

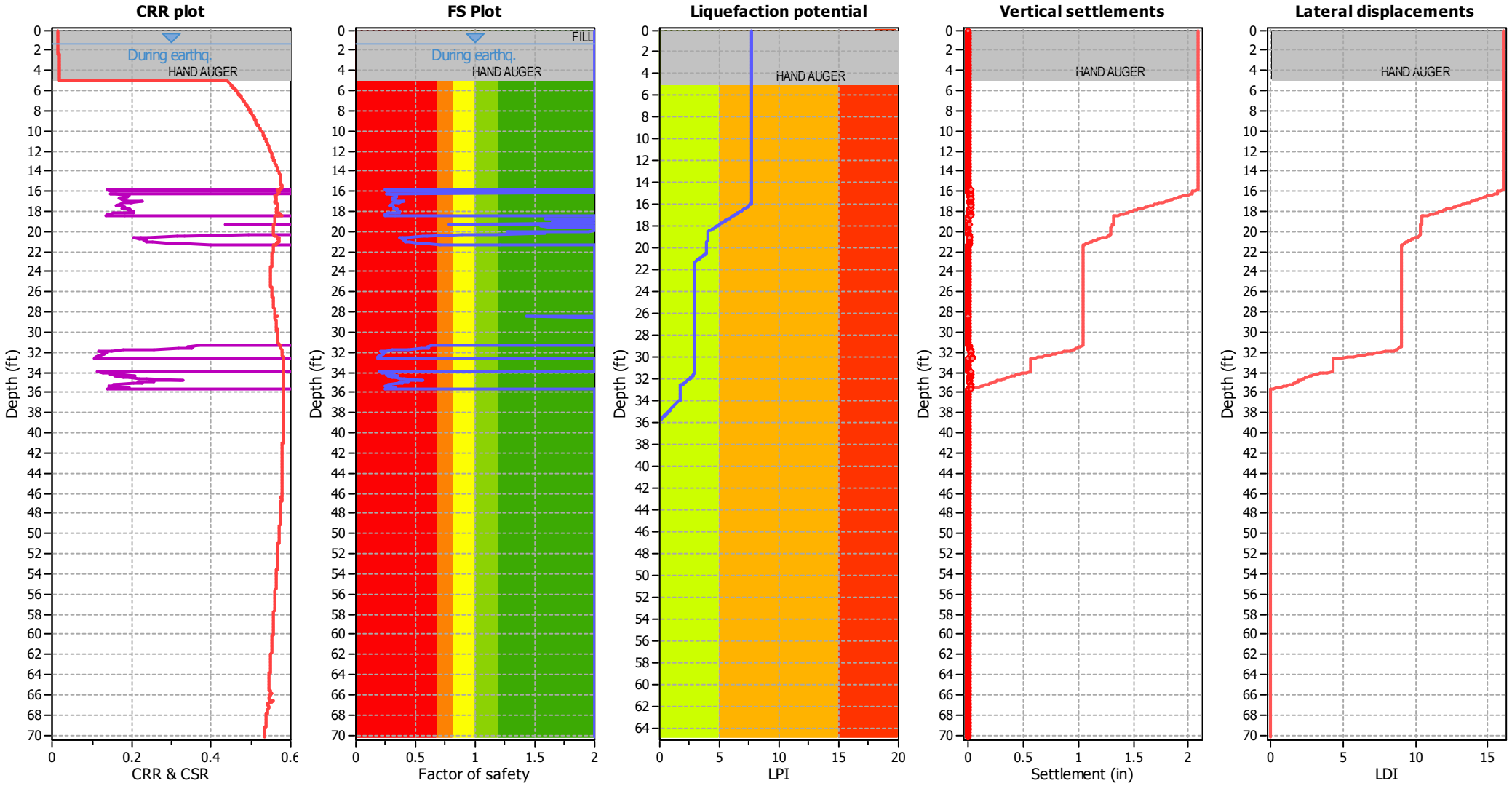






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	12.68 ft	Fill height:	0.80 ft	Limit depth:	N/A

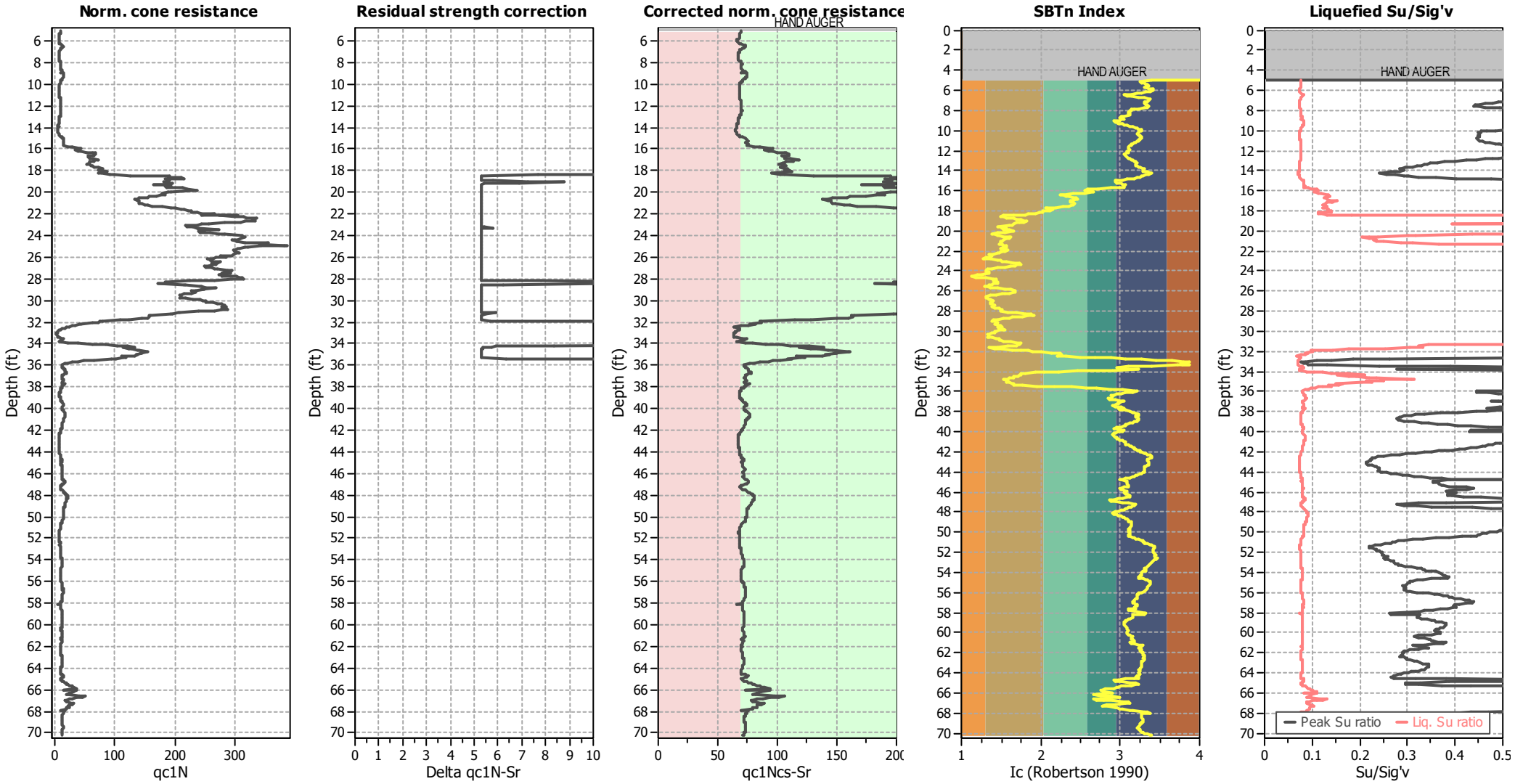
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

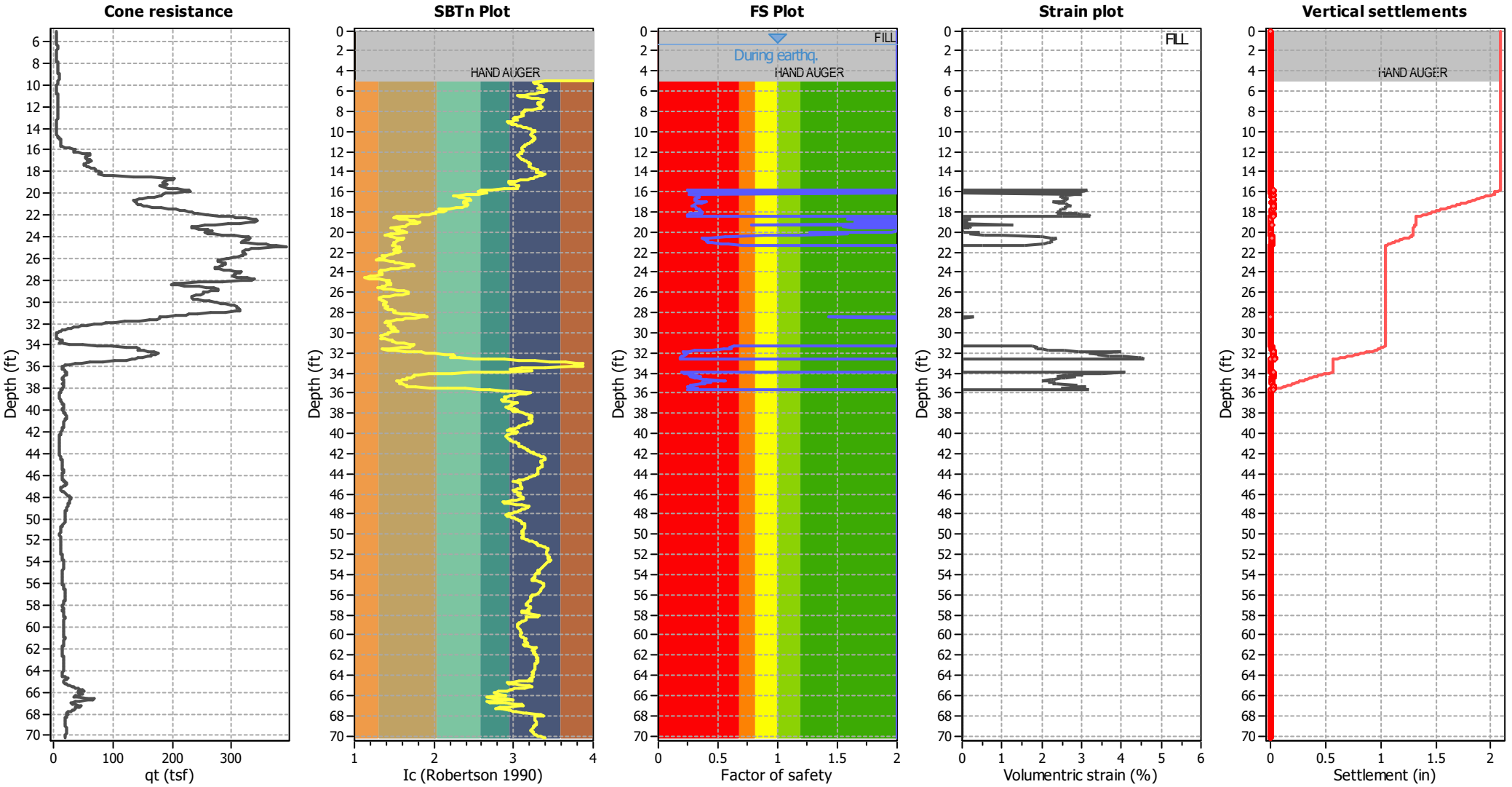
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	12.68 ft	Fill height:	0.80 ft	Limit depth:	N/A

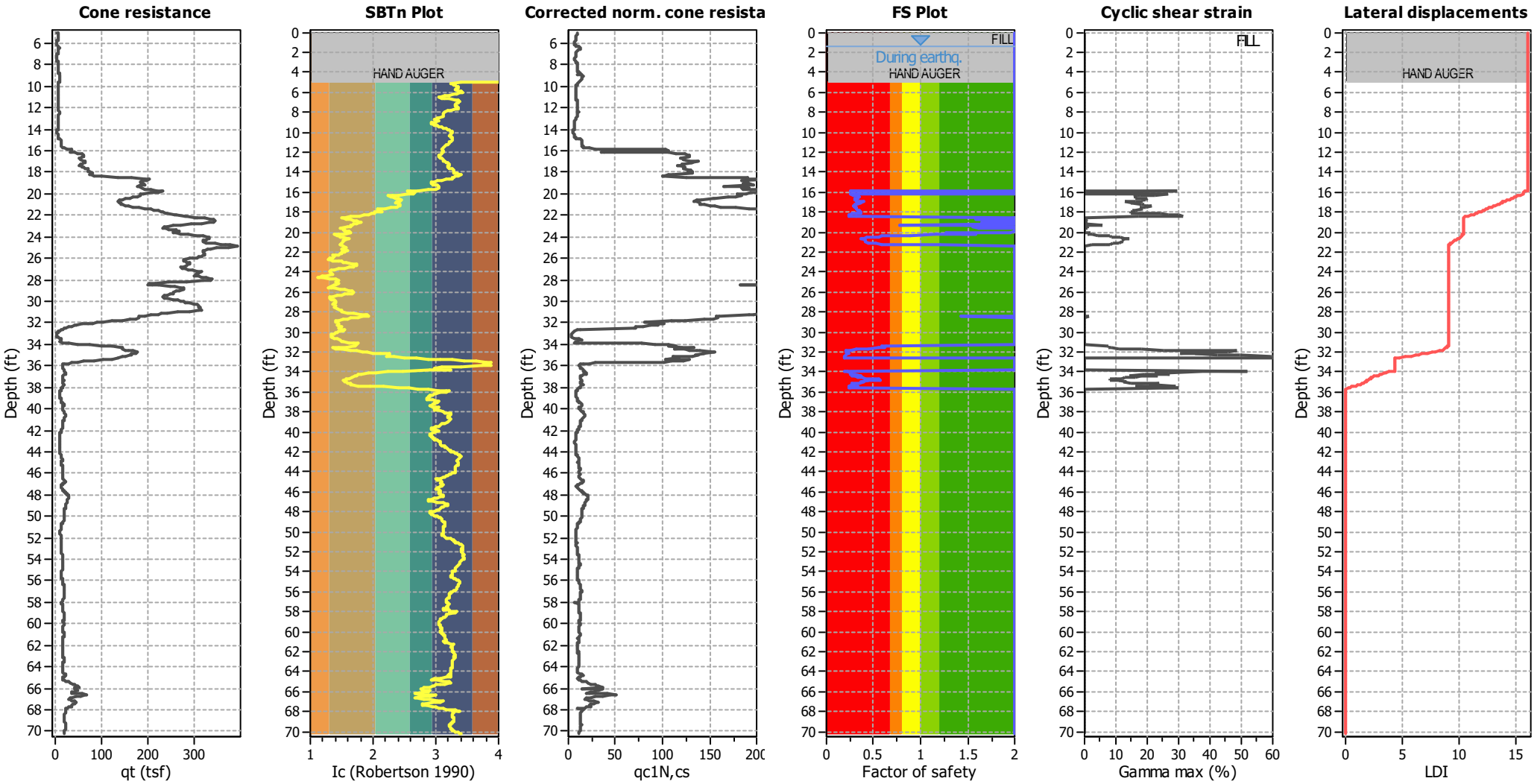
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
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$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Geosyntec Consultants, Inc.
SF Bay Area Branch

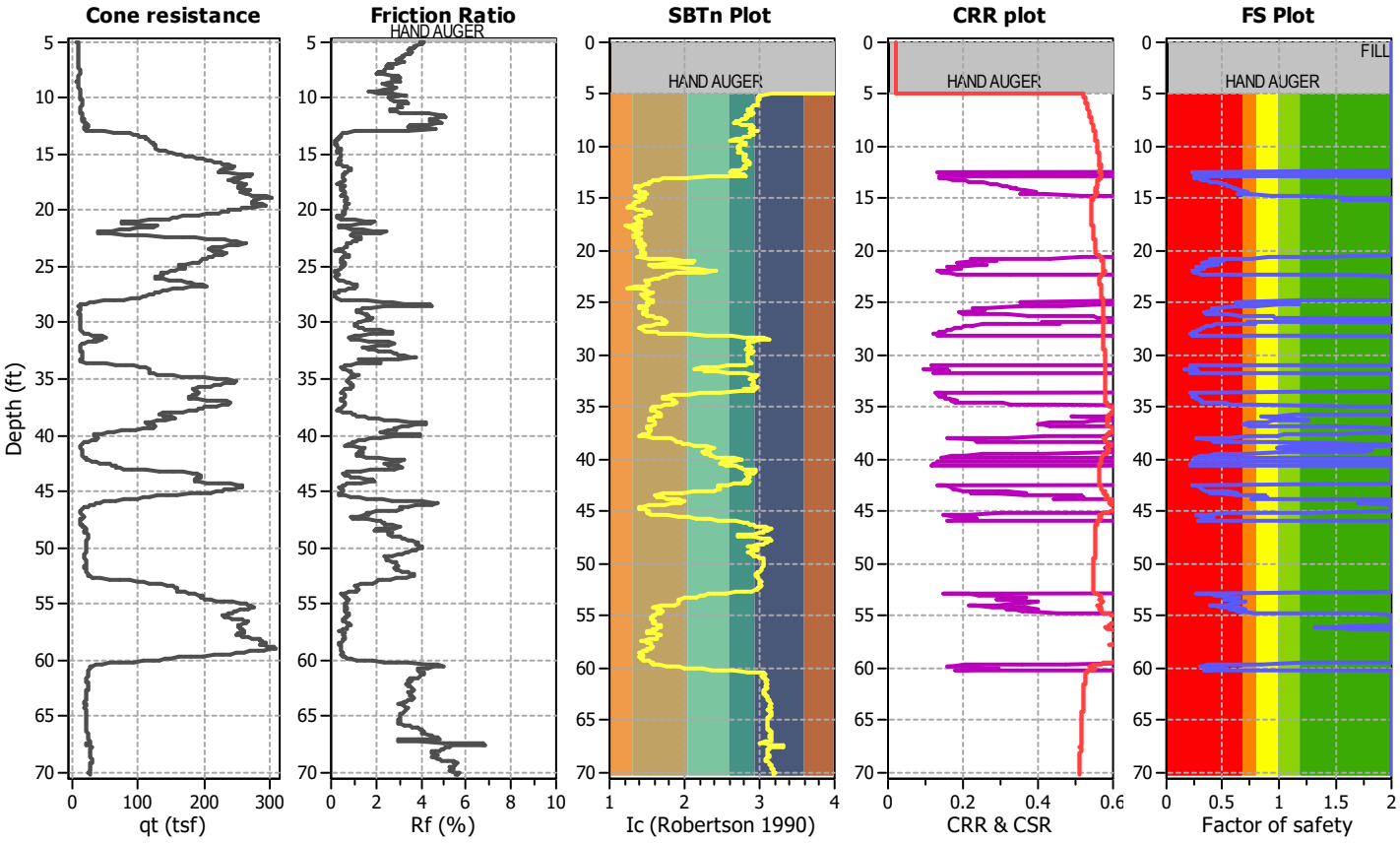
LIQUEFACTION ANALYSIS REPORT

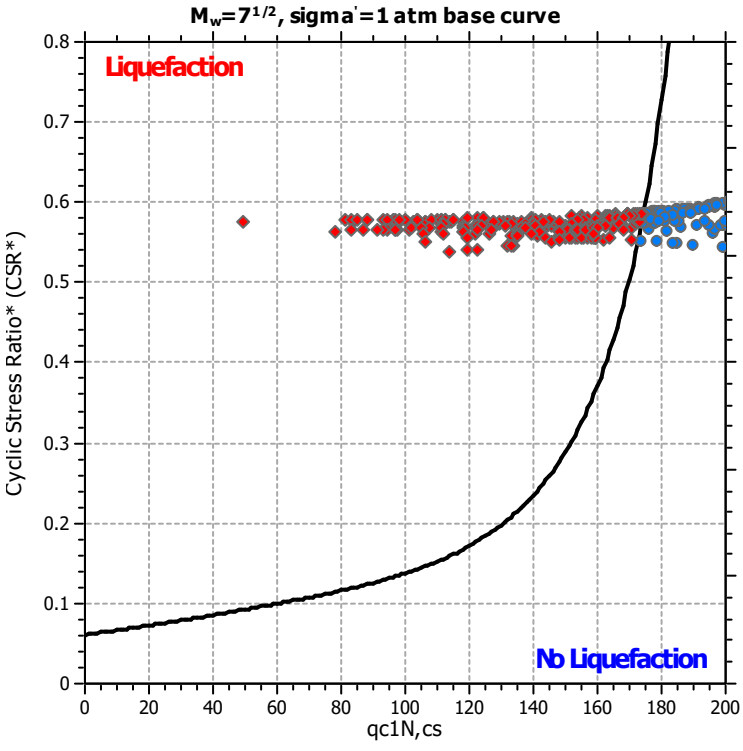
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

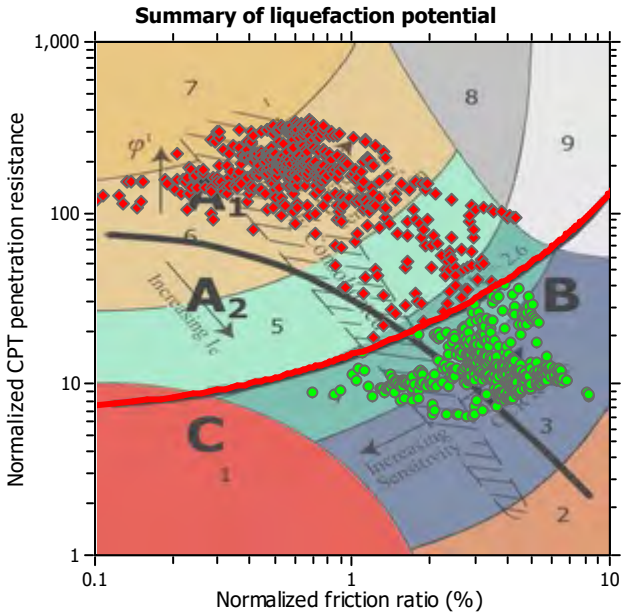
CPT file : GS26-CPT-04

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.23 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	8.10 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

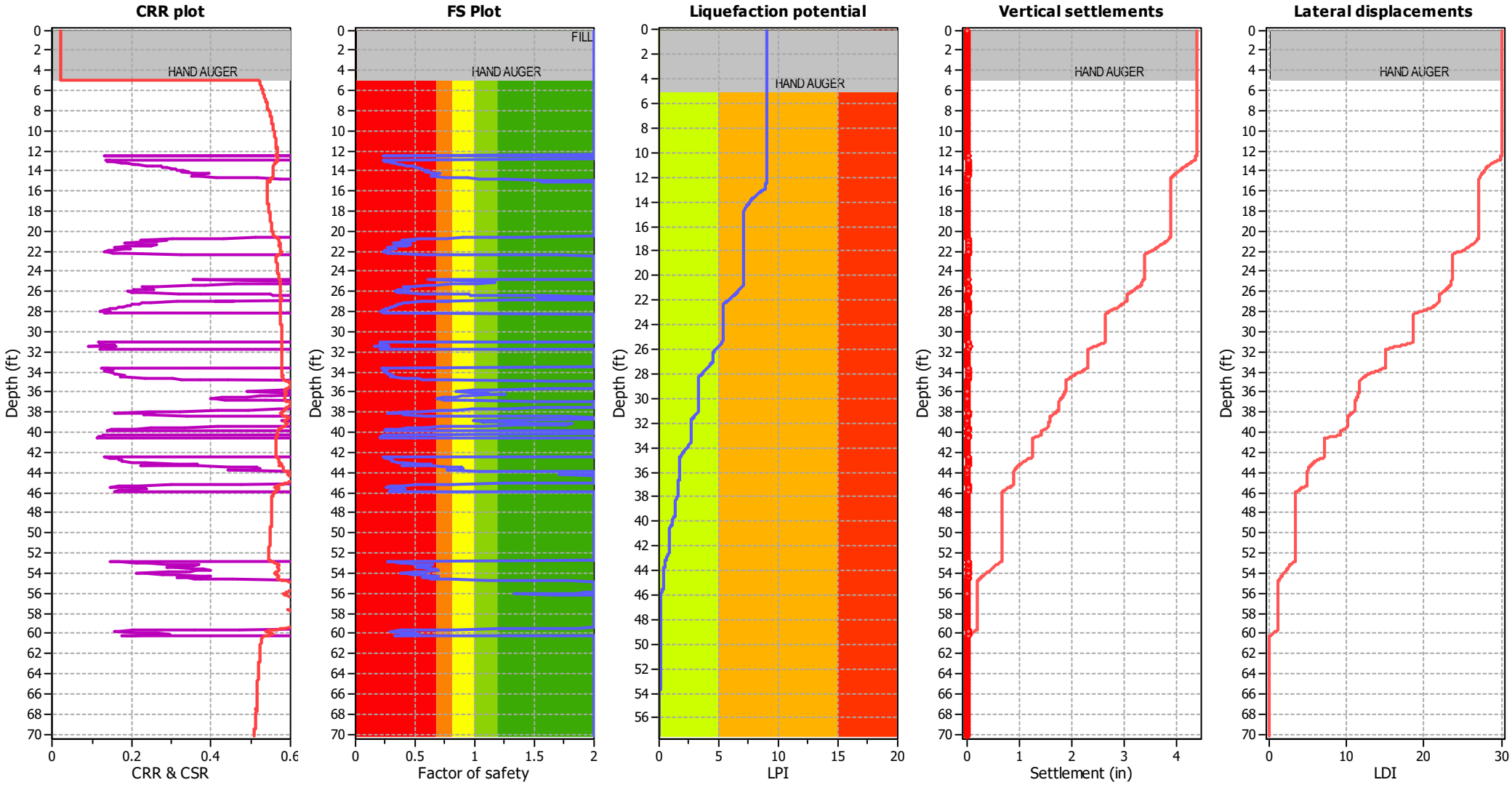






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.23 ft	Fill height:	8.10 ft	Limit depth:	N/A

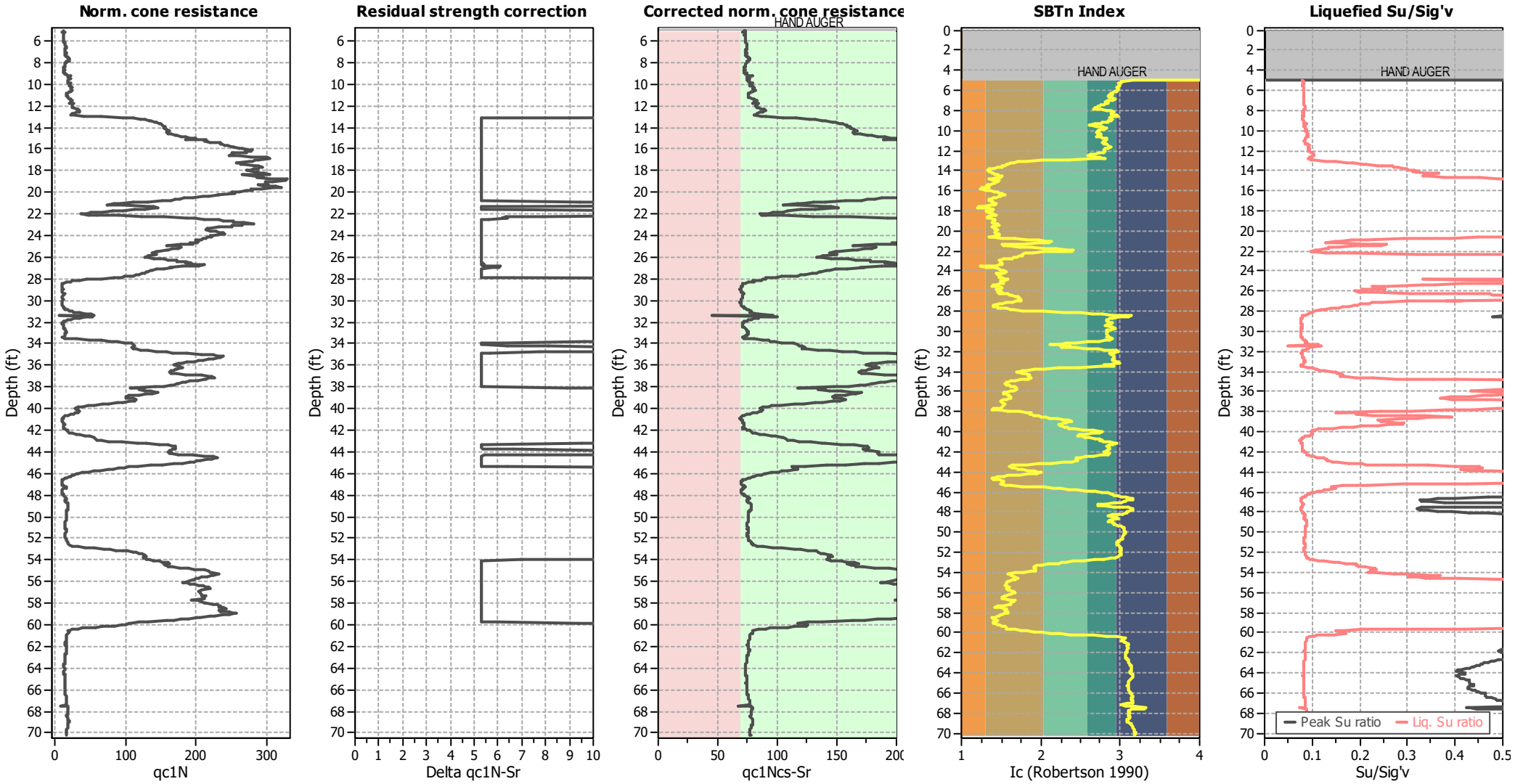
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

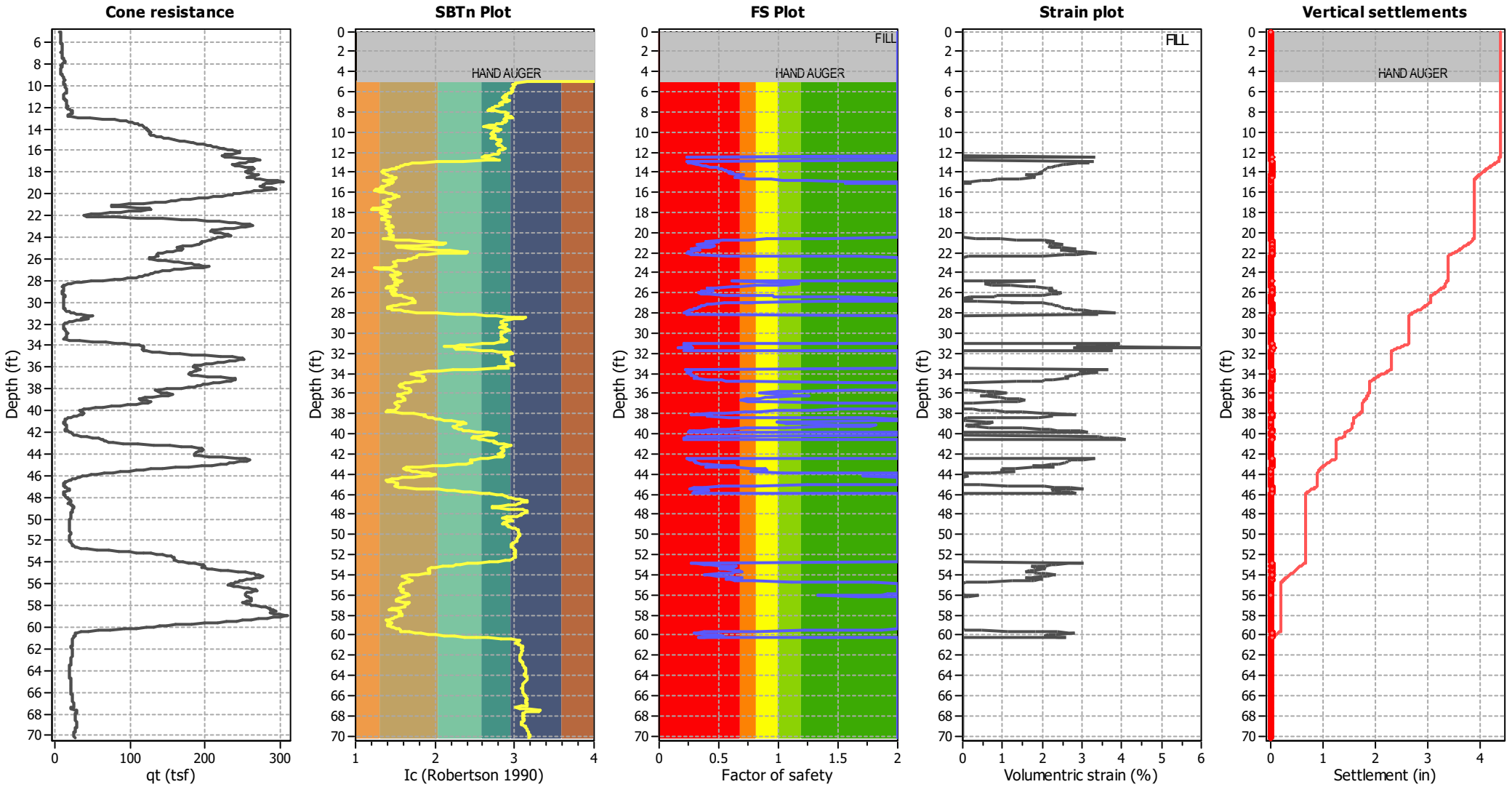
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.23 ft	Fill height:	8.10 ft	Limit depth:	N/A

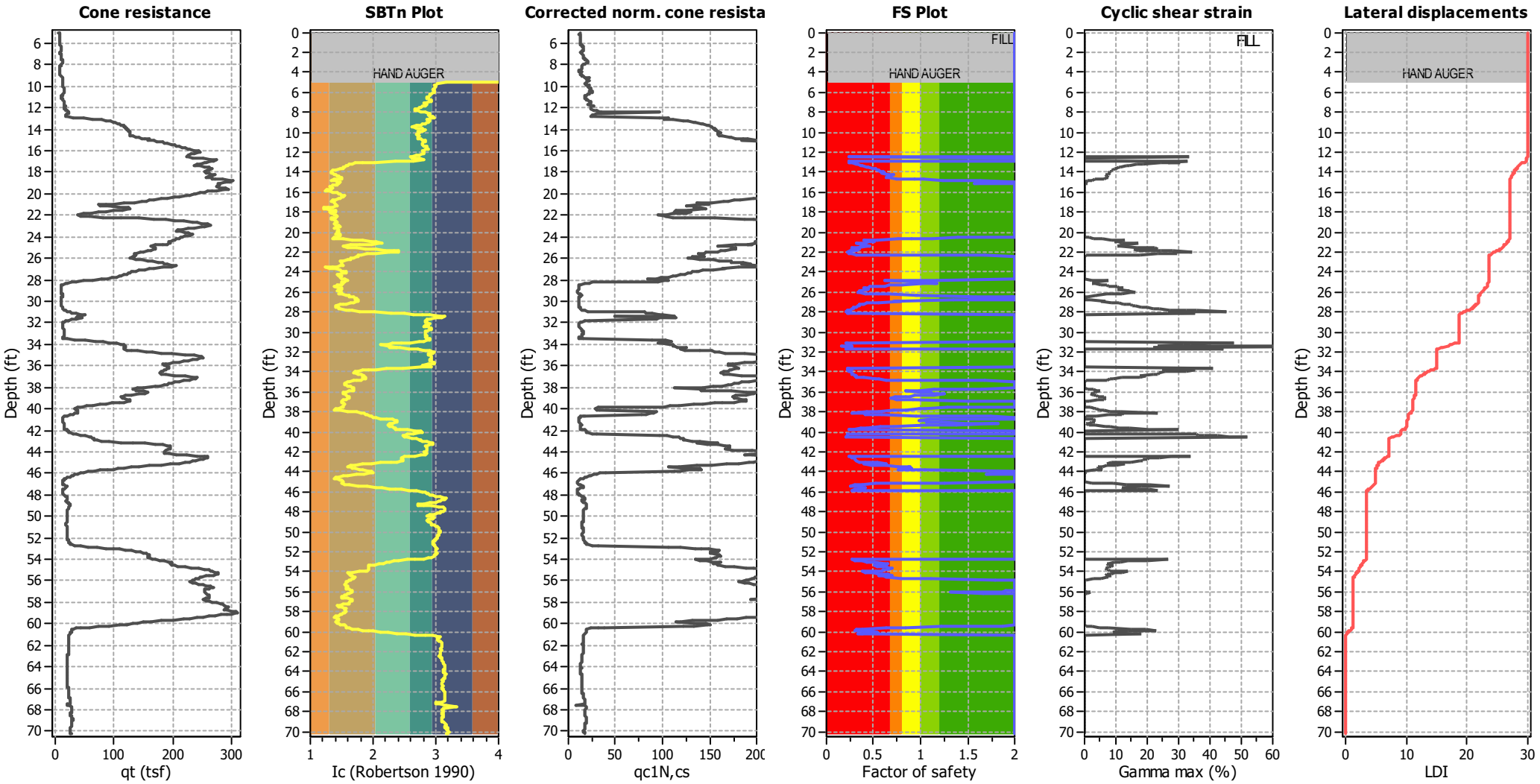
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

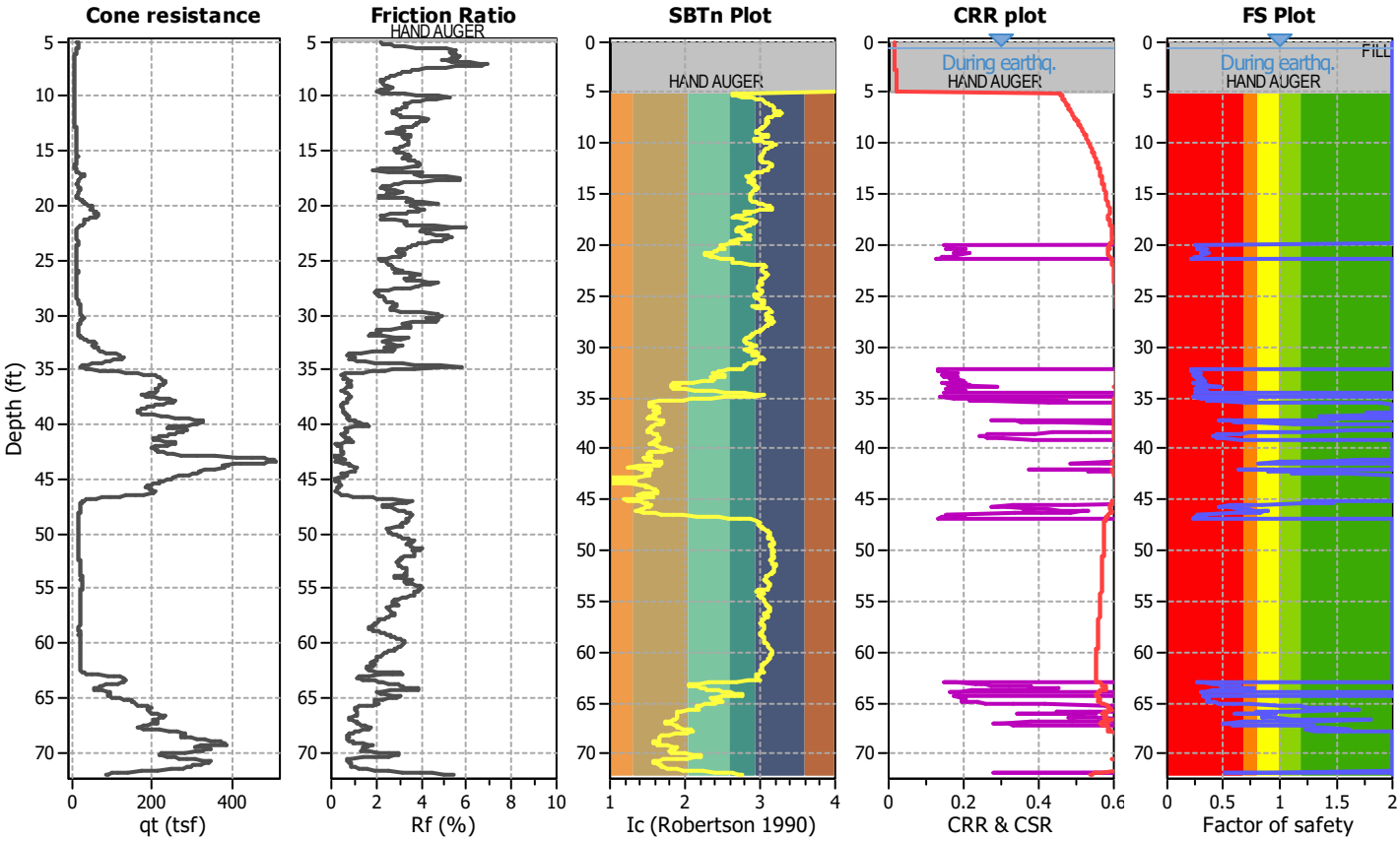
LIQUEFACTION ANALYSIS REPORT

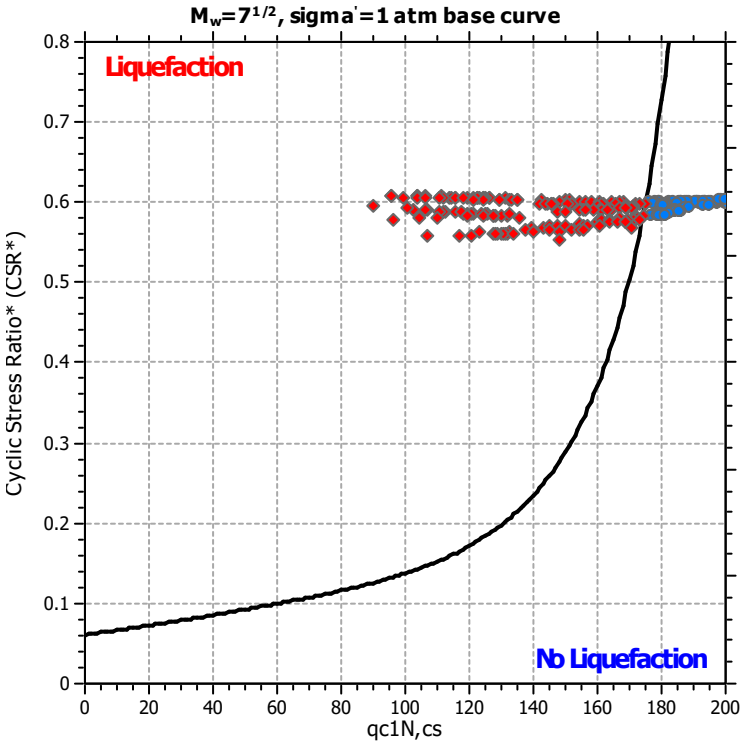
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

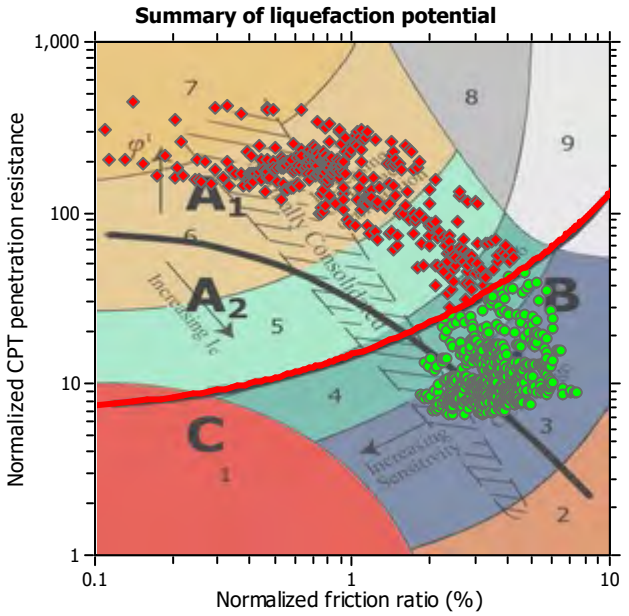
CPT file : GS26-CPT-05

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	9.67 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.60 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
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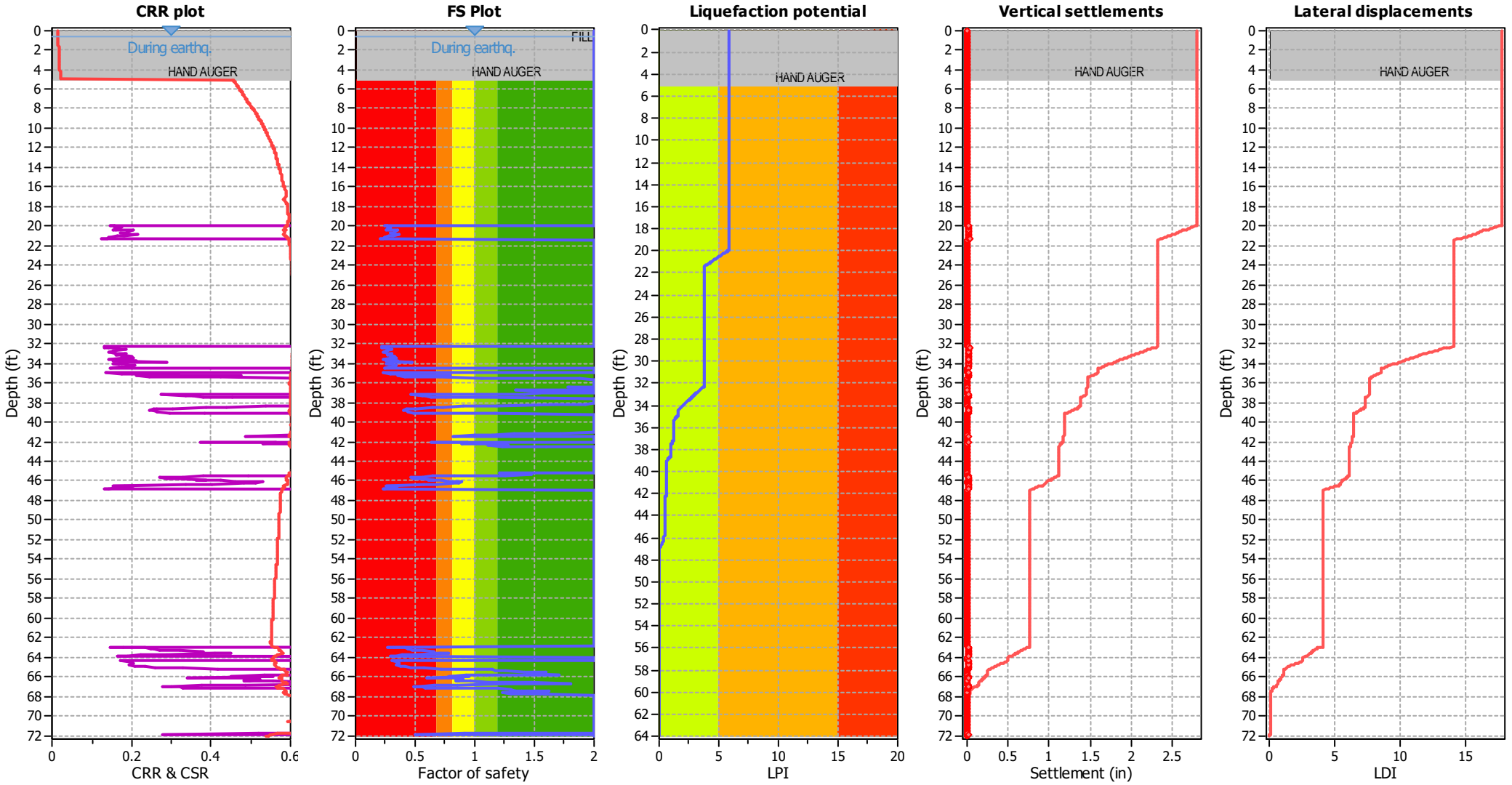
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	9.67 ft	Fill height:	1.60 ft	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
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LPI color scheme

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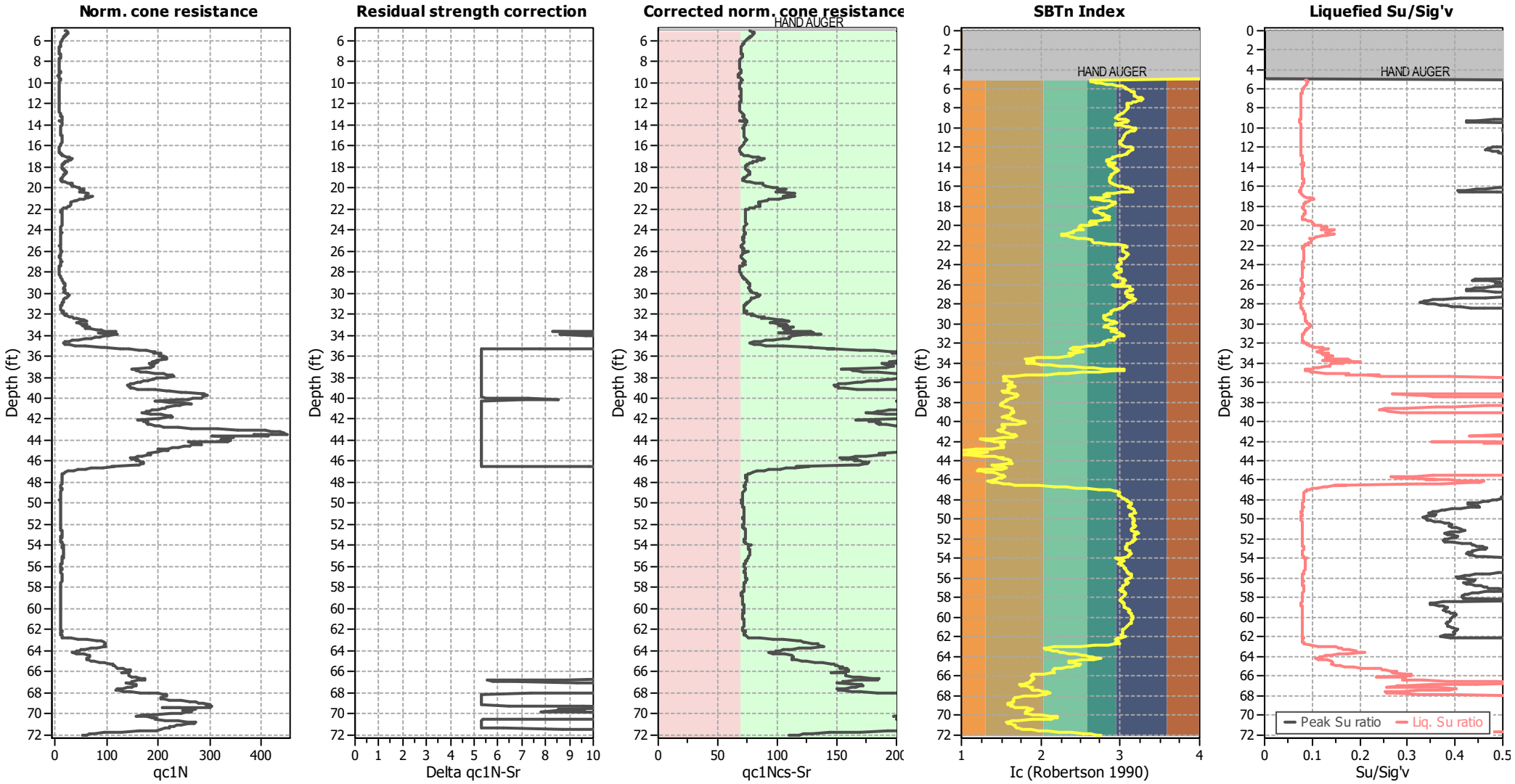
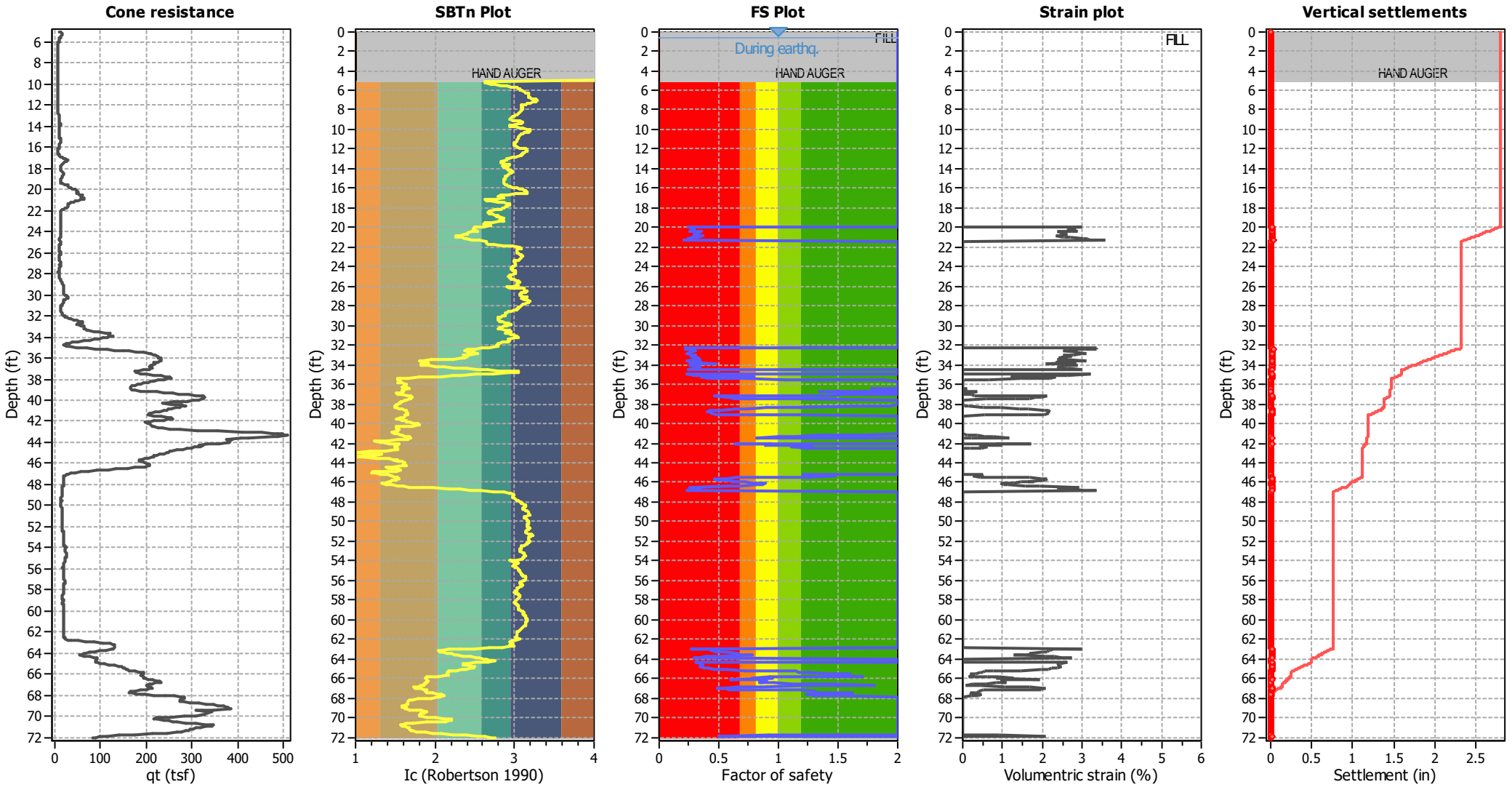
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	9.67 ft	Fill height:	1.60 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



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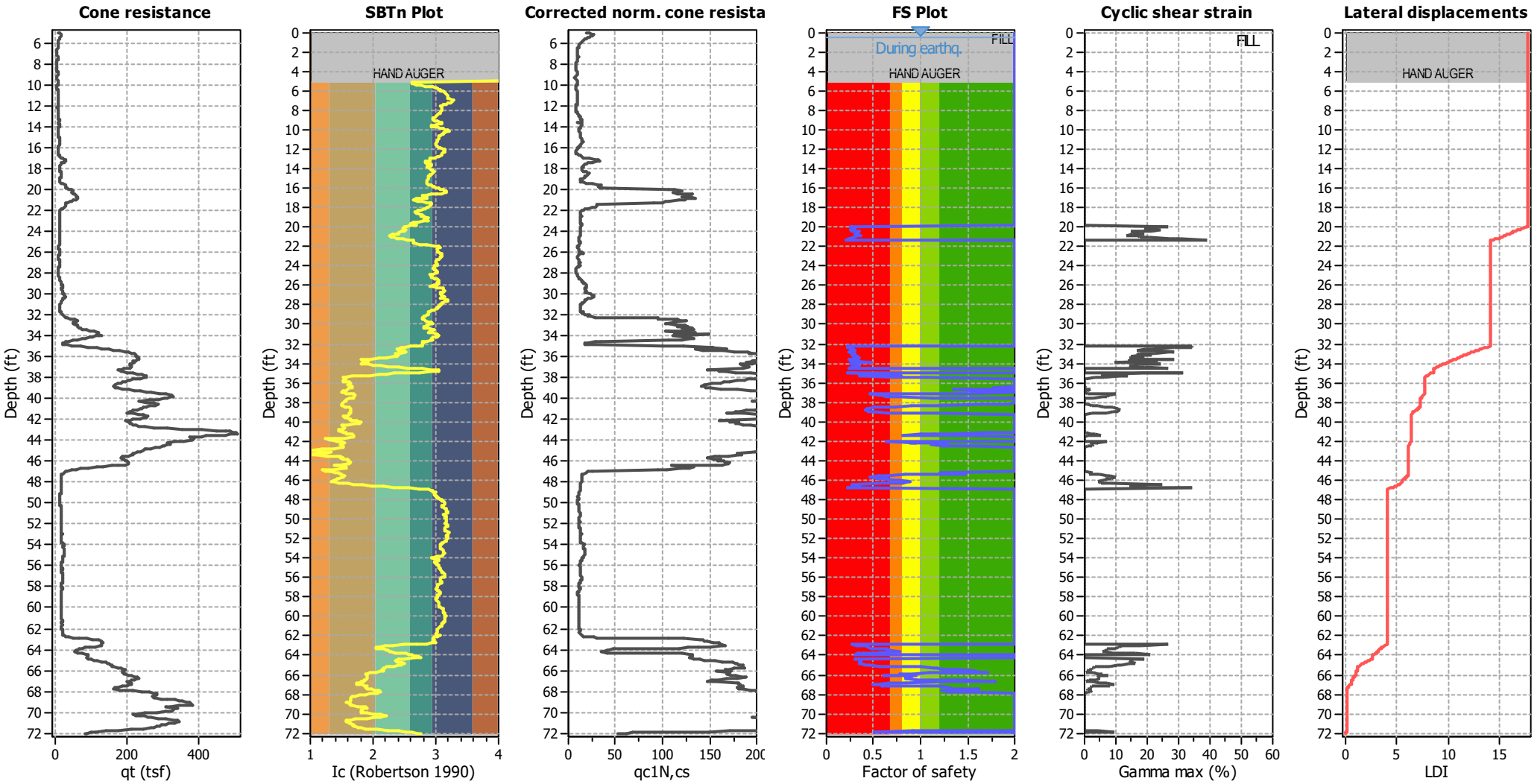
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

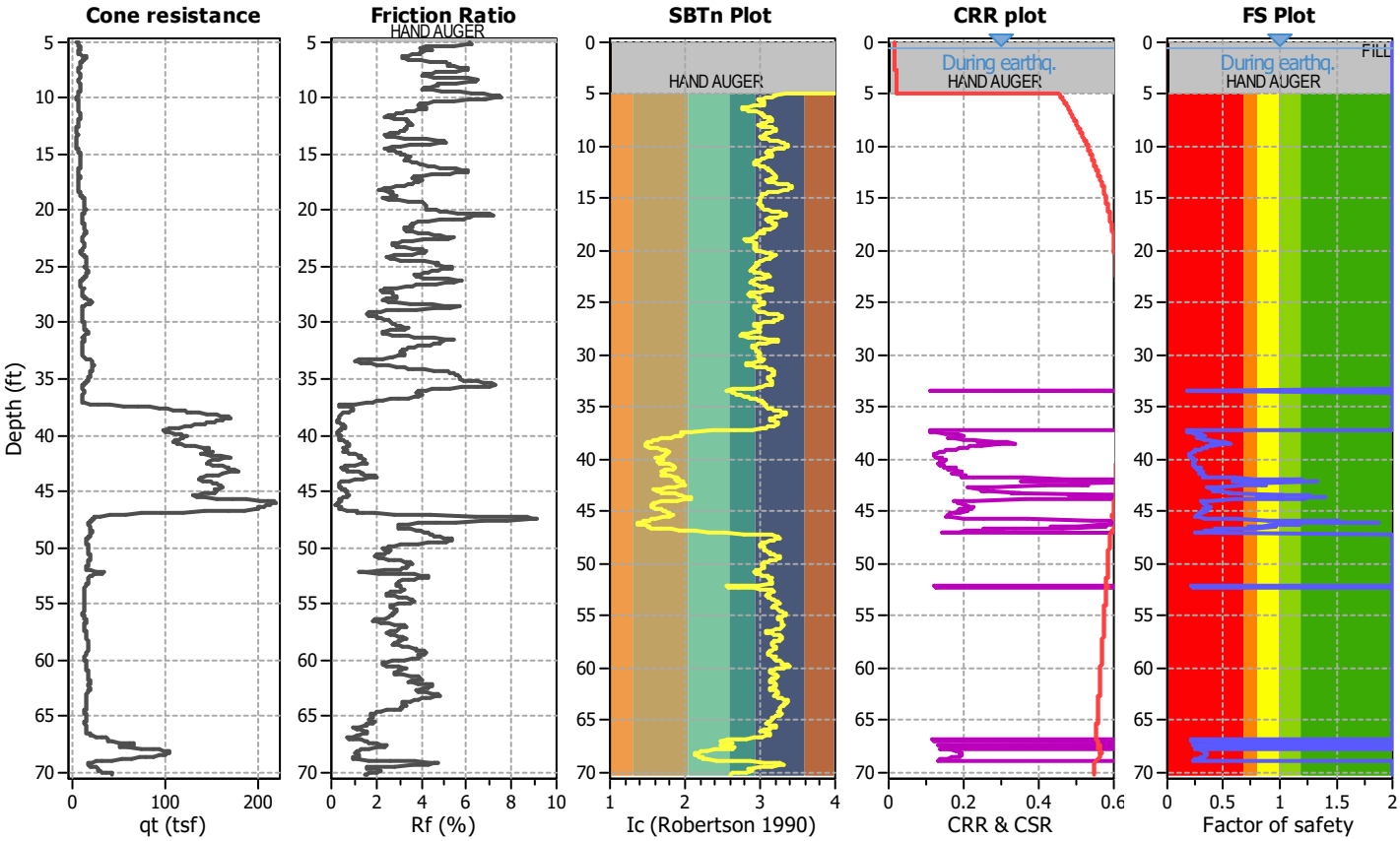
LIQUEFACTION ANALYSIS REPORT

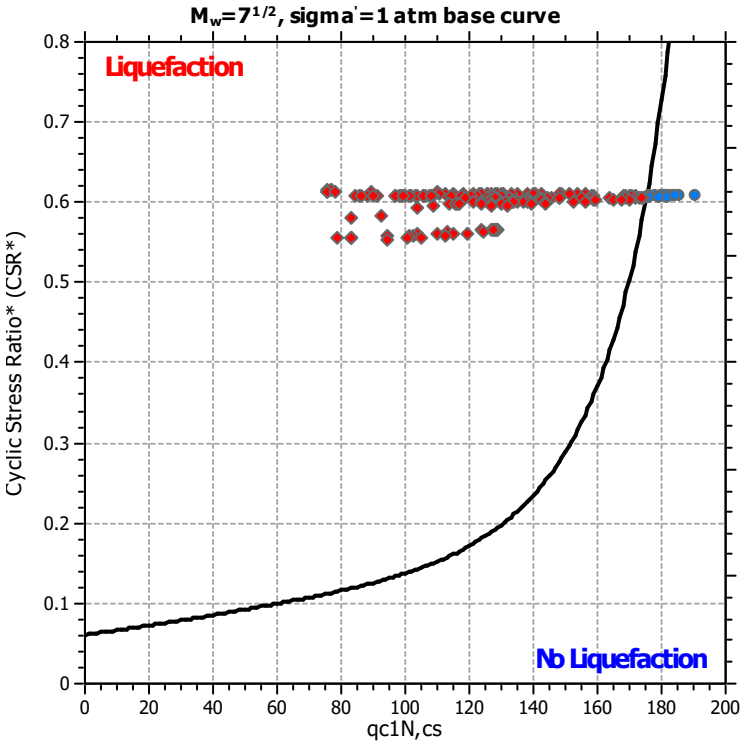
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

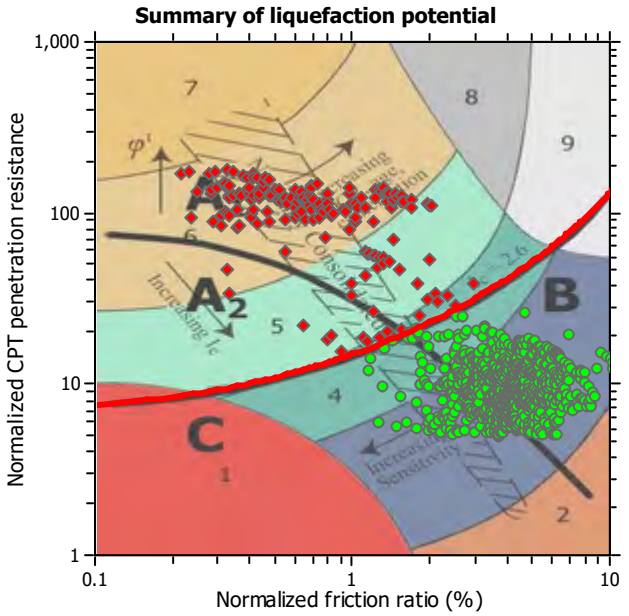
CPT file : GS26-CPT-06

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.75 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.60 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

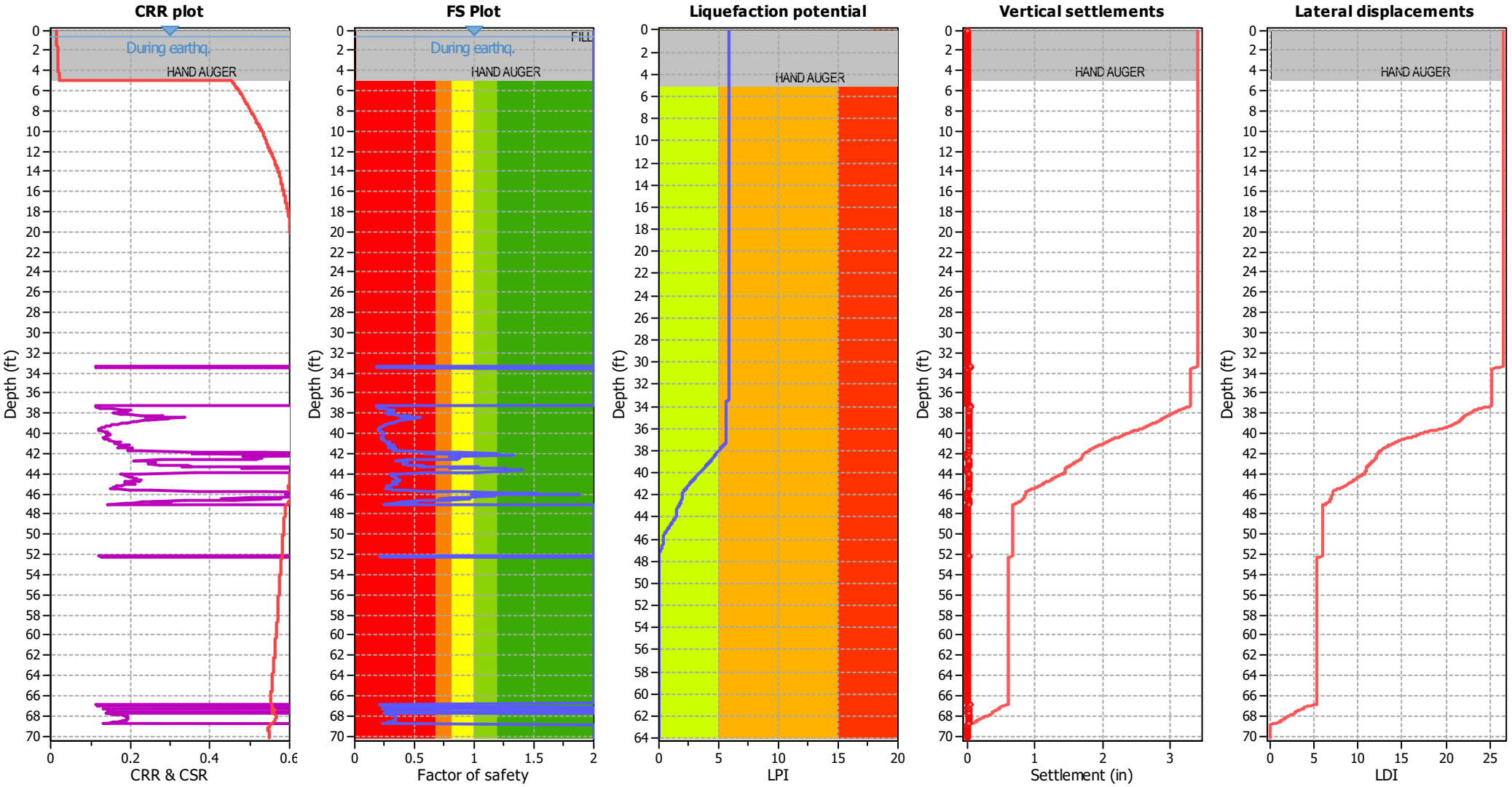






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	8.75 ft	Fill height:	1.60 ft	Limit depth:	N/A

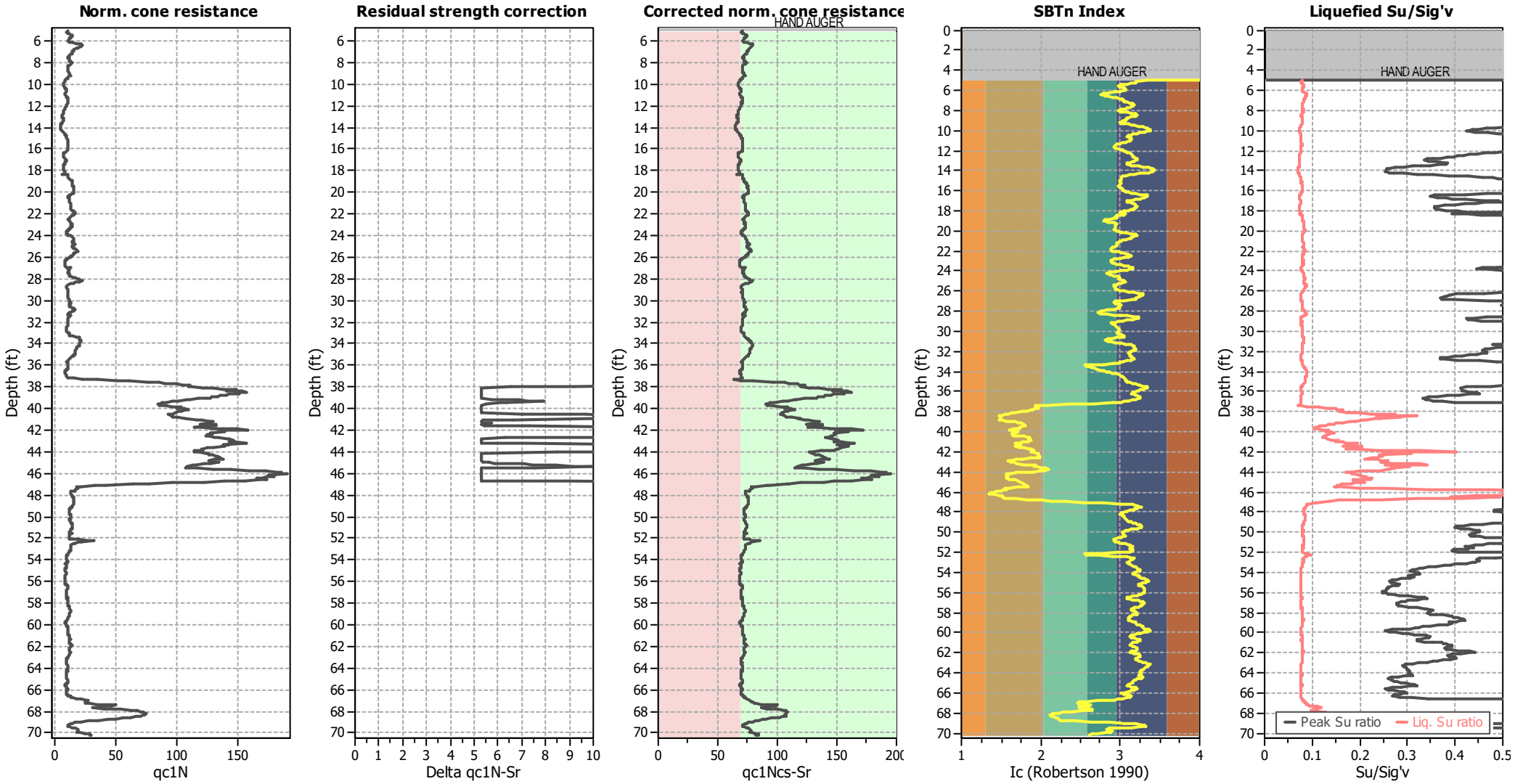
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

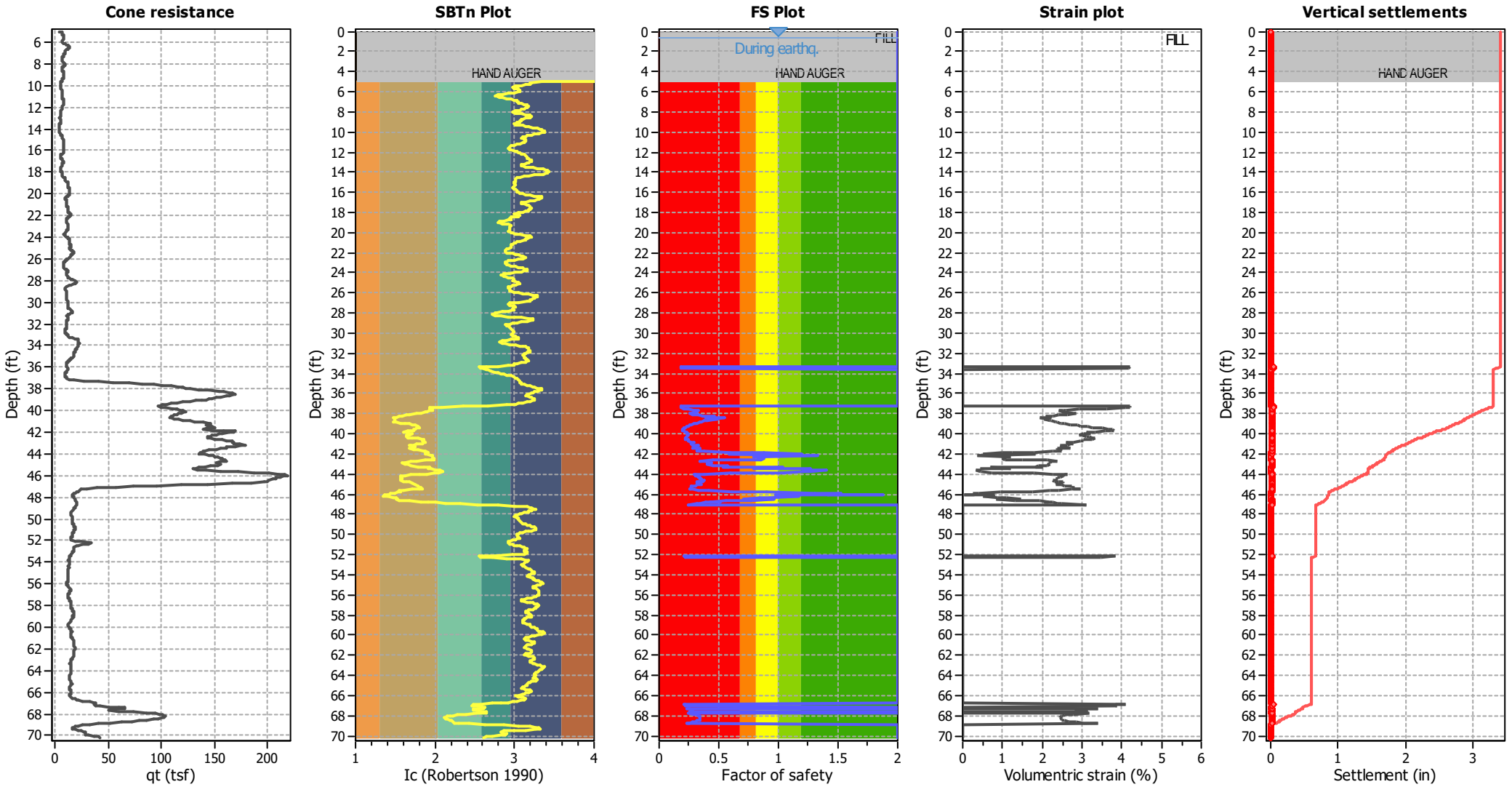
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	8.75 ft	Fill height:	1.60 ft	Limit depth:	N/A

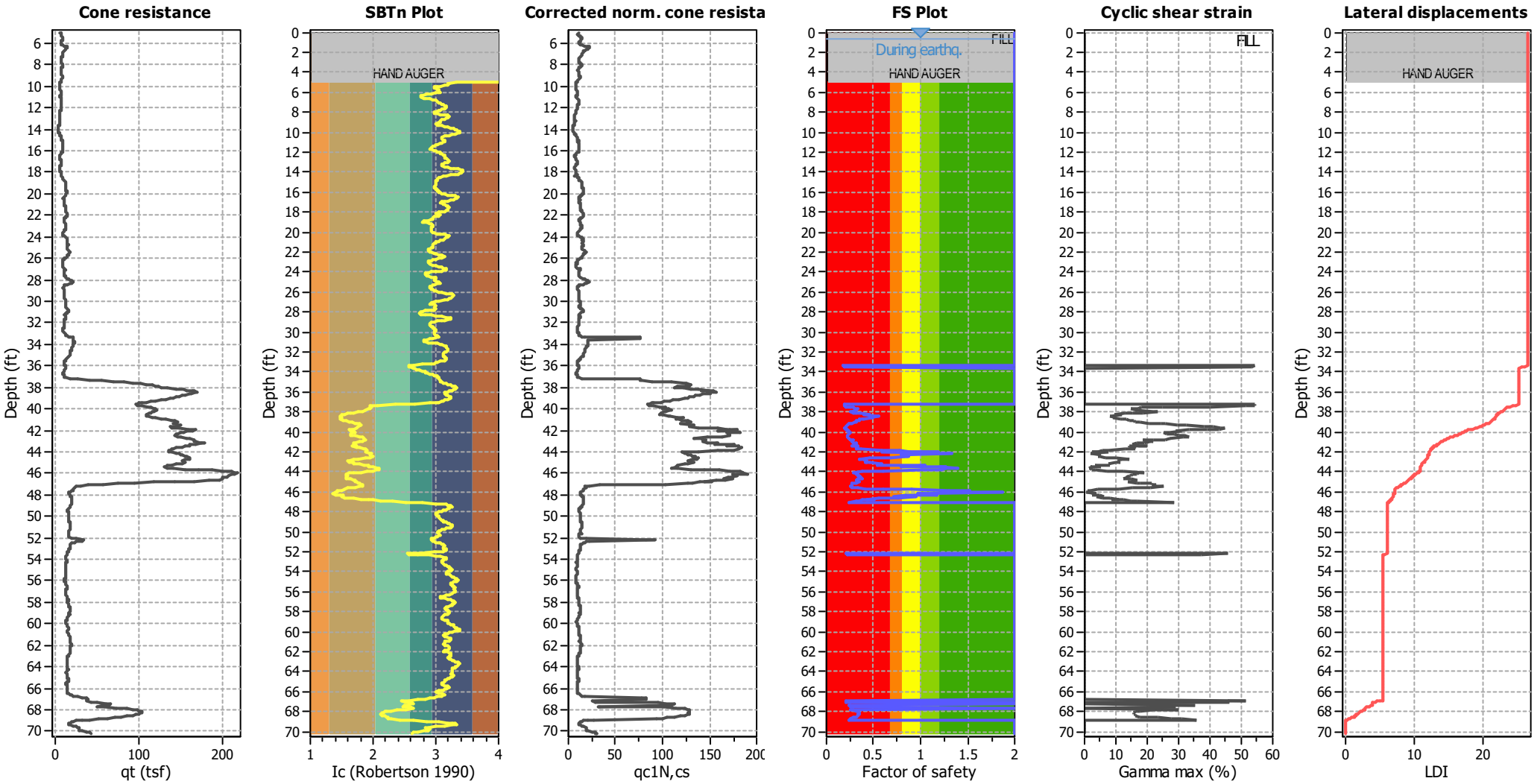
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

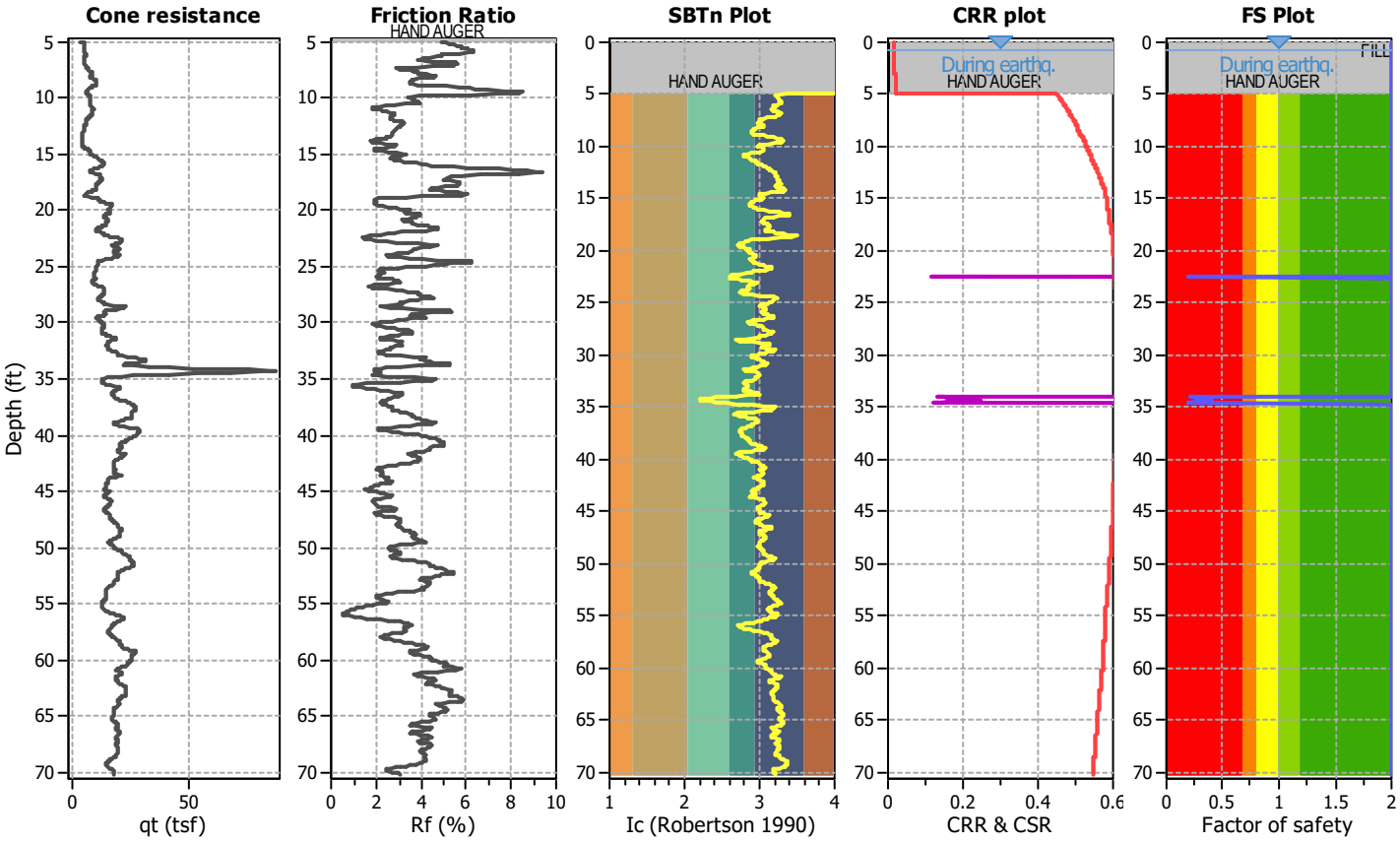
LIQUEFACTION ANALYSIS REPORT

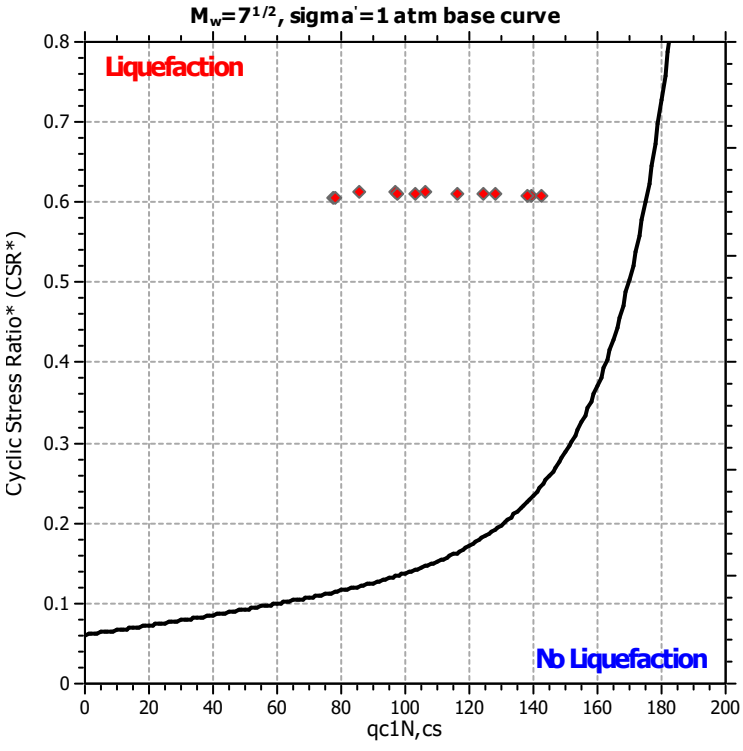
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

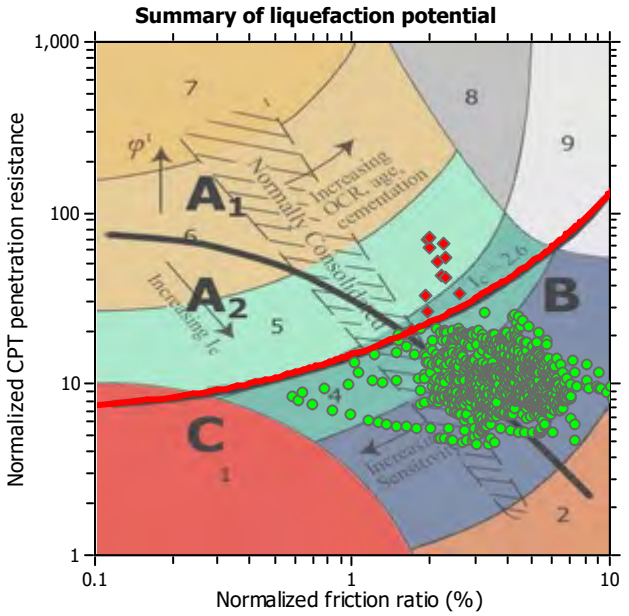
CPT file : GS26-CPT-07

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	12.98 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.30 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

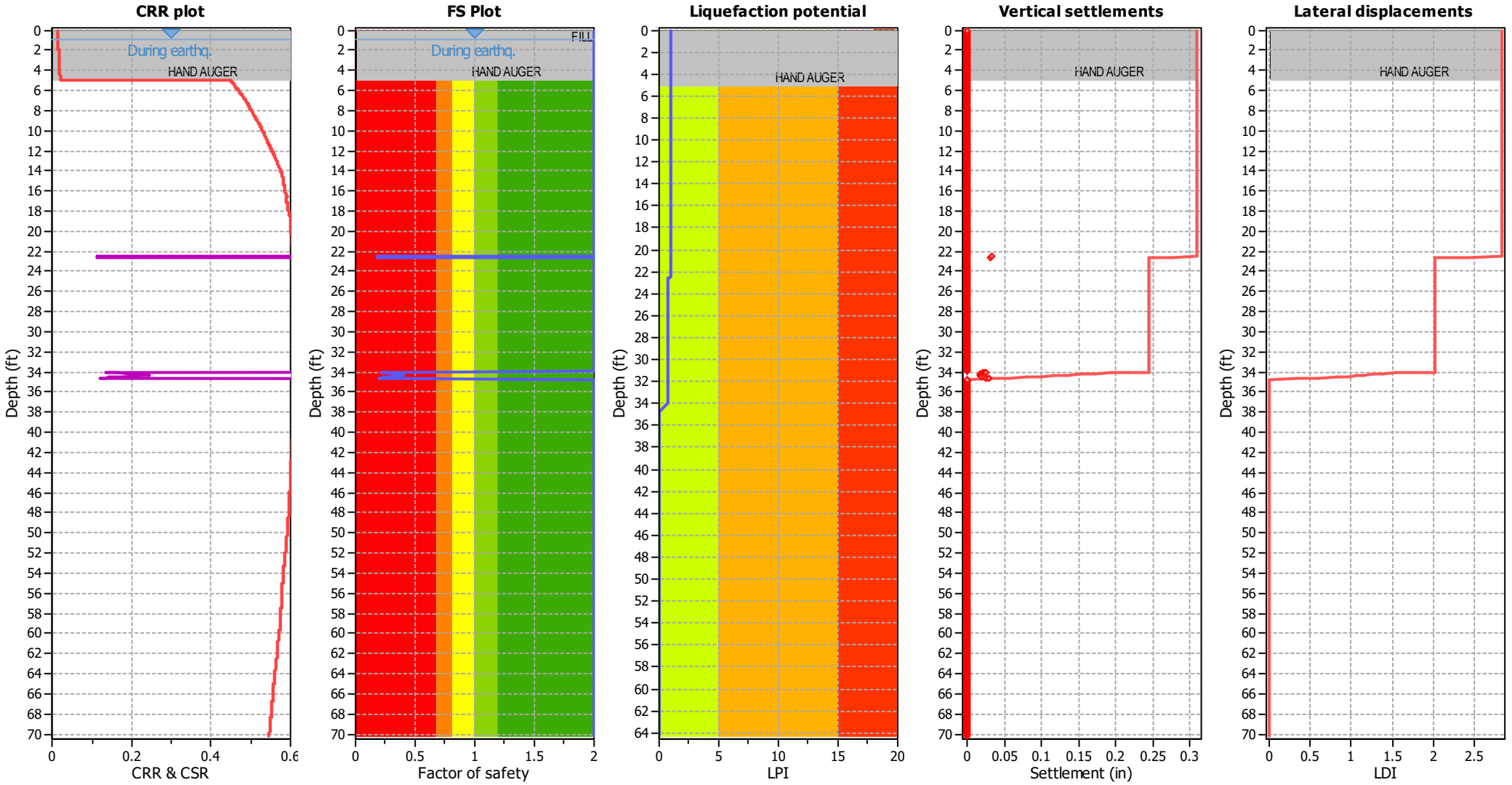






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	12.98 ft	Fill height:	1.30 ft	Limit depth:	N/A

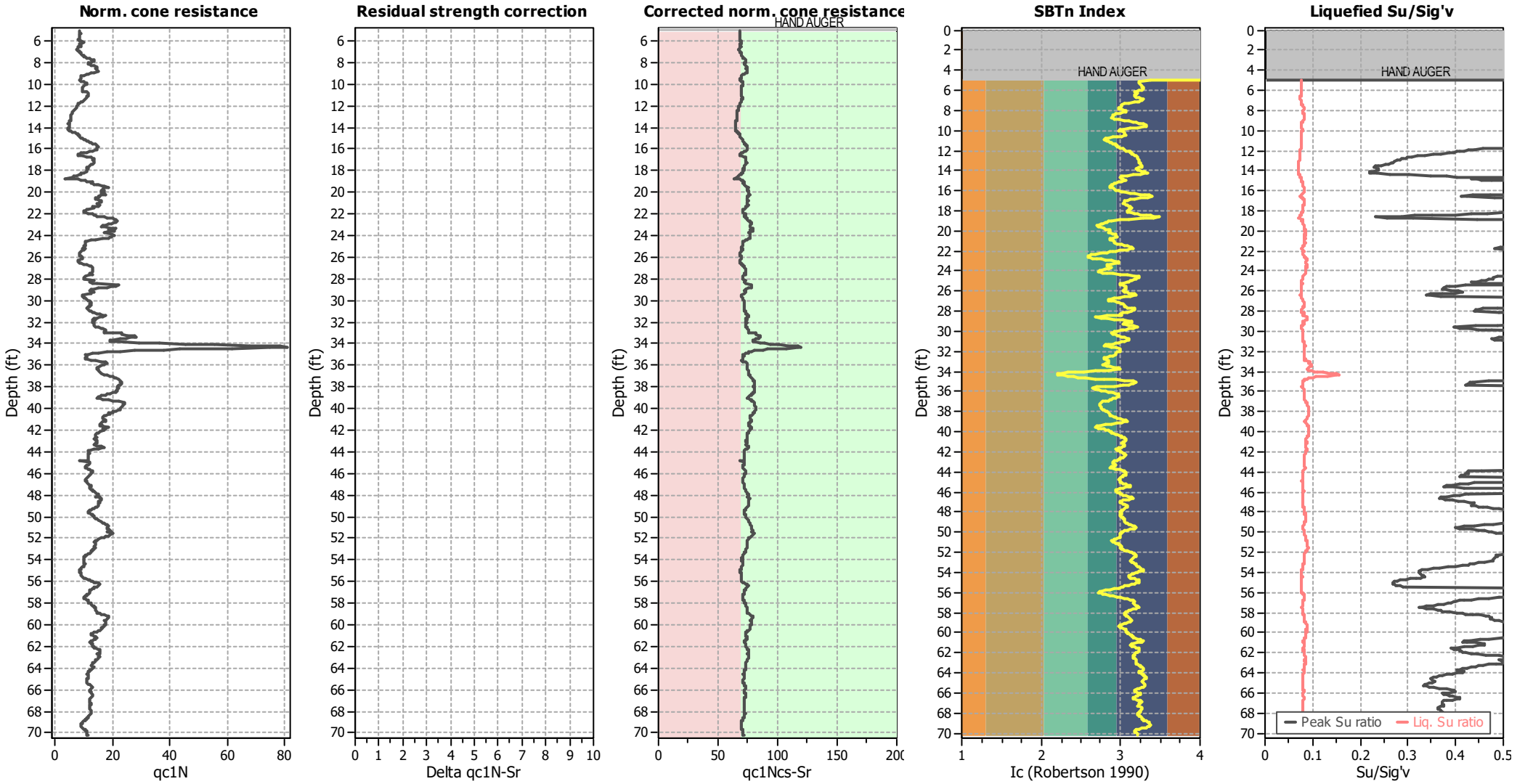
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

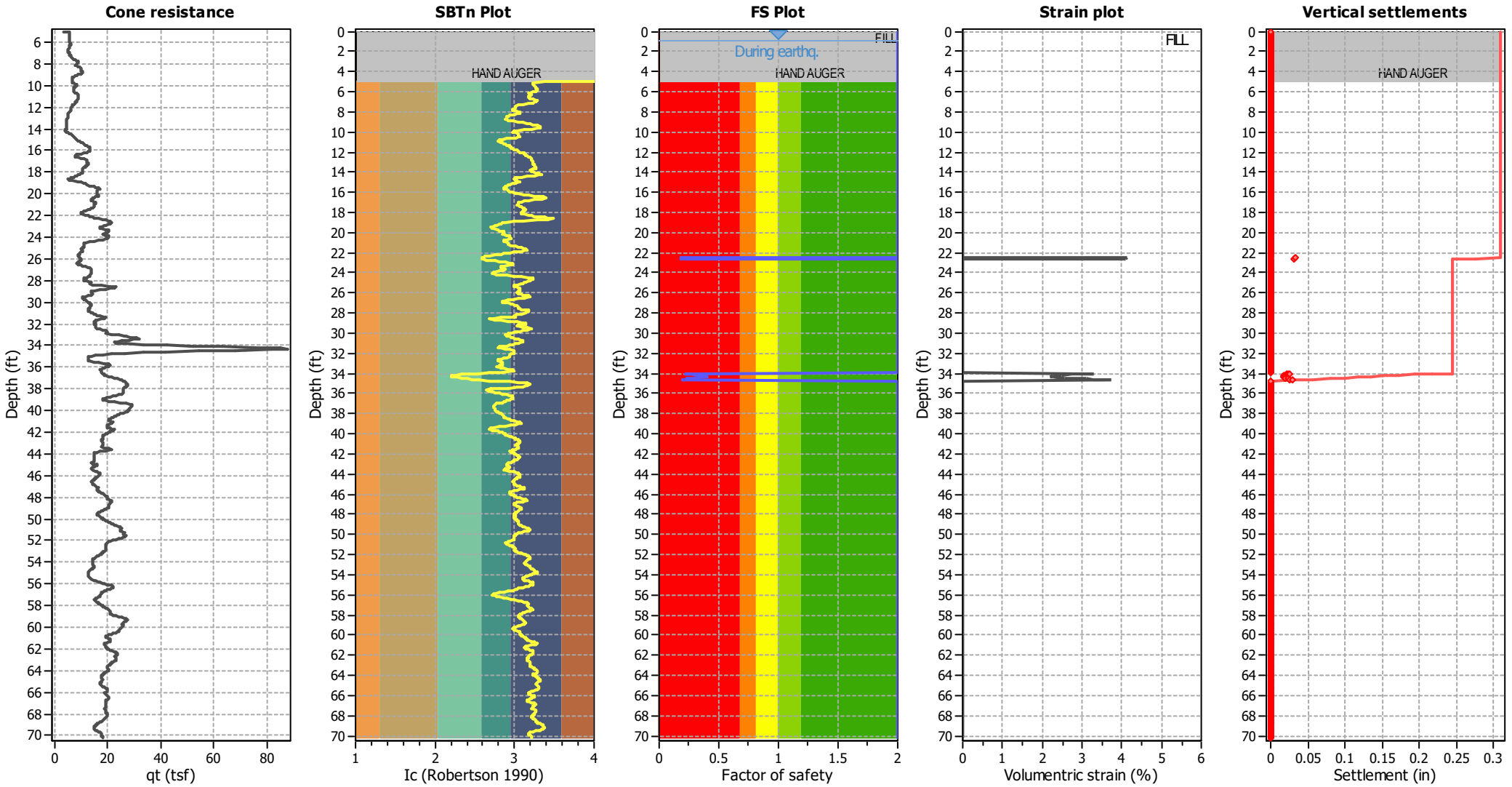
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	12.98 ft	Fill height:	1.30 ft	Limit depth:	N/A

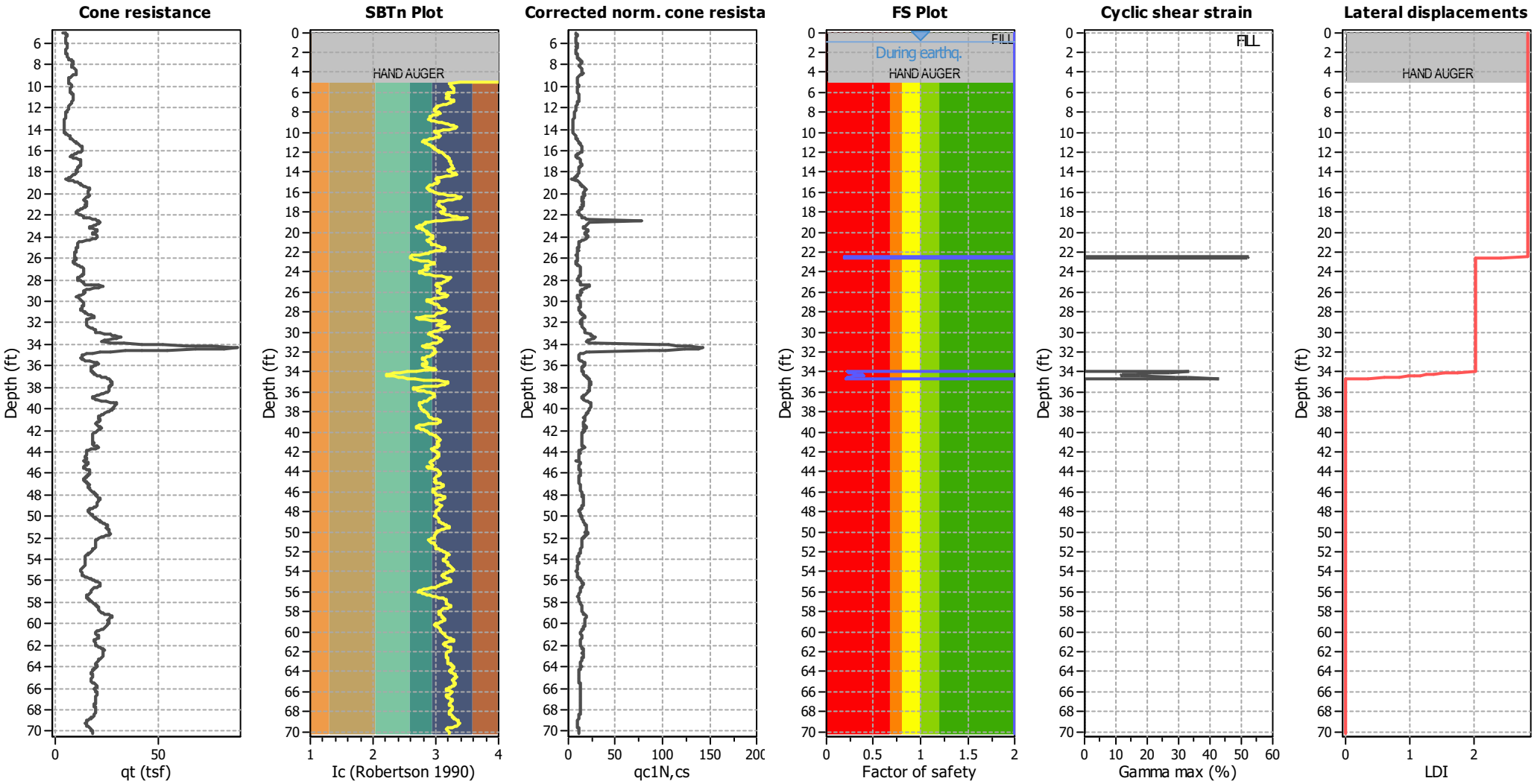
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

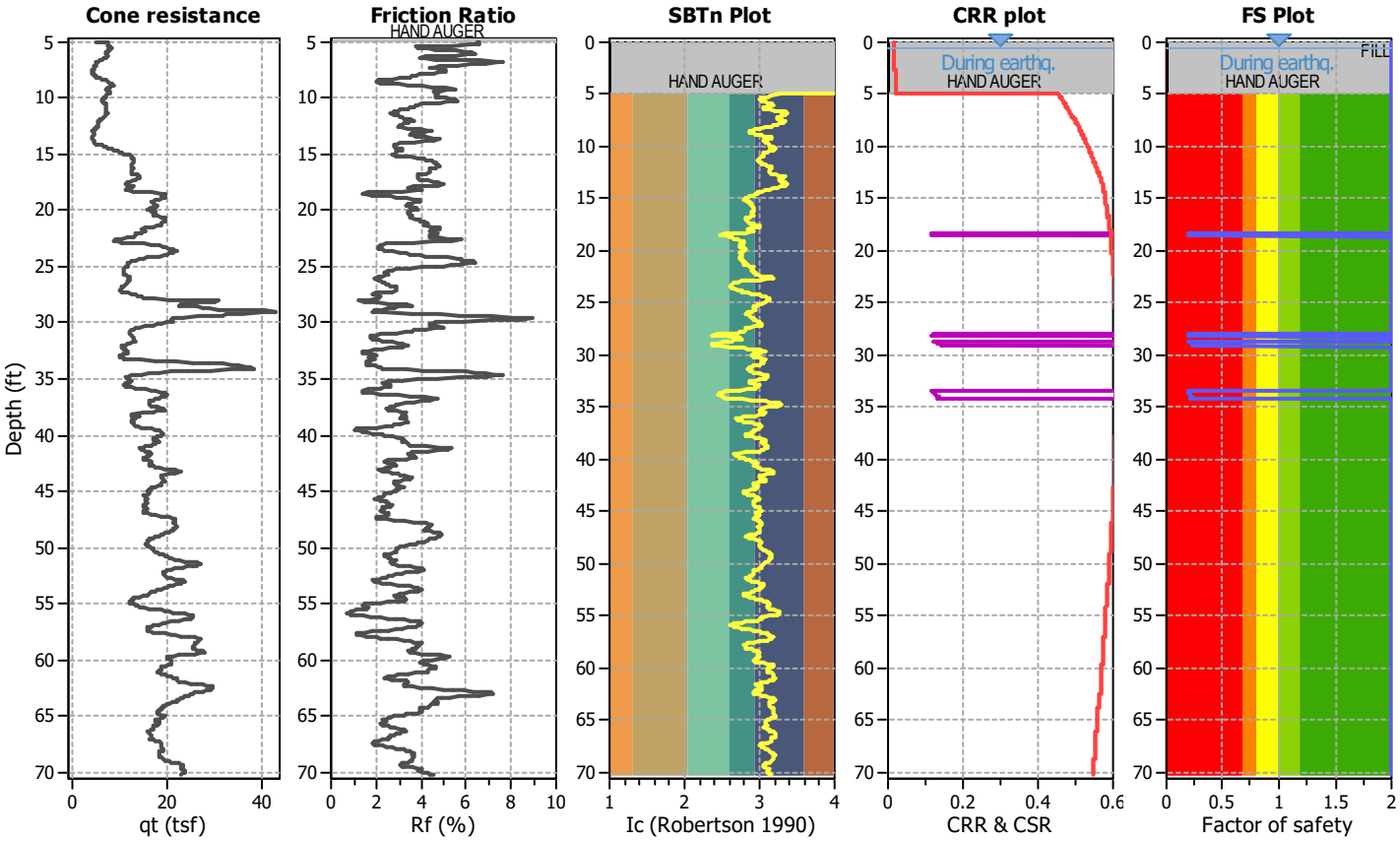
LIQUEFACTION ANALYSIS REPORT

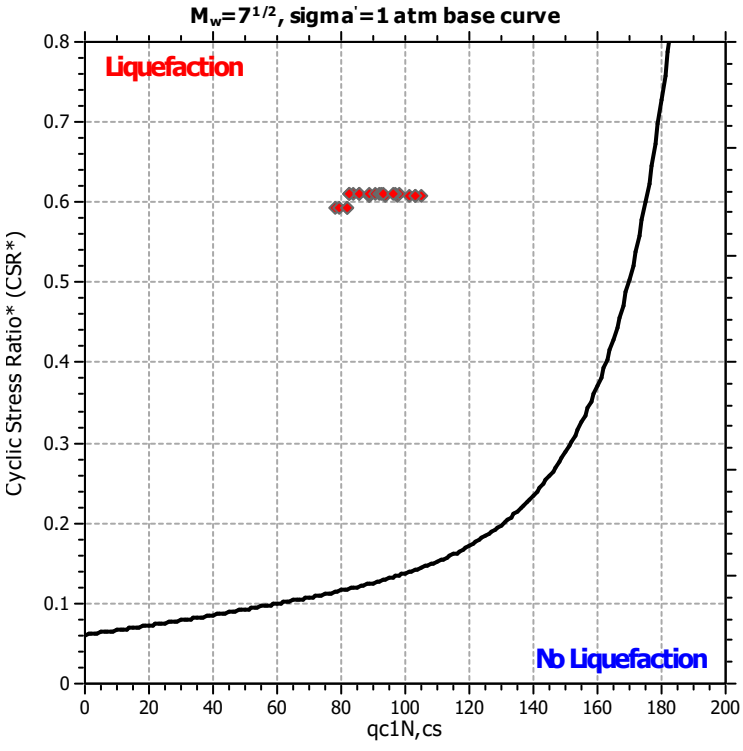
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

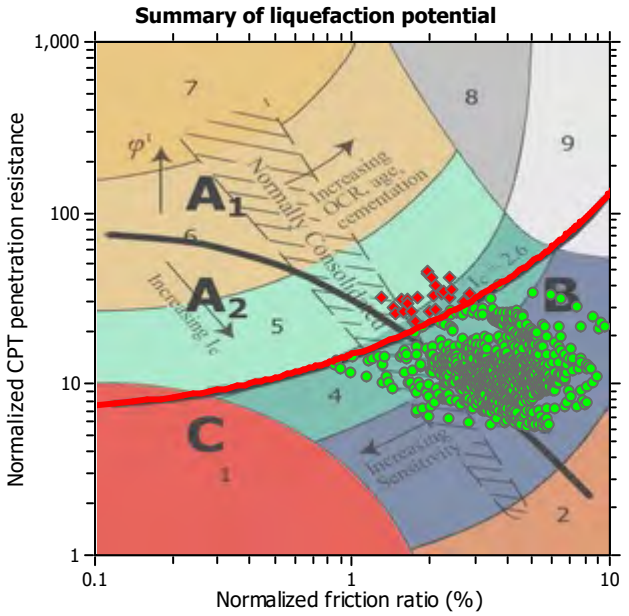
CPT file : GS26-CPT-08

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.99 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.60 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

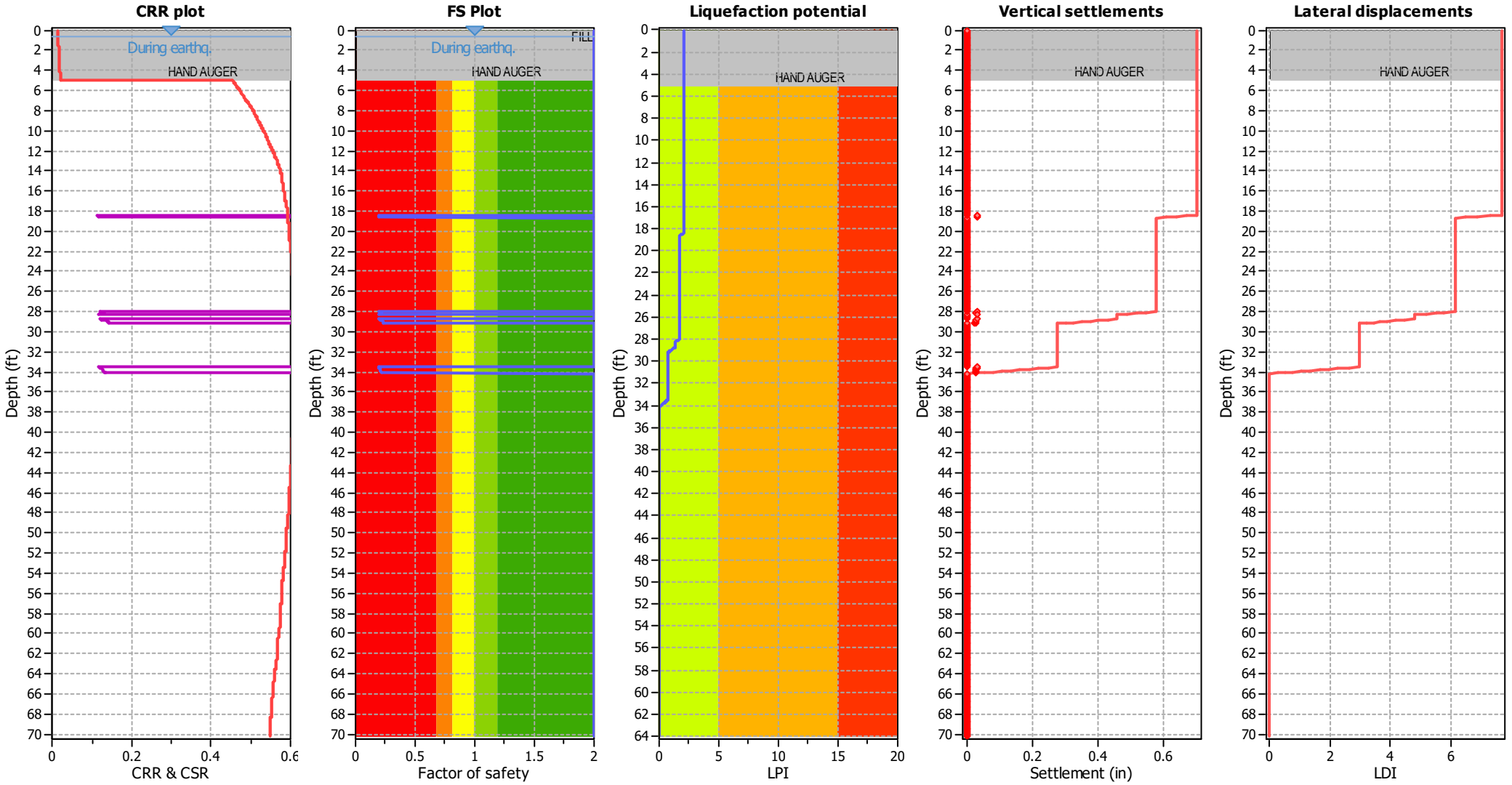






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.99 ft	Fill height:	1.60 ft	Limit depth:	N/A

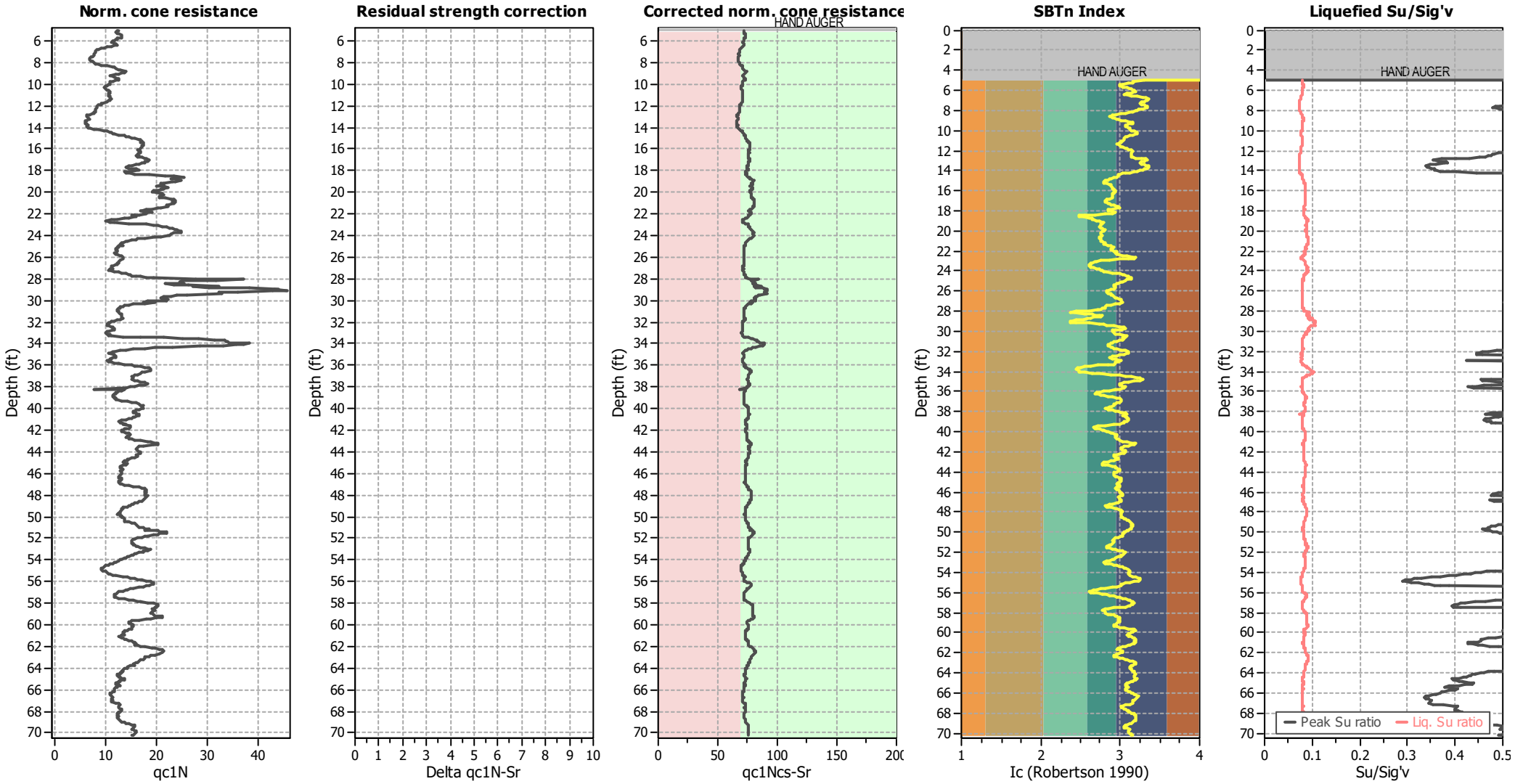
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

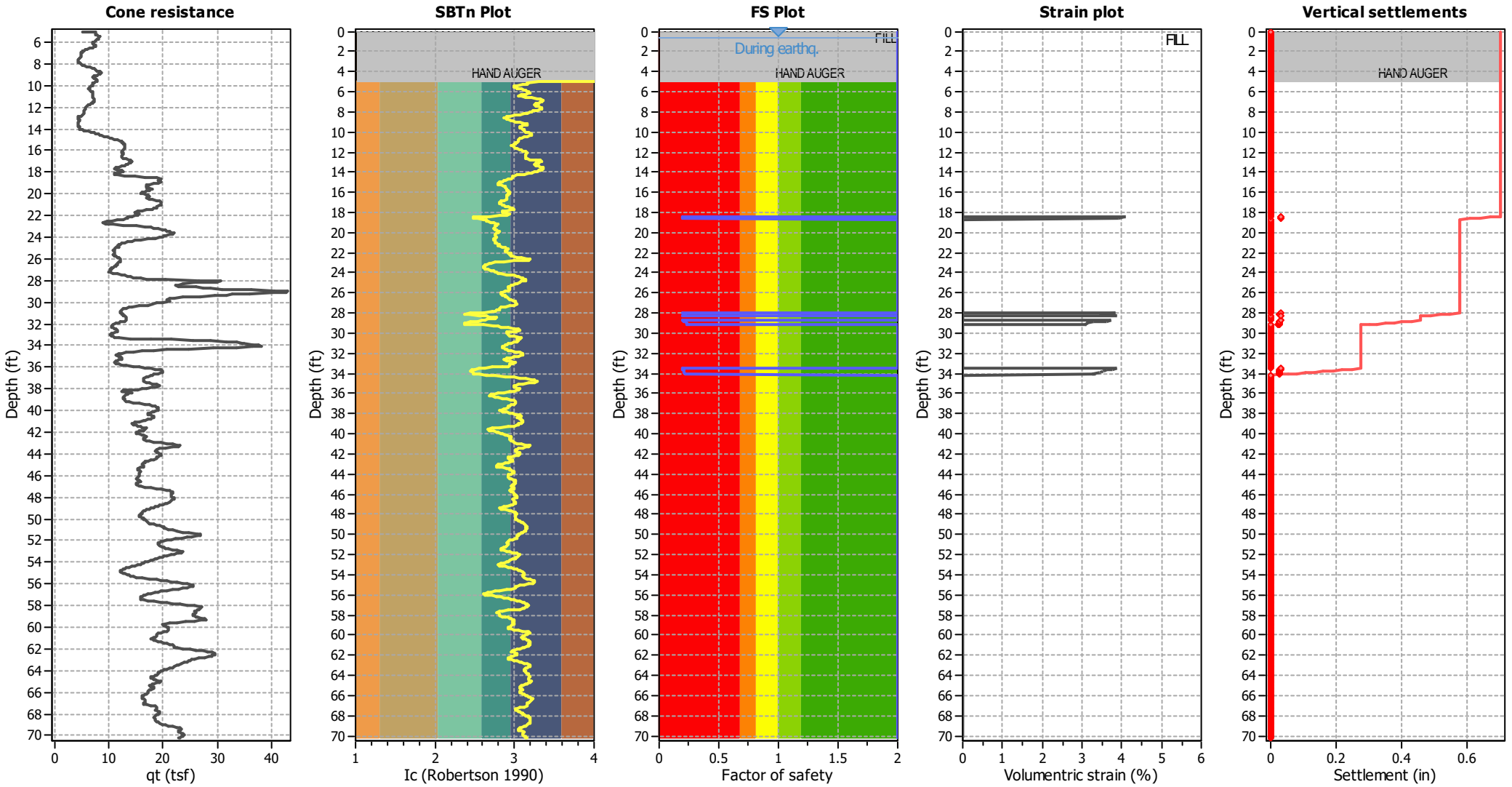
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.99 ft	Fill height:	1.60 ft	Limit depth:	N/A

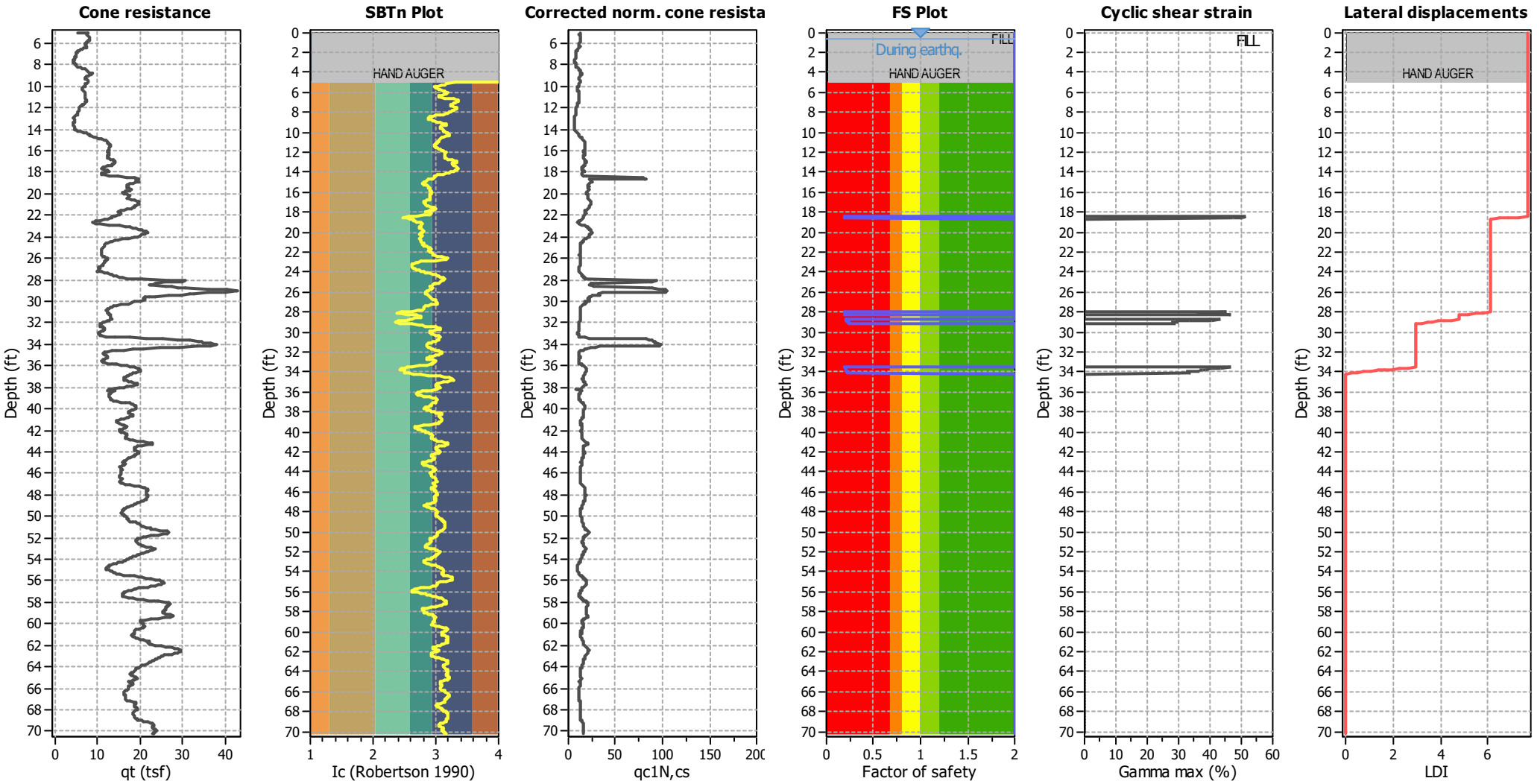
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

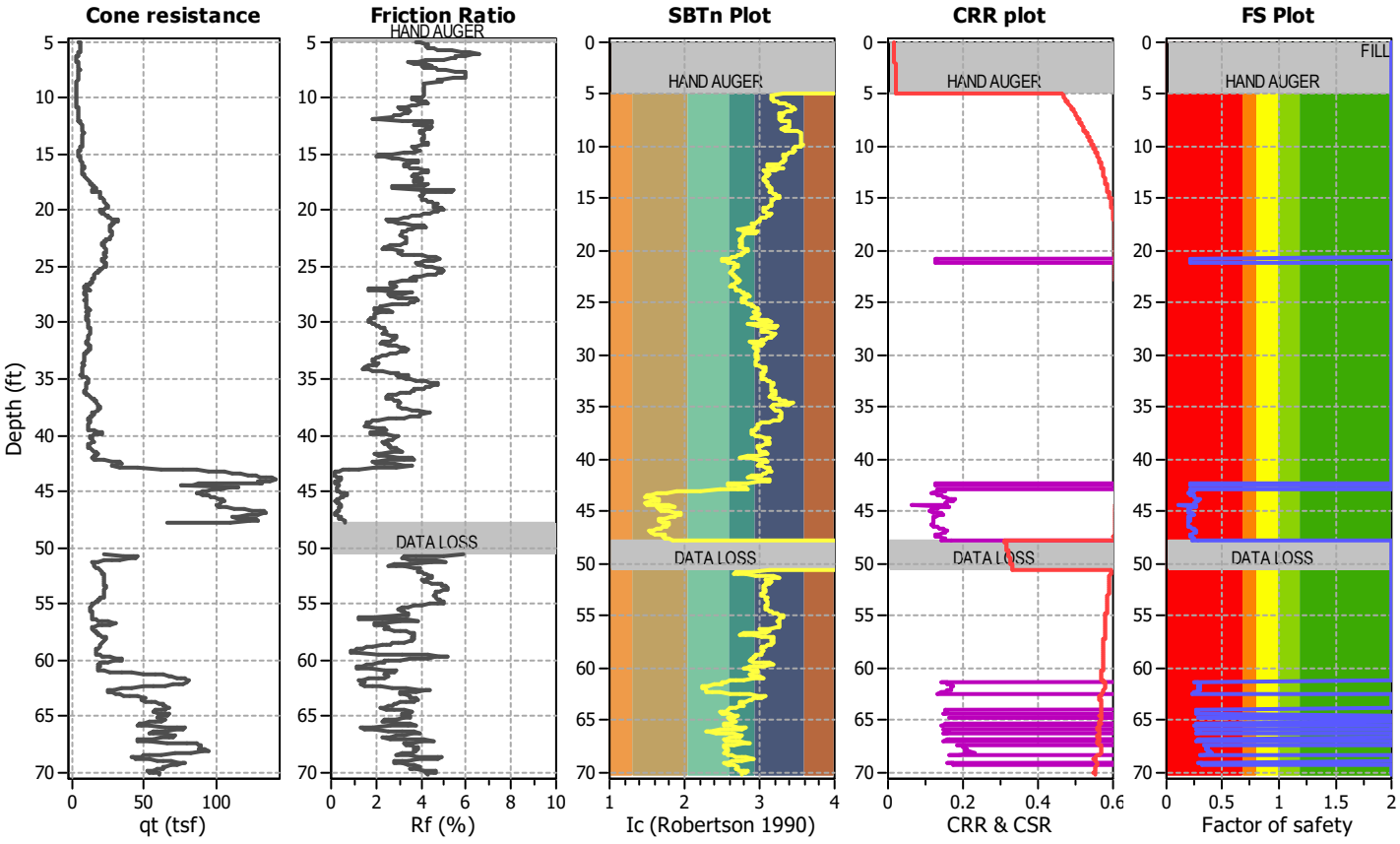
LIQUEFACTION ANALYSIS REPORT

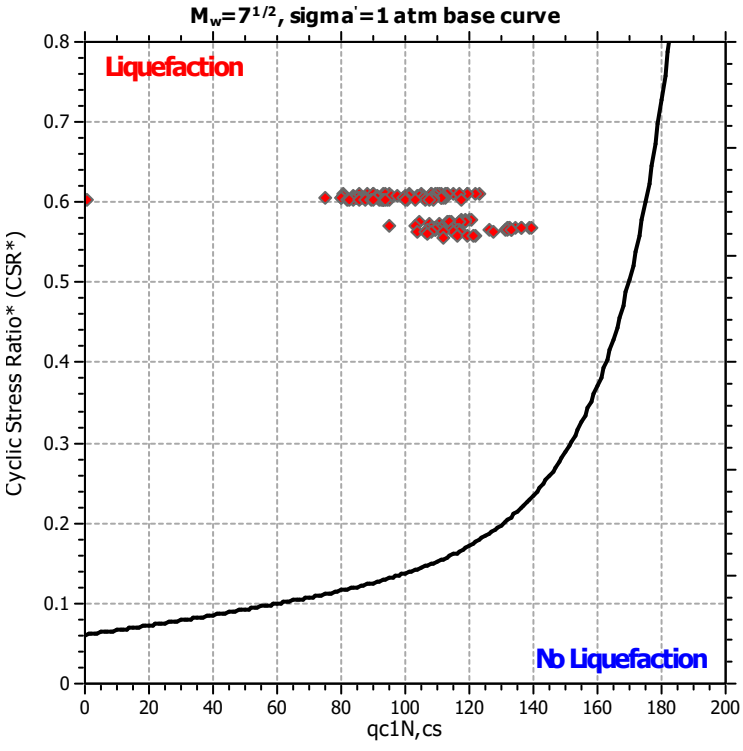
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

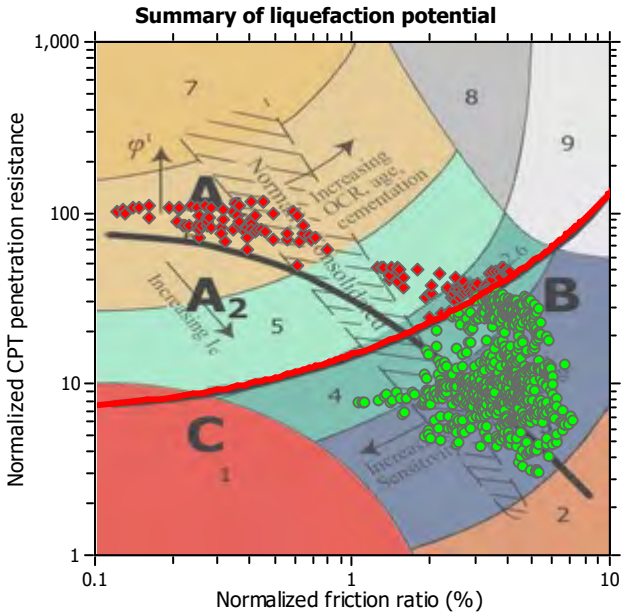
CPT file : GS26-CPT-09

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.94 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	2.30 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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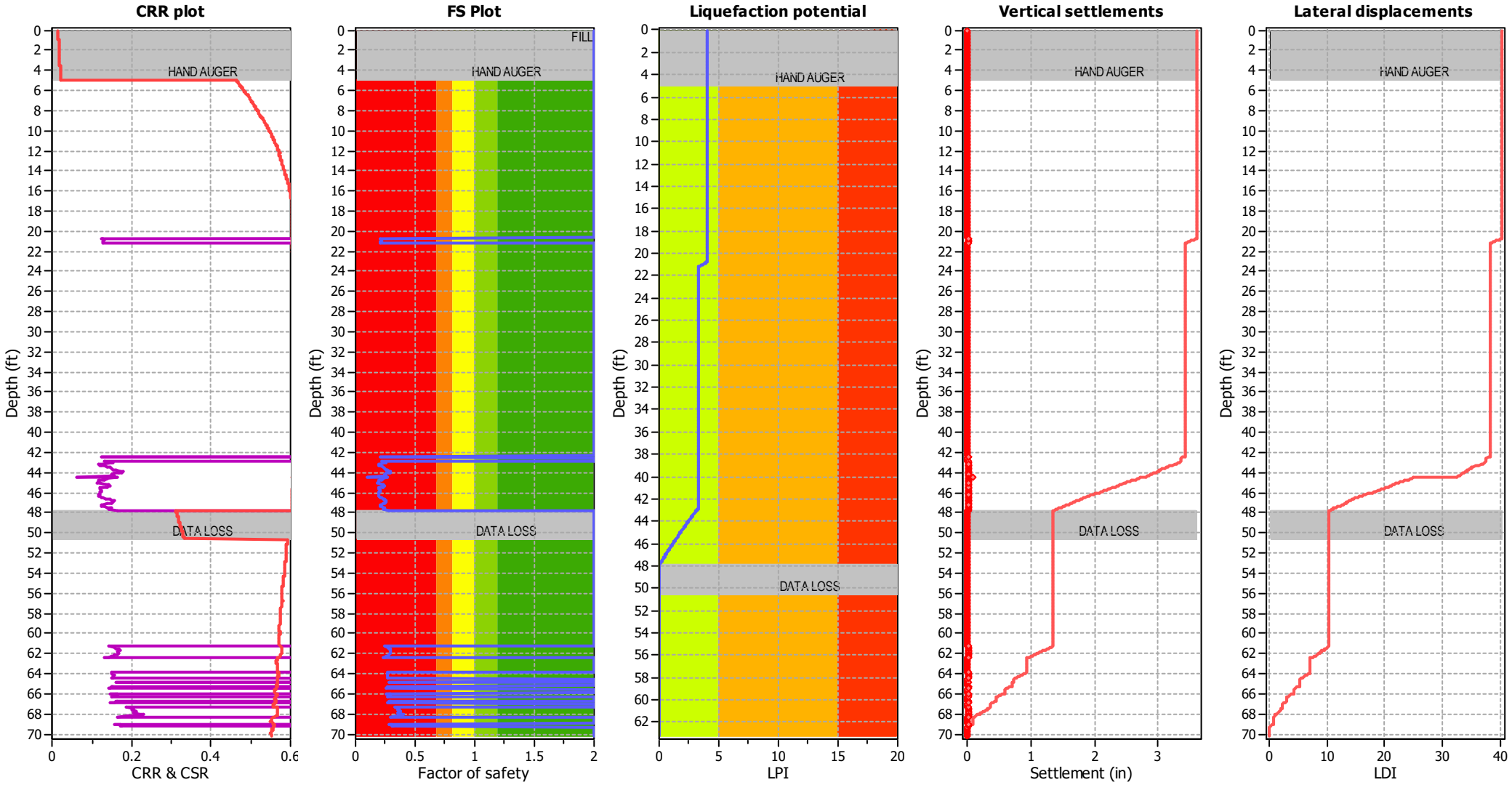
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	8.94 ft	Fill height:	2.30 ft	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
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LPI color scheme

- Very high risk
- High risk
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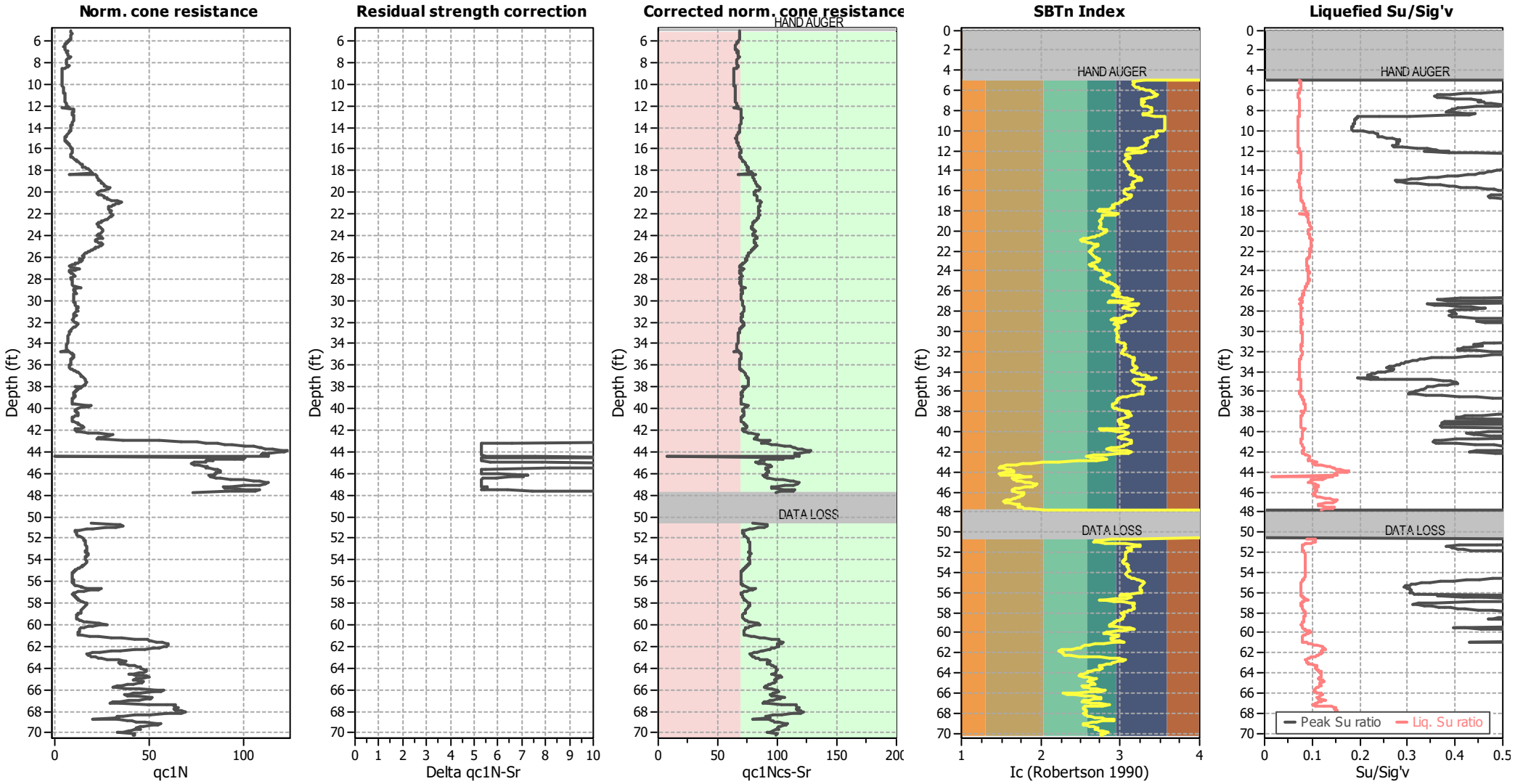
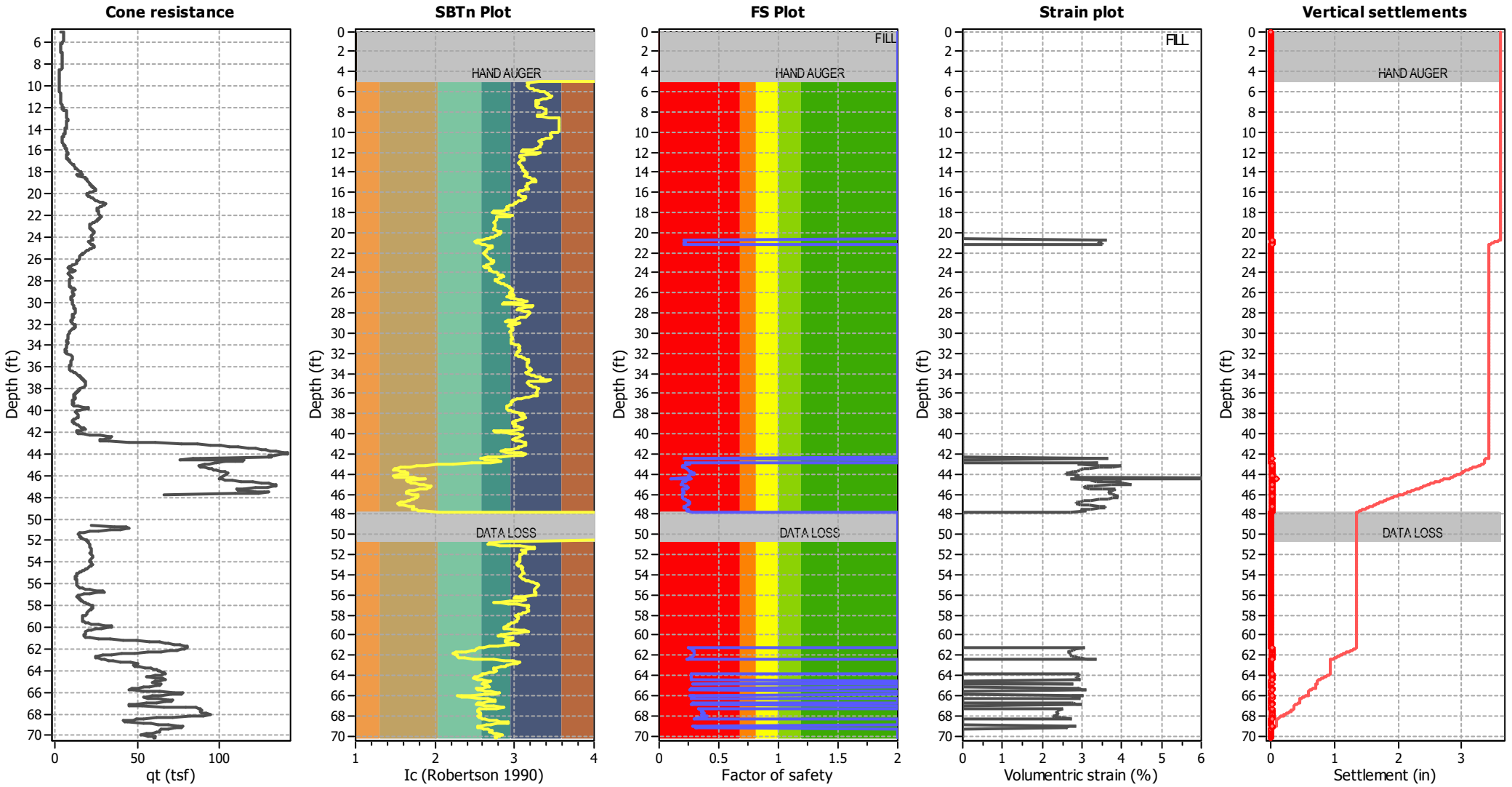
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	8.94 ft	Fill height:	2.30 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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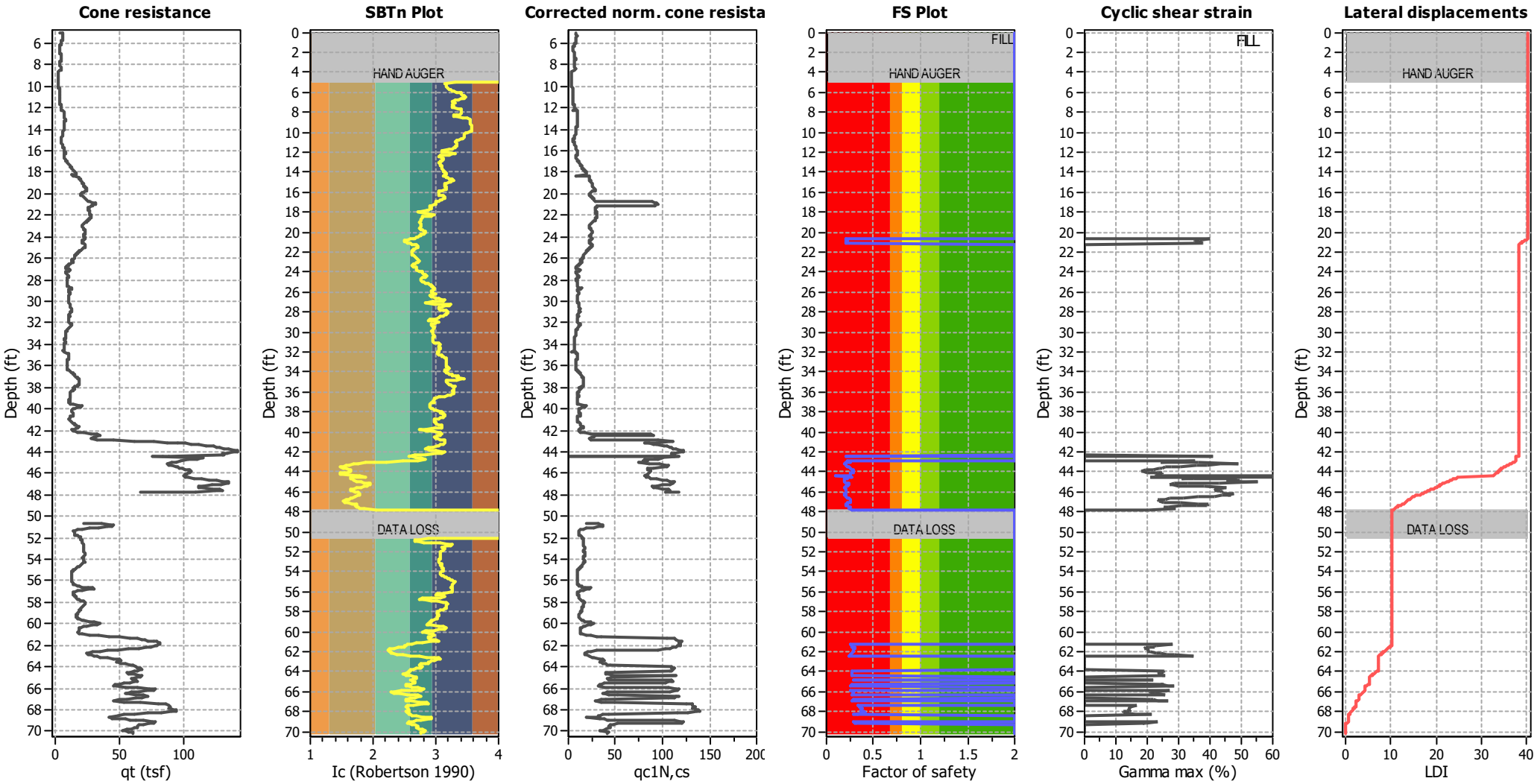
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

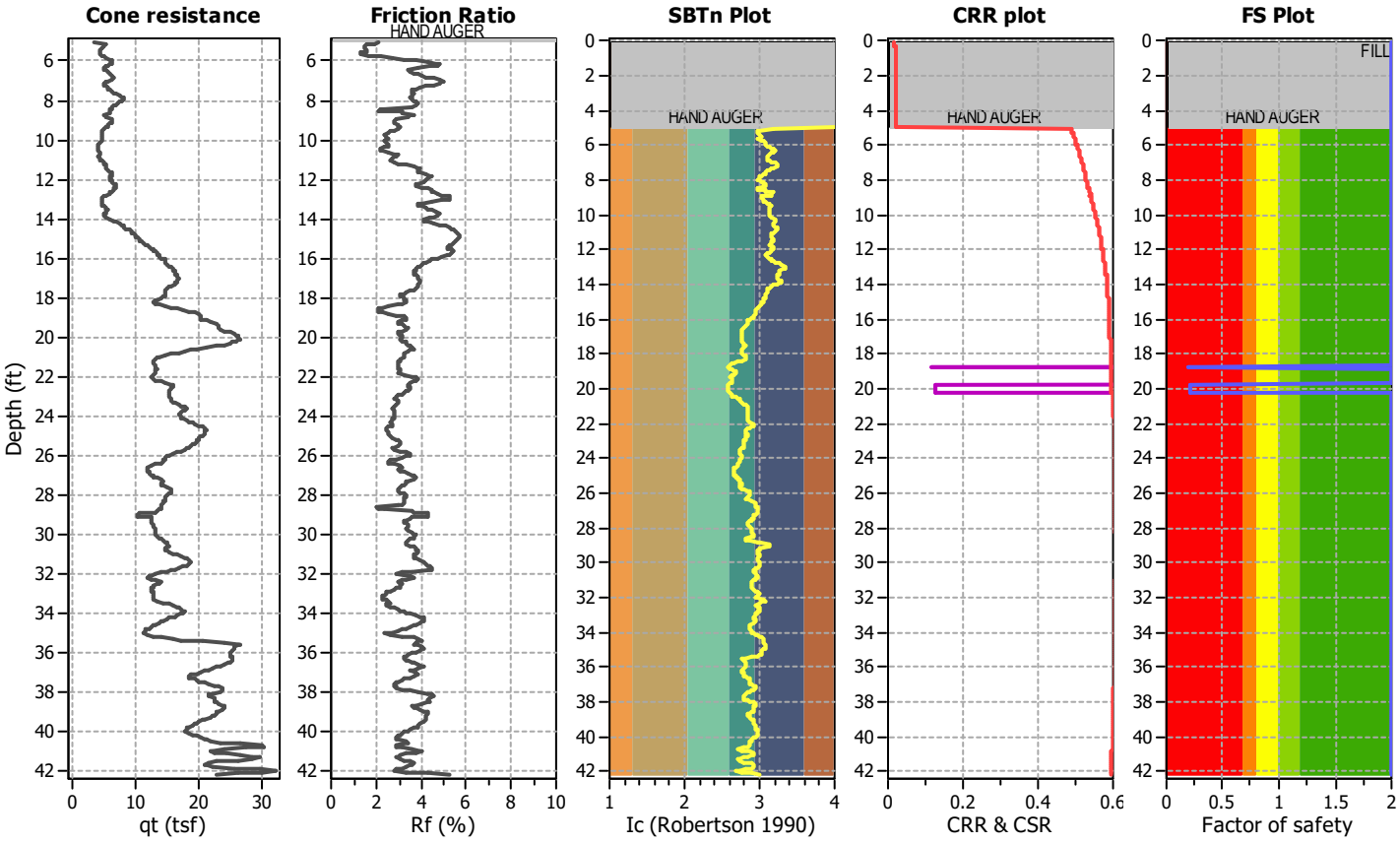
LIQUEFACTION ANALYSIS REPORT

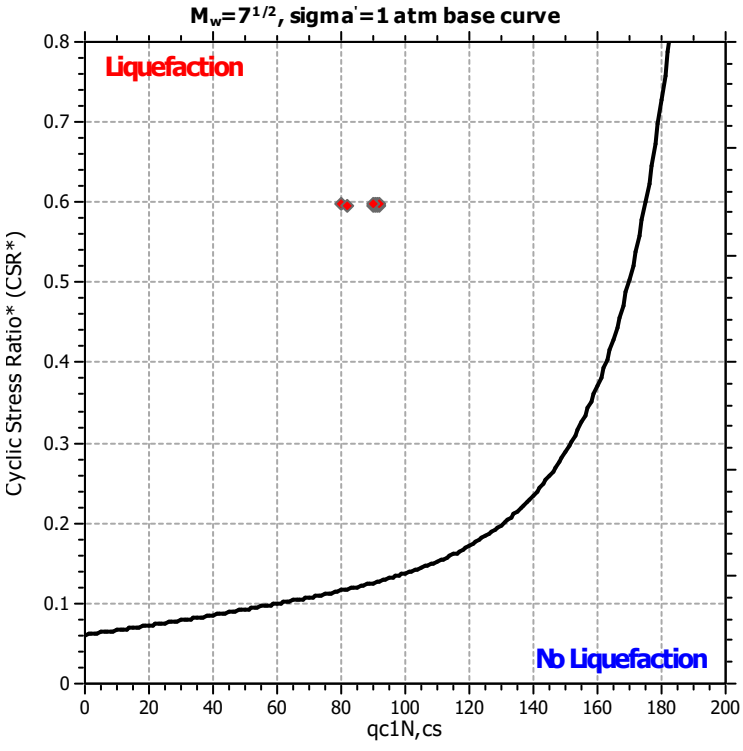
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

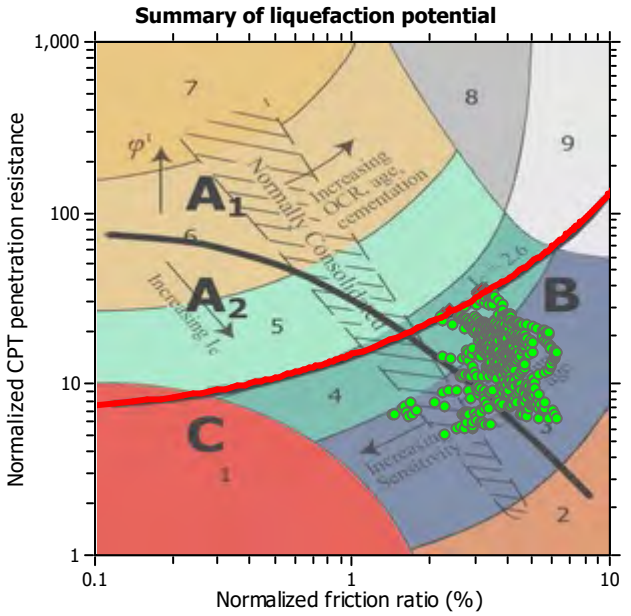
CPT file : GS26-CPT-10

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.01 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	4.10 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

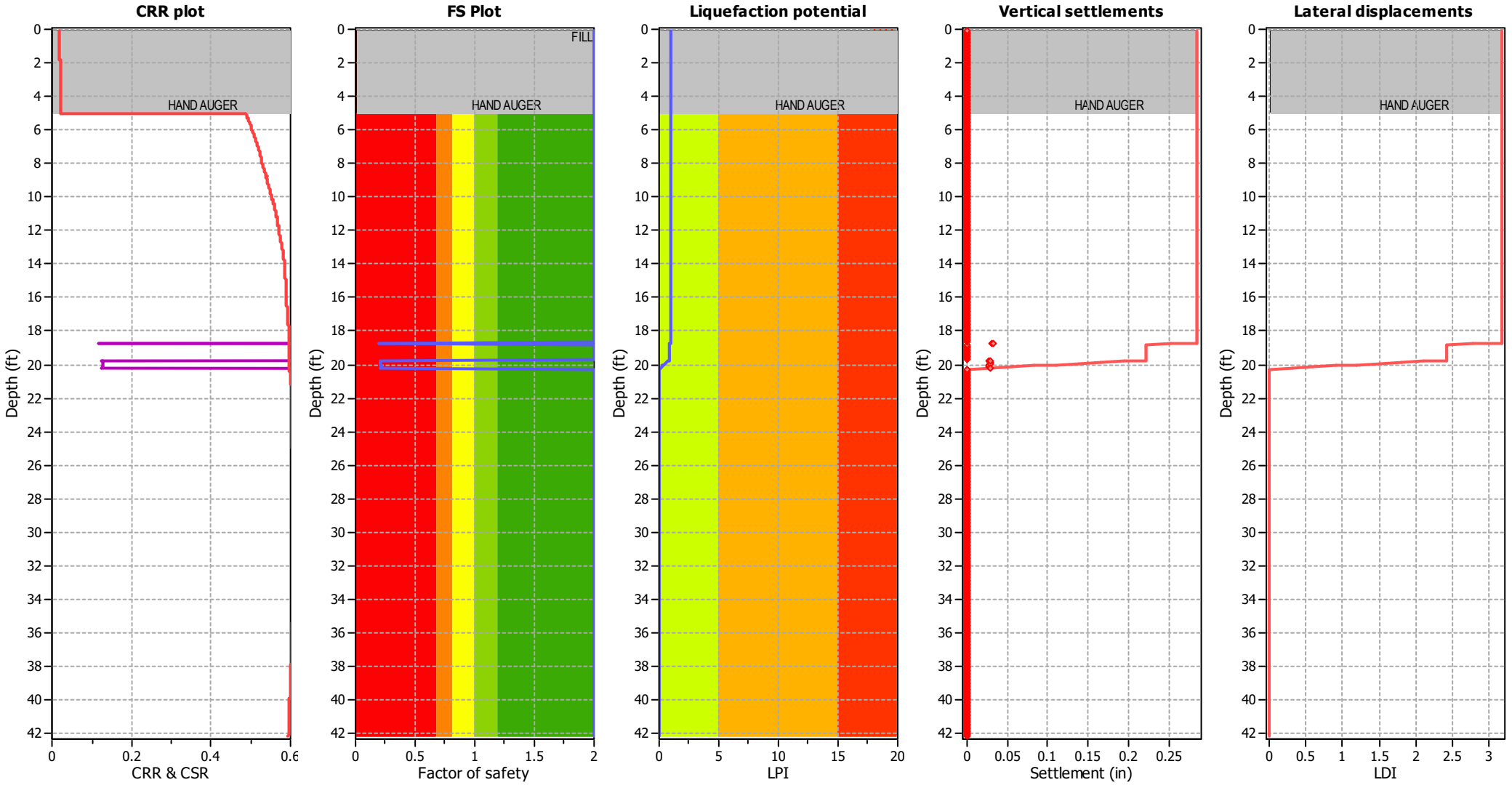






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.01 ft	Fill height:	4.10 ft	Limit depth:	N/A

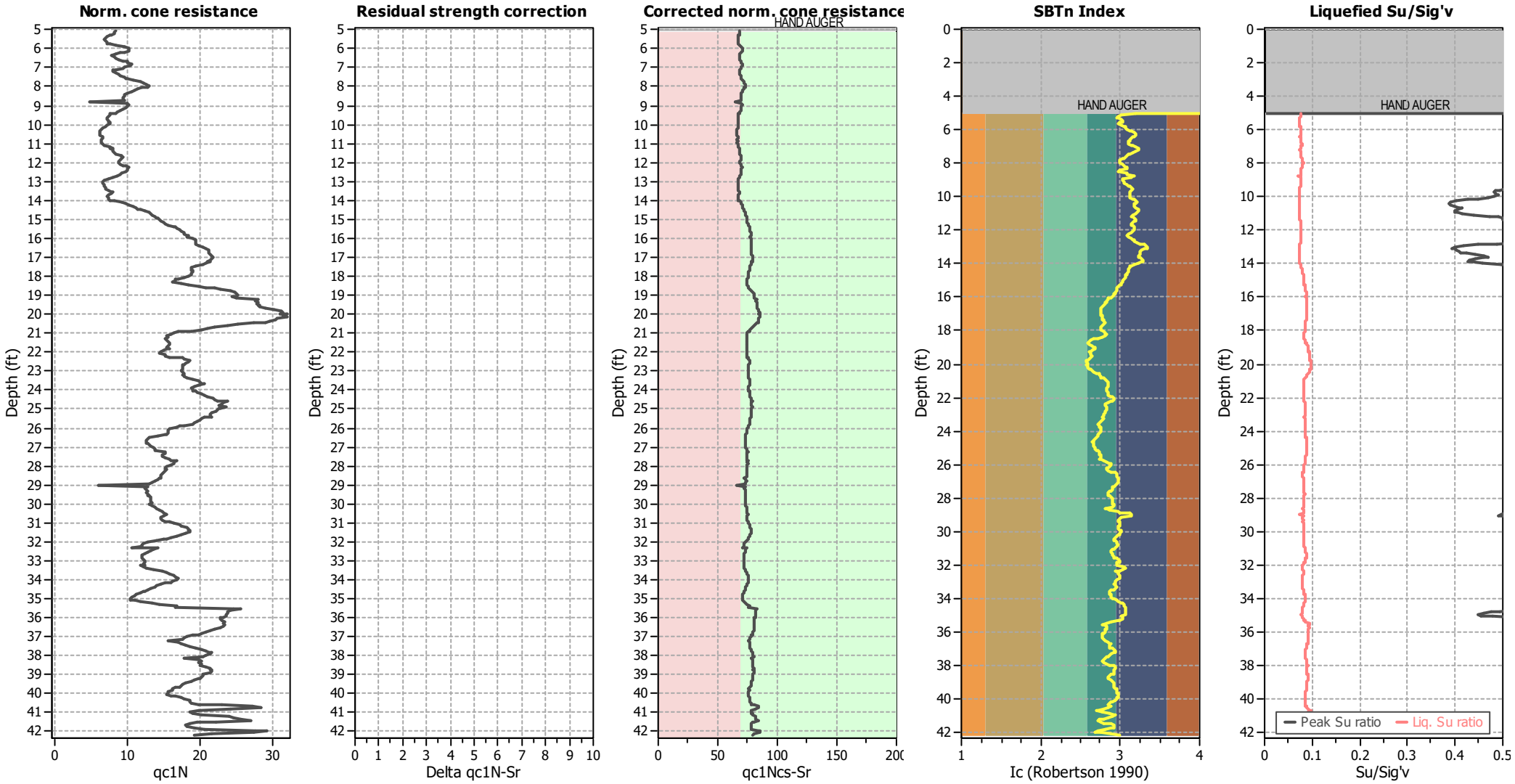
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

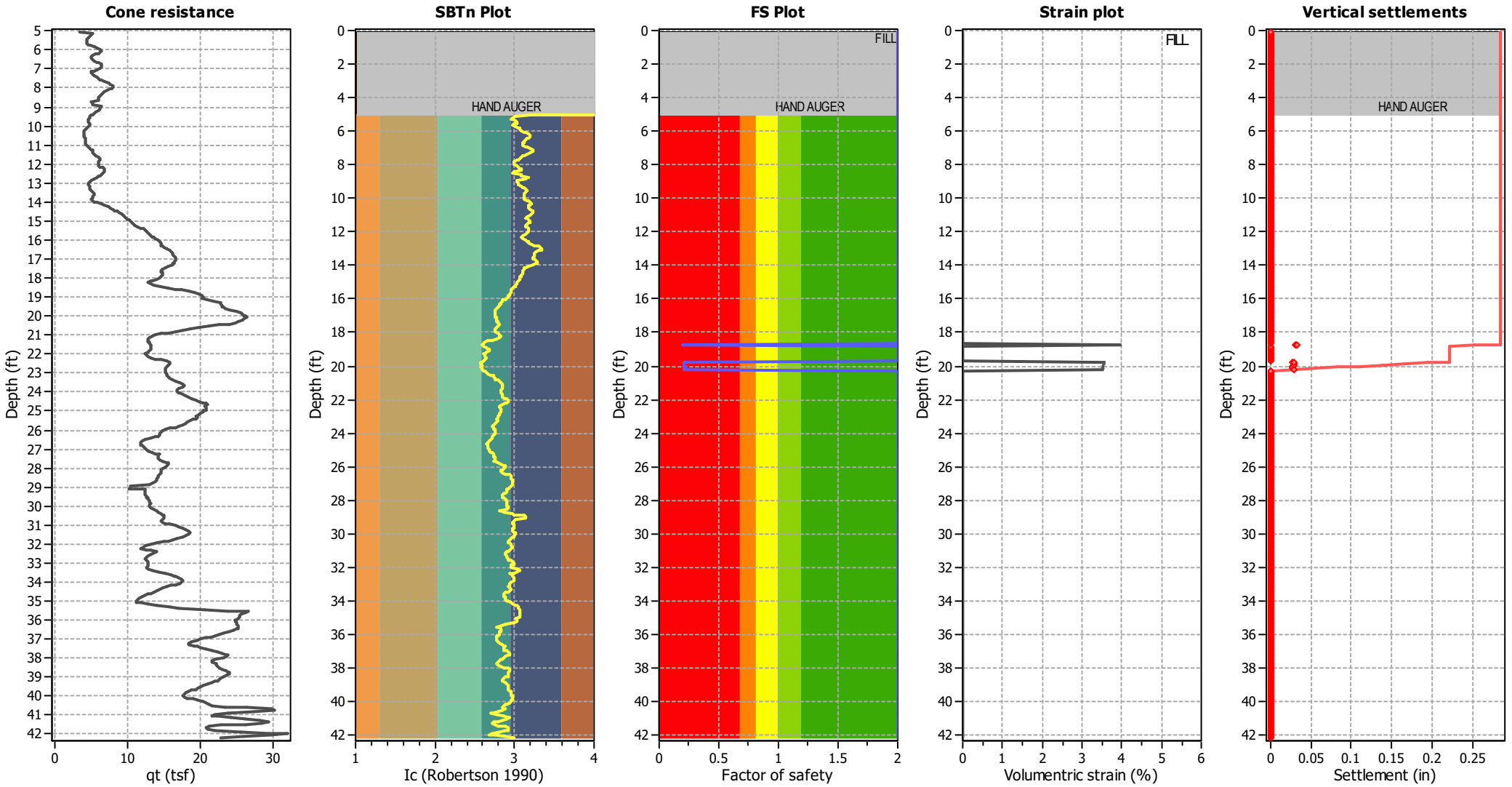
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.01 ft	Fill height:	4.10 ft	Limit depth:	N/A

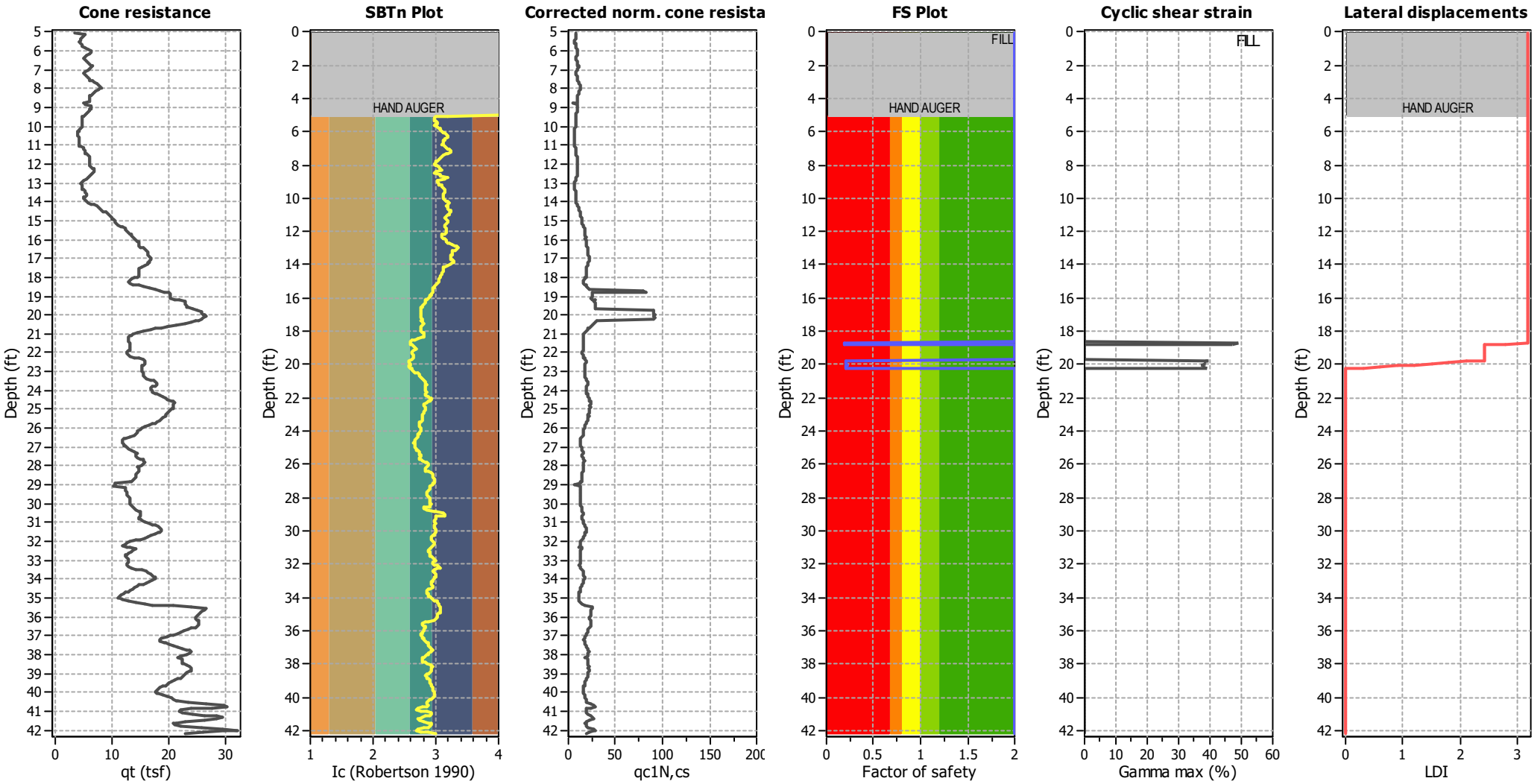
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

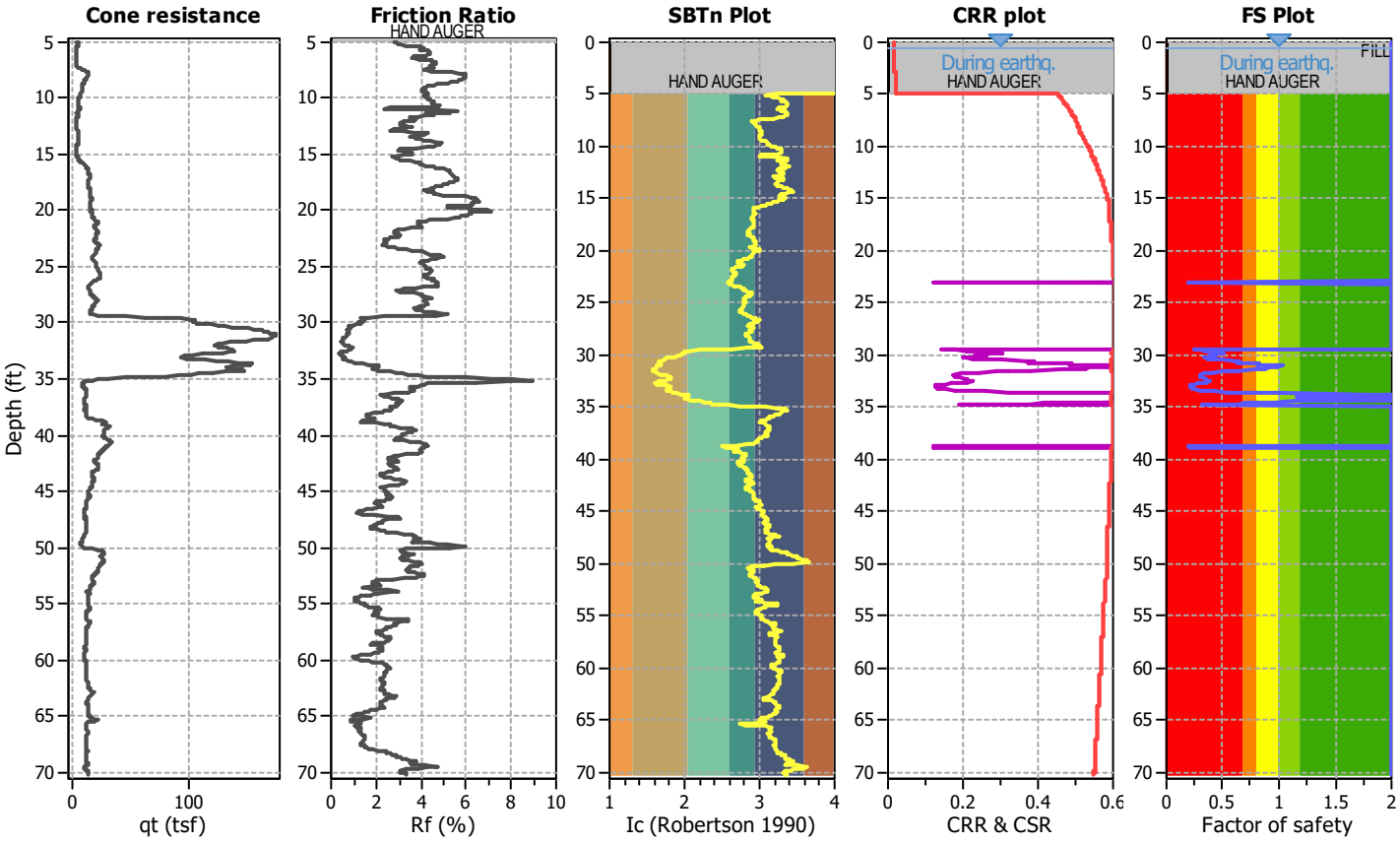
LIQUEFACTION ANALYSIS REPORT

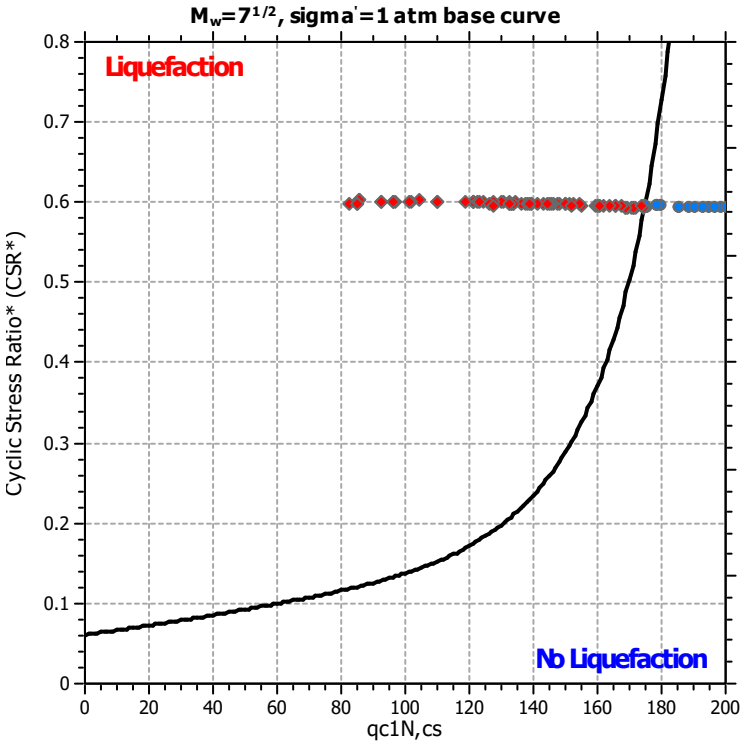
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

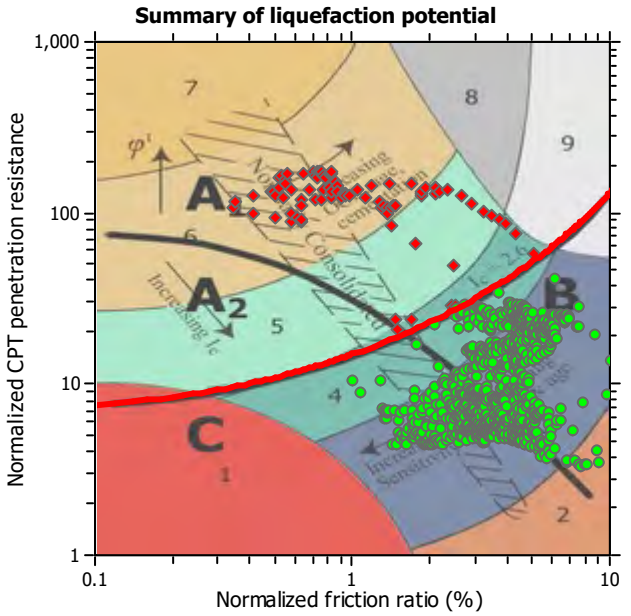
CPT file : GS26-CPT-11

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.05 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.50 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

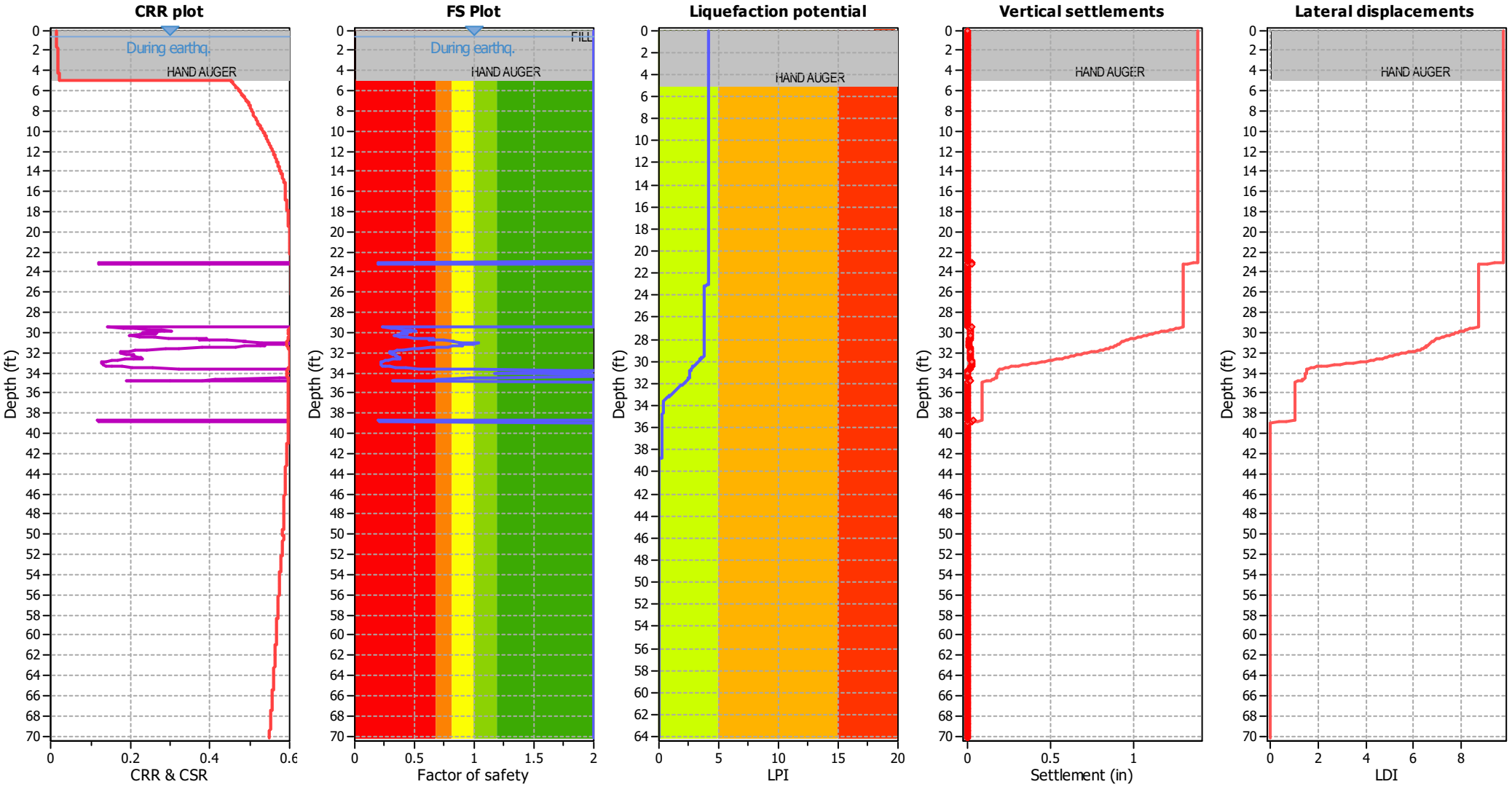






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.05 ft	Fill height:	1.50 ft	Limit depth:	N/A

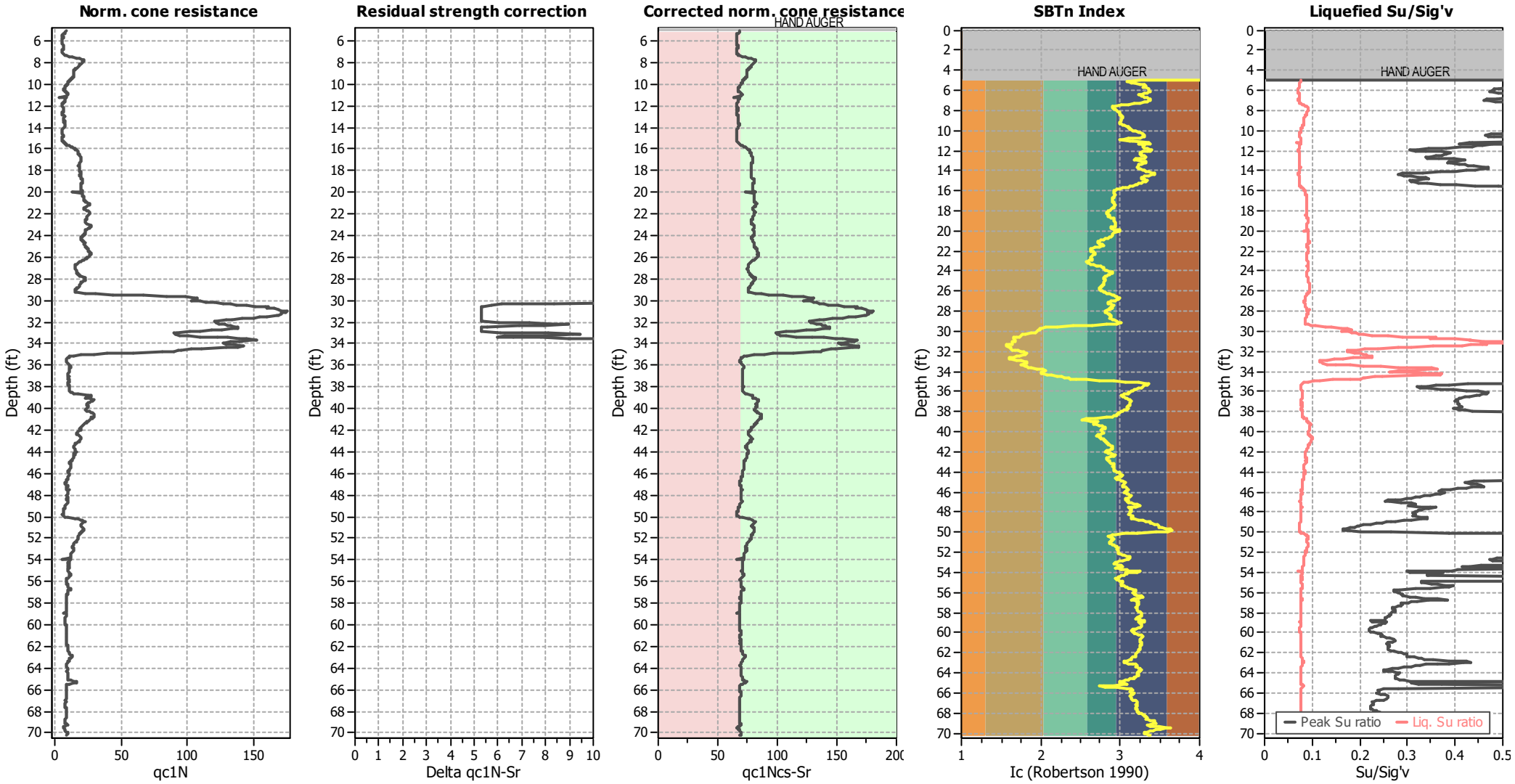
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

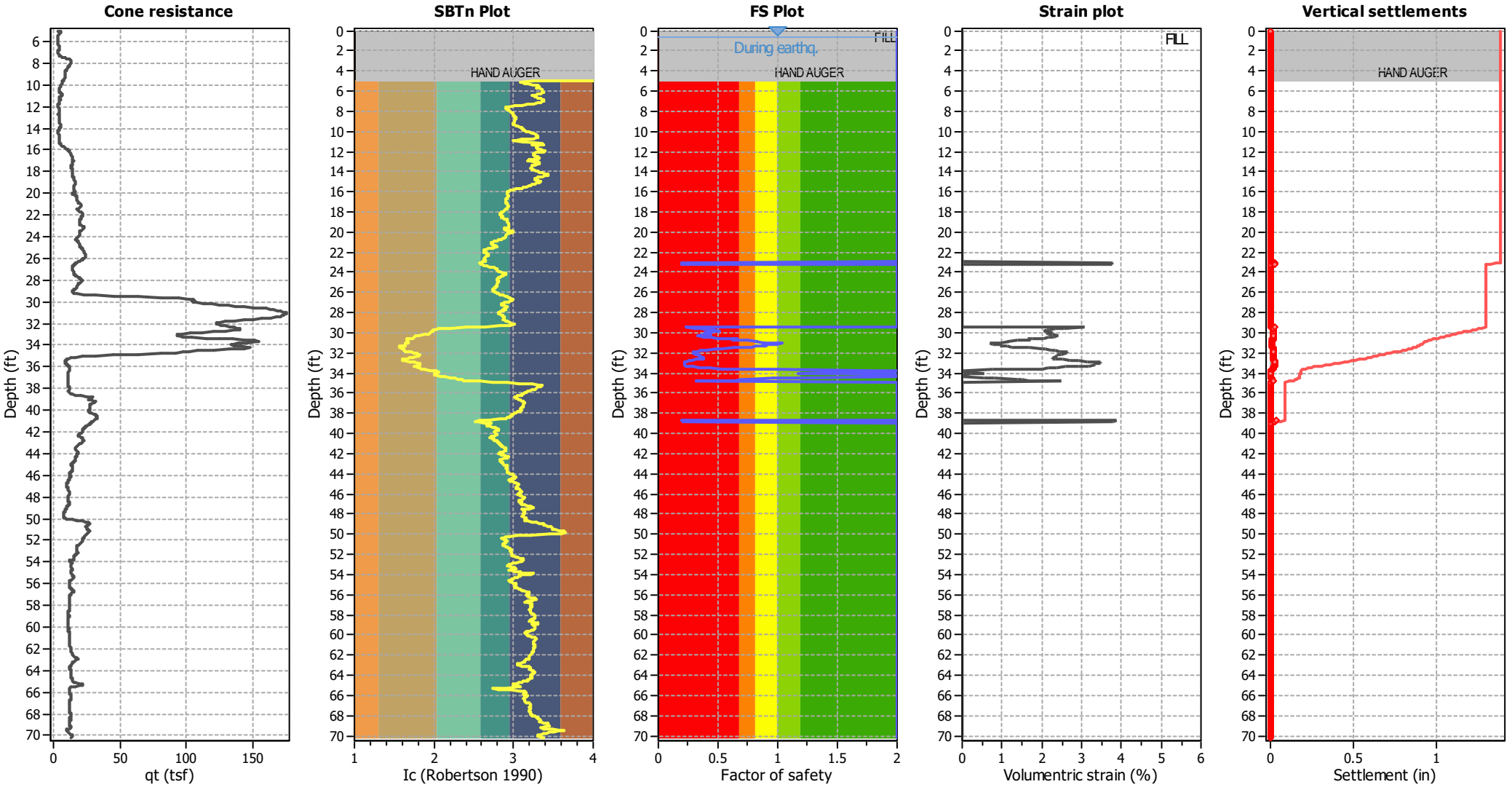
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.05 ft	Fill height:	1.50 ft	Limit depth:	N/A

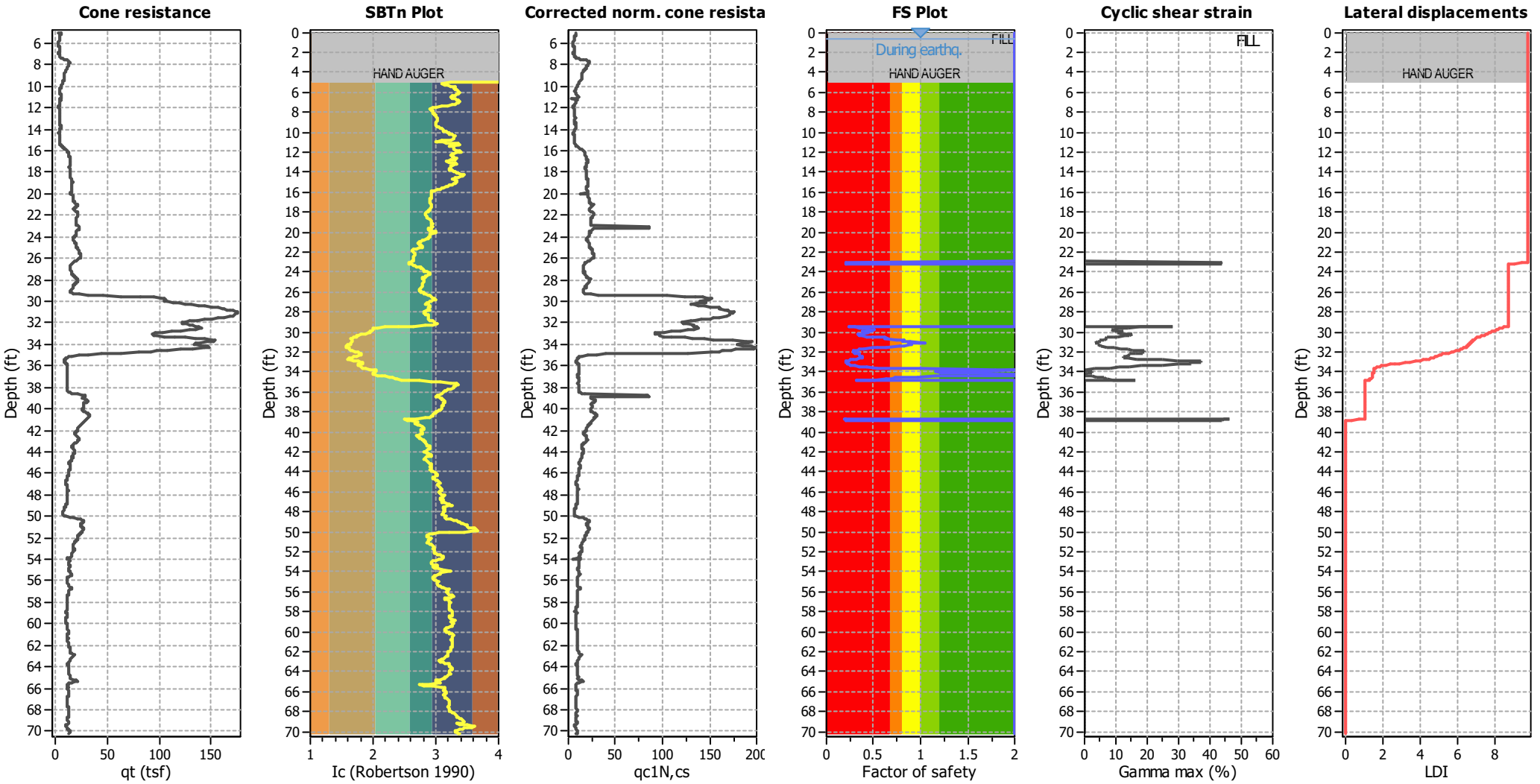
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

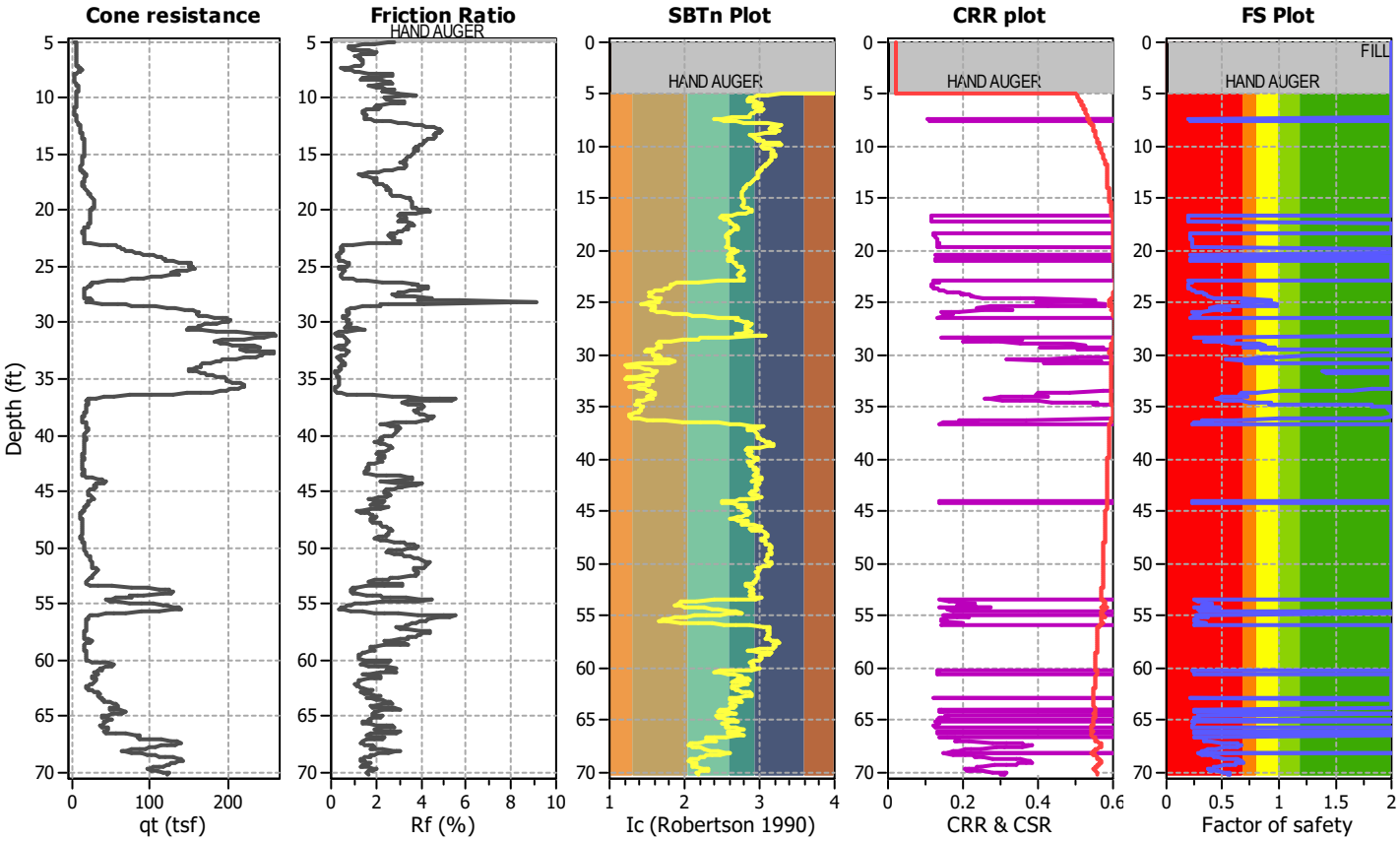
LIQUEFACTION ANALYSIS REPORT

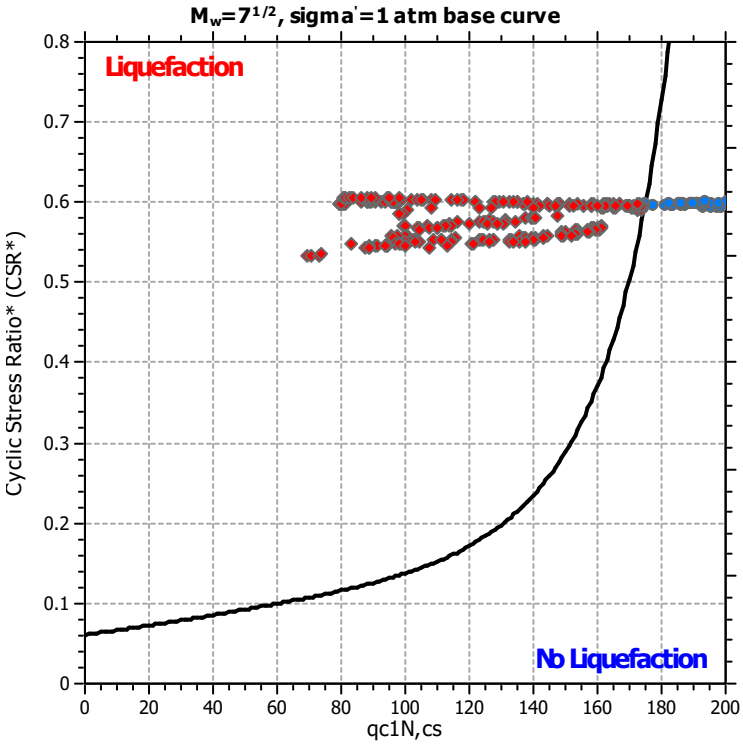
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

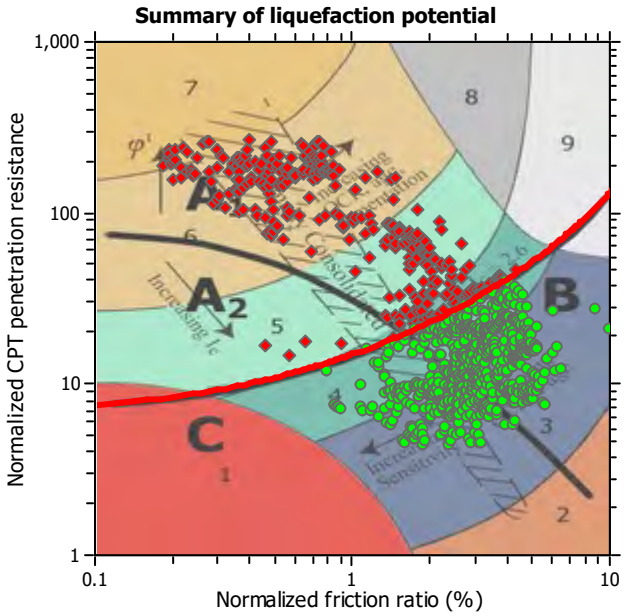
CPT file : GS26-CPT-12

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	3.02 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	5.40 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

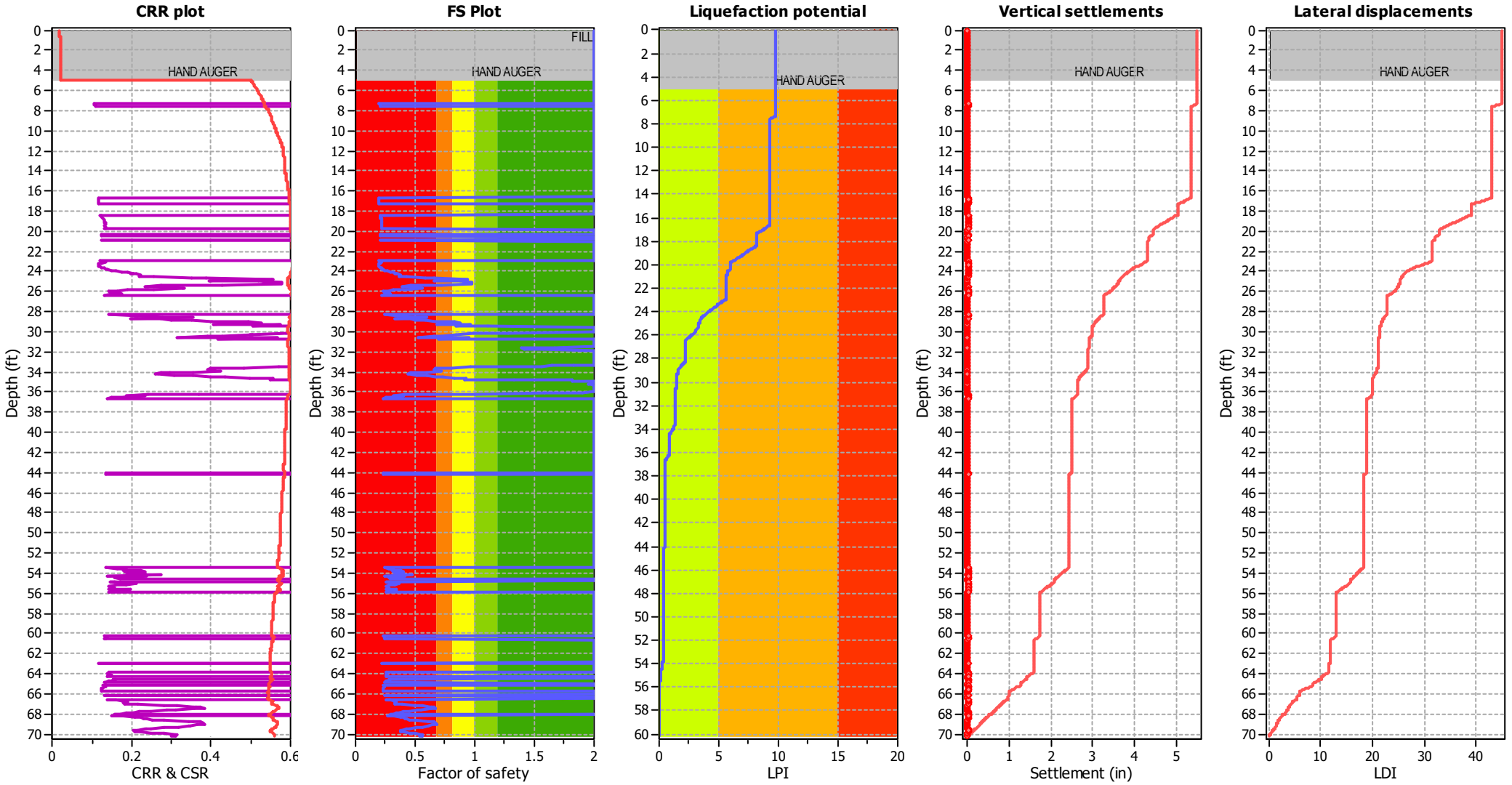






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	3.02 ft	Fill height:	5.40 ft	Limit depth:	N/A

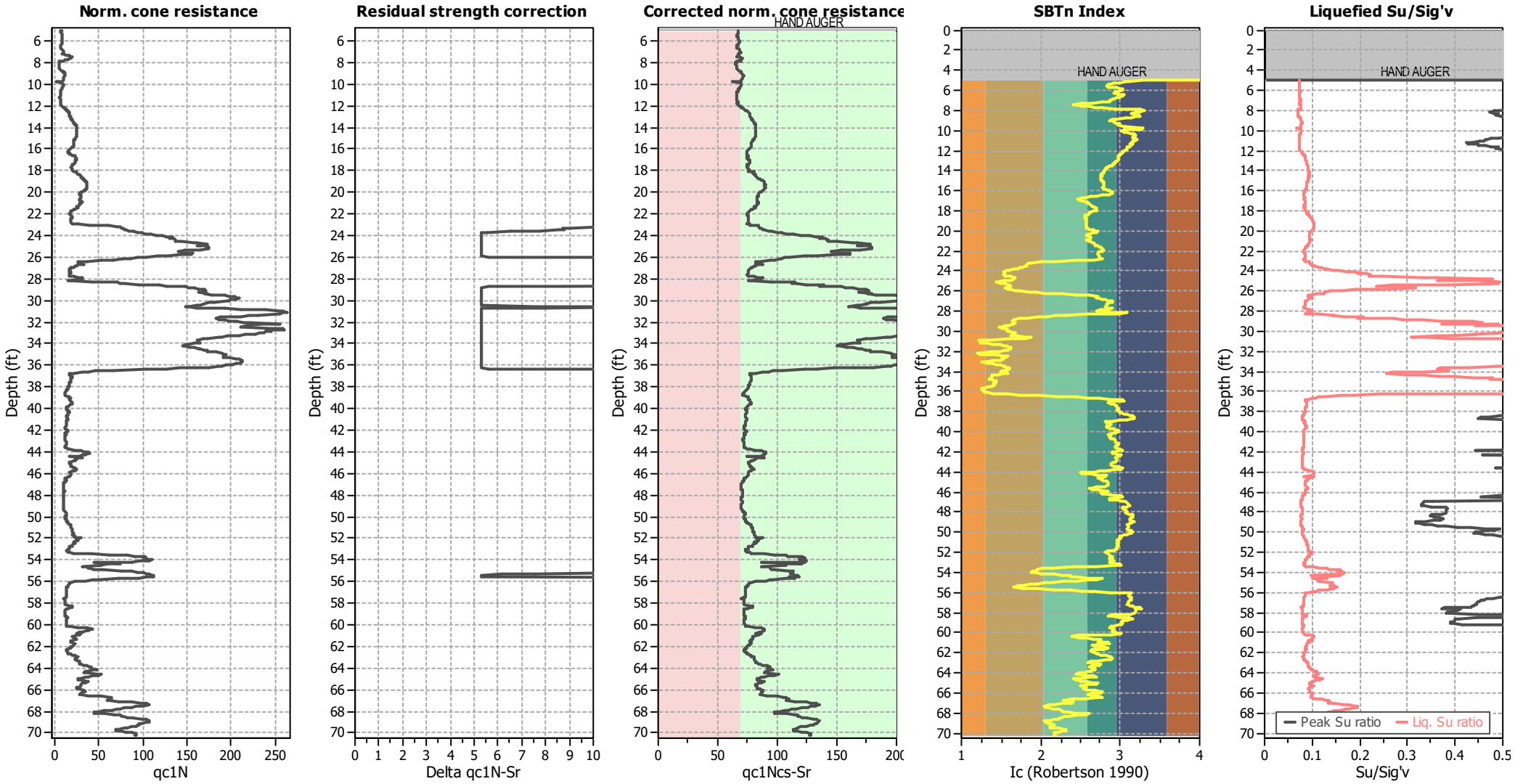
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

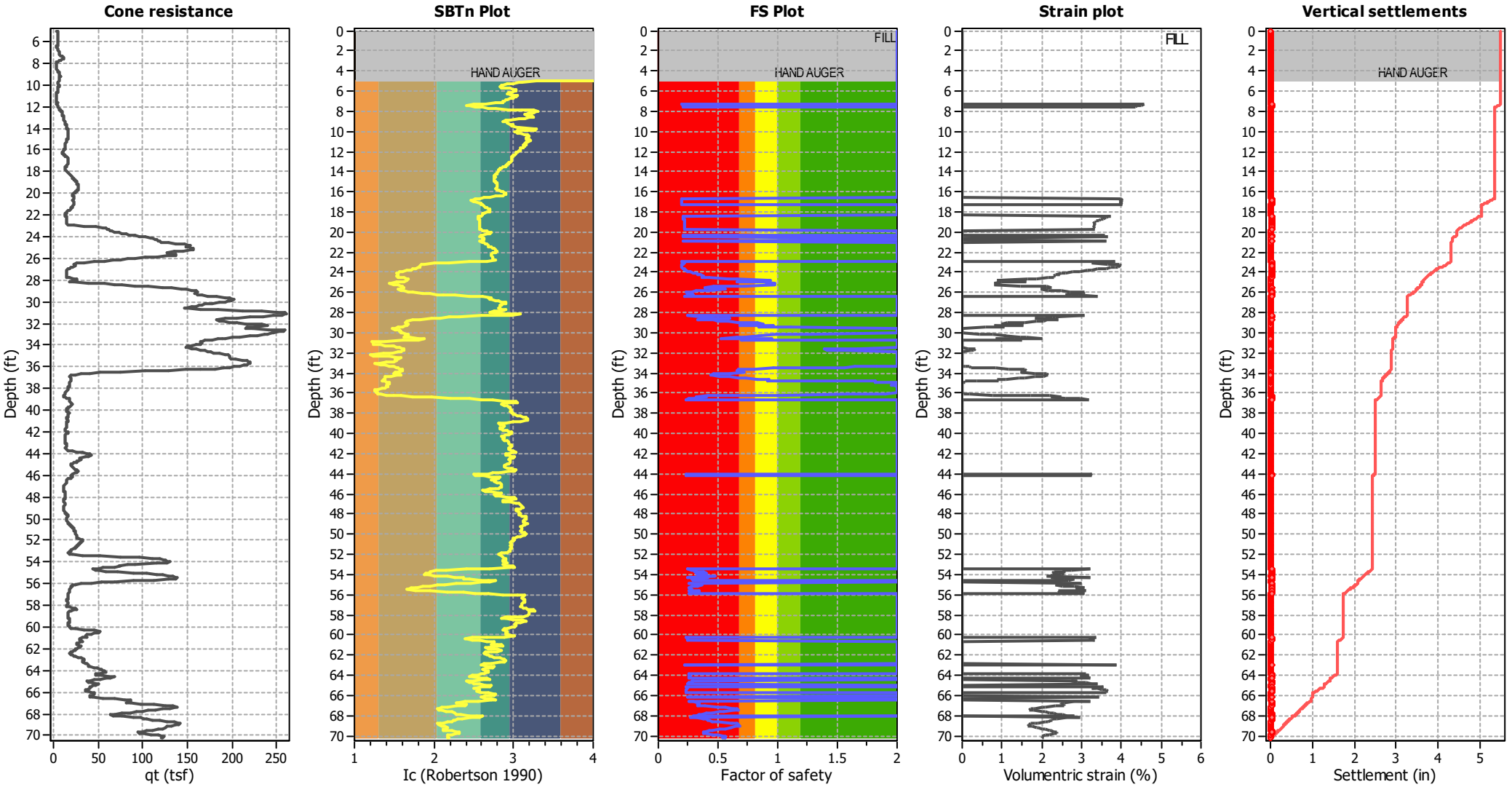
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	3.02 ft	Fill height:	5.40 ft	Limit depth:	N/A

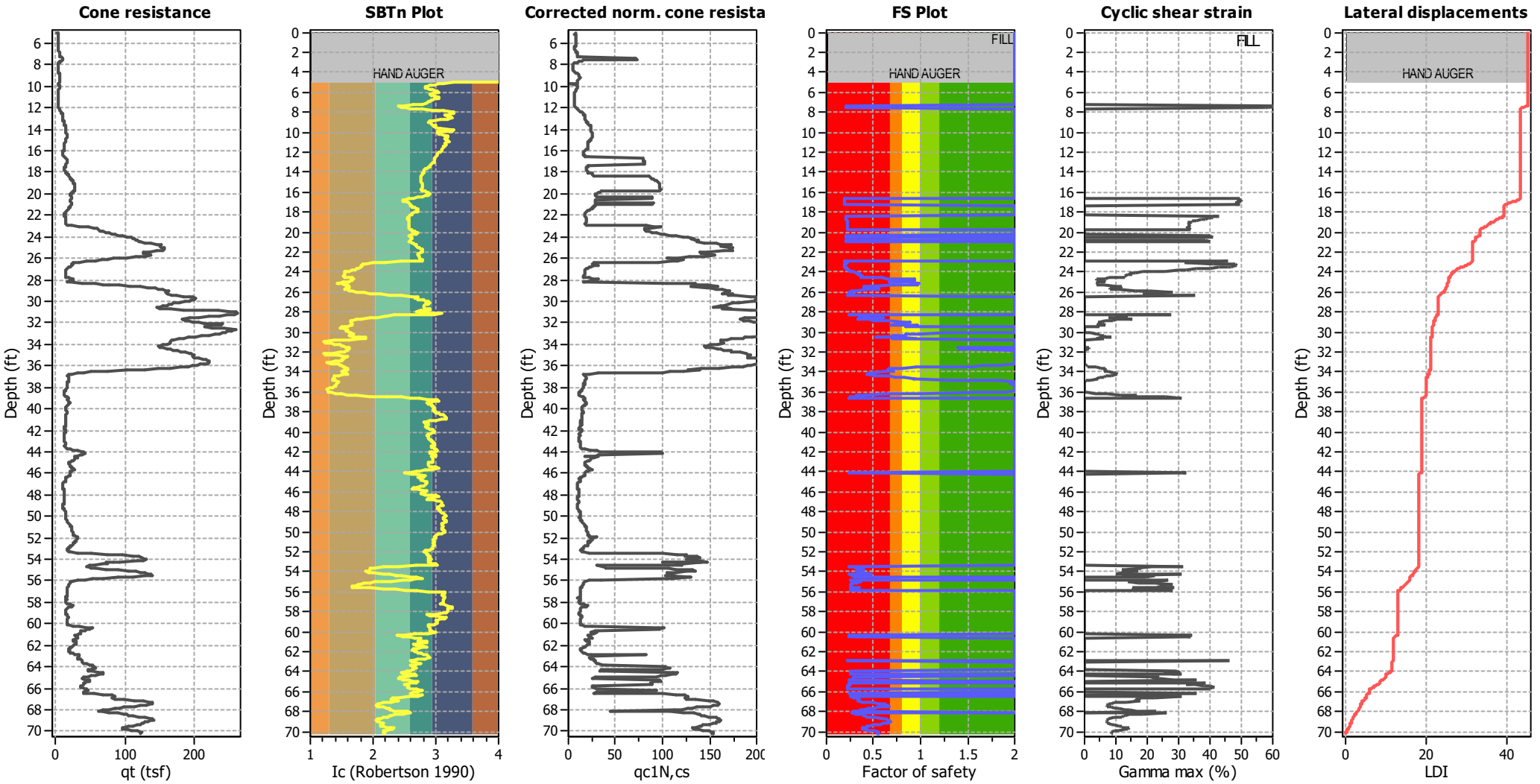
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

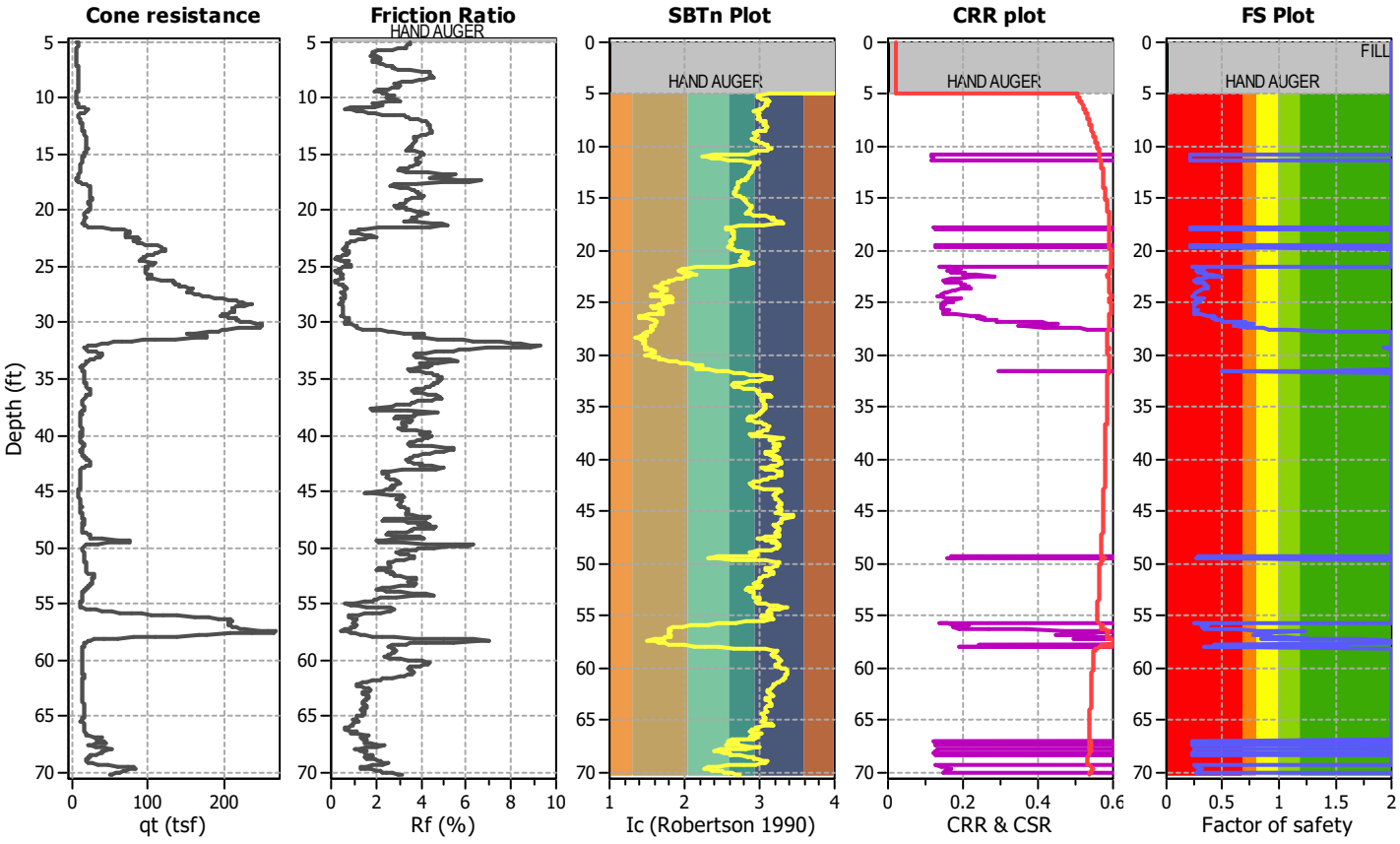
LIQUEFACTION ANALYSIS REPORT

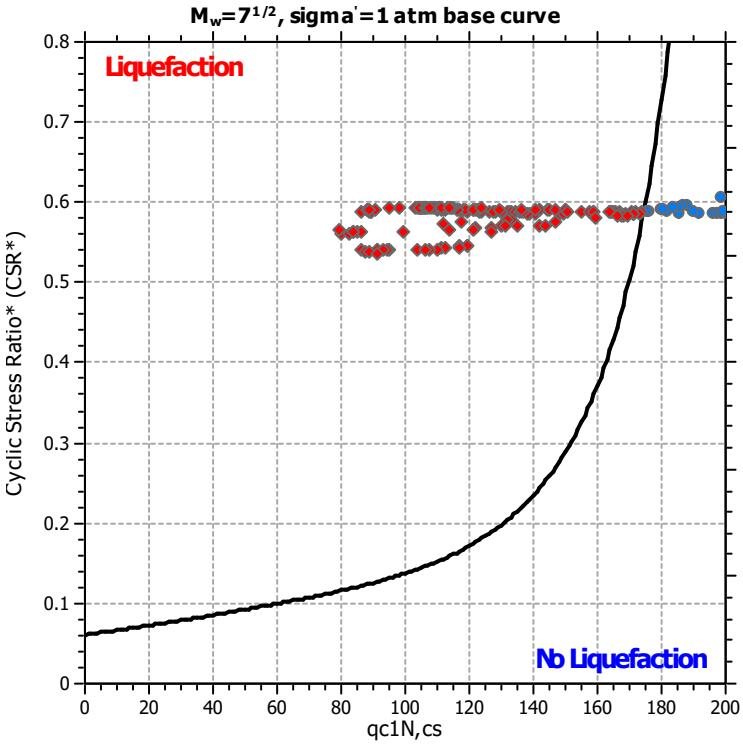
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

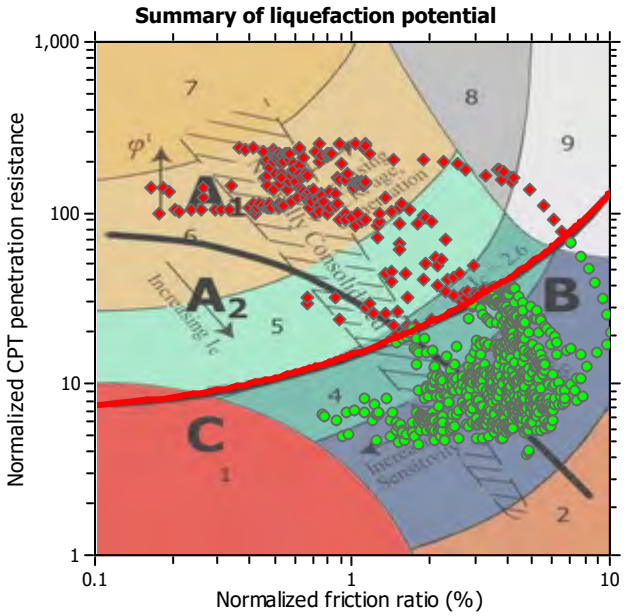
CPT file : GS26-CPT-13

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.51 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	5.80 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

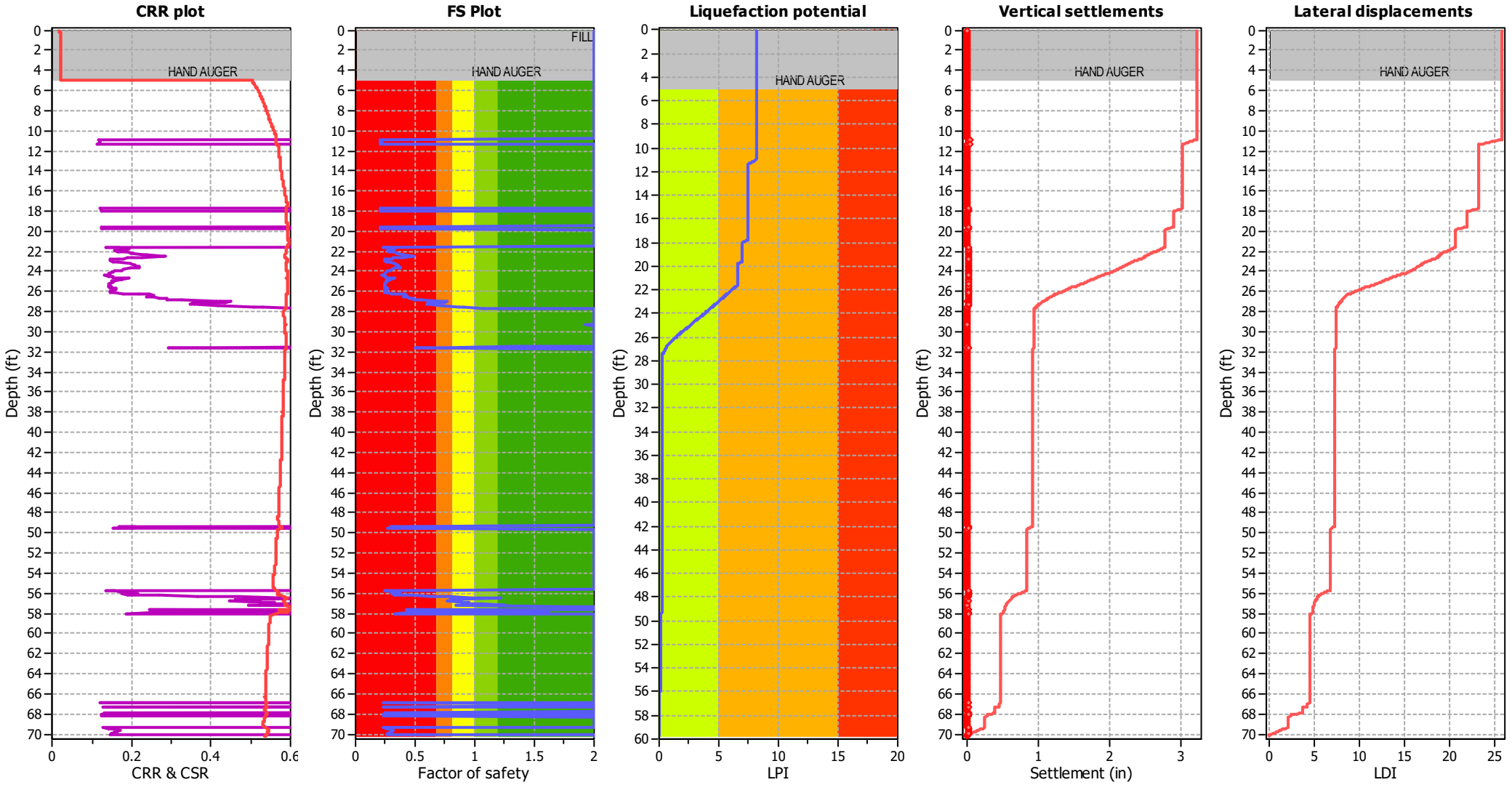






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.51 ft	Fill height:	5.80 ft	Limit depth:	N/A

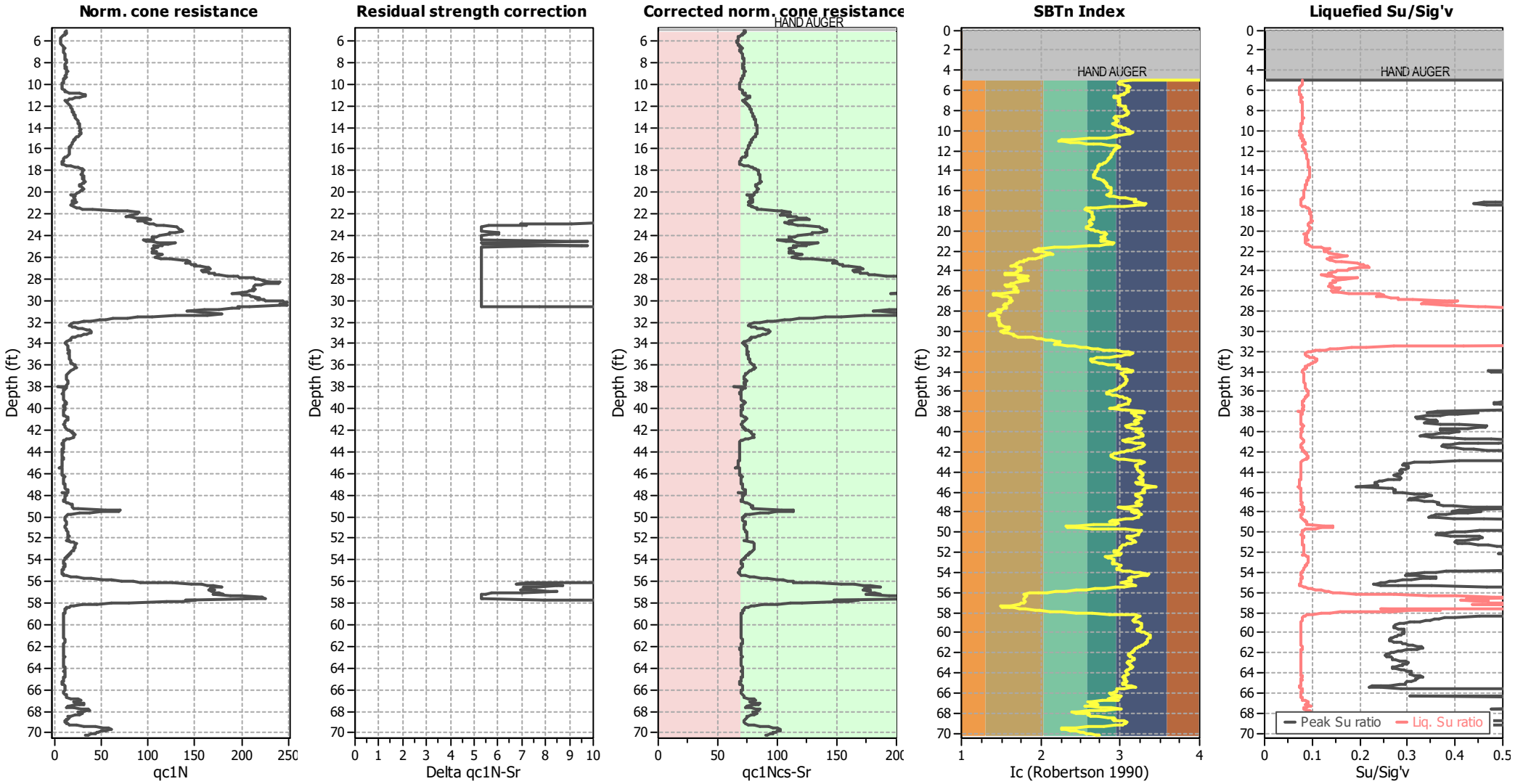
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

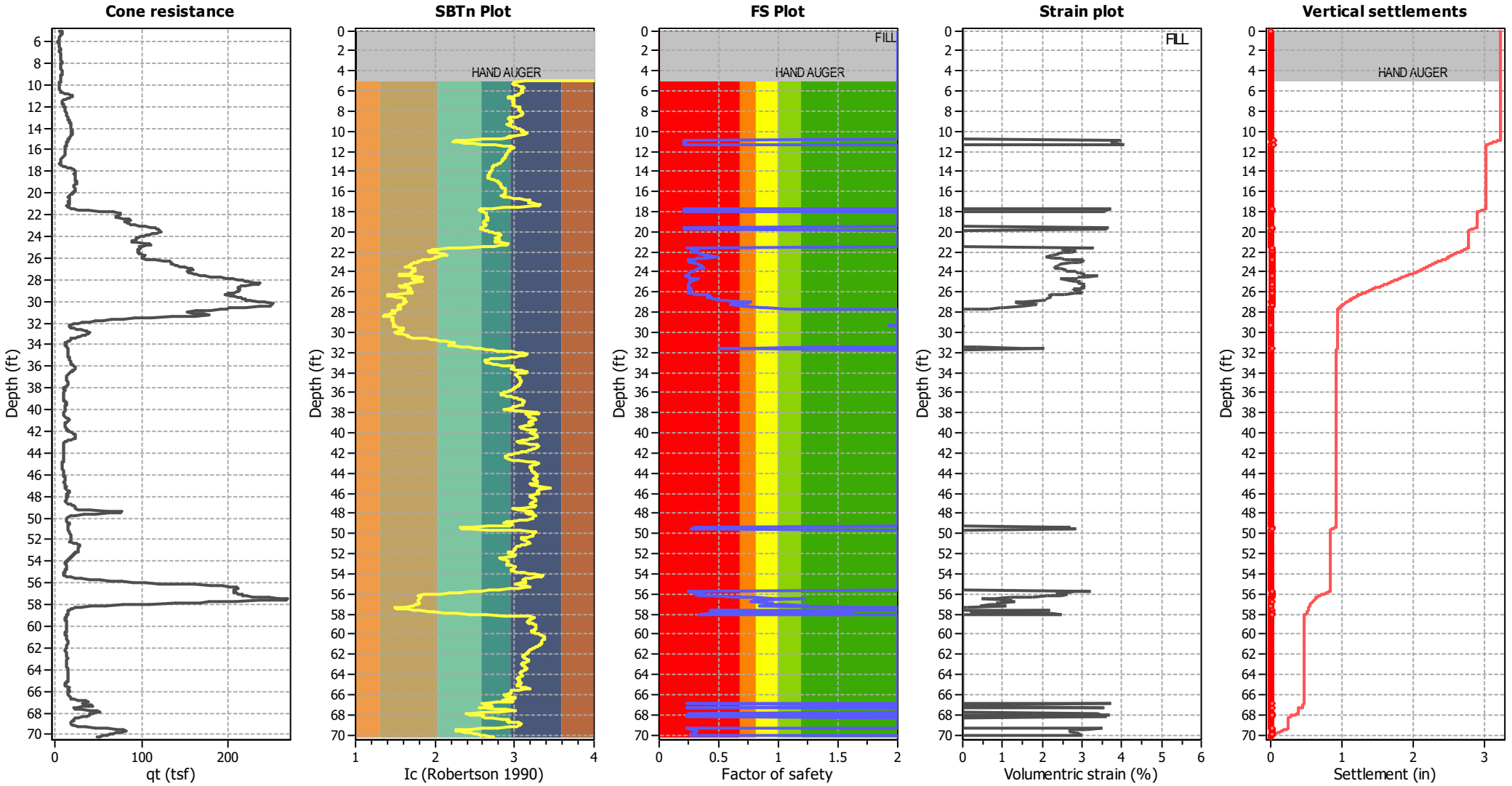
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.51 ft	Fill height:	5.80 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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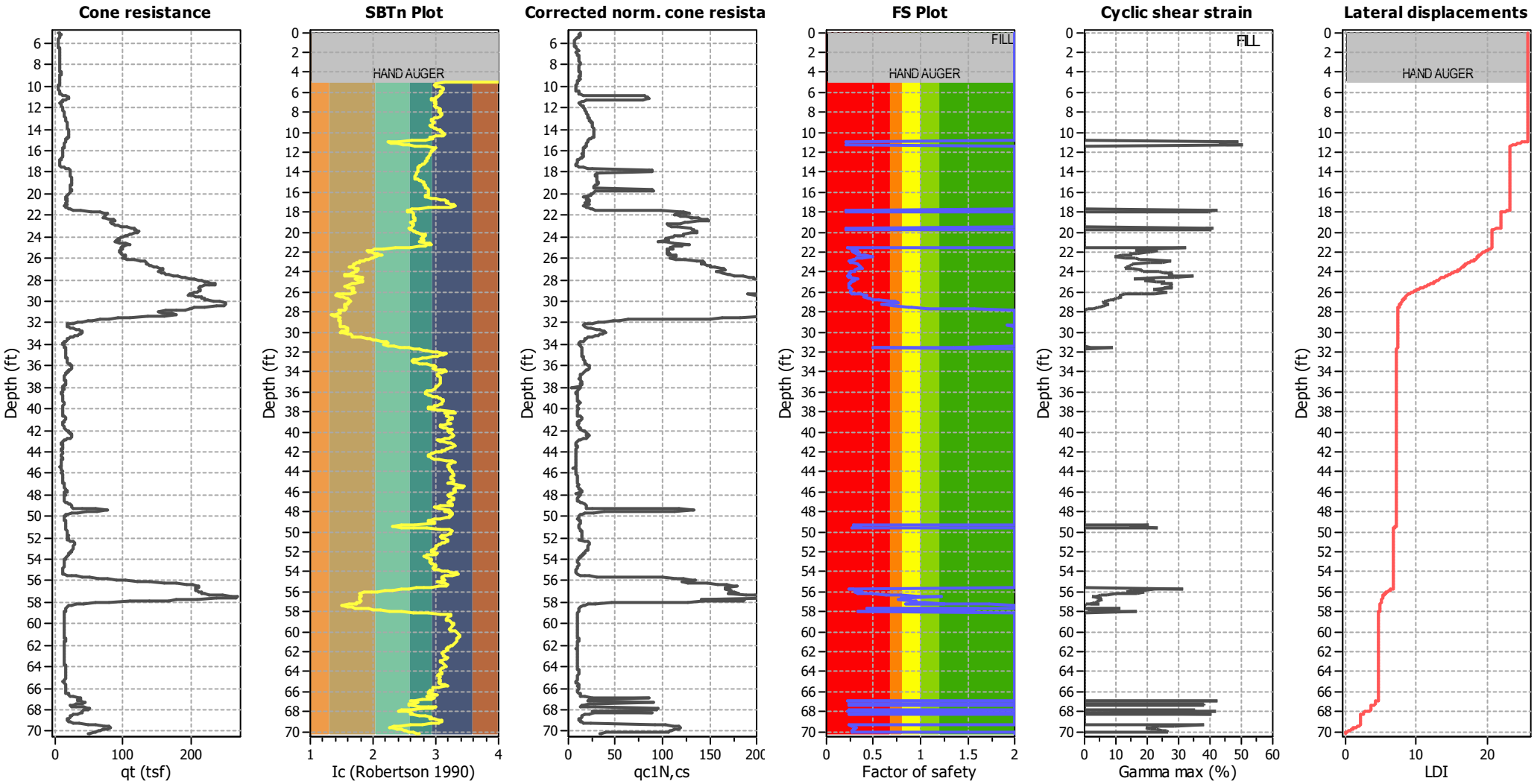
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

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I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

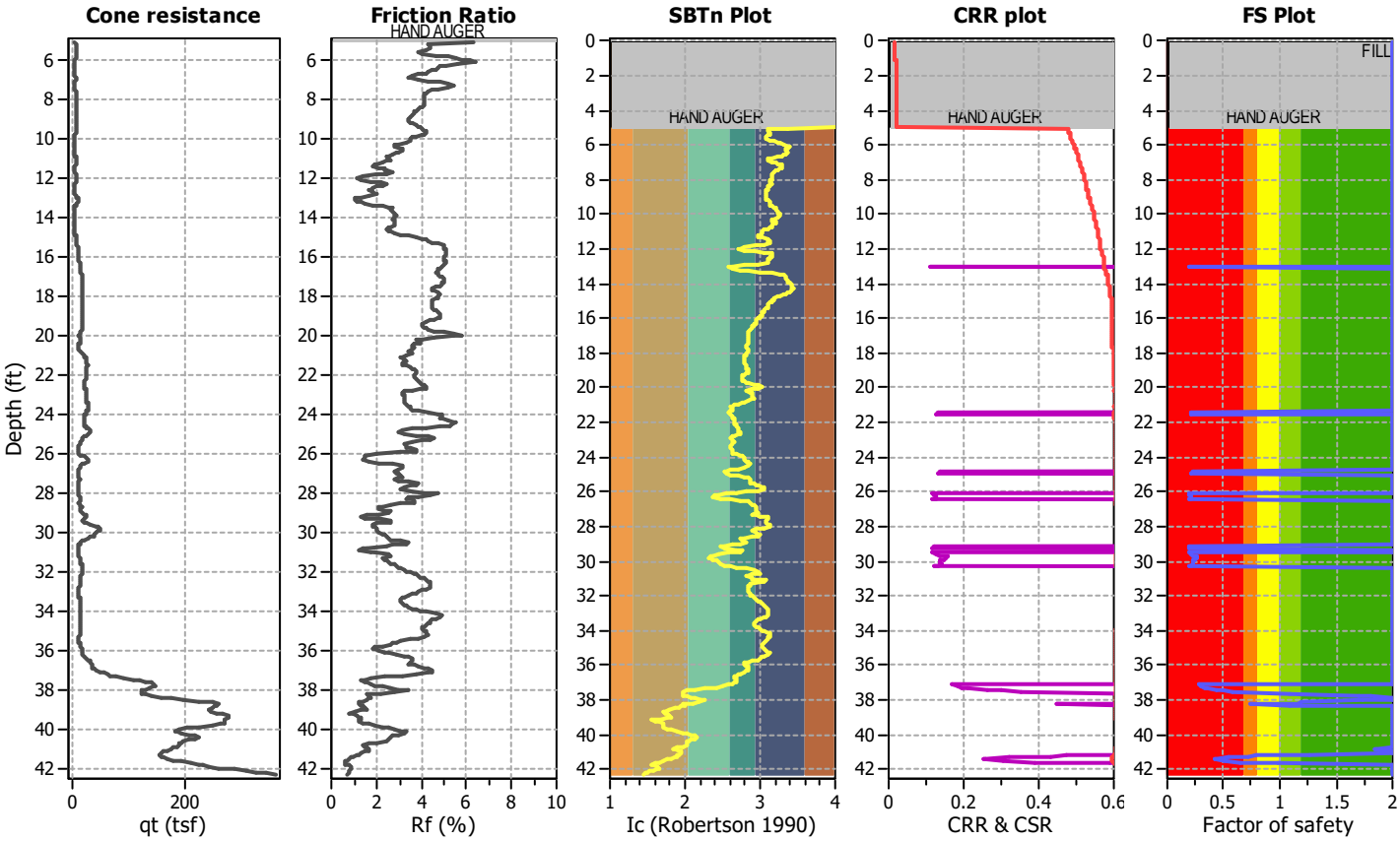
LIQUEFACTION ANALYSIS REPORT

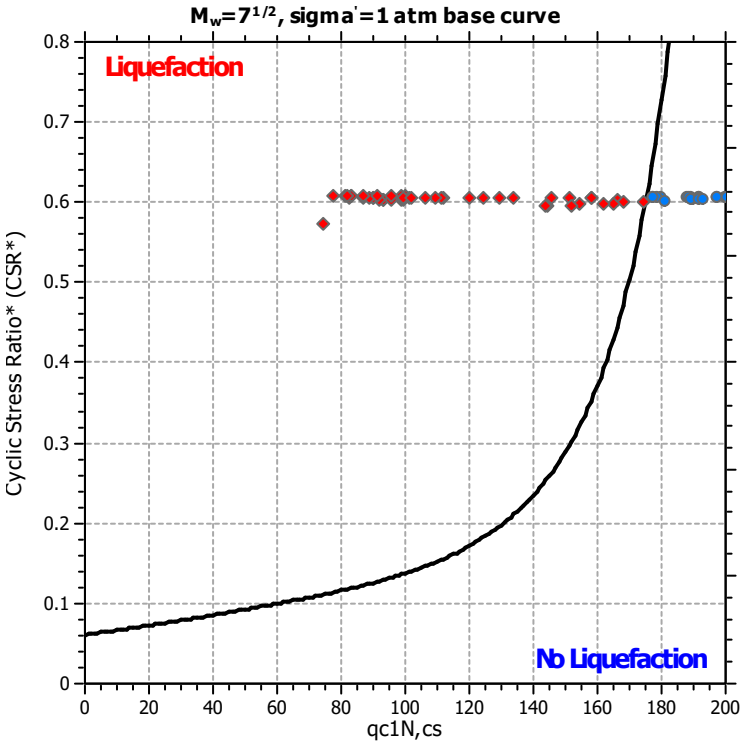
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

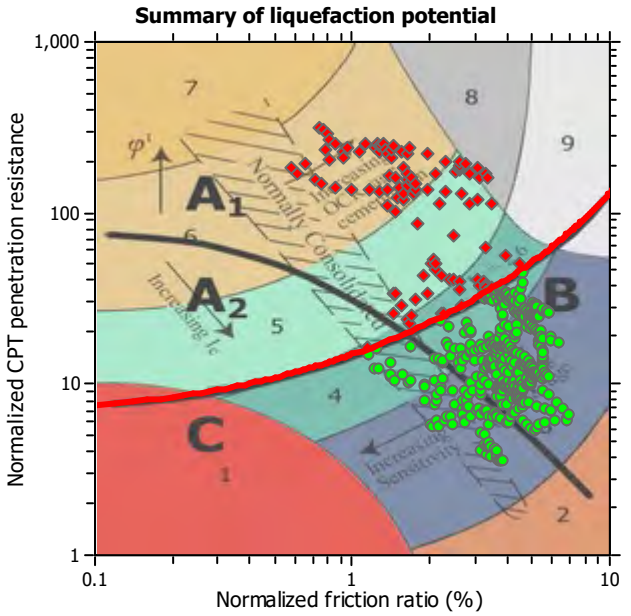
CPT file : GS26-CPT-14

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.72 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	3.30 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

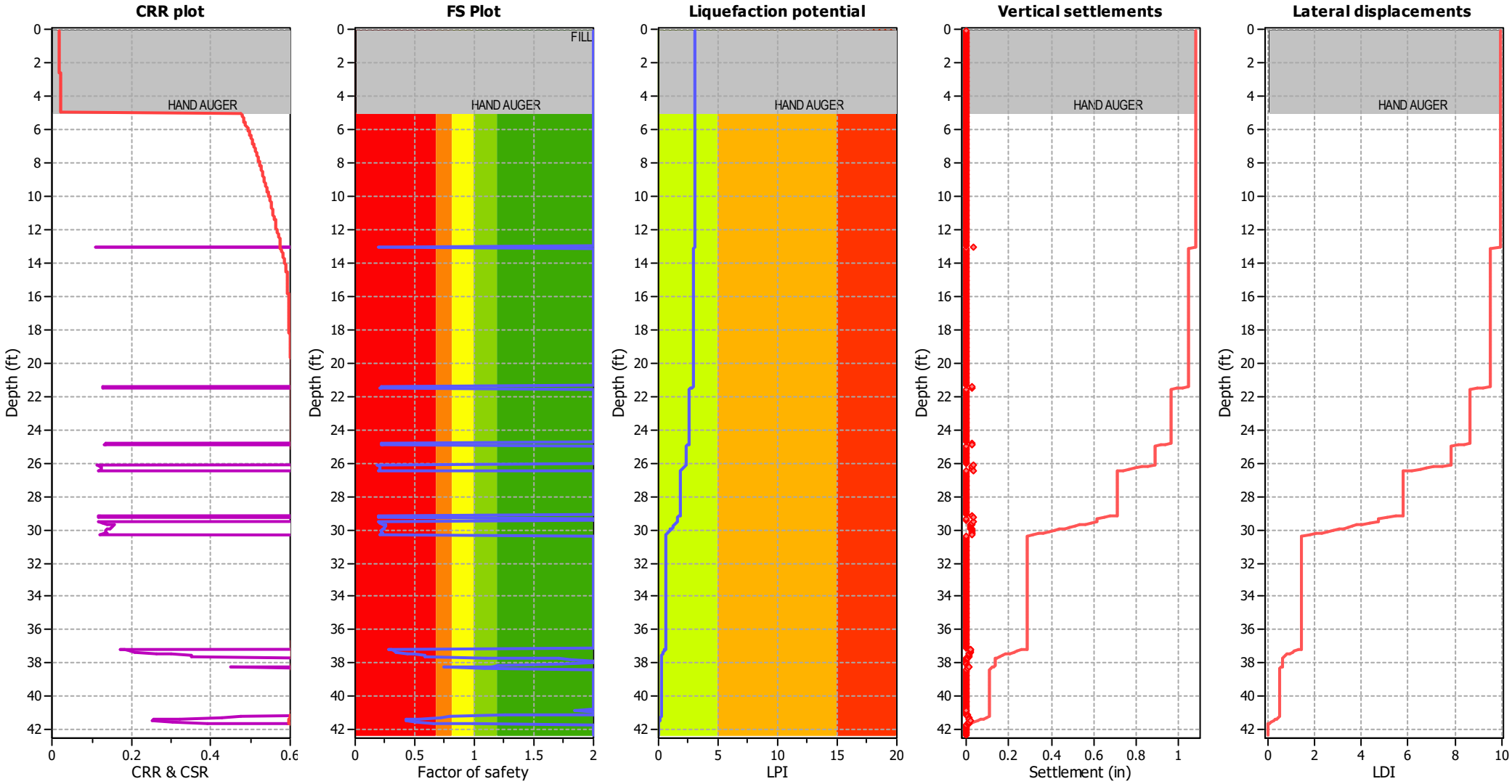






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.72 ft	Fill height:	3.30 ft	Limit depth:	N/A

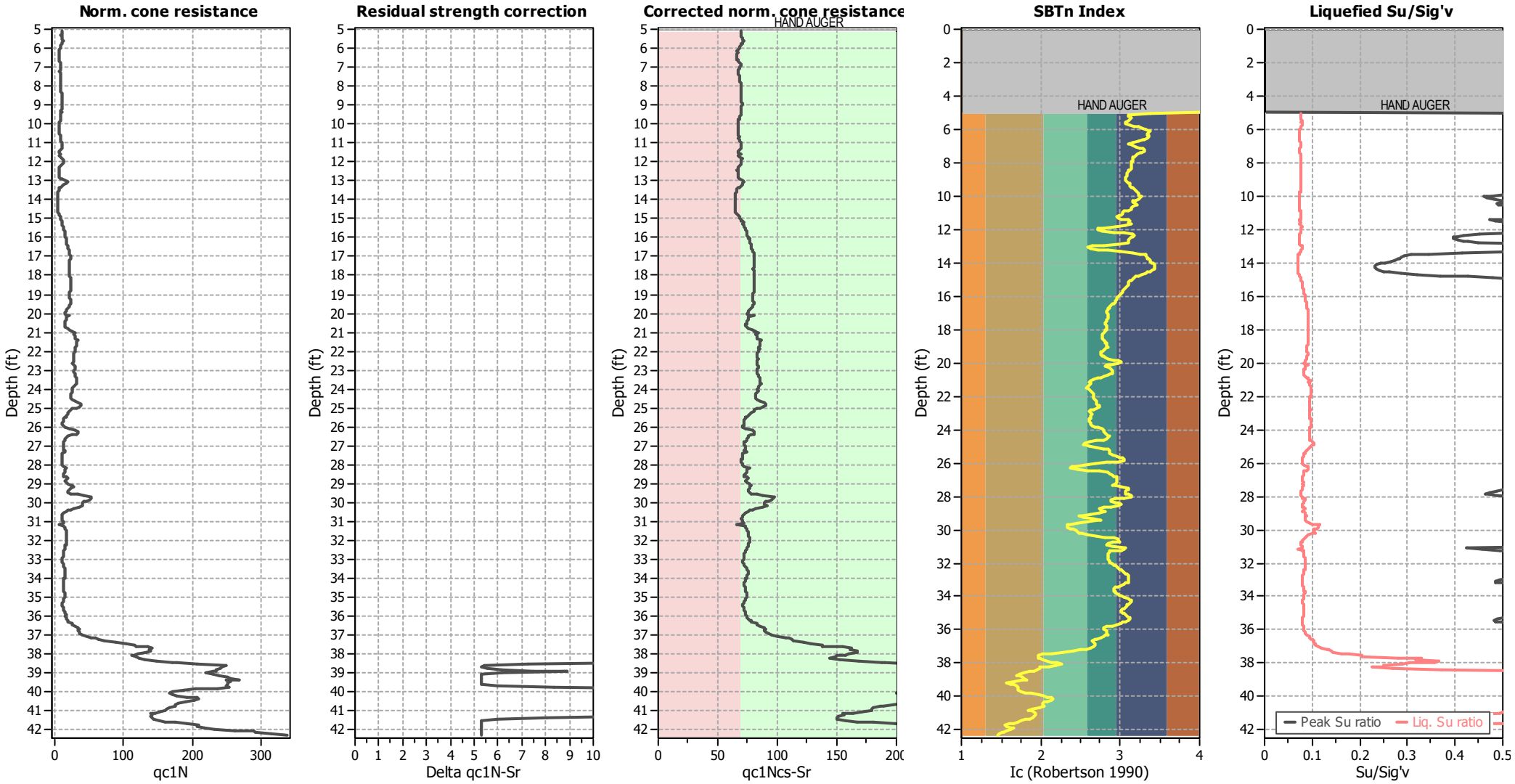
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

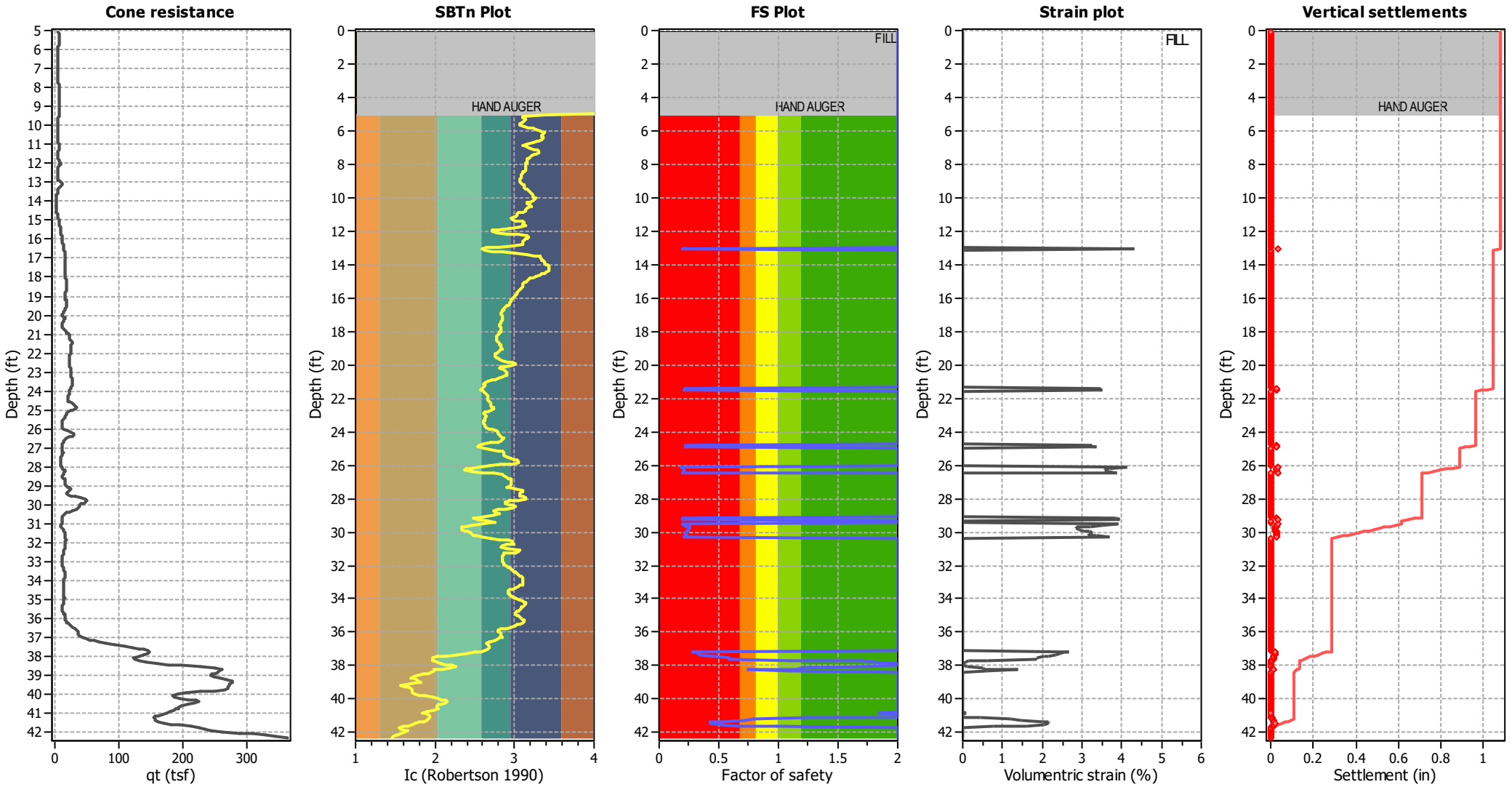
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.72 ft	Fill height:	3.30 ft	Limit depth:	N/A

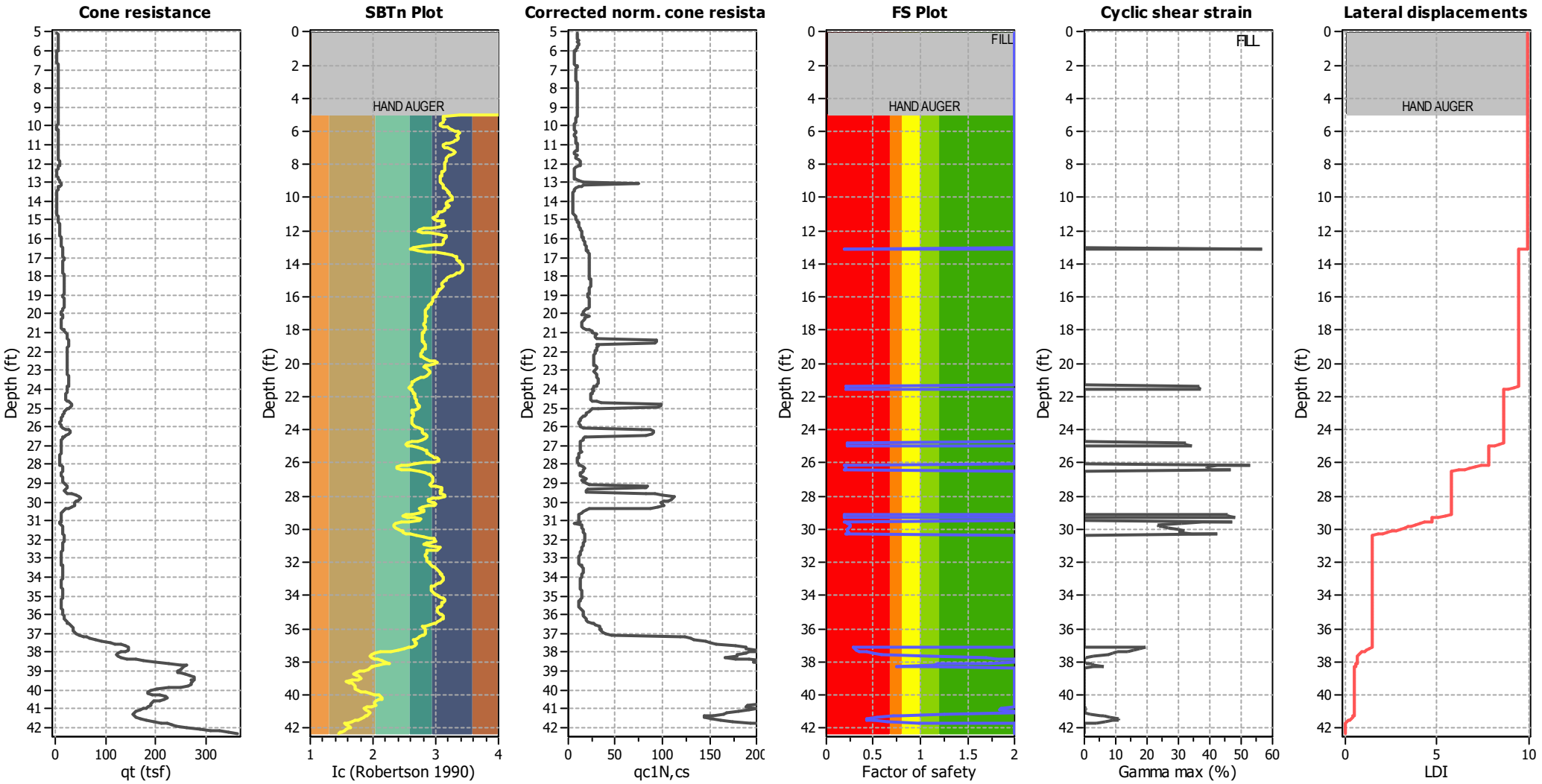
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

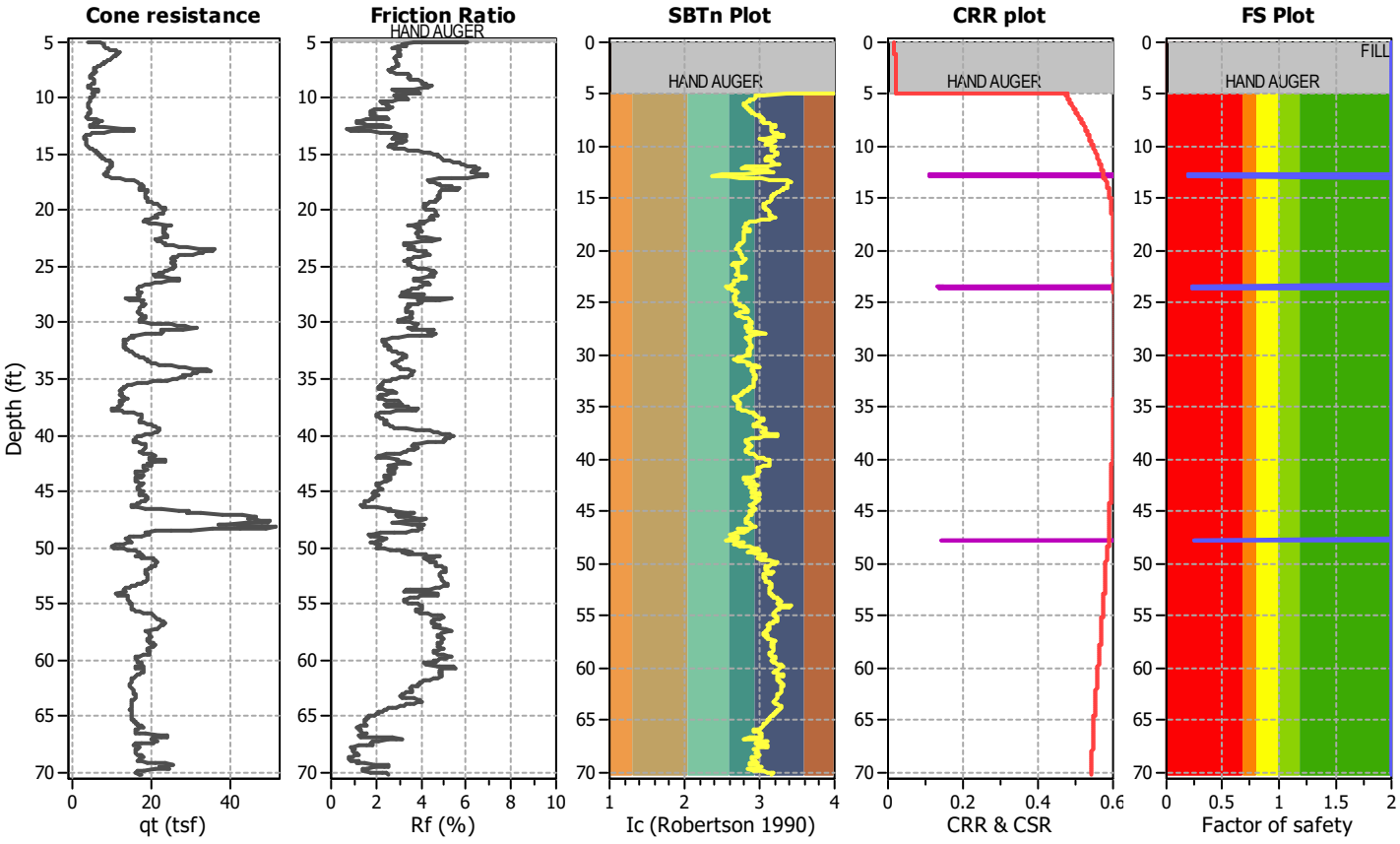
LIQUEFACTION ANALYSIS REPORT

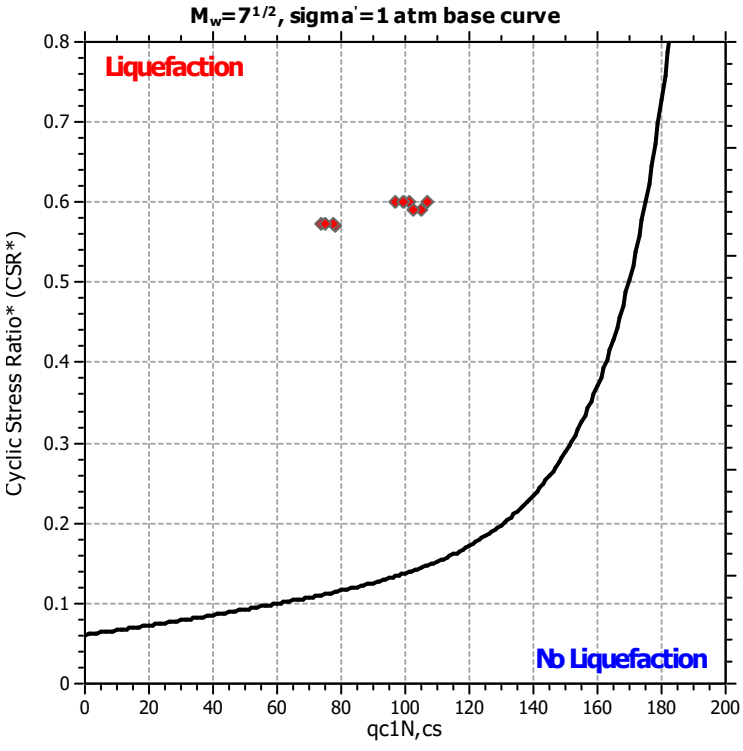
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

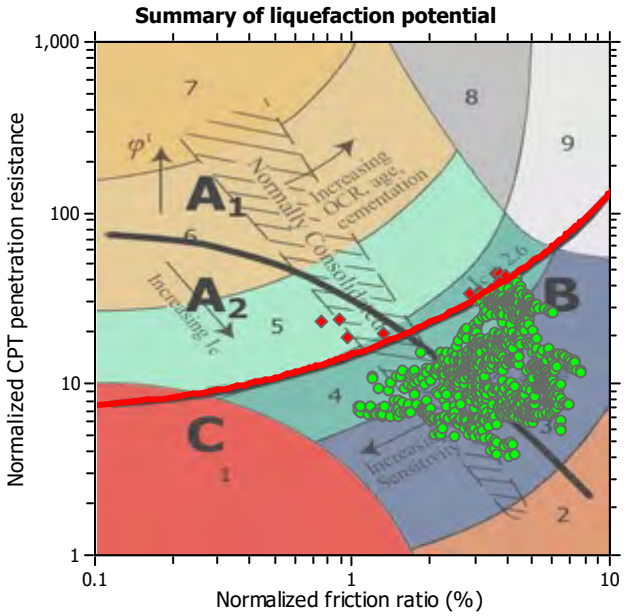
CPT file : GS26-CPT-15

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.96 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	3.20 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

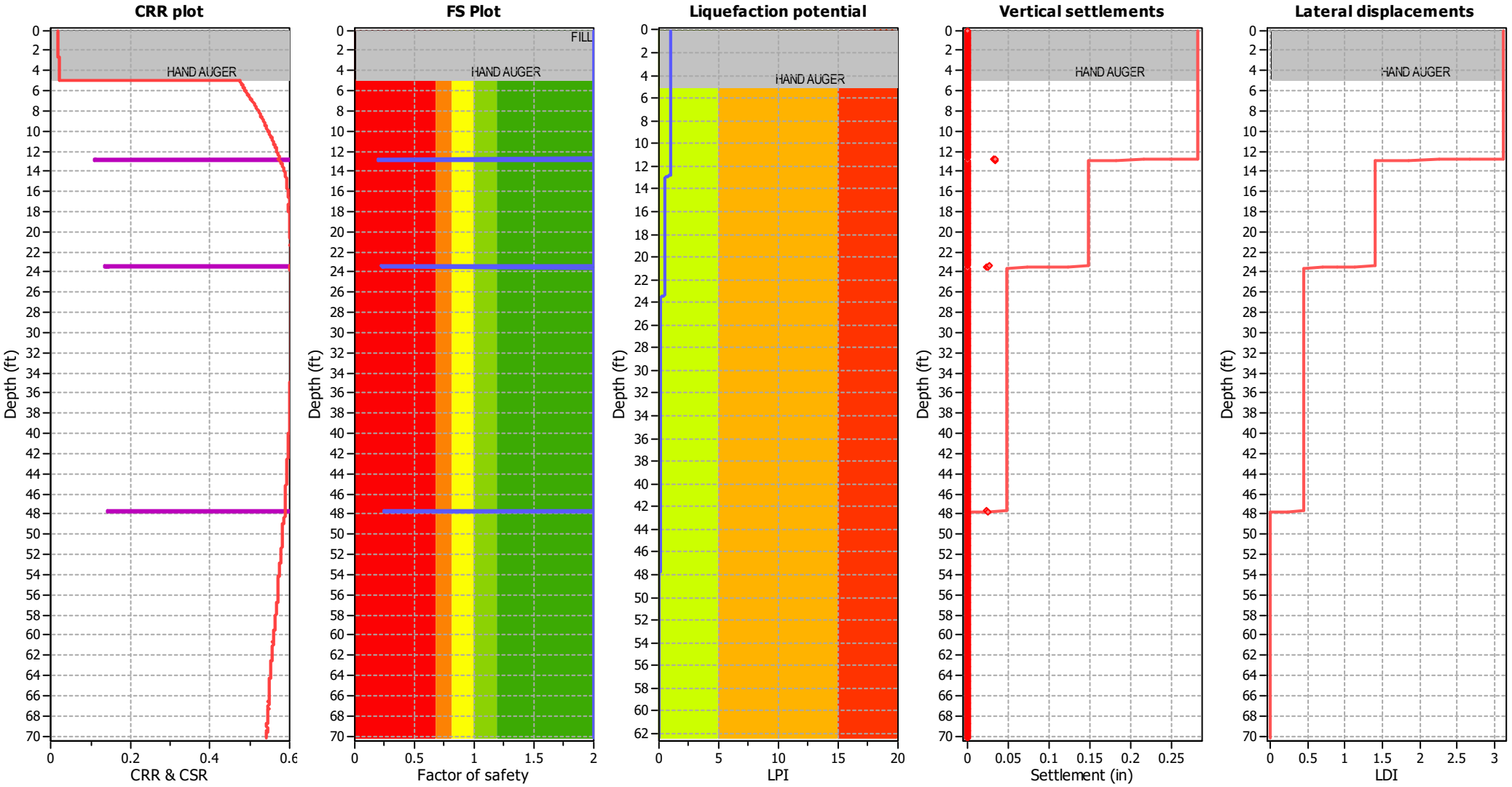






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.96 ft	Fill height:	3.20 ft	Limit depth:	N/A

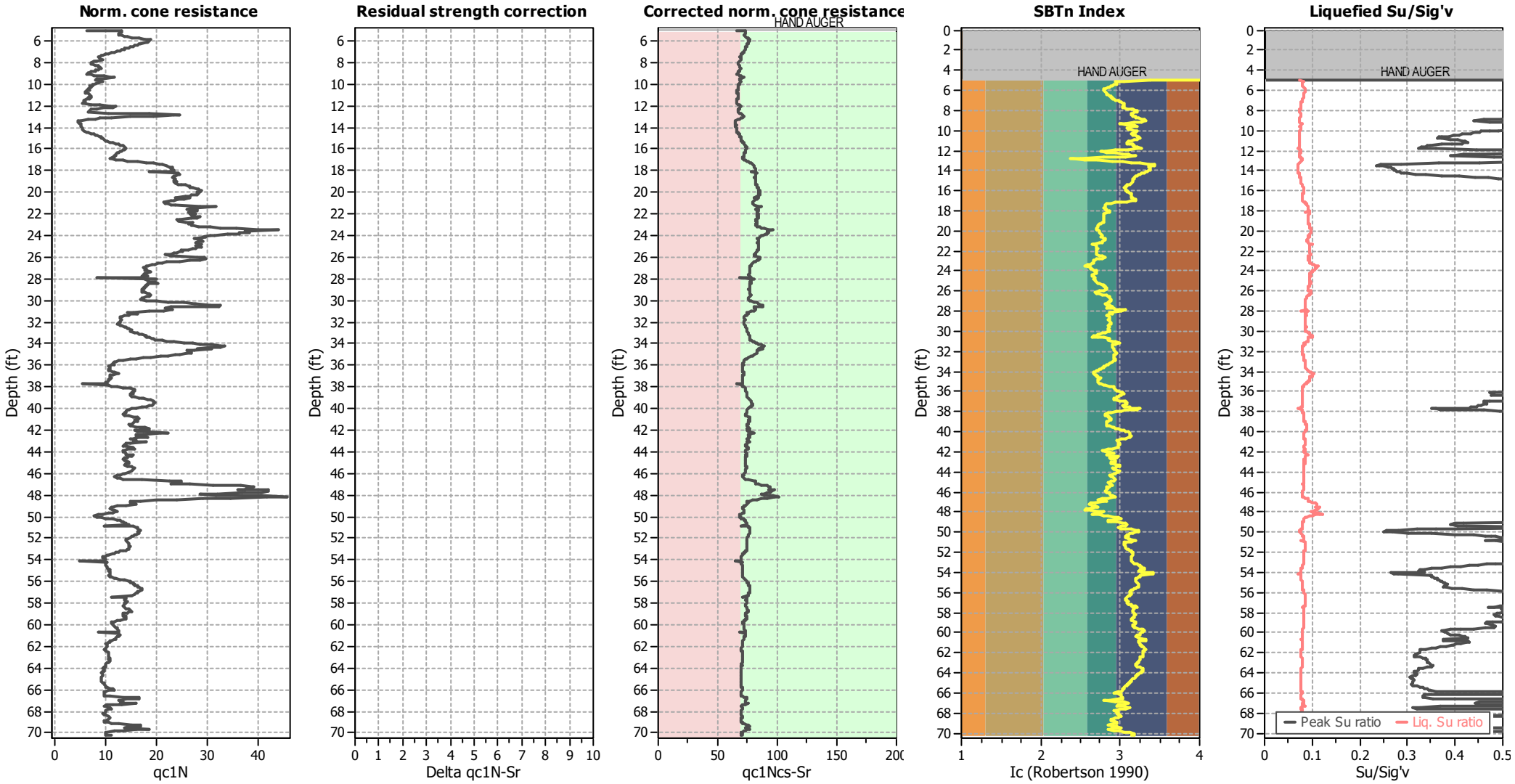
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

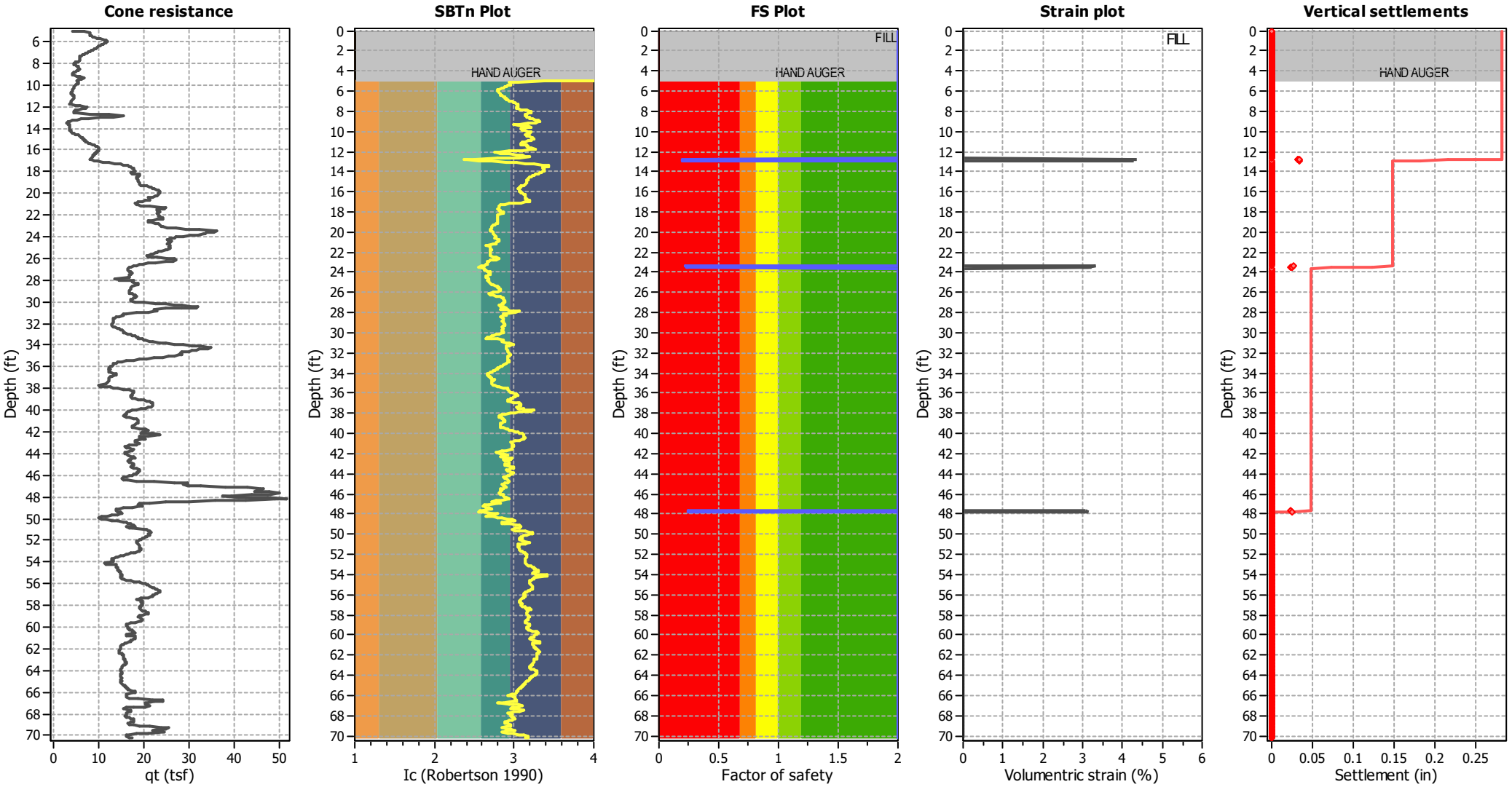
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.96 ft	Fill height:	3.20 ft	Limit depth:	N/A

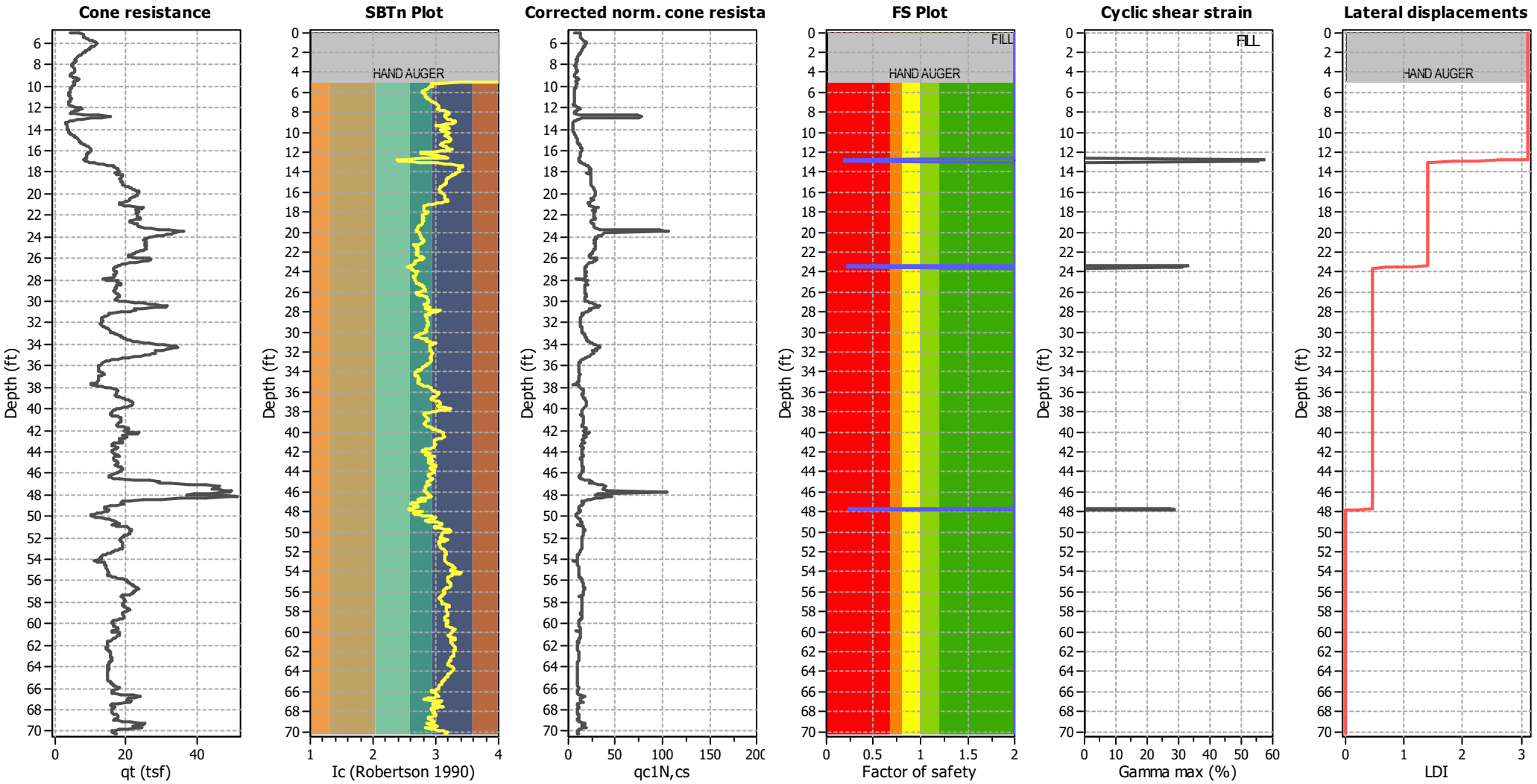
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

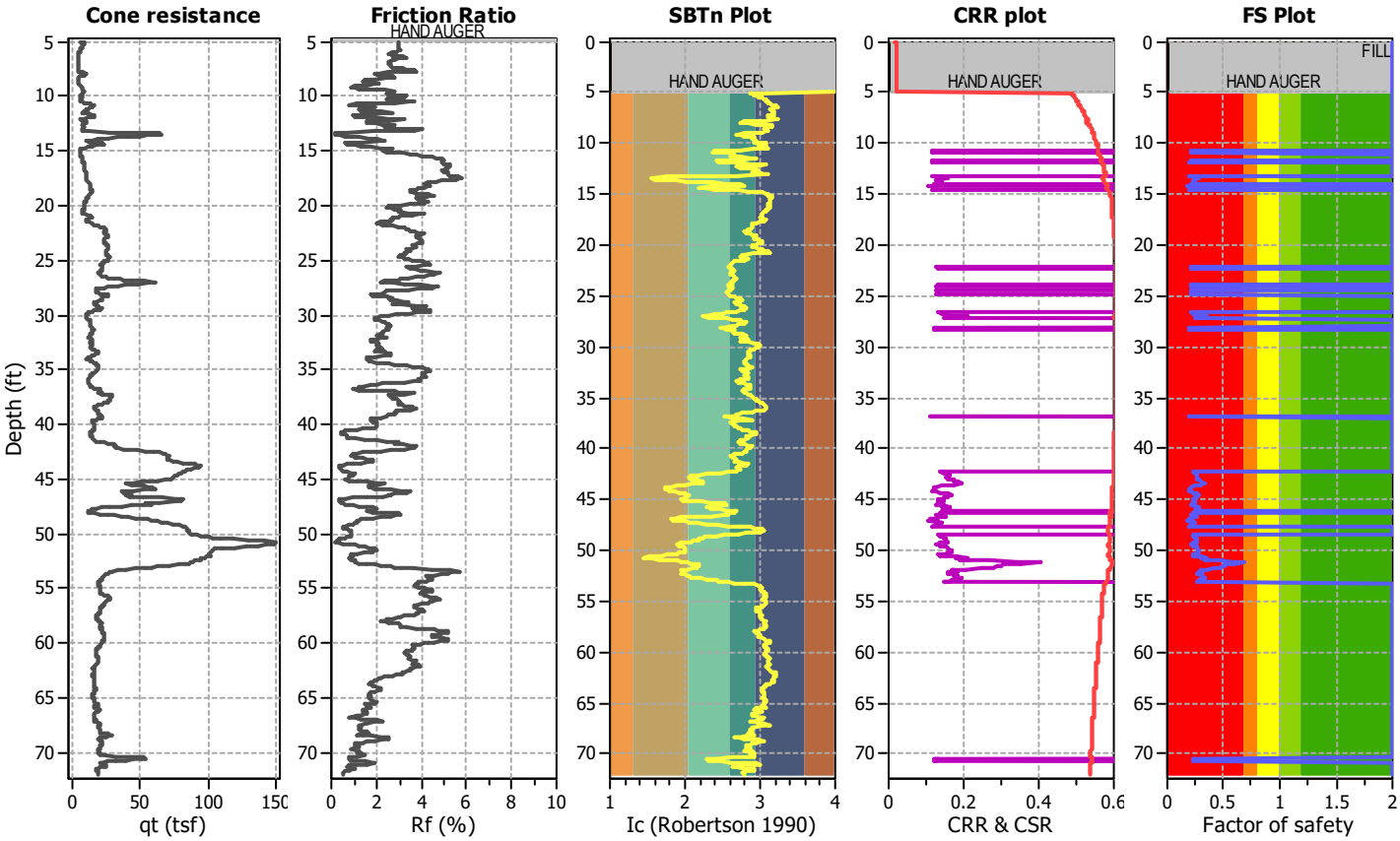
LIQUEFACTION ANALYSIS REPORT

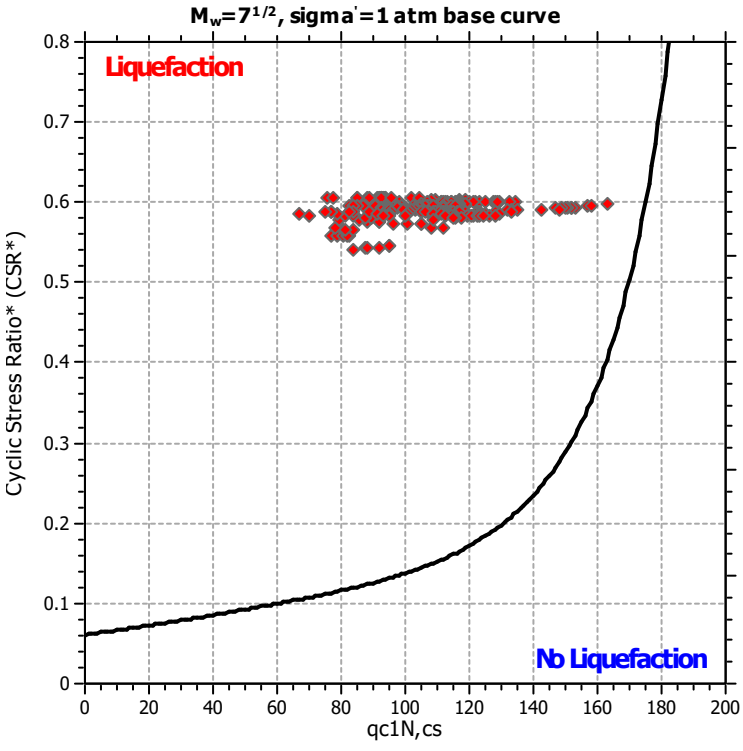
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

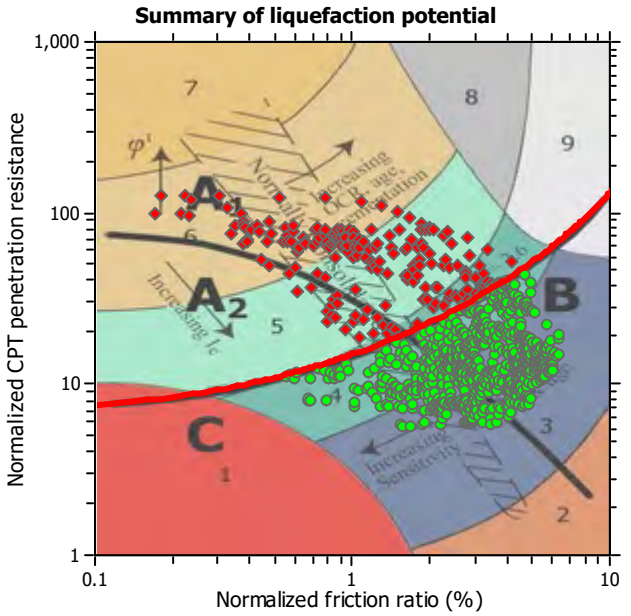
CPT file : GS26-CPT-16

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.70 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	4.30 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

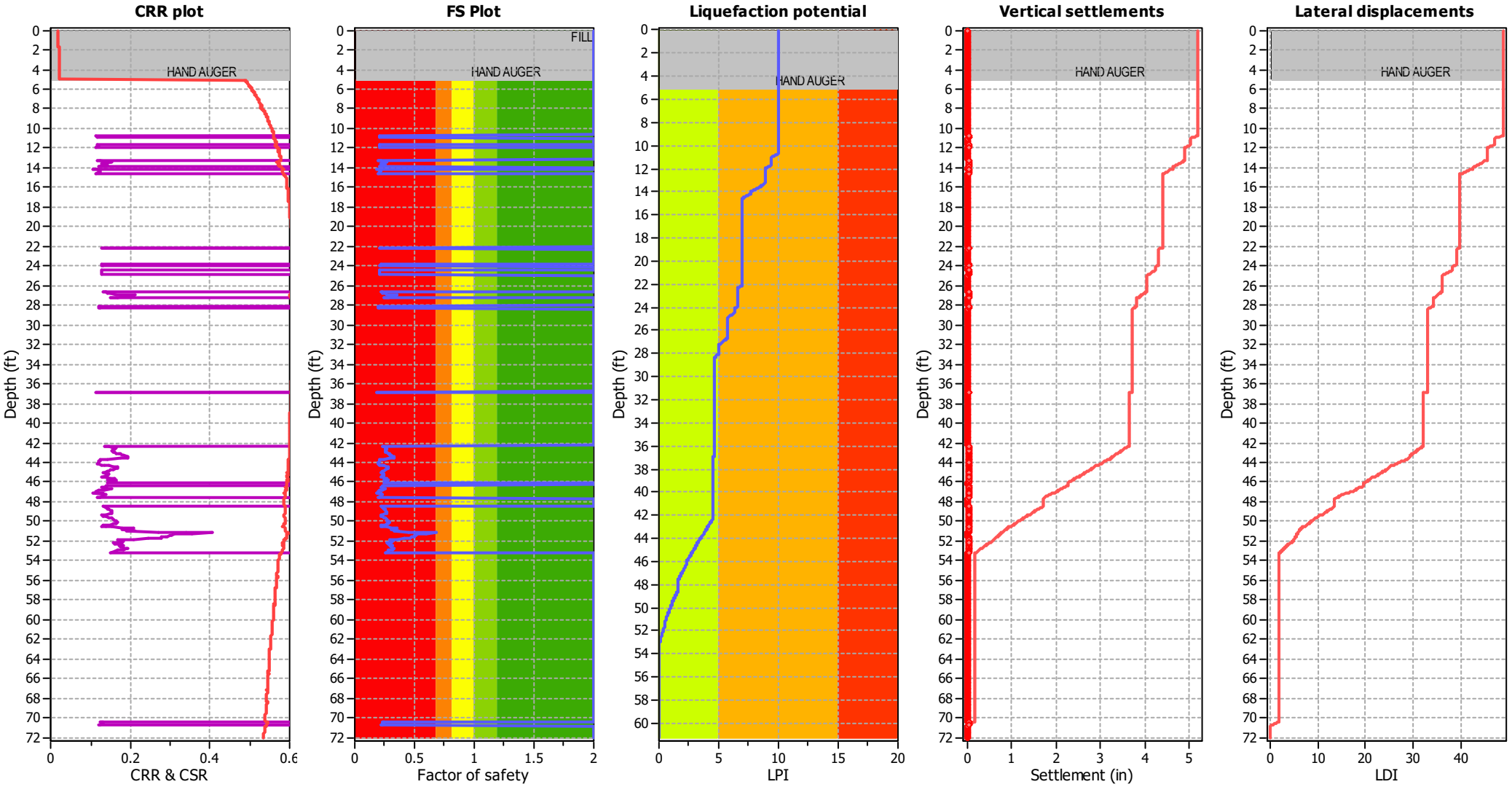






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	1.70 ft	Fill height:	4.30 ft	Limit depth:	N/A

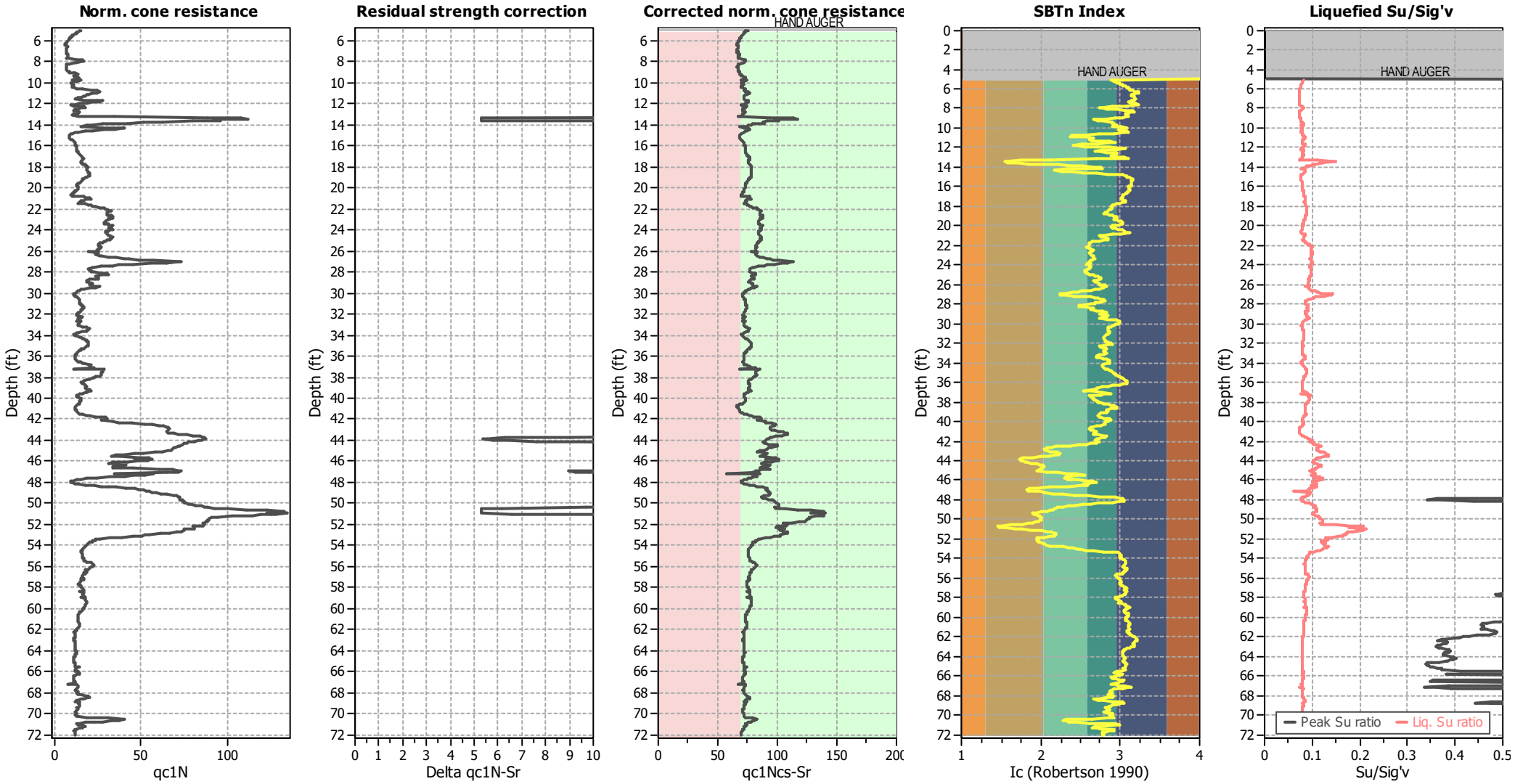
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

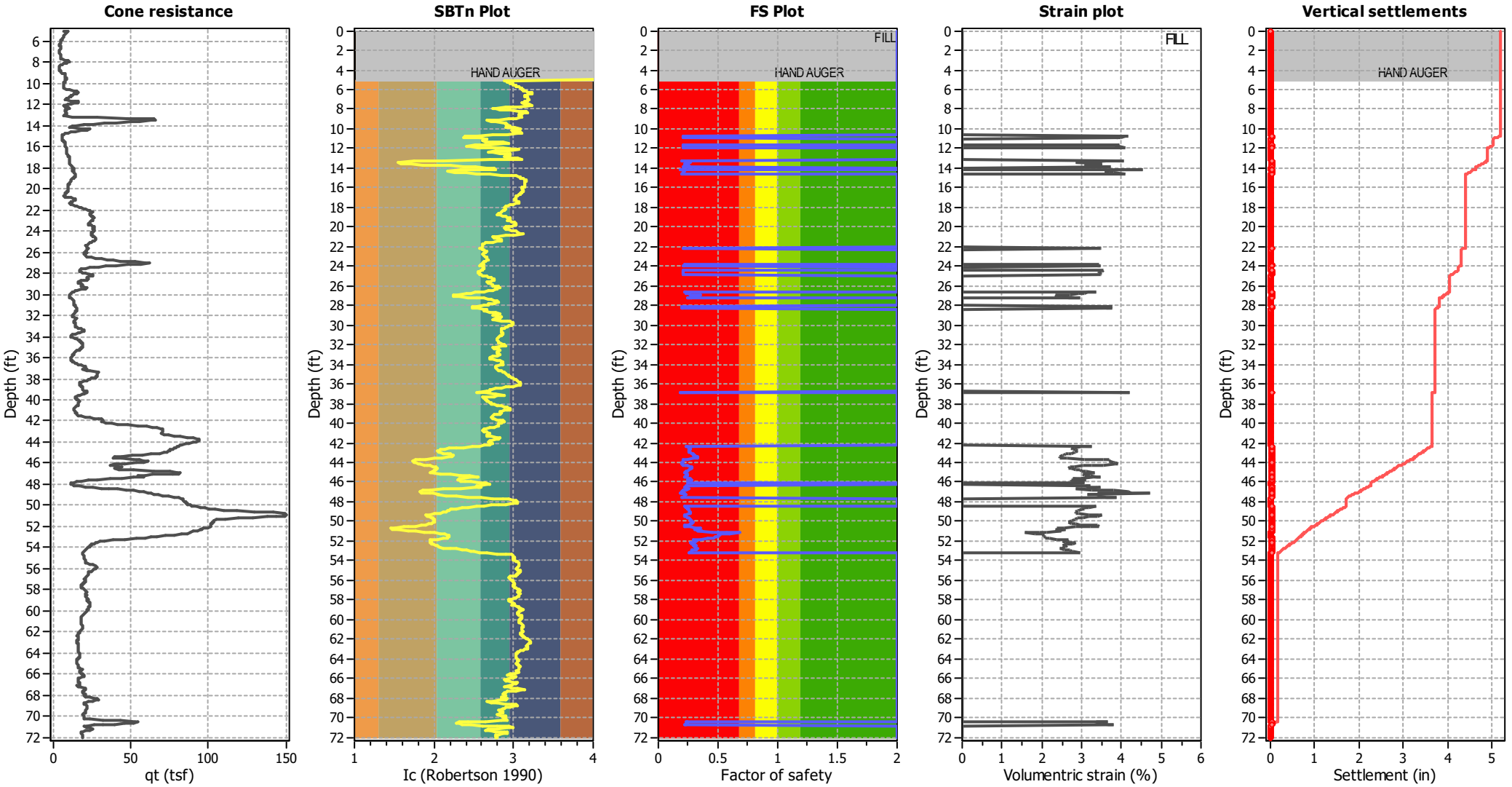
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	1.70 ft	Fill height:	4.30 ft	Limit depth:	N/A

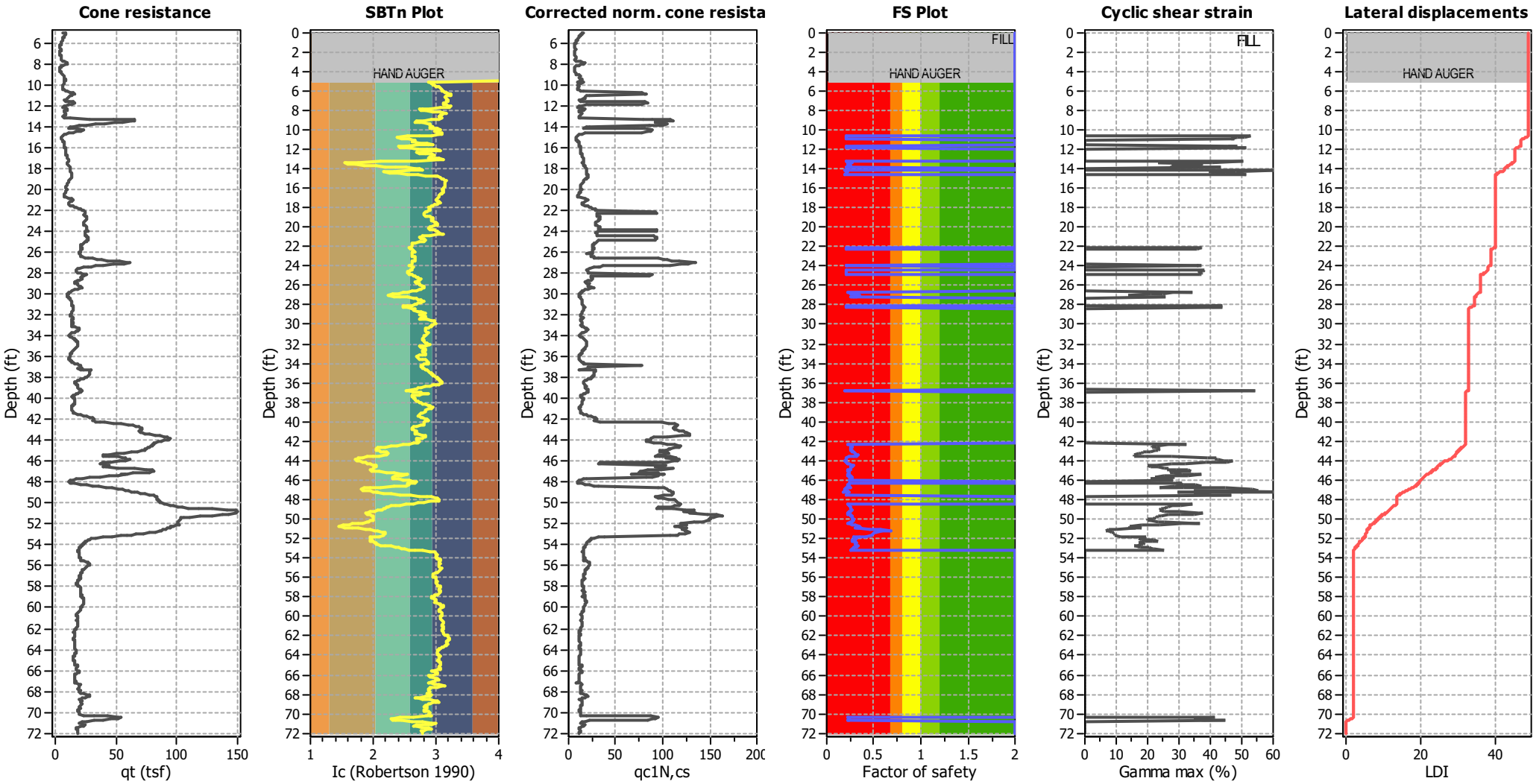
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

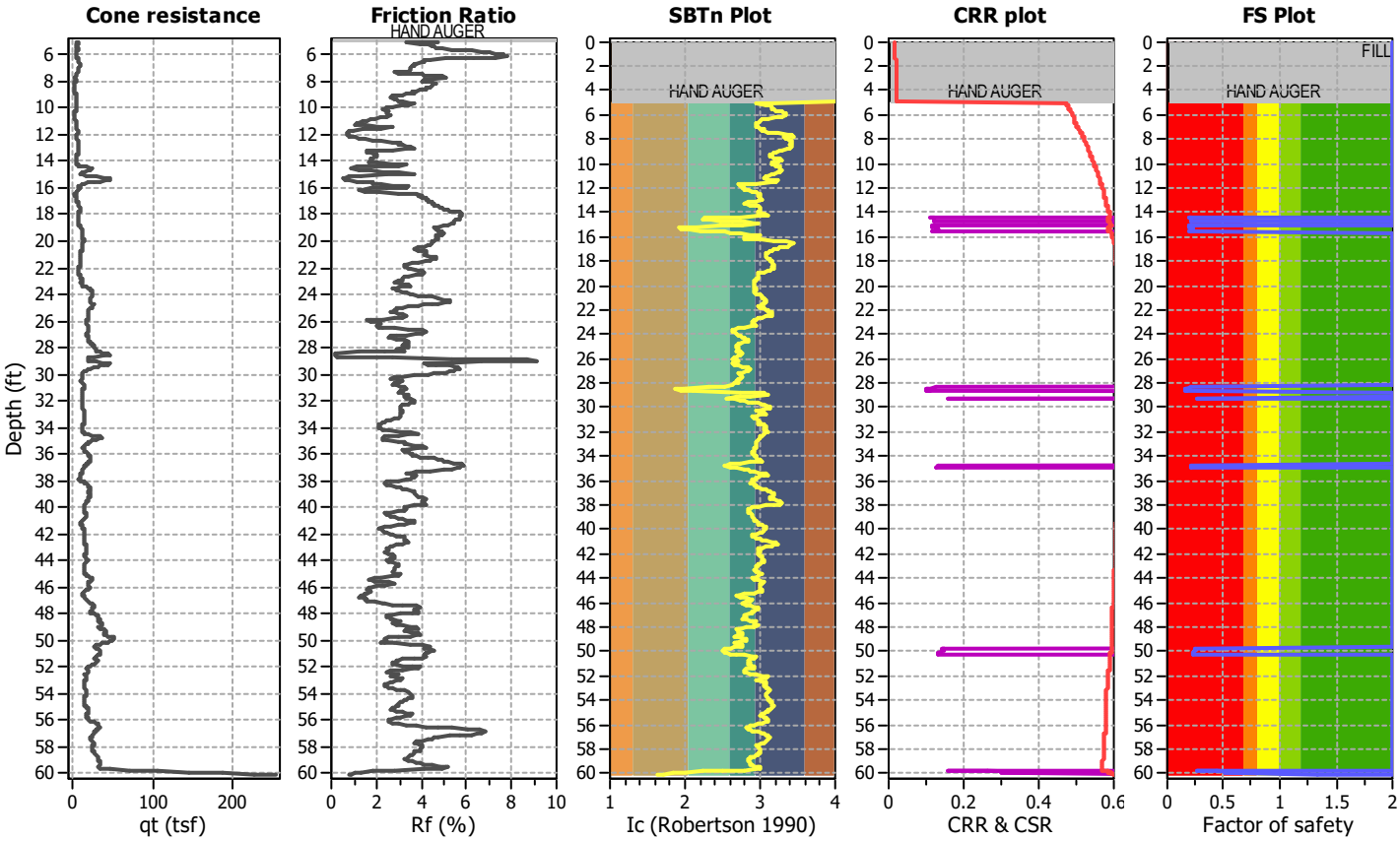
LIQUEFACTION ANALYSIS REPORT

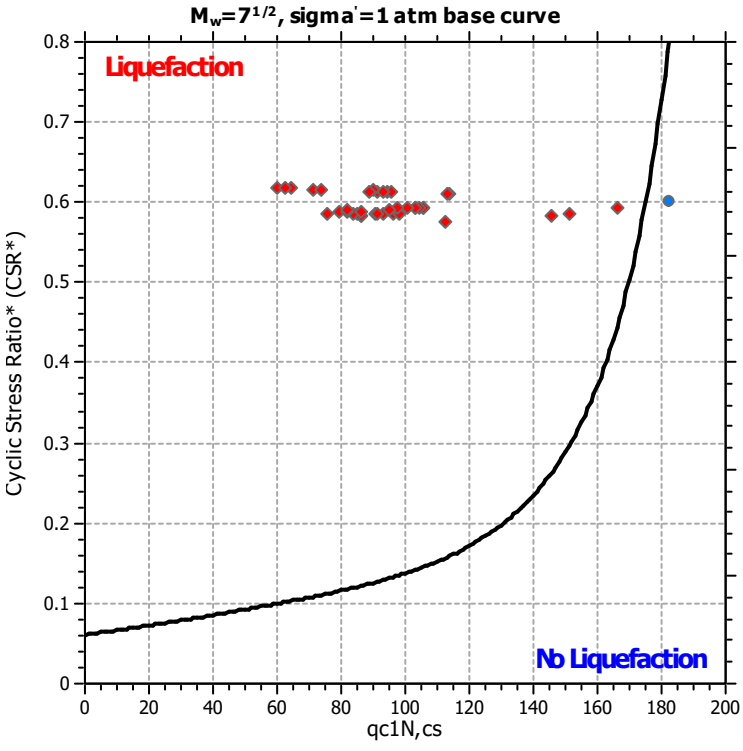
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

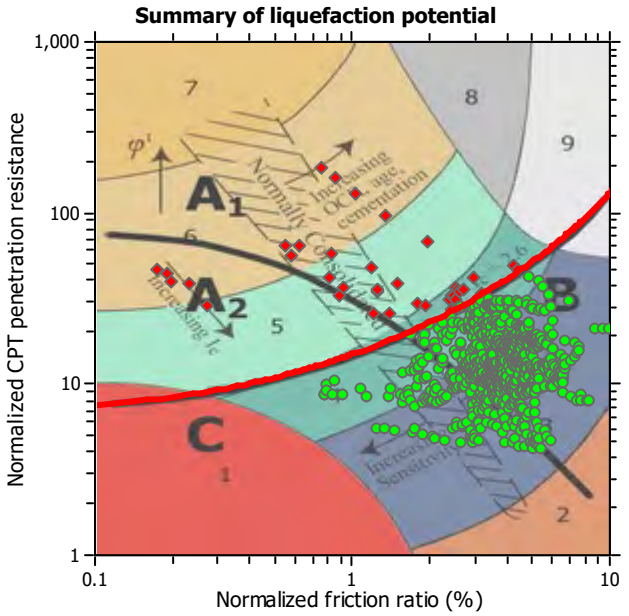
CPT file : GS26-CPT-17

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.98 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	3.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
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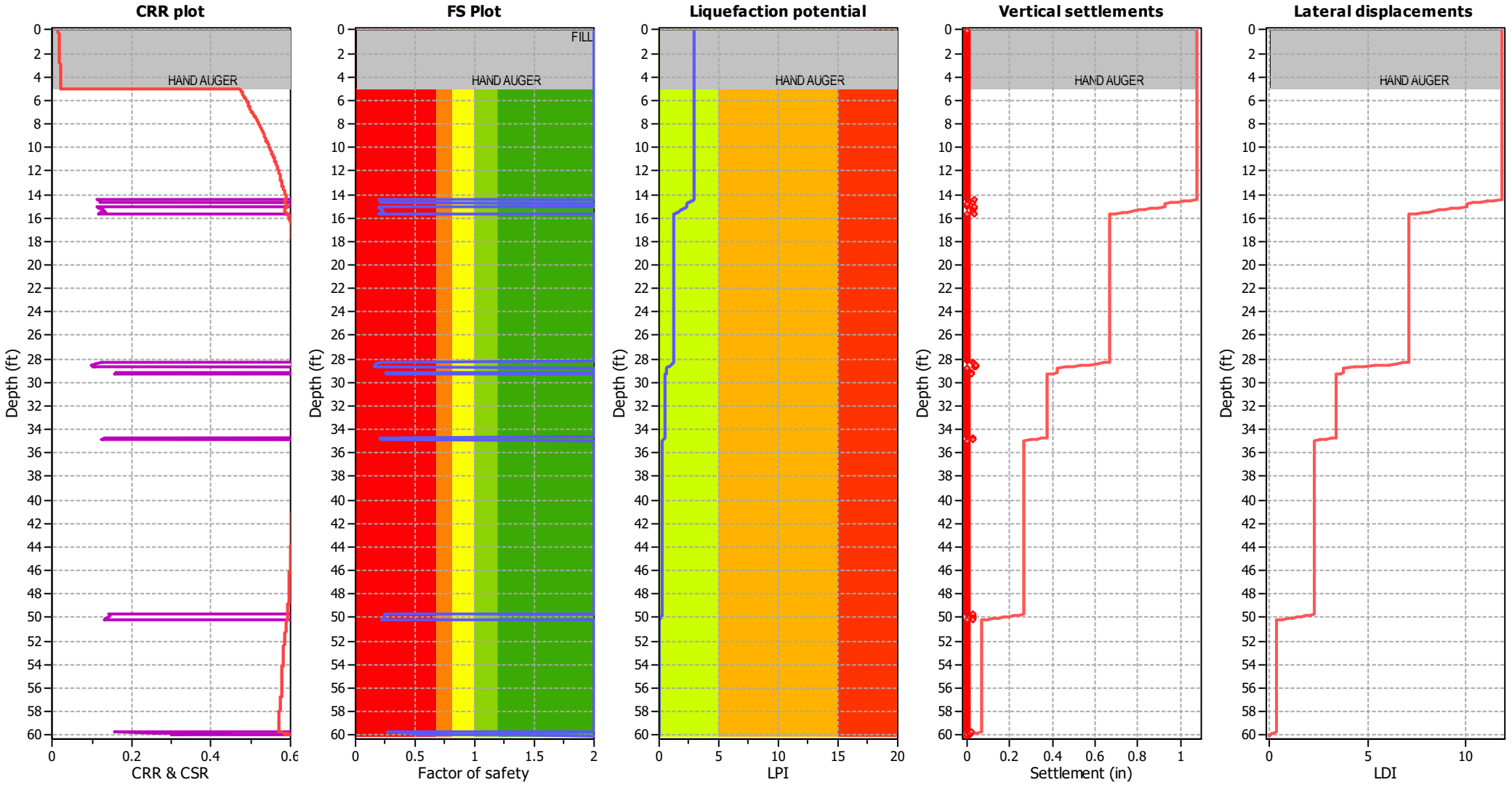
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.98 ft	Fill height:	3.00 ft	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
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LPI color scheme

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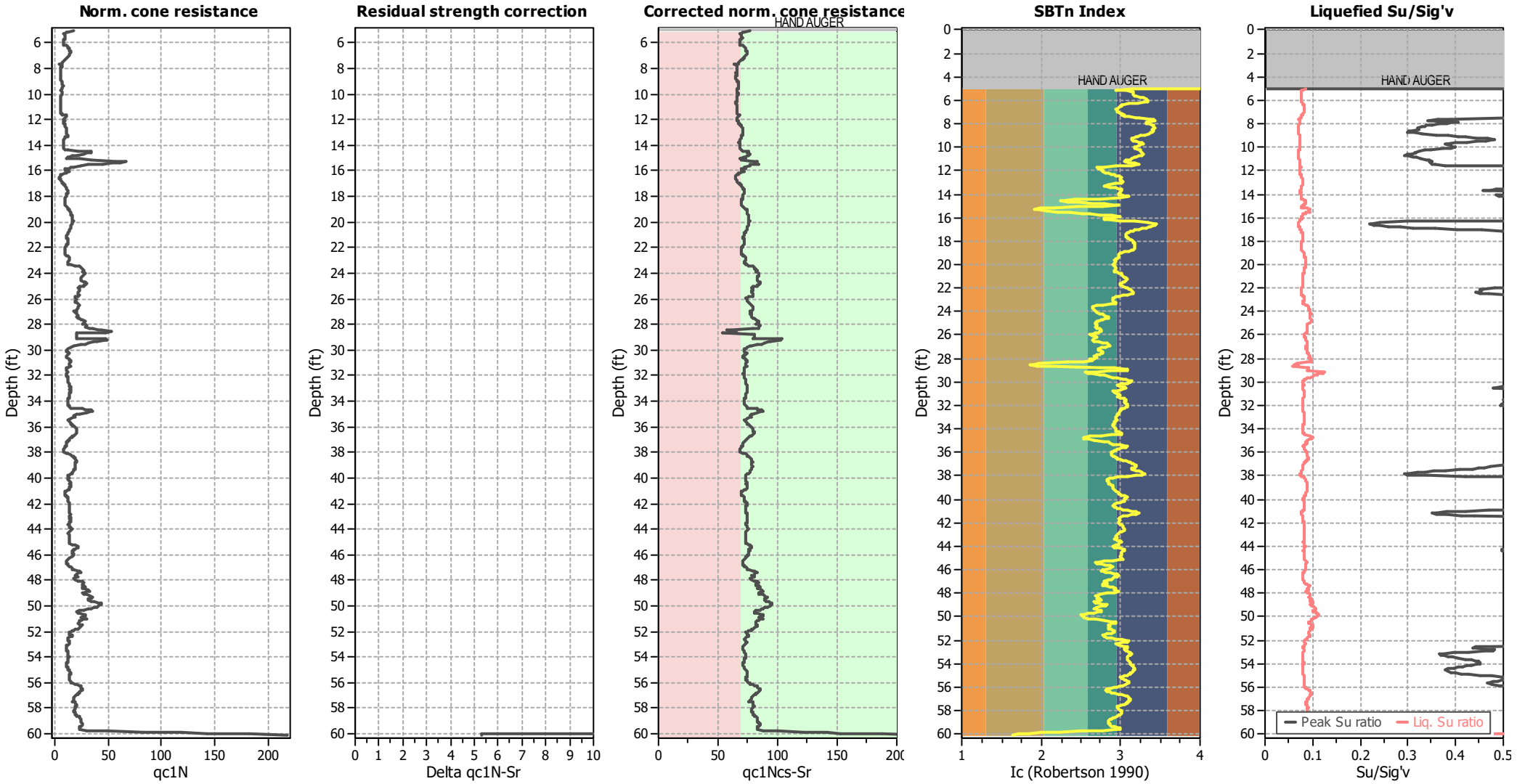
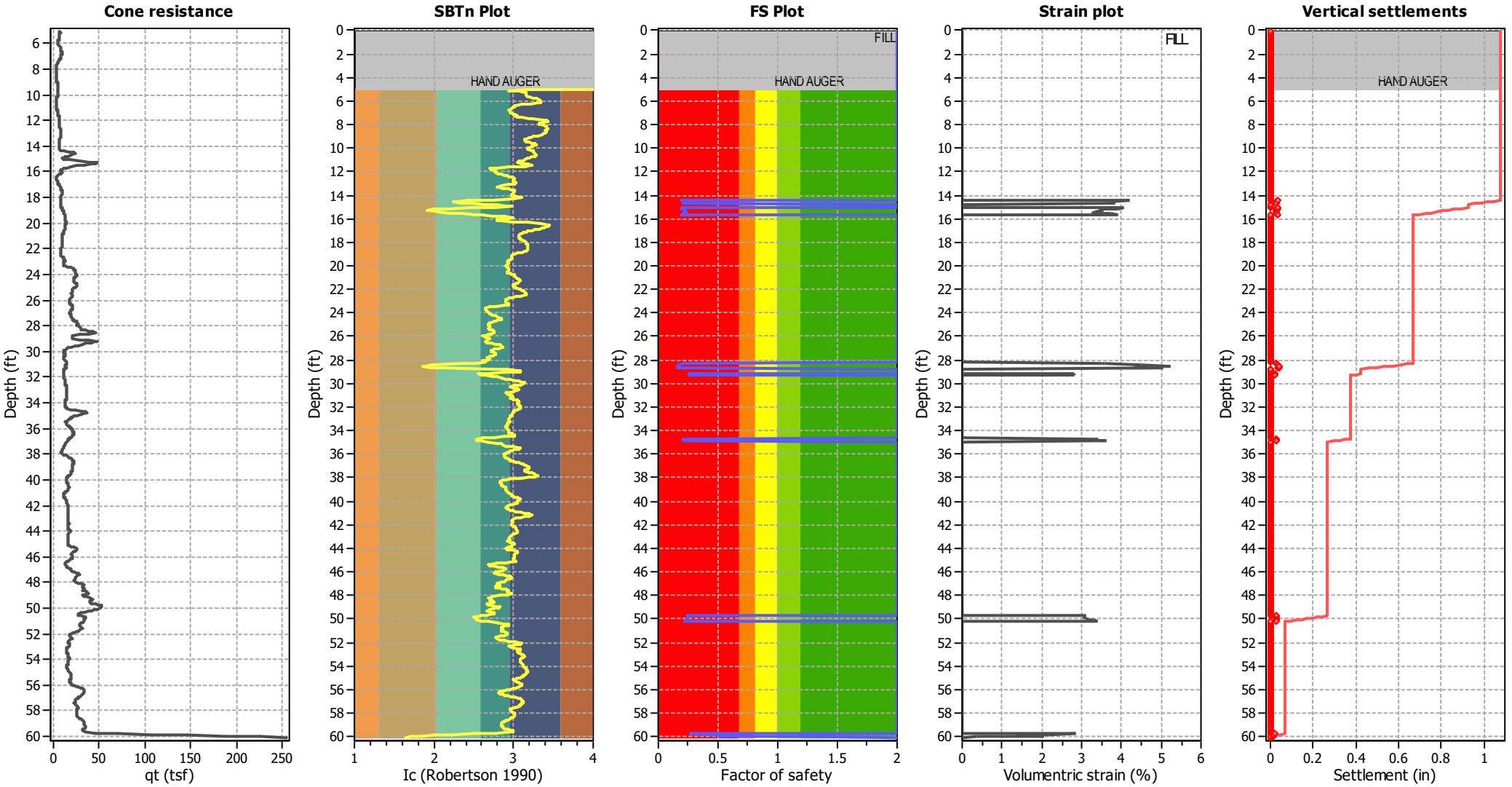
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.98 ft	Fill height:	3.00 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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I_c:	Soil Behaviour Type Index
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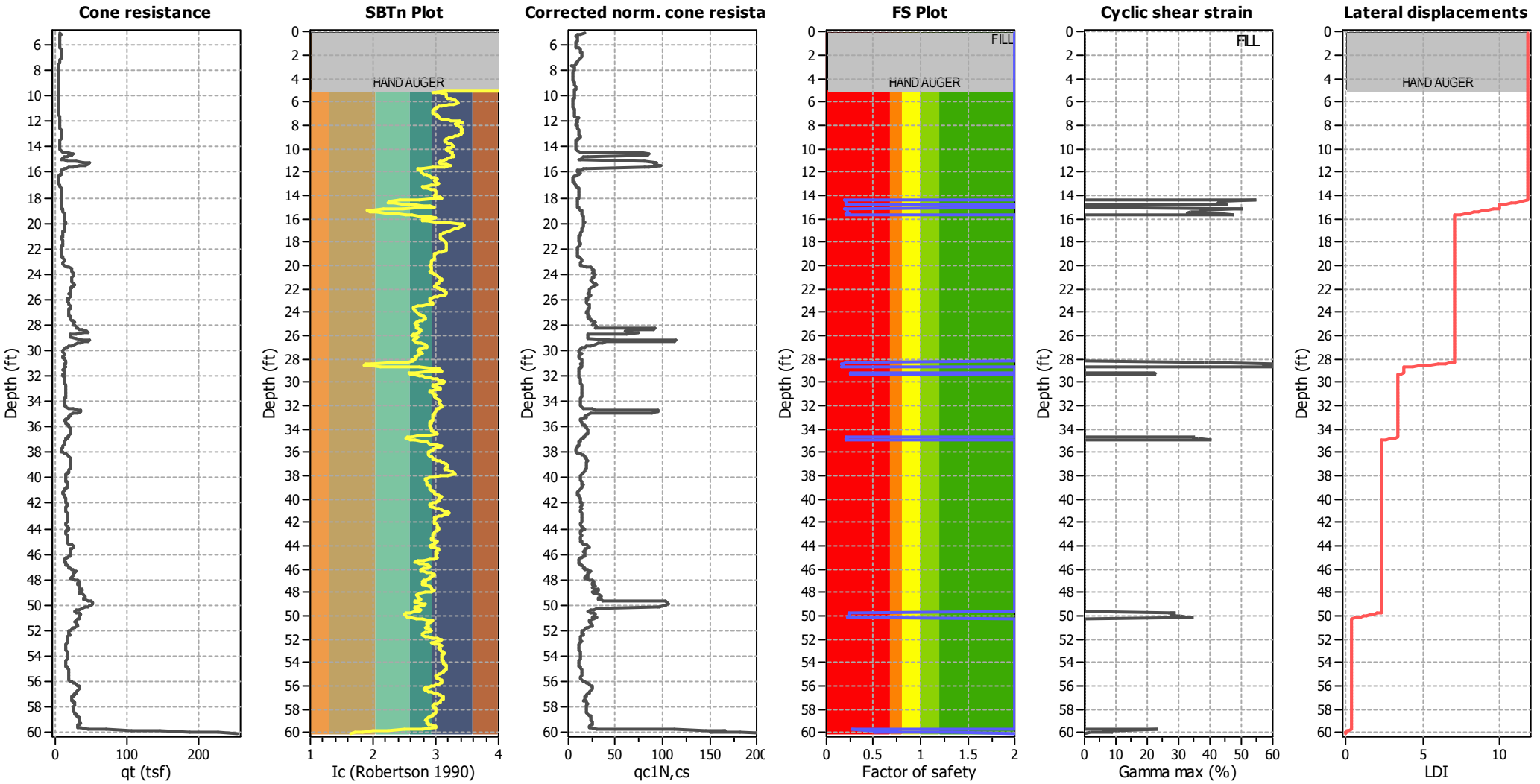
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

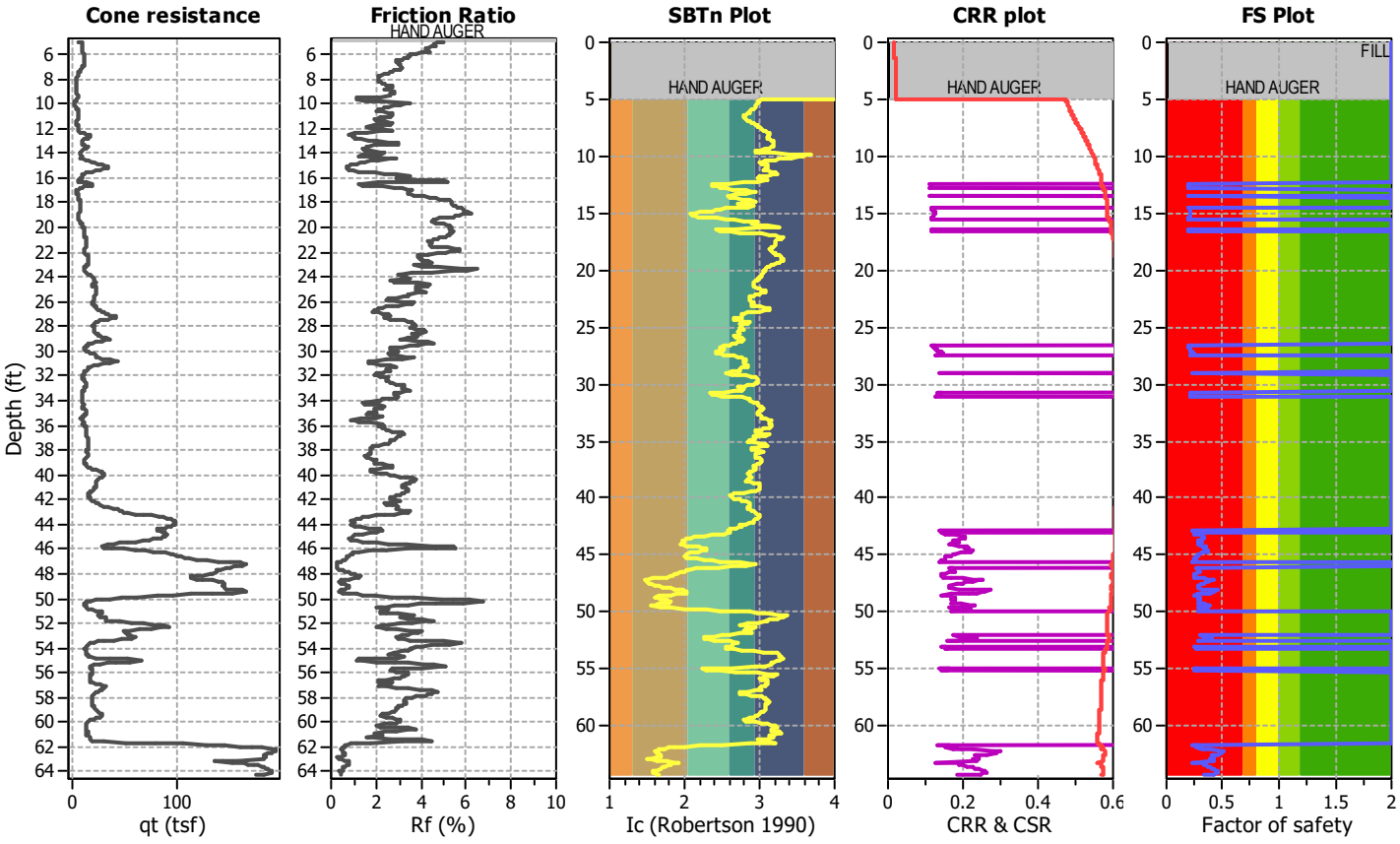
LIQUEFACTION ANALYSIS REPORT

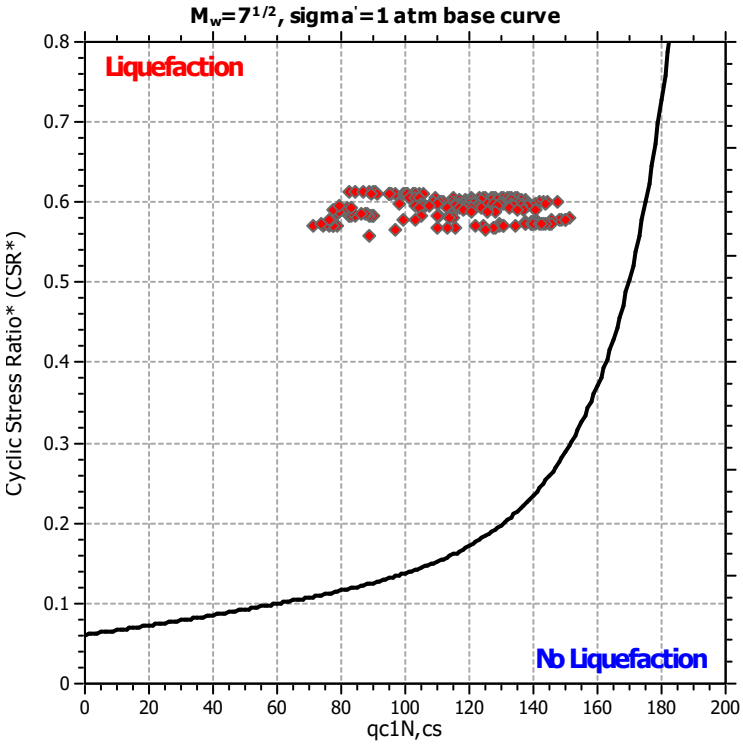
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

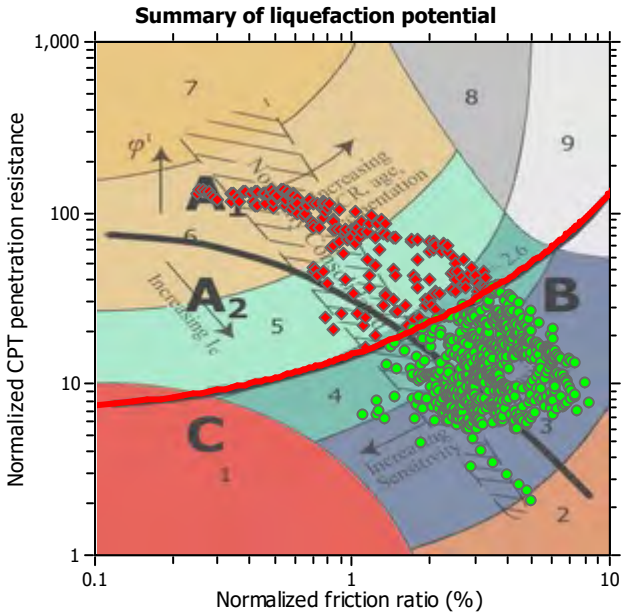
CPT file : GS26-CPT-18

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.64 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	3.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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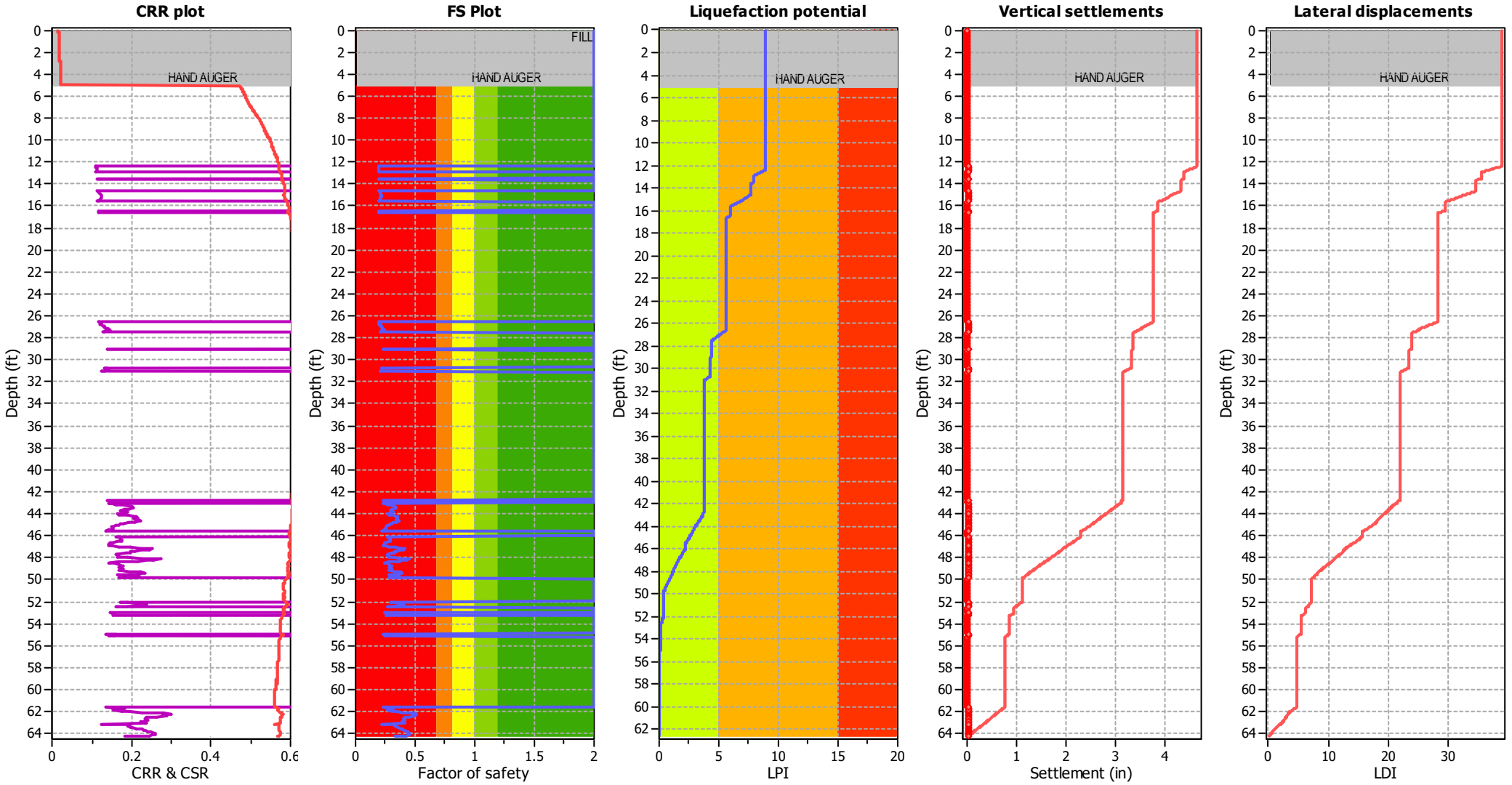
Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.64 ft	Fill height:	3.00 ft	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
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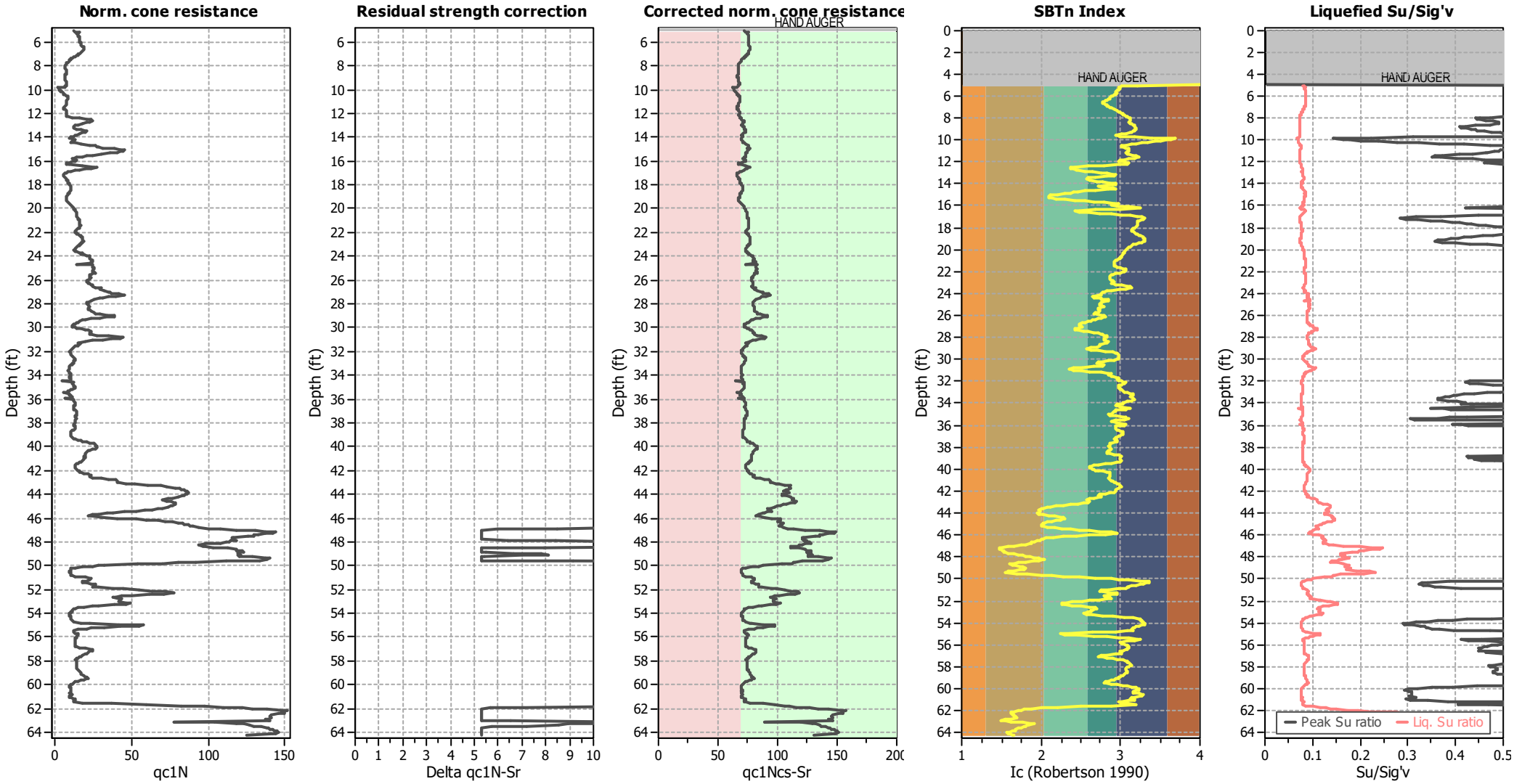
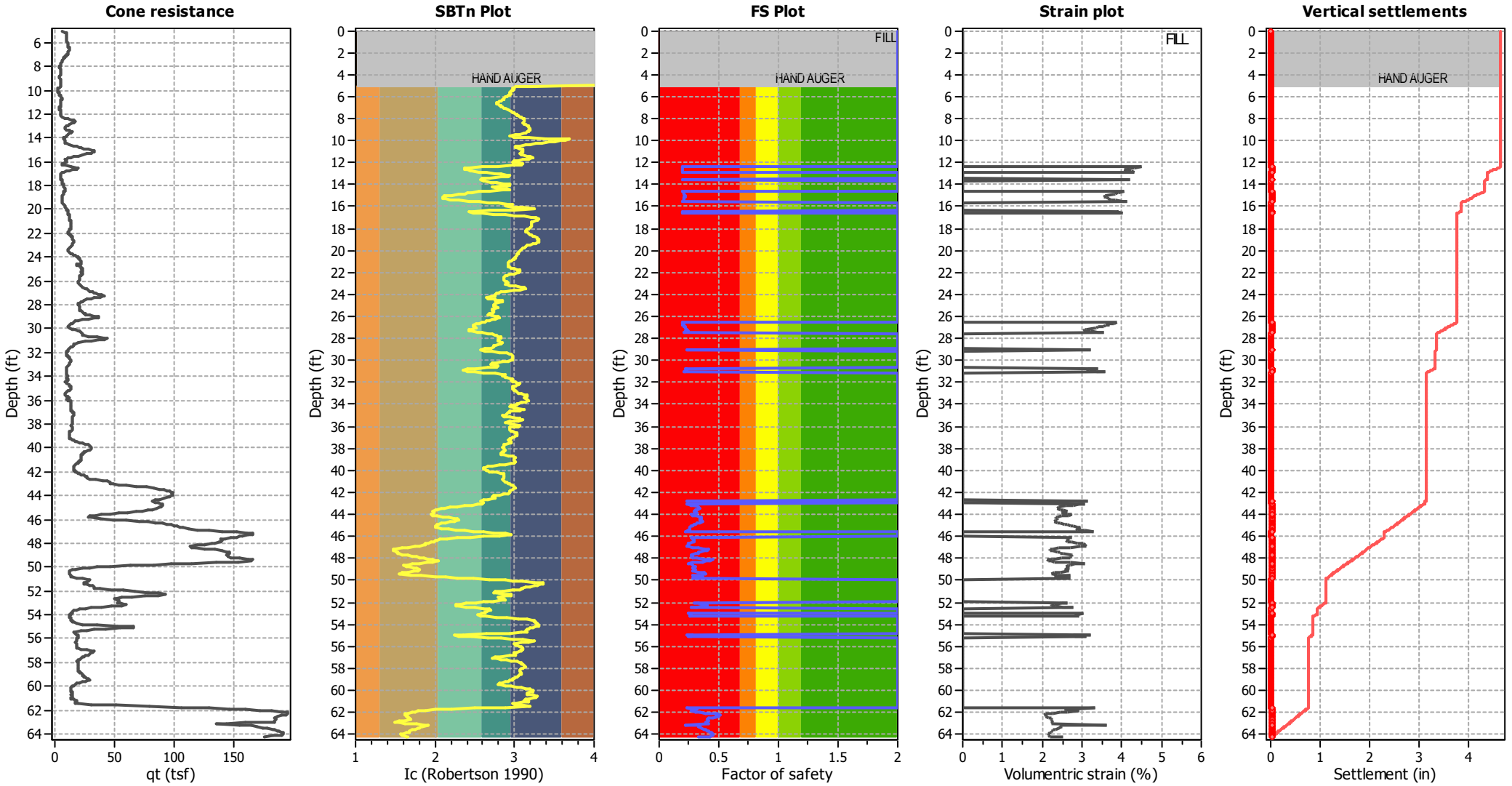
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.64 ft	Fill height:	3.00 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



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I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Operating Bases Earthquake Ground Motion (OBE-GM) CPT-based Liquefaction Outputs with Fines Content Adjustment

GS26-CPT-1

GS26-CPT-5

Geosyntec Consultants, Inc.
SF Bay Area Branch

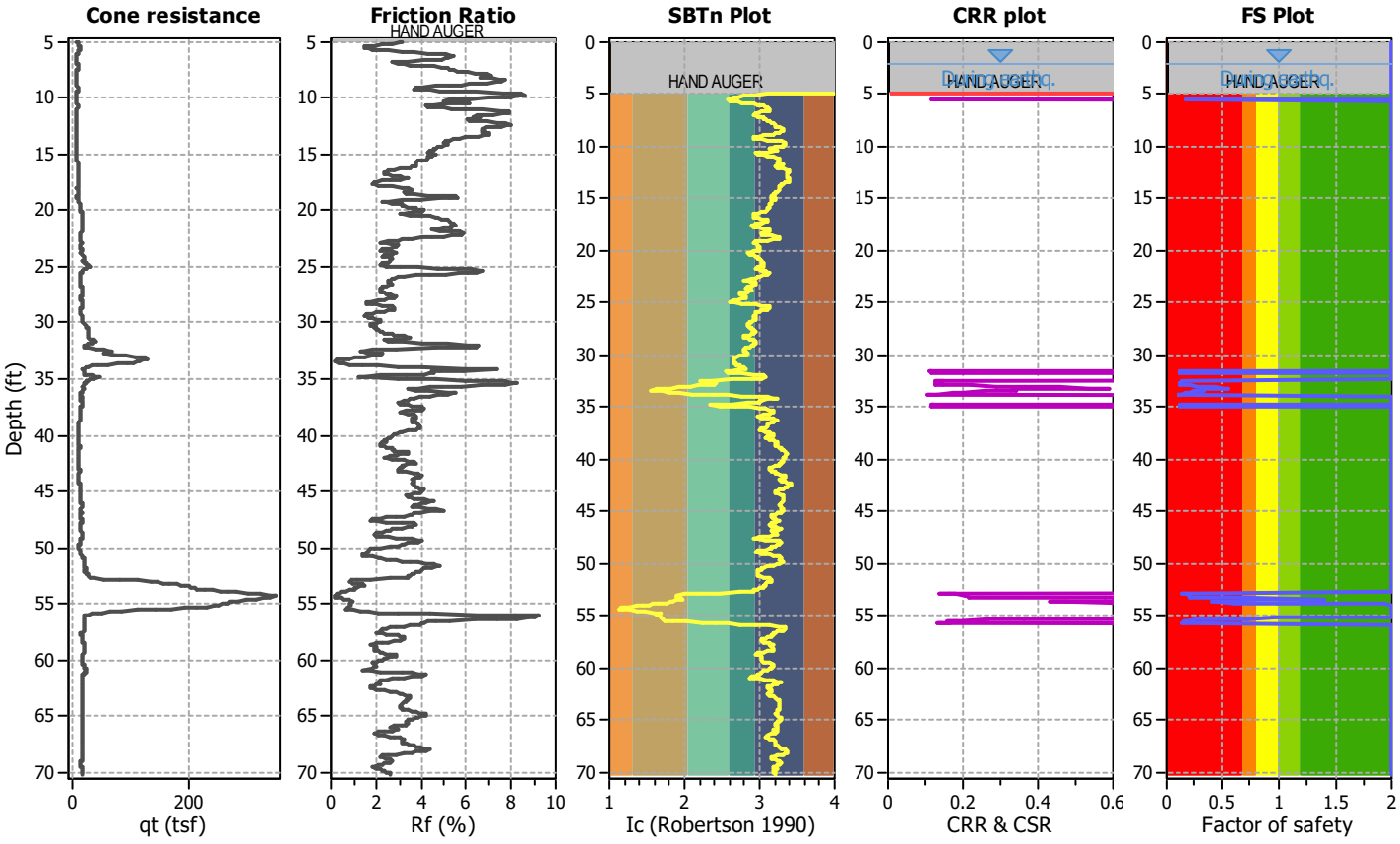
LIQUEFACTION ANALYSIS REPORT

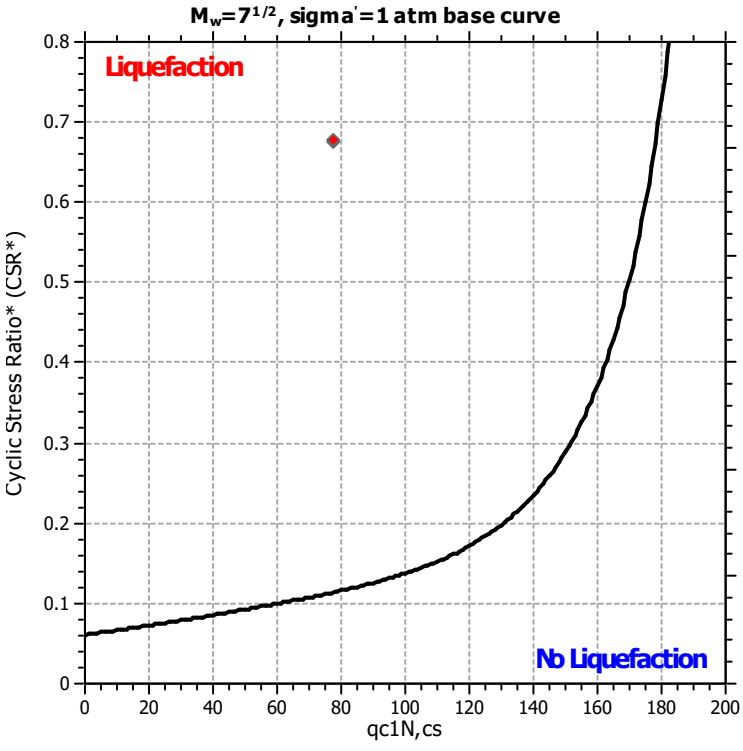
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

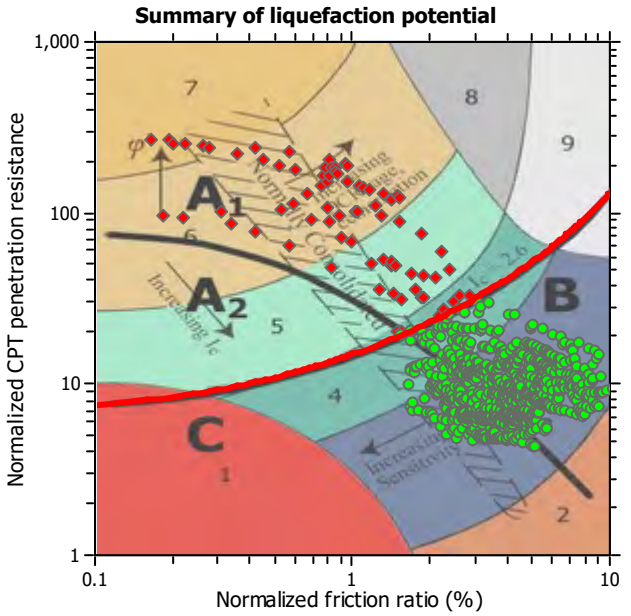
CPT file : GS26-CPT-01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.59 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	0.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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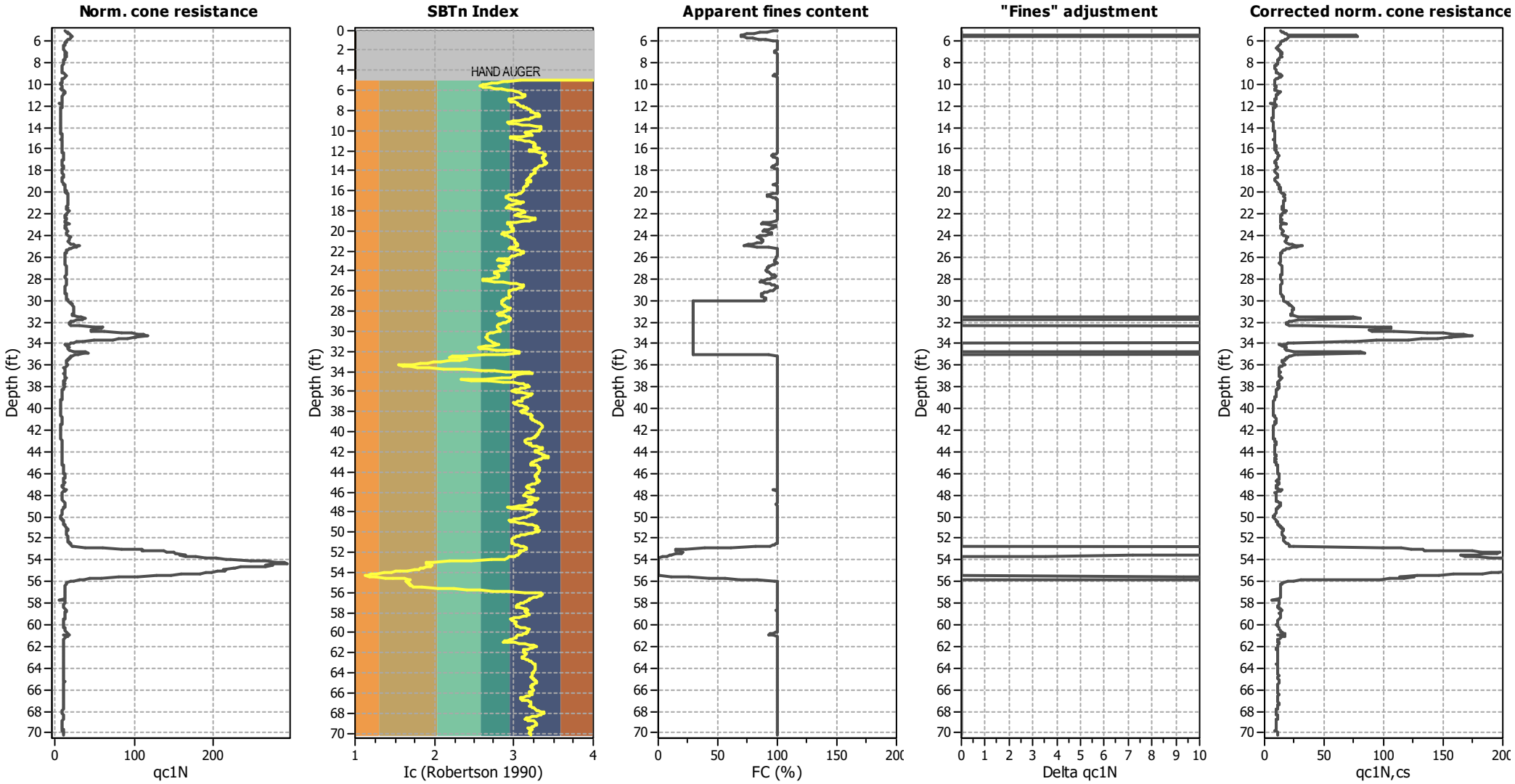
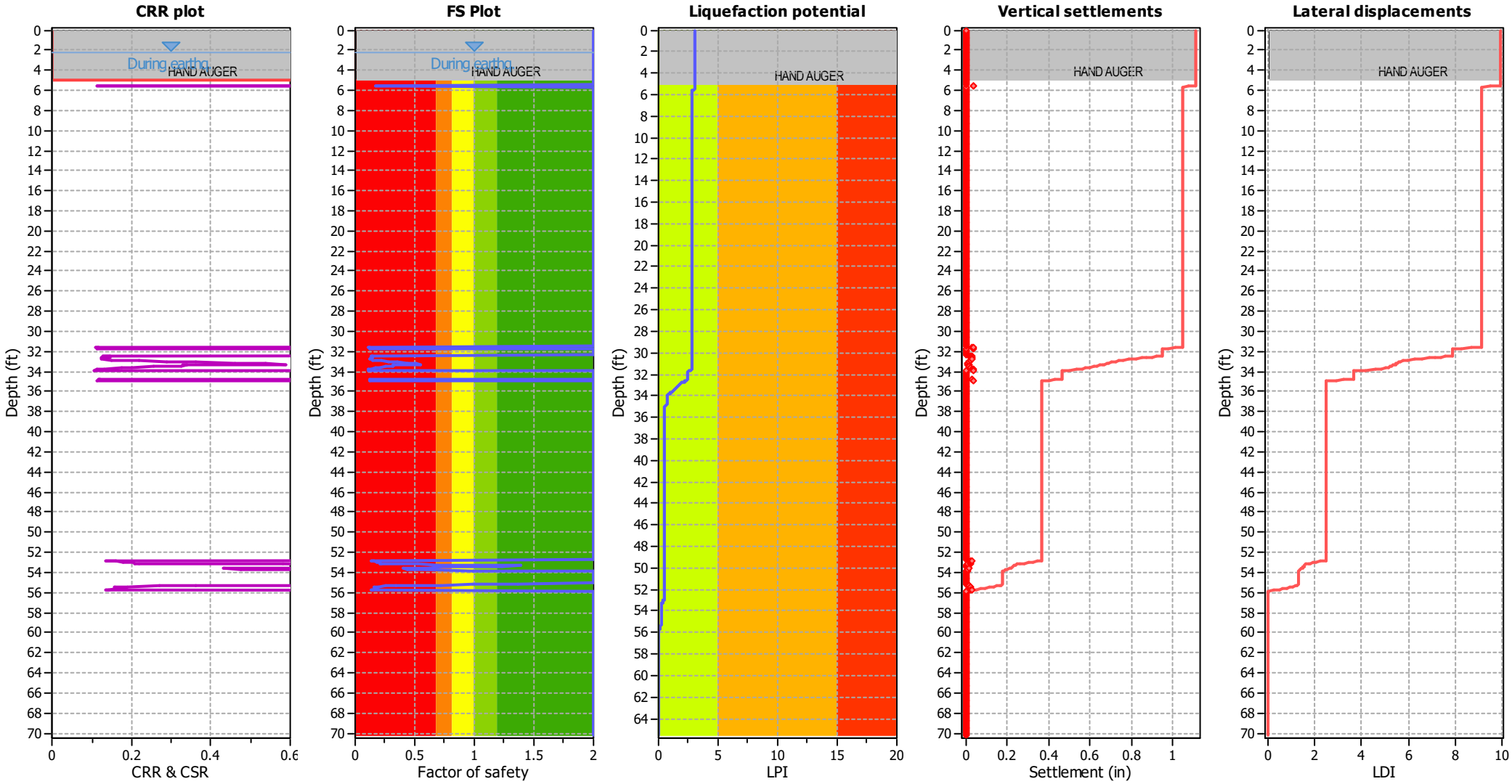
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
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F.S. color scheme

- Almost certain it will liquefy
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LPI color scheme

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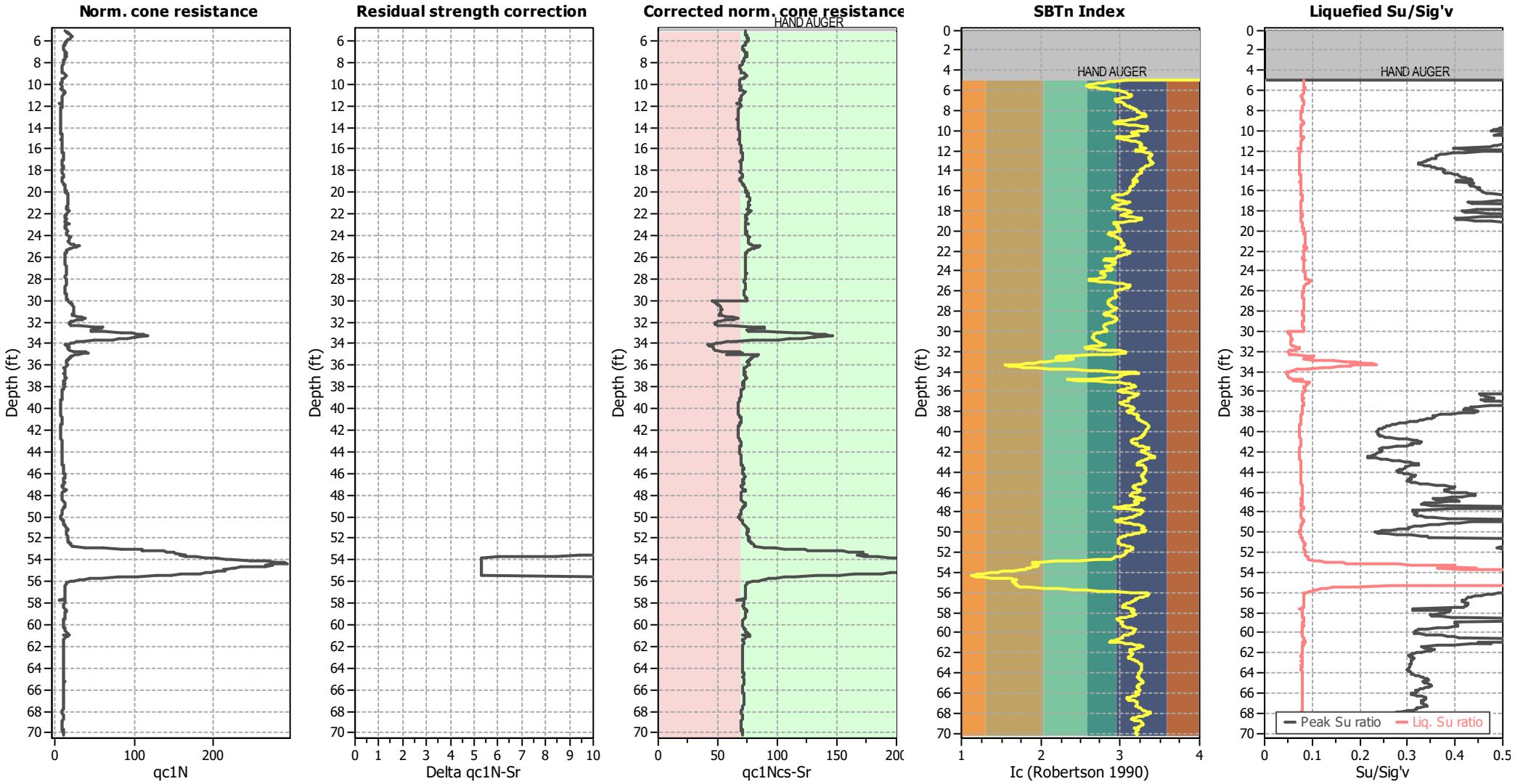
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

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Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
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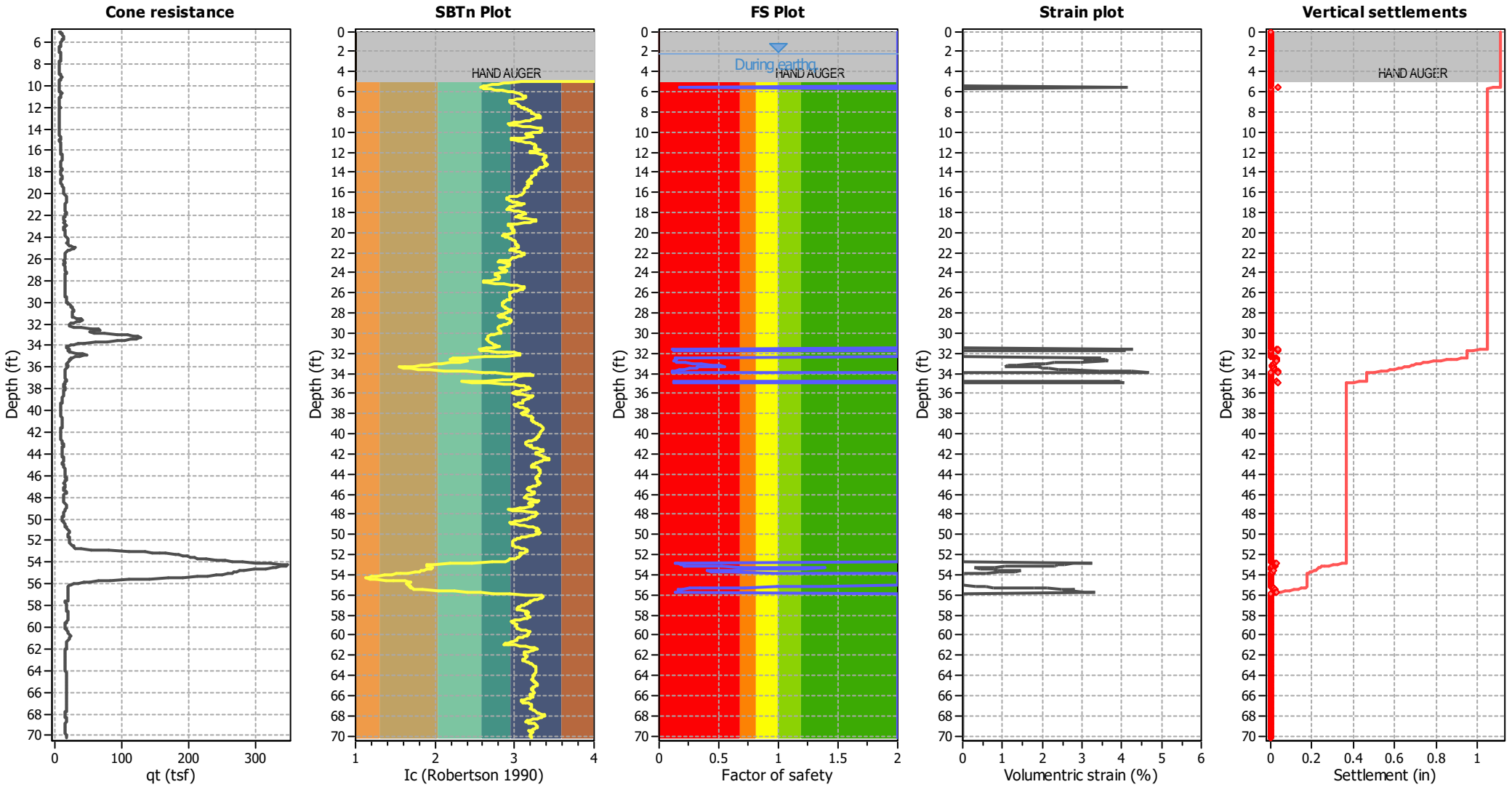
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
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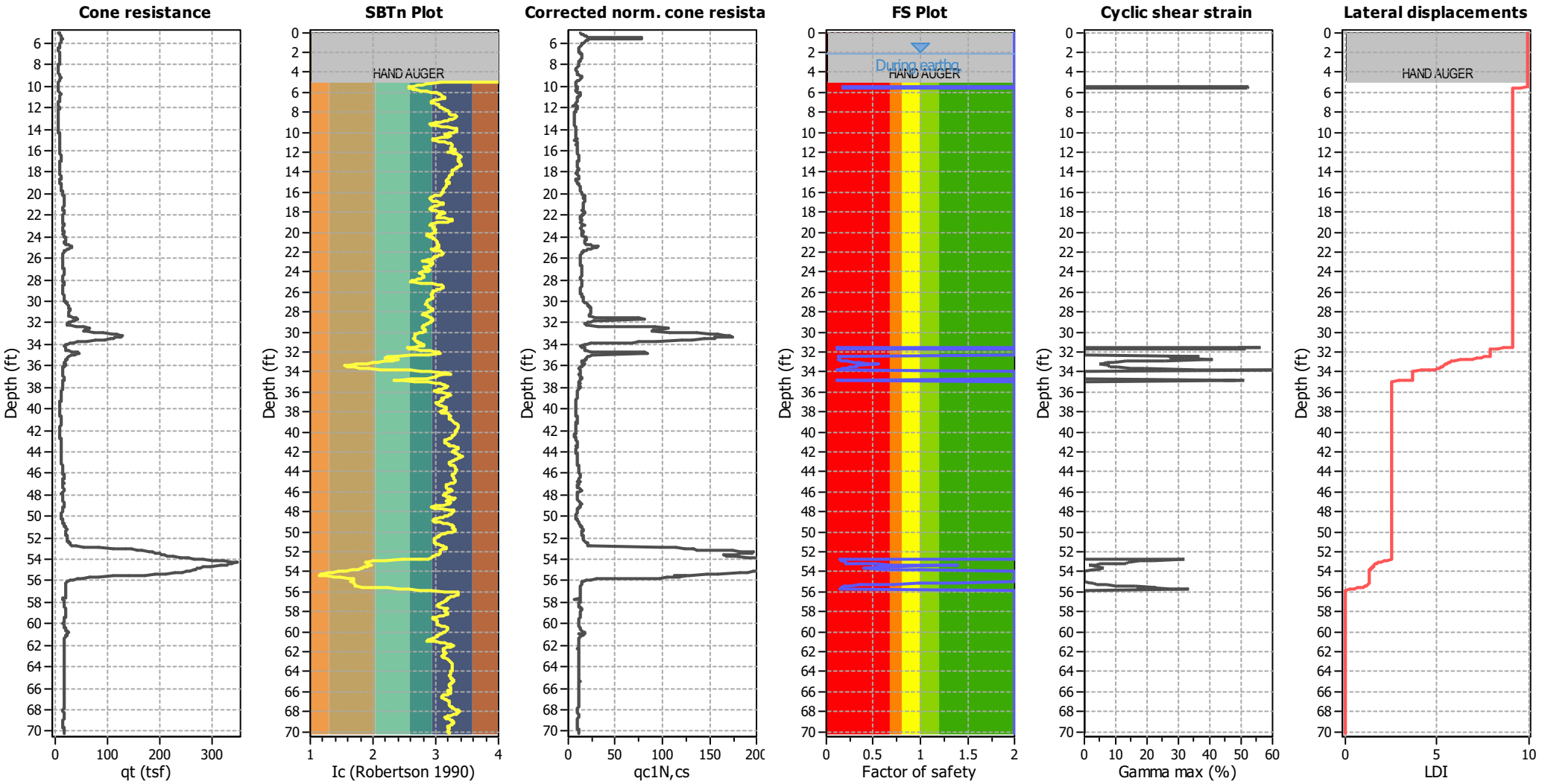
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



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$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

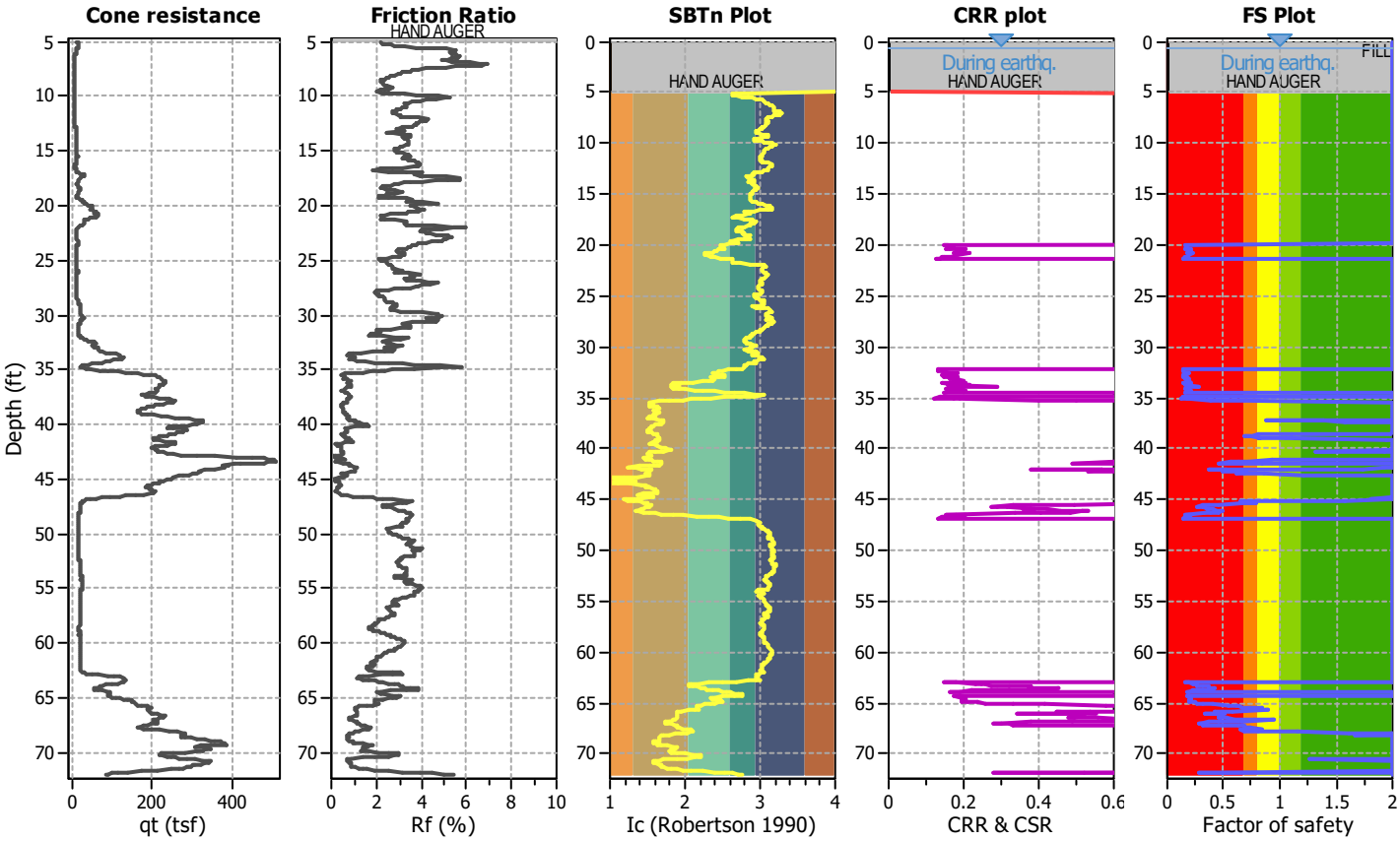
LIQUEFACTION ANALYSIS REPORT

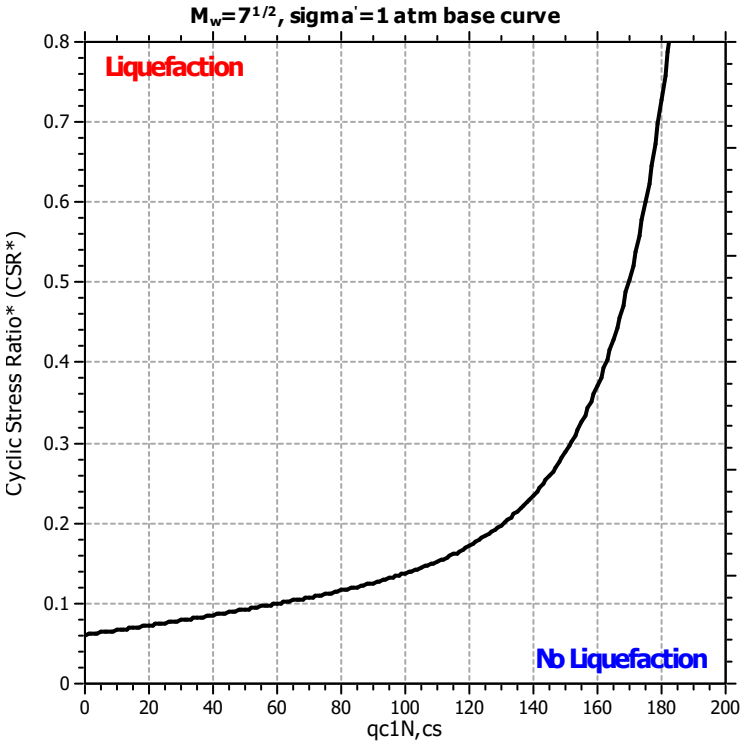
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

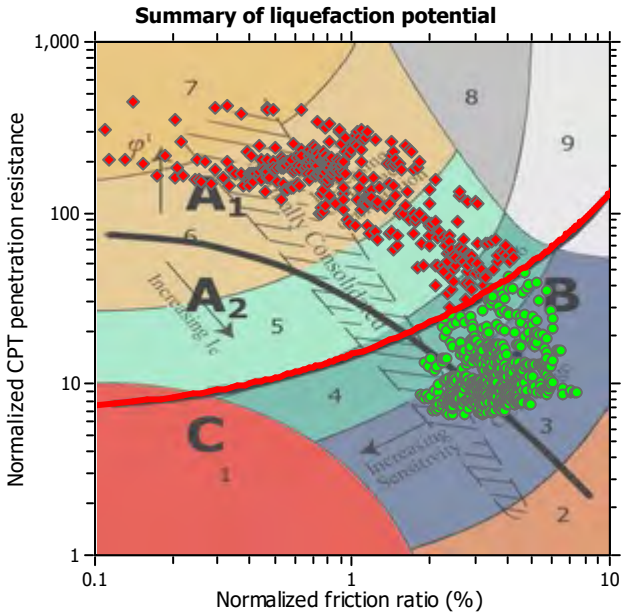
CPT file : GS26-CPT-05

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	9.67 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.60 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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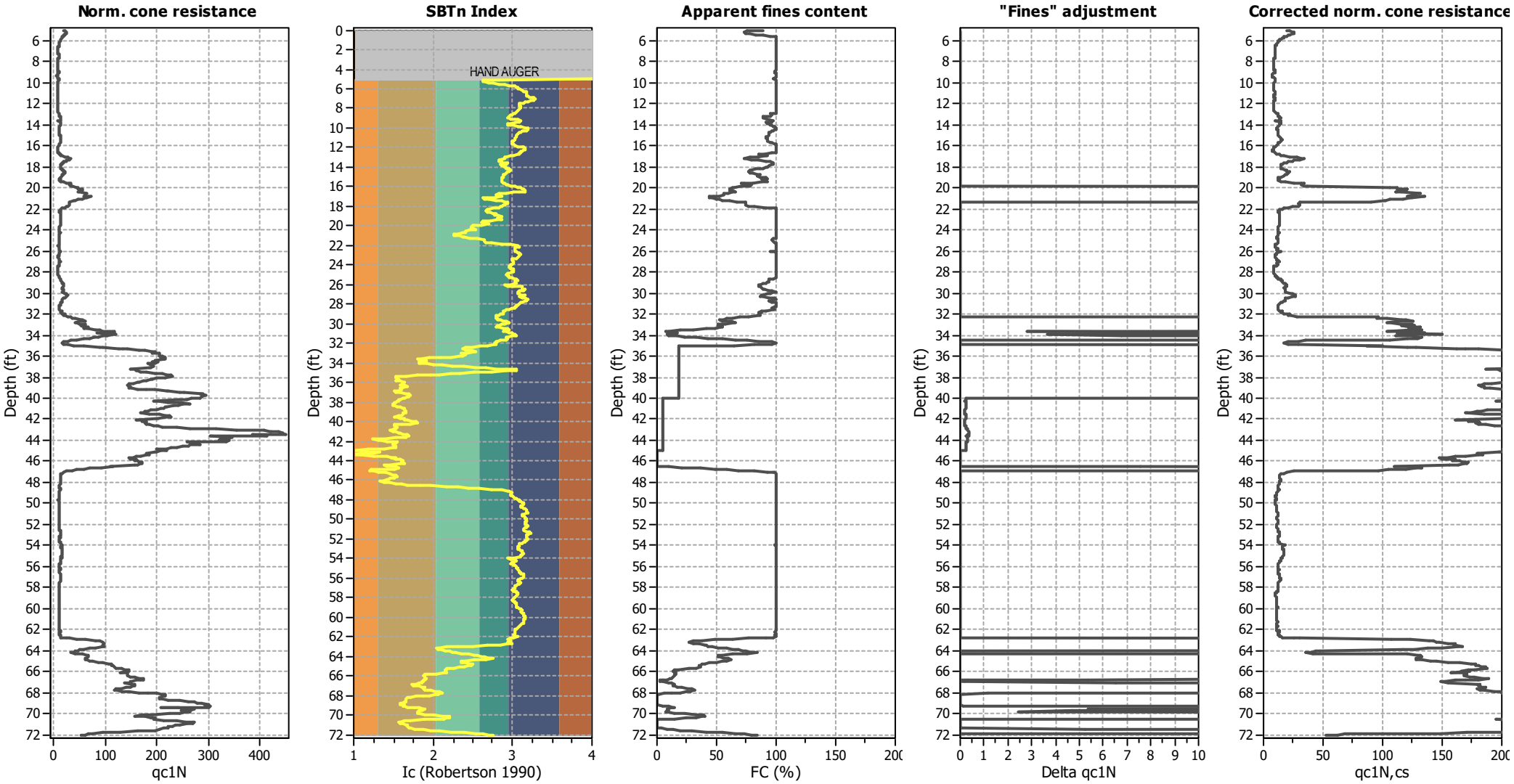
Liquefaction analysis overall plots (intermediate results)



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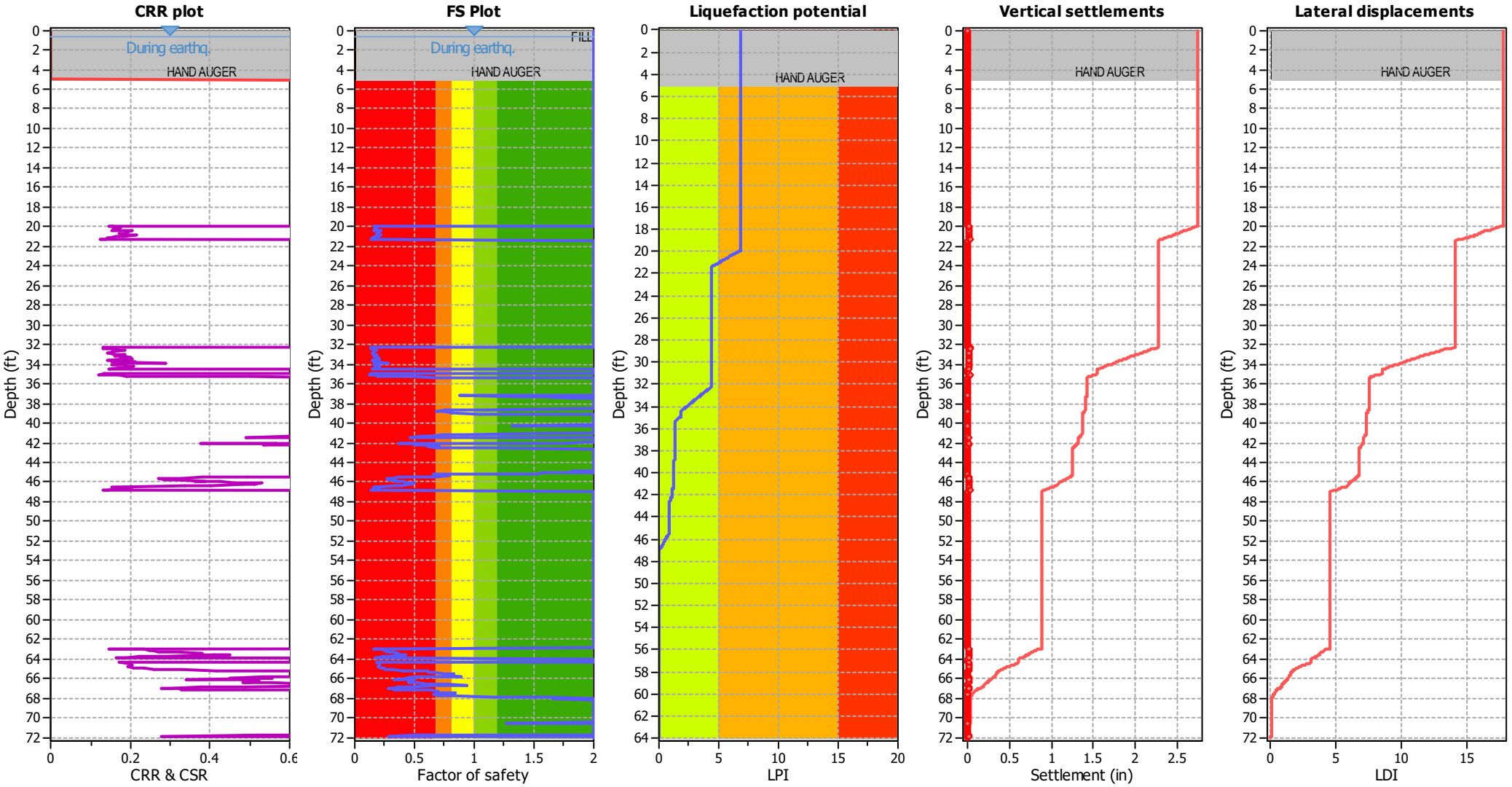
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	9.67 ft	Fill height:	1.60 ft	Limit depth:	N/A

Liquefaction analysis overall plots



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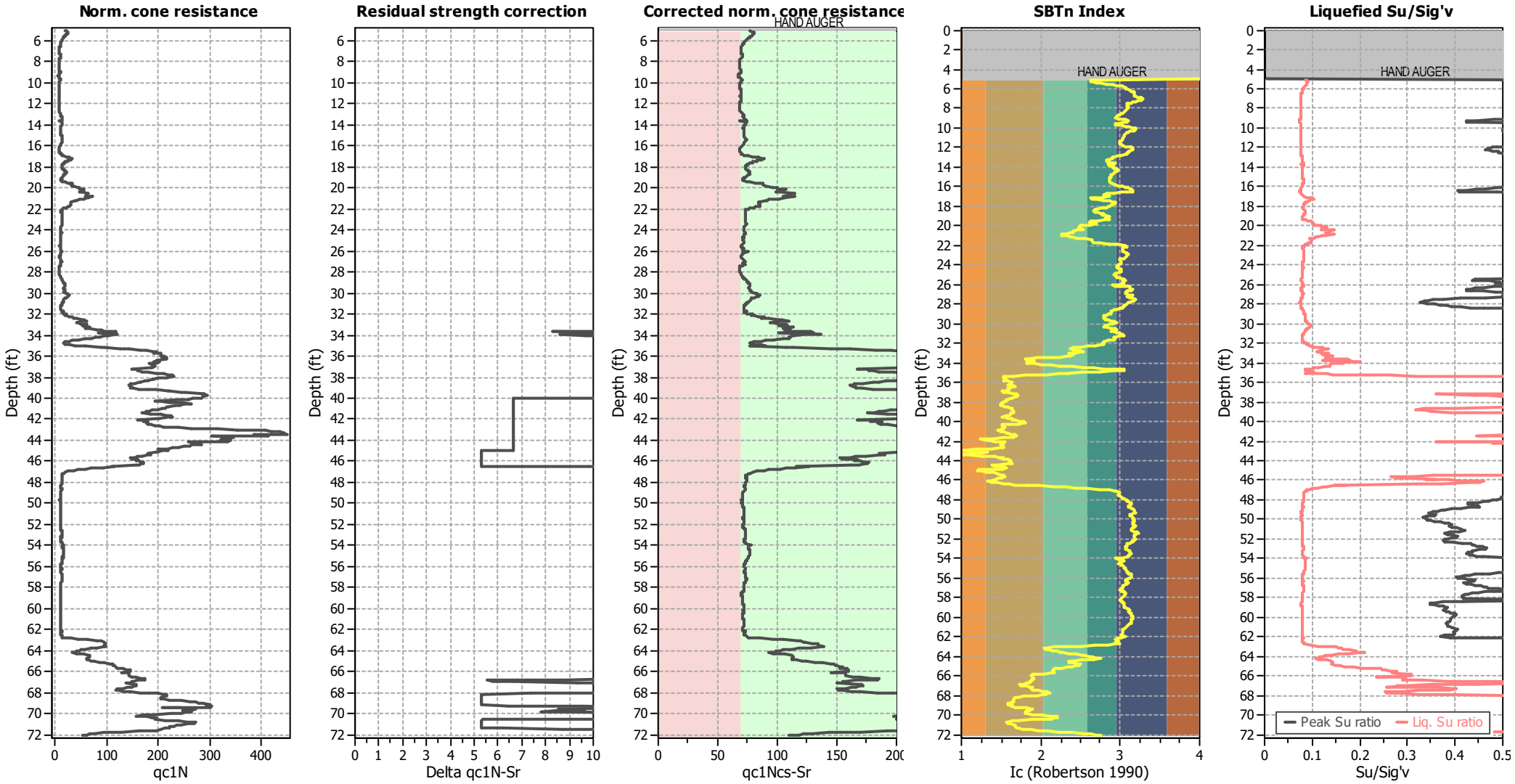

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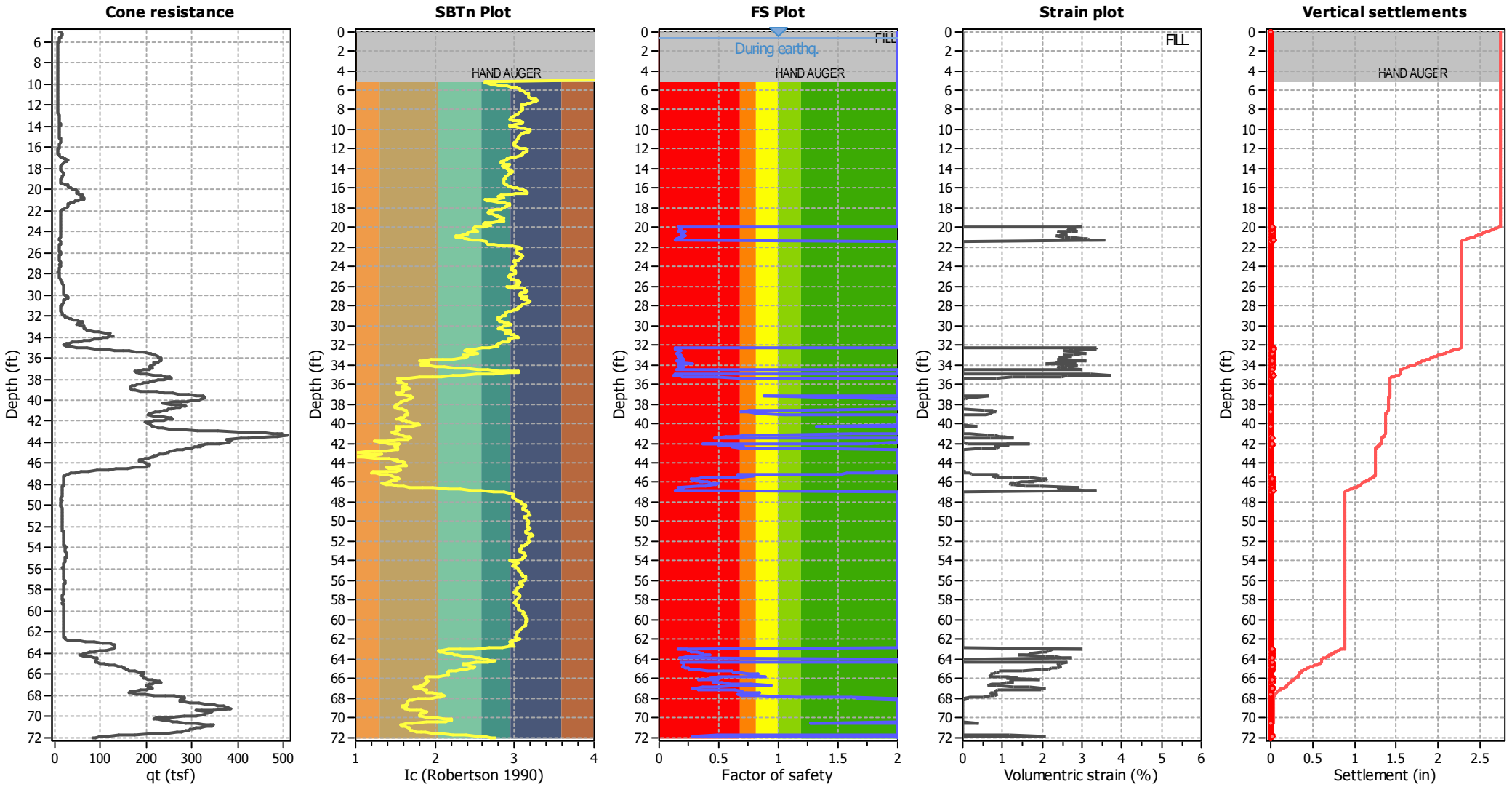
Estimation of post-earthquake settlements



Abbreviations

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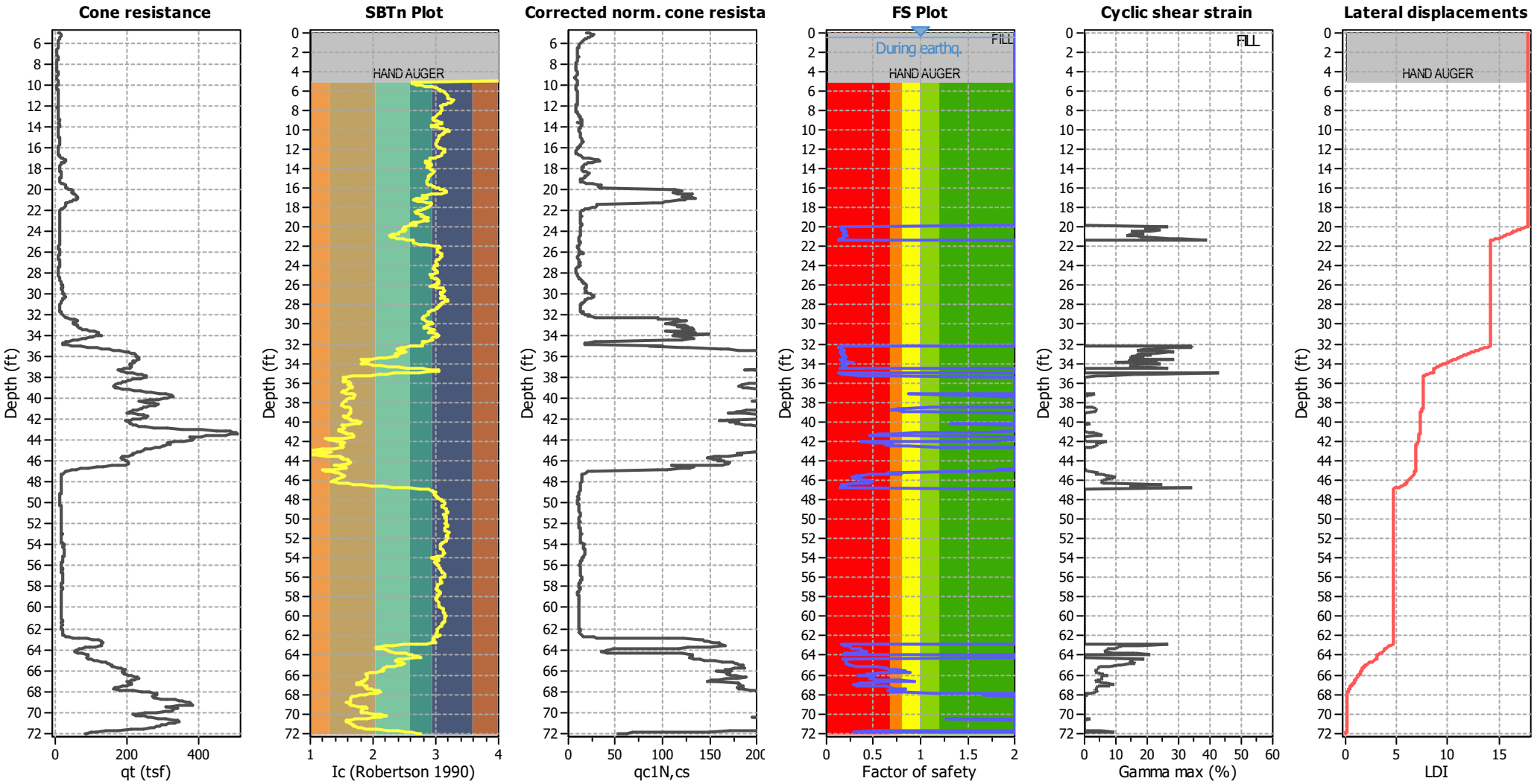
Estimation of post-earthquake lateral Displacements



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$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
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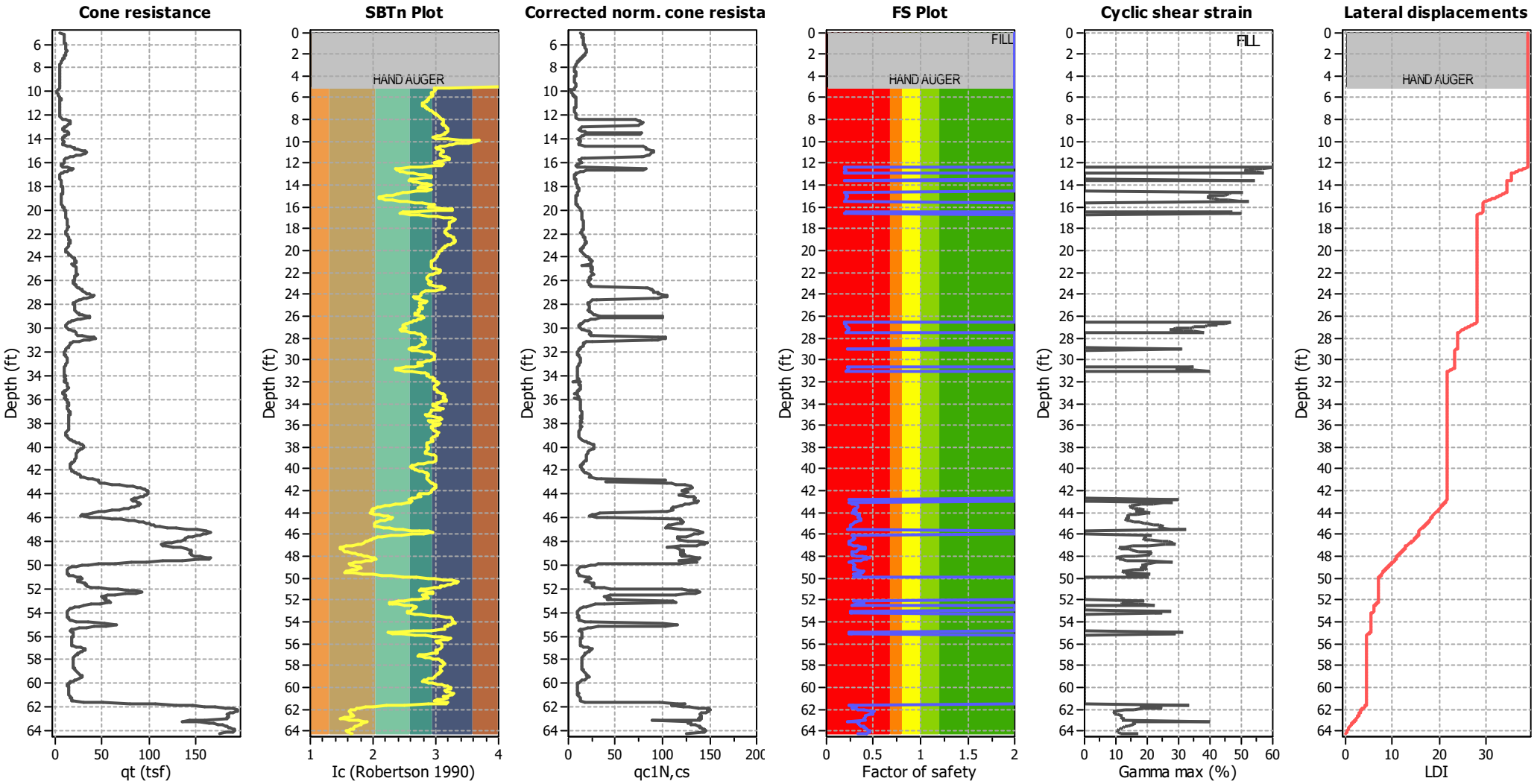
LDI: Lateral displacement index

Estimation of post-earthquake lateral Displacements



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Maximum Design Earthquake Ground Motion (MDE-GM) SPT and CPT-based Liquefaction Outputs

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: SAFER - East Palo Alto, South of Bay Road
Location: East Palo Alto, California

Project Number: WG3607
Prepared By: AK

Checked by: JM
Date: 5/21/2026

Boring: GS25-B-1
Date: 16-Jan-26
By: Geosyntec Consultants, Inc.

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Mud Rotary
Ground Elevation (ft)[2]: 13.62
Boring Coordinates[2]: N: 1994551.8, E: 6090246.2

$a_{max\ (ground\ surface)}$: 0.77 g[3]
Earthquake Magnitude: 7.9 [3]
MSF: 0.90 Idriss & Boulanger (2008)[4a]

Assumed depth to groundwater at time of earthquake (ft)[24]: 2.80
Assumed depth to groundwater at time of drilling (ft)[24]: 14.6

Potential Liquefaction: $FS_{liq} < 1.1$

Depth	Elevation	Soil Unit Weight	Soil Unit	USCS Class	Borehole Diameter	Sample Type[26]	ER[5]	N_{field}	σ_v	σ_v', during drilling	σ_v', during EQ[24]	N_{field} Correction Factors					N_{60}	Fines Content	Fines Content Method	[11]	[12]	$(N_1)_{60}$[13]	$\Delta(N_1)_{60}$[14]	$(N_1)_{60cs}$[15]	[4b]	[4c]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]
												C_{rod}[6]	$C_{sampler}$[7]	C_L[8]	C_S[9]	C_{SPT}[10]				m	C_N				MSF_{max}	MSF	α	β	r_d	C_σ	K_σ	$CRR_{M7.5,\sigma'vc}$	$CSR_{M7.5,\sigma'vc}$	NL/L
(ft)	(ft)	(pcf)			(mm)		(%)	(blows/ft)	(psf)	(psf)	(psf)						(blows/ft)	%				(blows/ft)		(blows/ft)										
0	13.6																																	
30.0	-16.4	125	Clayey SAND	SC	150.0	SPT	87	26	3545	2583	1848	0.95	1.45	1.05	1.00	1.00	38	29	Lab	0.298	0.94	35	5.3	40	2.20	0.85	-0.60	0.07	0.93	0.300	1.041	4.13	1.014	**NL**
55.0	-41.4	125	GRAVEL w/ Sand	GP	150.0	SPT	87	17	6570	4048	3313	1.00	1.45	1.05	1.00	1.00	26	0	Est	0.449	0.75	19	0.0	19	1.45	0.94	-1.32	0.15	0.84	0.128	0.942	0.19	0.943	**L**

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12"

[2] Horizontal coordinates are based on California State Plan, Zone III. The vertical datum is NAVD88.

[3] a_{max} and earthquake magnitude based on 2008 USGS data. See Seismic Parameter calculation package for details.

[4a] MSF=6.9exp(-M/4)-0.058≤1.8 is magnitude scaling factor (Equation 51-52, I&B 2008)

[4b] MSF_{max}=1.09+($(N_1)_{60cs}$/31.5)^2≤2.2 is maximum magnitude scaling factor (Equation 2.21, I&B 2014)

[4c] MSF=1+(MSF_{max}-1)[8.64exp(-M/4)-1.325] is magnitude scaling factor (Equation 2.19, I&B 2014)

[5] Hammer Energy Ratio based on site-specific calibration of drill rig used for project [Pitcher Drilling; East Palo Alto, California], for SPT and MOD-CAL samples

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] $C_{sampler}$ accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_L accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_S accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. C_{SPT} is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] m=0.784-0.0768sqrt($(N_1)_{60cs}$)≥0.264 is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] C_N=$(P_a/\sigma'_v)^m$≤1.7 accounts for effective overburden stress (Equation 33, I&B 2008)

[13] $(N_1)_{60}$=N_{60}*C_N is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] $\Delta(N_1)_{60}$=exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2] represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)

[15] $(N_1)_{60cs}$=$(N_1)_{60}$ + $\Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] α(z) = -1.012-1.126sin((z/11.73)+5.133) in which z is depth in meters (Equation 23, I&B 2008)

[17] β(z) = 0.106+0.118sin((z/11.28)+5.142) in which z is depth in meters (Equation 24, I&B 2008)

[18] r_d=exp[α(z)+β(z)M] is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] C_σ=1/(18.9-2.55sqrt[$(N_1)_{60cs}$])≤0.3 is the coefficient for K_σ (Equation 56, I&B 2008)

[20] K_σ = 1-$C_\sigma$ln(σ'_{vc}/P_a)≤1.1 is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'vc}$=0.65(a_{max}/g)(σ_v/σ'_v)r_d(1/MSF)(1/K_σ) is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable, defined as factor of safety against the triggering of liquefaction (FS_{liq}) of 1.3 or below

[24] Groundwater is conservatively assumed to be at a depth observed during the field investigation (for blow count correction), and at Mean Higher-High Water Level (MHHW) + 3.5ft Sea-Level Rise (SLR) during an earthquake event.

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: SAFER - East Palo Alto, South of Bay Road
Location: East Palo Alto, California

Project Number: WG3607
Prepared By: AK

Checked by: JM
Date: 5/21/2026

Boring: GS25-B-2
Date: 16-Jan-26
By: Geosyntec Consultants, Inc.

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Mud Rotary
Ground Elevation (ft)[2]: 11.39
Boring Coordinates[2]: N: 1995502.5, E: 6090021.0

$a_{max\ (ground\ surface)}$: 0.77 g[3]
Earthquake Magnitude: 7.9 [3]
MSF: 0.90 Idriss & Boulanger (2008)[4a]

Assumed depth to groundwater at time of earthquake (ft)[24]: 0.60
Assumed depth to groundwater at time of drilling (ft)[24]: 9.7

Potential Liquefaction: FS_{liq} < 1.1

Depth	Elevation	Soil Unit Weight	Soil Unit	USCS Class	Borehole Diameter	Sample Type[20]	ER[5]	N_{field}	σ_v	σ_v', during drilling	σ_v', during EQ[24]	N_{field} Correction Factors					N_{60}	Fines Content	Fines Content Method	[11]	[12]	$(N_1)_{60}$[13]	$\Delta(N_1)_{60}$[14]	$(N_1)_{60cs}$[15]	[4b]	[4c]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]
												C_{rod}[6]	C_{energy}[7]	C_b[8]	C_S[9]	C_{SPT}[10]				m	C_N				MSF_{max}	MSF	α	β	r_d	C_σ	K_σ	$CRR_{M7.5,\sigma'vc}$	$CSR_{M7.5,\sigma'vc}$	NL/L
(ft)	(ft)	(pcf)			(mm)		(%)	(blows/ft)	(psf)	(psf)	(psf)						(blows/ft)	%				(blows/ft)		(blows/ft)										
0	11.4																																	
20.0	-8.6	125	Clayey SAND	SC	150.0	SPT	87	10	2546	1901	1335	0.95	1.45	1.05	1.00	1.00	14	32	Lab	0.441	1.05	15	5.4	20	1.49	0.94	-0.35	0.04	0.96	0.133	1.061	0.21	0.923	L
35.0	-23.6	125	Clayey SAND w/ Gravel	SC	150.0	SPT	87	28	4346	2765	2199	1.00	1.45	1.05	1.00	1.00	43	18	Lab	0.275	0.93	40	4.1	44	2.20	0.85	-0.74	0.08	0.92	0.300	0.988	19.43	1.081	NL
40.0	-28.6	125	Poorly graded GRAVEL w/ Clay	GP	150.0	SPT	87	45	4986	3093	2527	1.00	1.45	1.05	1.00	1.00	69	5	Lab	0.264	0.90	62	0.0	62	2.20	0.85	-0.89	0.10	0.90	0.300	0.947	50.00	1.105	NL
45.0	-33.6	125	SAND with Gravel	SP	150.0	MOD-CAL	87	49	5636	3431	2865	1.00	1.45	1.05	1.00	0.60	45	0	Est	0.304	0.86	39	0.0	39	2.20	0.85	-1.03	0.11	0.88	0.300	0.909	3.02	1.124	NL
70.0	-58.6	125	Silty SAND	SM	150.0	SPT	87	54	8746	4981	4415	1.00	1.45	1.05	1.00	1.00	82	12	Est	0.264	0.80	66	2.1	68	2.20	0.85	-1.71	0.19	0.79	0.300	0.779	50.00	1.183	NL

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes* , EERI Monograph MNO-12"

[2] Horizontal coordinates are based on California State Plan, Zone III. The vertical datum is NAVD88.

[3] a_{max} and earthquake magnitude based on 2008 USGS data. See Seismic Parameter calculation package for details.

[4a] MSF=6.9exp(-M/4)-0.058≤1.8 is magnitude scaling factor (Equation 51-52, I&B 2008)

[4b] MSF_{max}=1.09+(((N_1)$_{60cs}$/31.5)^2≤2.2 is maximum magnitude scaling factor (Equation 2.21, I&B 2014)

[4c] MSF=1+(MSF_{max}-1)[8.64exp(-M/4)-1.325] is magnitude scaling factor (Equation 2.19, I&B 2014)

[5] Hammer Energy Ratio based on site-specific calibration of drill rig used for project [Pitcher Drilling: East Palo Alto, California], for SPT and MOD-CAL samples

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] C_{energy} accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. C_{SPT} is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] m=0.784-0.0768sqrt((N_1)$_{60cs}$)≥0.264 is iteratively calculated until (N_1)$_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] C_N=(P_a/σ'_v)^m≤1.7 accounts for effective overburden stress (Equation 33, I&B 2008)

[13] (N_1)$_{60}$=N_{60}*C_N is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] Δ(N_1)$_{60}$=exp{1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2} represents the change in (N_1)$_{60}$ with fines content (Equation 76, I&B 2008)

[15] (N_1)$_{60cs}$=(N_1)$_{60}$ + Δ(N_1)$_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] α(z) = -1.012-1.126sin((z/11.73)+5.133) in which z is depth in meters (Equation 23, I&B 2008)

[17] β(z) = 0.106+0.118sin((z/11.28)+5.142) in which z is depth in meters (Equation 24, I&B 2008)

[18] r_d=exp[α(z)+β(z)M] is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] C_σ=1/(18.9-2.55sqrt((N_1)$_{60cs}$))≤0.3 is the coefficient for K_σ (Equation 56, I&B 2008)

[20] K_σ = 1-$C_\sigma$ln(σ'_{vc}/P_a)≤1.1 is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'vc}$=0.65(a_{max}/g)(σ_v/σ_v')r_d(1/MSF)(1/K_σ) is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable, defined as factor of safety against the triggering of liquefaction (FS_{liq}) of 1.3 or below

[24] Groundwater is conservatively assumed to be at a depth observed during the field investigation (for blow count correction), and at Mean Higher-High Water Level (MHHW) + 3.5ft Sea-Level Rise (SLR) during an earthquake event.

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project:	SAFER - East Palo Alto, South of Bay Road	Project Number:	WG3607	Checked by:	JM
Location:	East Palo Alto, California	Prepared By:	AK	Date:	5/21/2026

Boring:	GS25-B-3	Hammer Type:	Automatic 140 lb./30-in.	$a_{max\ (ground\ surface)}$:	0.77 g[3]	Assumed depth to groundwater at time of earthquake (ft)[24]:	-1.90
Date:	16-Jan-26	Drilling Method:	Mud Rotary	Earthquake Magnitude:	7.9 [3]	Assumed depth to groundwater at time of drilling (ft)[24]:	6.0
By:	Geosyntec Consultants, Inc.	Ground Elevation (ft)[2]:	8.90	MSF:	0.90 Idriss & Boulanger (2008)[4a]	Potential Liquefaction:	FS_{liq} < 1.1
		Boring Coordinates[2]:	N: 1997202.5, E: 6089734.2				

Depth	Elevation	Soil Unit Weight	Soil Unit	USCS Class	Borehole Diameter	Sample Type[26]	ER[5]	N_{field}	σ_v	σ_v', during drilling	σ_v', during EQ[24]	N_{field} Correction Factors					N_{60}	Fines Content	Fines Content Method	[11]	[12]	$(N_1)_{60}$[13]	$\Delta(N_1)_{60}$[14]	$(N_1)_{60cs}$[15]	[4b]	[4c]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]
(ft)	(ft)	(pcf)			(mm)		(%)	(blows/ft)	(psf)	(psf)	(psf)	C_{rod}[6]	C_{energy}[7]	C_b[8]	C_S[9]	C_{SPT}[10]	(blows/ft)	%		m	C_N	(blows/ft)		(blows/ft)	MSF_{max}	MSF	α	β	r_d	C_σ	K_σ	$CRR_{M7.5,\sigma'_{vc}}$	$CSR_{M7.5,\sigma'_{vc}}$	NL/L
0	8.9																																
65.0	-56.1	125	Silty SAND	SM	150.0	SPT	87	31	8267	4586	4092	1.00	1.45	1.05	1.00	1.00	47	35	Lab	0.280	0.81	38	5.5	43	2.20	0.85	-1.59	0.17	0.81	0.300	0.802	12.57	1.199	**NL**
70.0	-61.1	125	Silty SAND	SM	150.0	SPT	87	24	8892	4899	4405	1.00	1.45	1.05	1.00	1.00	37	35	Lab	0.343	0.75	27	5.5	33	2.19	0.85	-1.71	0.19	0.79	0.235	0.828	0.76	1.133	**L**

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12"

[2] Horizontal coordinates are based on California State Plan, Zone III. The vertical datum is NAVD88.

[3] a_{max} and earthquake magnitude based on 2008 USGS data. See Seismic Parameter calculation package for details.

[4a] MSF=6.9exp(-M/4)-0.058≤1.8 is magnitude scaling factor (Equation 51-52, I&B 2008)

[4b] MSF_{max}=1.09+((N$_1$)$_{60cs}$/31.5)^2≤2.2 is maximum magnitude scaling factor (Equation 2.21, I&B 2014)

[4c] MSF=1+(MSF$_{max}$-1)[8.64exp(-M/4)-1.325] is magnitude scaling factor (Equation 2.19, I&B 2014)

[5] Hammer Energy Ratio based on site-specific calibration of drill rig used for project [Pitcher Drilling: East Palo Alto, California], for SPT and MOD-CAL samples

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] C_{energy} accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. C_{SPT} is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] m=0.784-0.0768sqrt((N$_1$)$_{60cs}$)≥0.264 is iteratively calculated until (N$_1$)$_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] C_N=(P_a/σ'_v)^m≤1.7 accounts for effective overburden stress (Equation 33, I&B 2008)

[13] $(N_1)_{60}$=N_{60}*C_N is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] $\Delta(N_1)_{60}$=exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2] represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)

[15] $(N_1)_{60cs}$=$(N_1)_{60}$ + $\Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] α(z) = -1.012-1.126sin((z/11.73)+5.133) in which z is depth in meters (Equation 23, I&B 2008)

[17] β(z) = 0.106+0.118sin((z/11.28)+5.142) in which z is depth in meters (Equation 24, I&B 2008)

[18] r_d=exp[α(z)+β(z)M] is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] C_σ=1/(18.9-2.55sqrt($(N_1)_{60cs}$])≤0.3 is the coefficient for K_σ (Equation 56, I&B 2008)

[20] K_σ = 1-$C_\sigma$ln(σ'_{vc}/P_a)≤1.1 is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'_{vc}}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'_{vc}}$=0.65(a_{max}/g)(σ_v/σ'_v)r_d(1/MSF)(1/K_σ) is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable, defined as factor of safety against the triggering of liquefaction (FS_{liq}) of 1.3 or below

[24] Groundwater is conservatively assumed to be at a depth observed during the field investigation (for blow count correction), and at Mean Higher-High Water Level (MHHW) + 3.5ft Sea-Level Rise (SLR) during an earthquake event.

LIQUEFACTION SUSCEPTIBILITY EVALUATION[1]

Geosyntec consultants

Project: SAFER - East Palo Alto, South of Bay Road
Location: East Palo Alto, California

Project Number: WG3607
Prepared By: AK

Checked by: JM
Date: 5/21/2026

Boring: GS25-B-4
Date: 16-Jan-26
By: Geosyntec Consultants, Inc.

Hammer Type: Automatic 140 lb./30-in.
Drilling Method: Mud Rotary
Ground Elevation (ft)[2]: 8.71
Boring Coordinates[2]: N: 1999210.6, E: 6089580.6

$a_{max\ (ground\ surface)}$: 0.77 g[3]
Earthquake Magnitude: 7.9 [3]
MSF: 0.90 Idriss & Boulanger (2008)[4a]

Assumed depth to groundwater at time of earthquake (ft)[24]: -2.10
Assumed depth to groundwater at time of drilling (ft)[24]: 1.7

Potential Liquefaction: $FS_{liq} <$ 1.1

Depth	Elevation	Soil Unit Weight	Soil Unit	USCS Class	Borehole Diameter	Sample Type[26]	ER[5]	N_{field}	σ_v	σ_v', during drilling	σ_v', during EQ[24]	N_{field} Correction Factors					N_{60}	Fines Content	Fines Content Method	[11]	[12]	$(N_1)_{60}$[13]	$\Delta(N_1)_{60}$[14]	$(N_1)_{60cs}$[15]	[4b]	[4c]	[16]	[17]	[18]	[19]	[20]	[21]	[22]	[23]
												C_{rod}[6]	C_{sample}[7]	C_b[8]	C_S[9]	C_{SPT}[10]				m	C_N				MSF_{max}	MSF	α	β	r_d	C_σ	K_σ	$CRR_{M7.5,\sigma'vc}$	$CSR_{M7.5,\sigma'vc}$	NL/L
(ft)	(ft)	(pcf)			(mm)		(%)	(blows/ft)	(psf)	(psf)	(psf)						(blows/ft)	%				(blows/ft)		(blows/ft)										
0	8.7																																	
45.0	-36.3	125	Clayey SAND w/ Gravel	SC	150.0	MOD-CAL	87	11	5866	3164	2927	1.00	1.45	1.05	1.00	0.60	10	12	Est	0.541	0.80	8	2.1	10	1.19	0.98	-1.03	0.11	0.88	0.092	0.970	0.12	0.934	L
70.0	-61.3	125	Clayey SAND	SC	150.0	SPT	87	15	8876	4614	4377	1.00	1.45	1.05	1.00	1.00	23	12	Est	0.458	0.70	16	2.1	18	1.42	0.95	-1.71	0.19	0.79	0.124	0.910	0.18	0.929	L

Notes:

[1] Evaluation is based on: "Idriss and Boulanger (2008), *Soil Liquefaction During Earthquakes*, EERI Monograph MNO-12"

[2] Horizontal coordinates are based on California State Plan, Zone III. The vertical datum is NAVD88.

[3] a_{max} and earthquake magnitude based on 2008 USGS data. See Seismic Parameter calculation package for details.

[4a] MSF=6.9exp(-M/4)-0.058≤1.8 is magnitude scaling factor (Equation 51-52, I&B 2008)

[4b] MSF_{max}=1.09+($(N_1)_{60cs}$/31.5)^2≤2.2 is maximum magnitude scaling factor (Equation 2.21, I&B 2014)

[4c] MSF=1+(MSF_{max}-1)[8.64exp(-M/4)-1.325] is magnitude scaling factor (Equation 2.19, I&B 2014)

[5] Hammer Energy Ratio based on site-specific calibration of drill rig used for project [Pitcher Drilling; East Palo Alto, California], for SPT and MOD-CAL samples

[6] C_{rod} accounts for short rod correction (<1 if rod length < 10 meters) (Table 3, I&B 2008)

[7] $C_{sampler}$, accounts for rod energy delivered to sampler (Table 3, I&B 2008)

[8] C_b accounts for the effect of the size of the borehole (Table 3, I&B 2008)

[9] C_s accounts for the effect of the liners in the SPT/MODCAL sampler (Table 3, I&B 2008)

[10] C_{SPT} is a correction factor to adjust the blow counts recorded with MOD-CAL samplers to equivalent SPT blow count values. C_{SPT} is assumed to be 1.0 for SPT samples and 0.60 for MOD-CAL samples based on an outside diameter of 3.0 inches and an inside diameter of 2.4 inches (Burmister, 1948)

[11] m=0.784-0.0768sqrt($(N_1)_{60cs}$)≥0.264 is iteratively calculated until $(N_1)_{60cs}$ converges (Equation 33 and 39, I&B 2008)

[12] C_N=$(P_a/\sigma'_v)^m$≤1.7 accounts for effective overburden stress (Equation 33, I&B 2008)

[13] $(N_1)_{60}$=N_{60}*C_N is the overburden corrected penetration resistance (Equation 31, I&B 2008)

[14] $\Delta(N_1)_{60}$=exp[1.63+(9.7/(FC+0.1))-(15.7/(FC+0.01))^2] represents the change in $(N_1)_{60}$ with fines content (Equation 76, I&B 2008)

[15] $(N_1)_{60cs}$=$(N_1)_{60}$ + $\Delta(N_1)_{60}$ is the equivalent clean-sand SPT penetration resistance (Equation 75, I&B 2008)

[16] α(z) = -1.012-1.126sin((z/11.73)+5.133) in which z is depth in meters (Equation 23, I&B 2008)

[17] β(z) = 0.106+0.118sin((z/11.28)+5.142) in which z is depth in meters (Equation 24, I&B 2008)

[18] r_d=exp[α(z)+β(z)M] is shear stress reduction coefficient (Equation 22, I&B 2008)

[19] C_σ=1/(18.9-2.55sqrt($(N_1)_{60cs}$))≤0.3 is the coefficient for K_σ (Equation 56, I&B 2008)

[20] K_σ = 1-$C_\sigma$ln(σ'_{vc}/P_a)≤1.1 is the overburden correction factor (Equation 54, I&B 2008)

[21] $CRR_{M7.5,\sigma'vc}$ is the derived correlation between CRR and corrected penetration resistance (Equation 70, I&B 2008)

[22] $CSR_{M7.5,\sigma'vc}$=0.65(a_{max}/g)(σ_v/σ'_v)r_d(1/MSF)(1/K_σ) is the equivalent CSR for the reference values of M=7.5 and σ'_{vc}=1 atm (Equation 69, I&B 2008)

[23] NL = non-liquefiable; L = potentially liquefiable, defined as factor of safety against the triggering of liquefaction (FS_{liq}) of 1.3 or below

[24] Groundwater is conservatively assumed to be at a depth observed during the field investigation (for blow count correction), and at Mean Higher-High Water Level (MHHW) + 3.5ft Sea-Level Rise (SLR) during an earthquake event.

Geosyntec Consultants, Inc.
SF Bay Area Branch

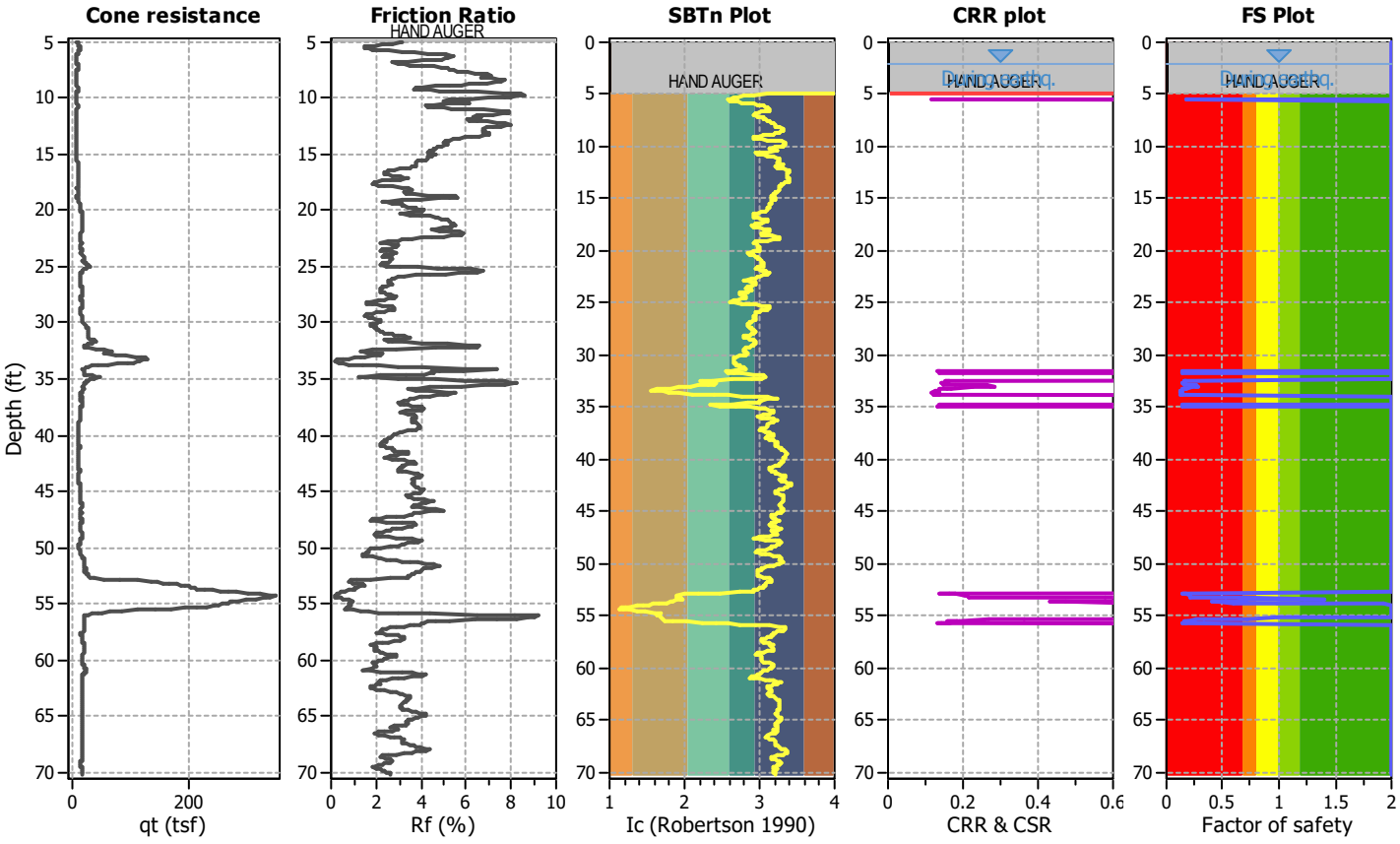
LIQUEFACTION ANALYSIS REPORT

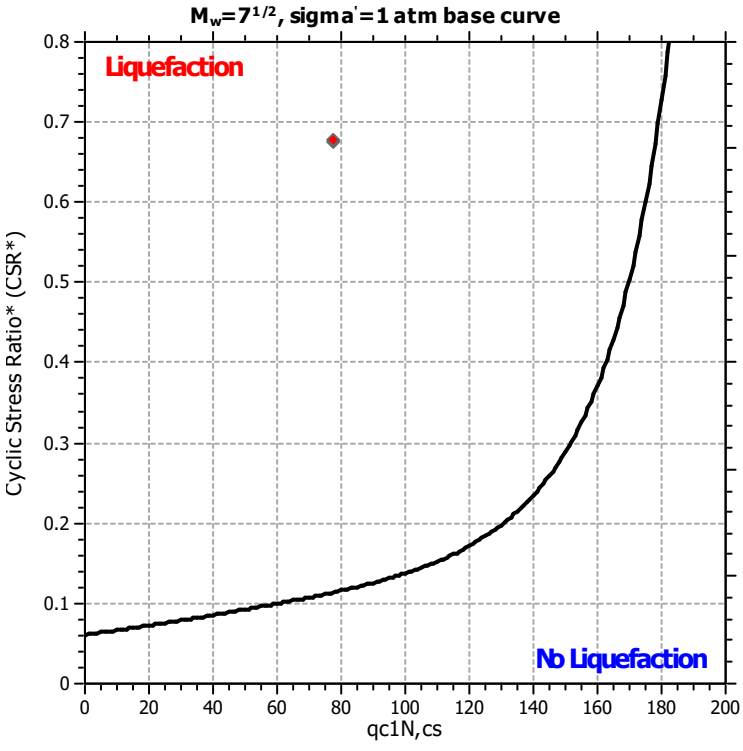
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

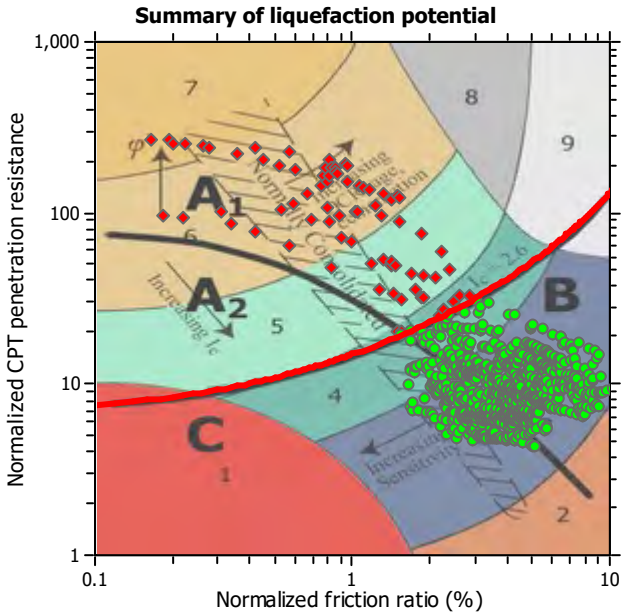
CPT file : GS26-CPT-01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.59 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	0.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

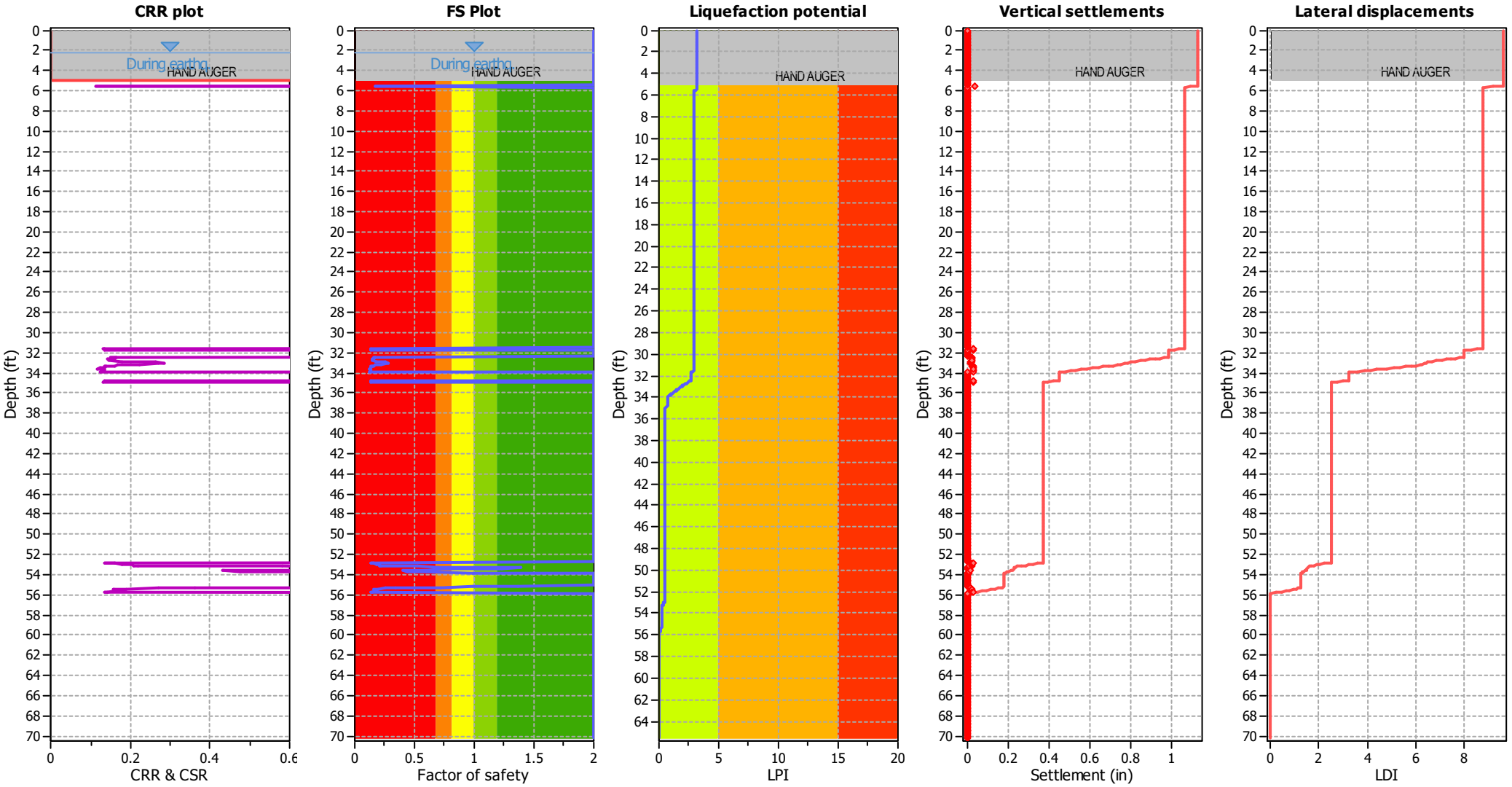






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

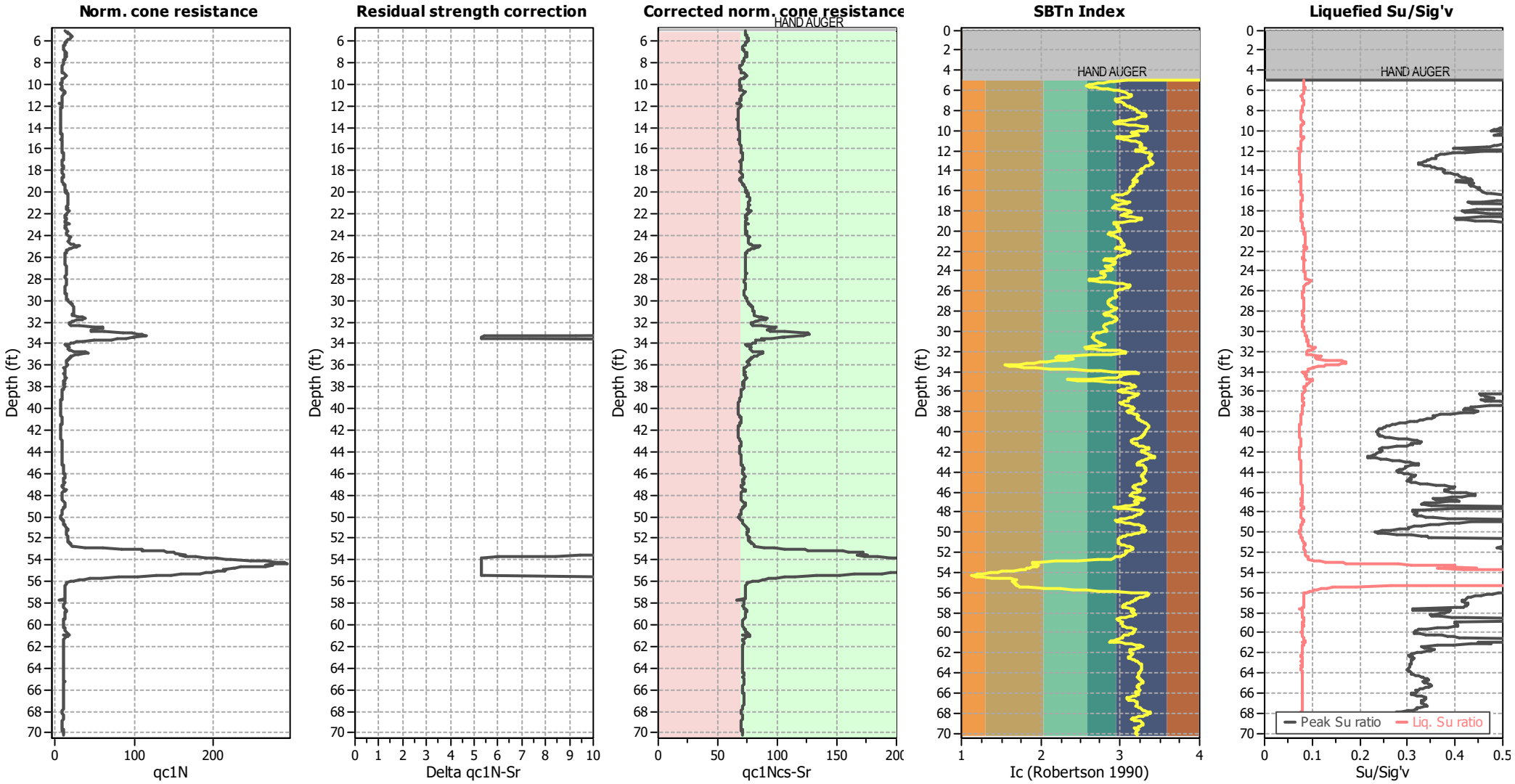
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

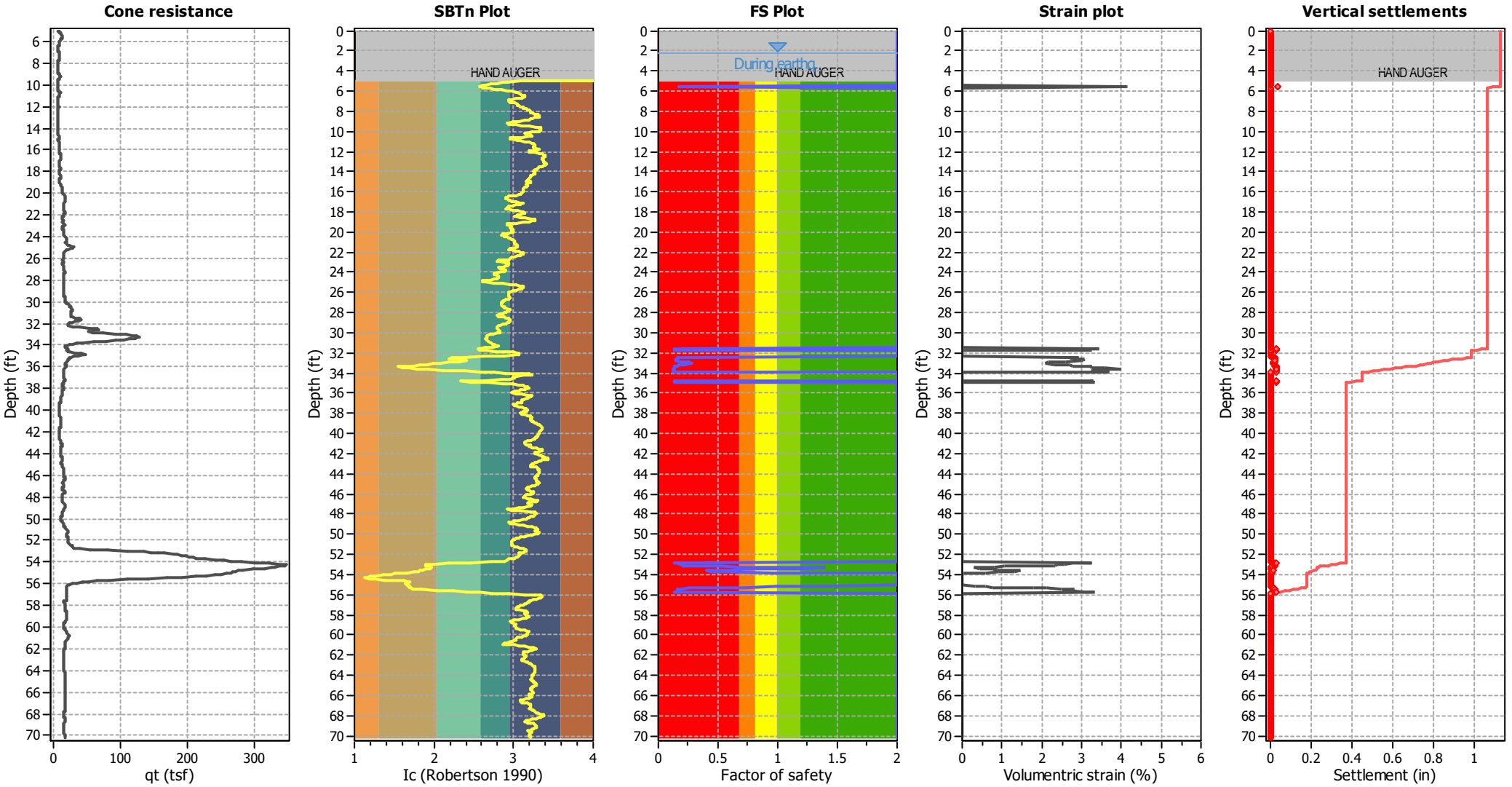
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

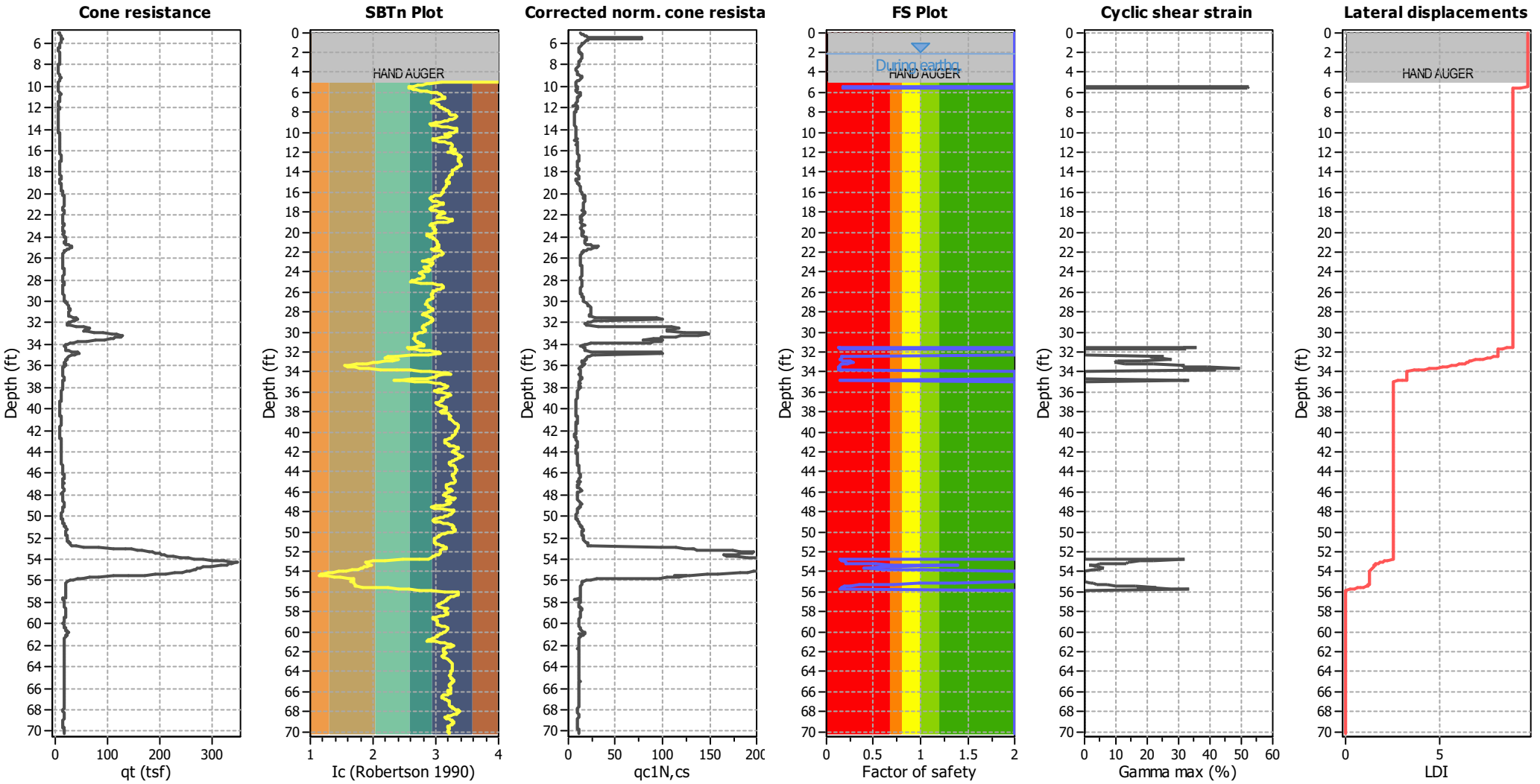
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

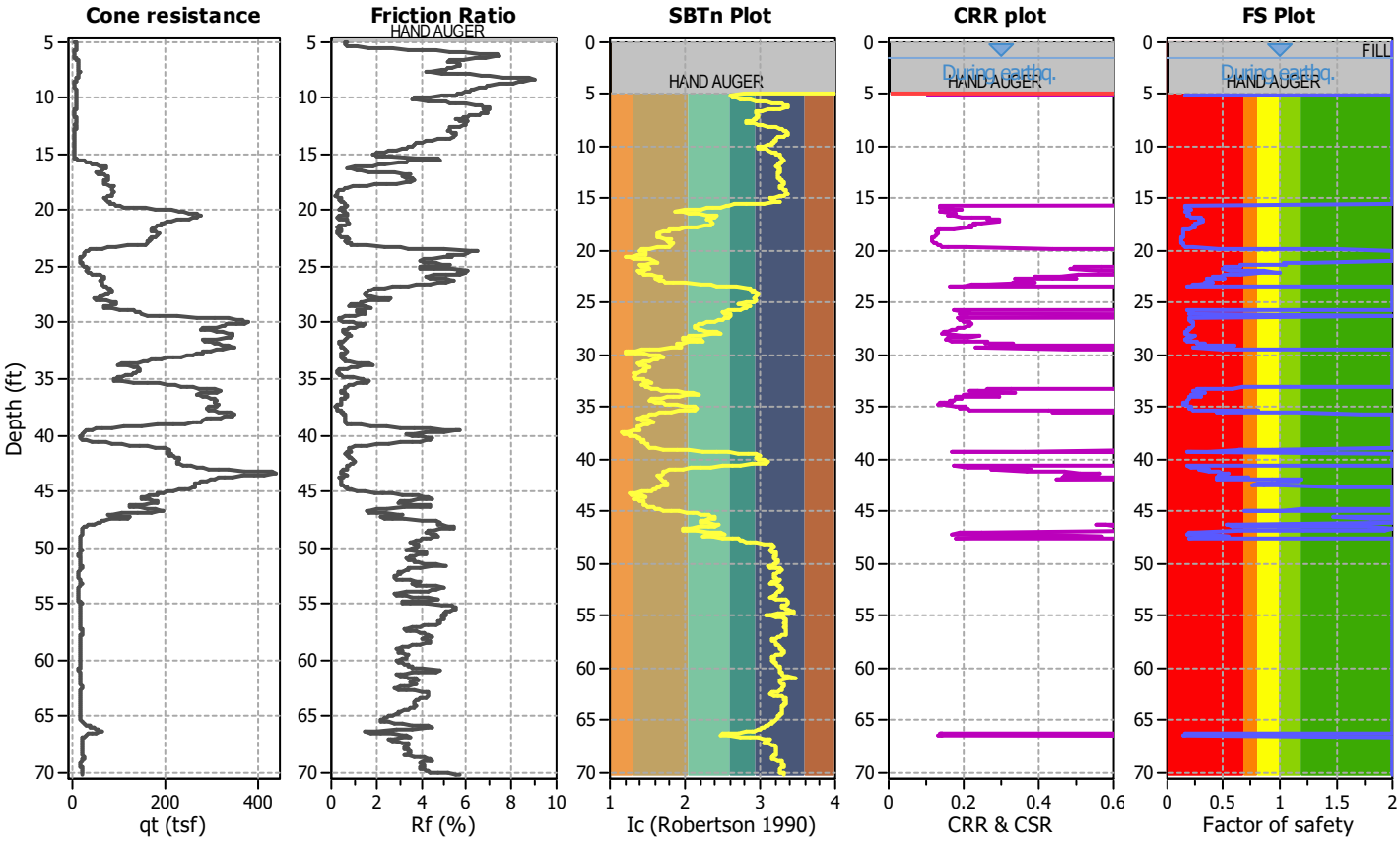
LIQUEFACTION ANALYSIS REPORT

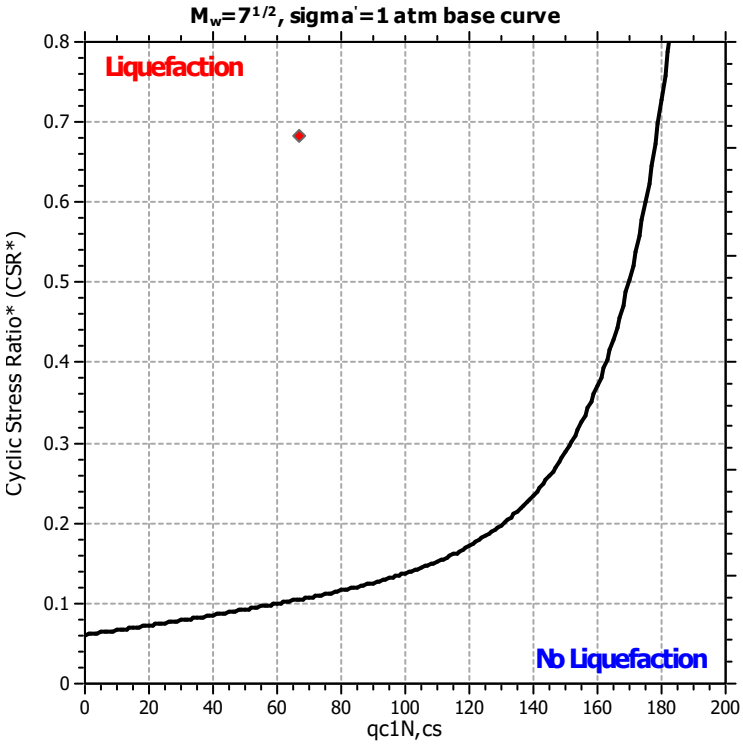
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

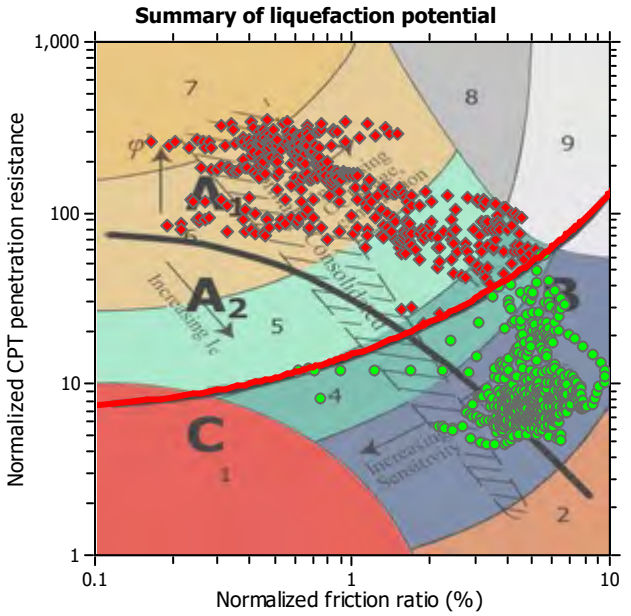
CPT file : GS26-CPT-02

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	11.71 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	0.70 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

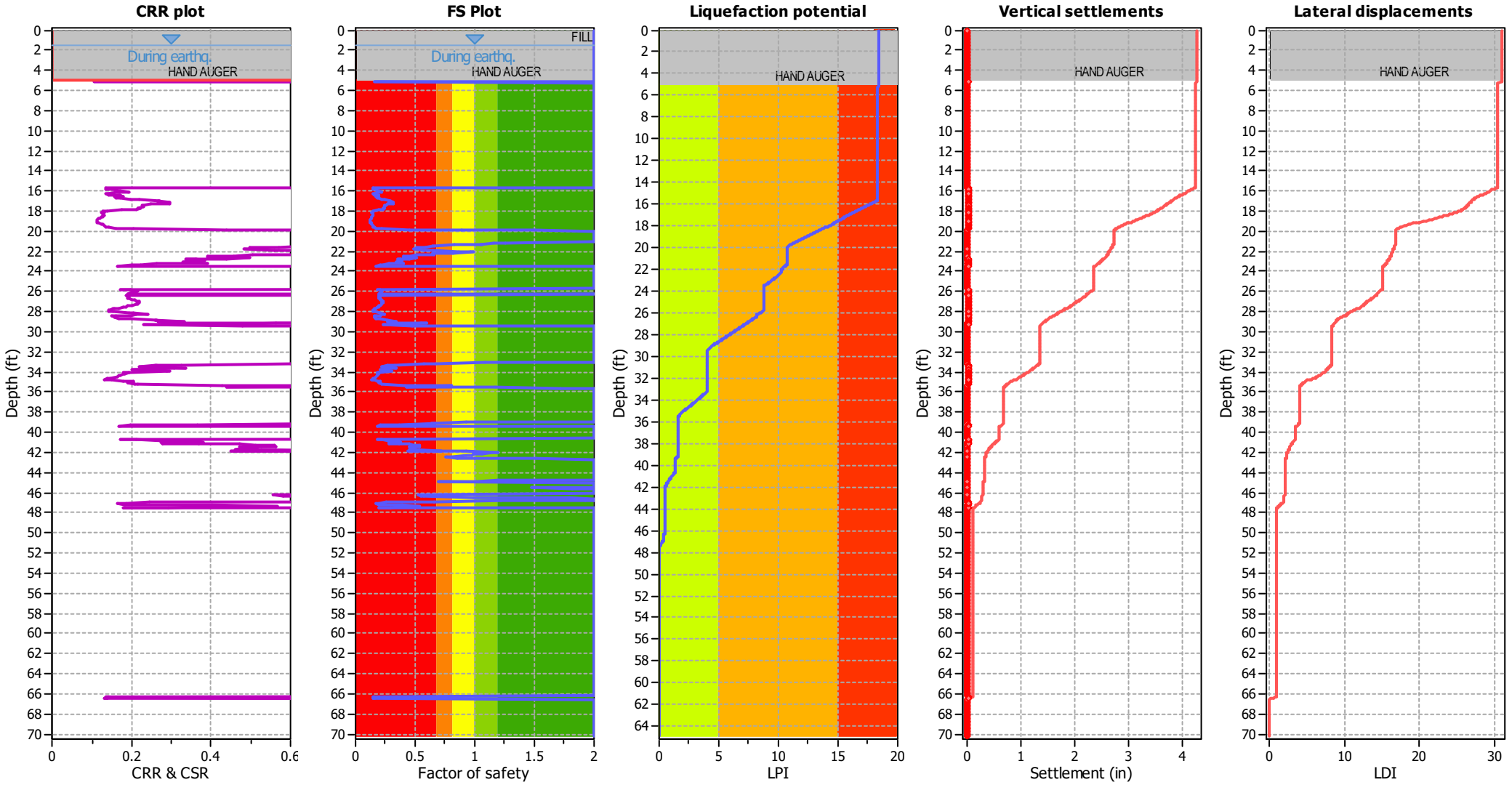






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	11.71 ft	Fill height:	0.70 ft	Limit depth:	N/A

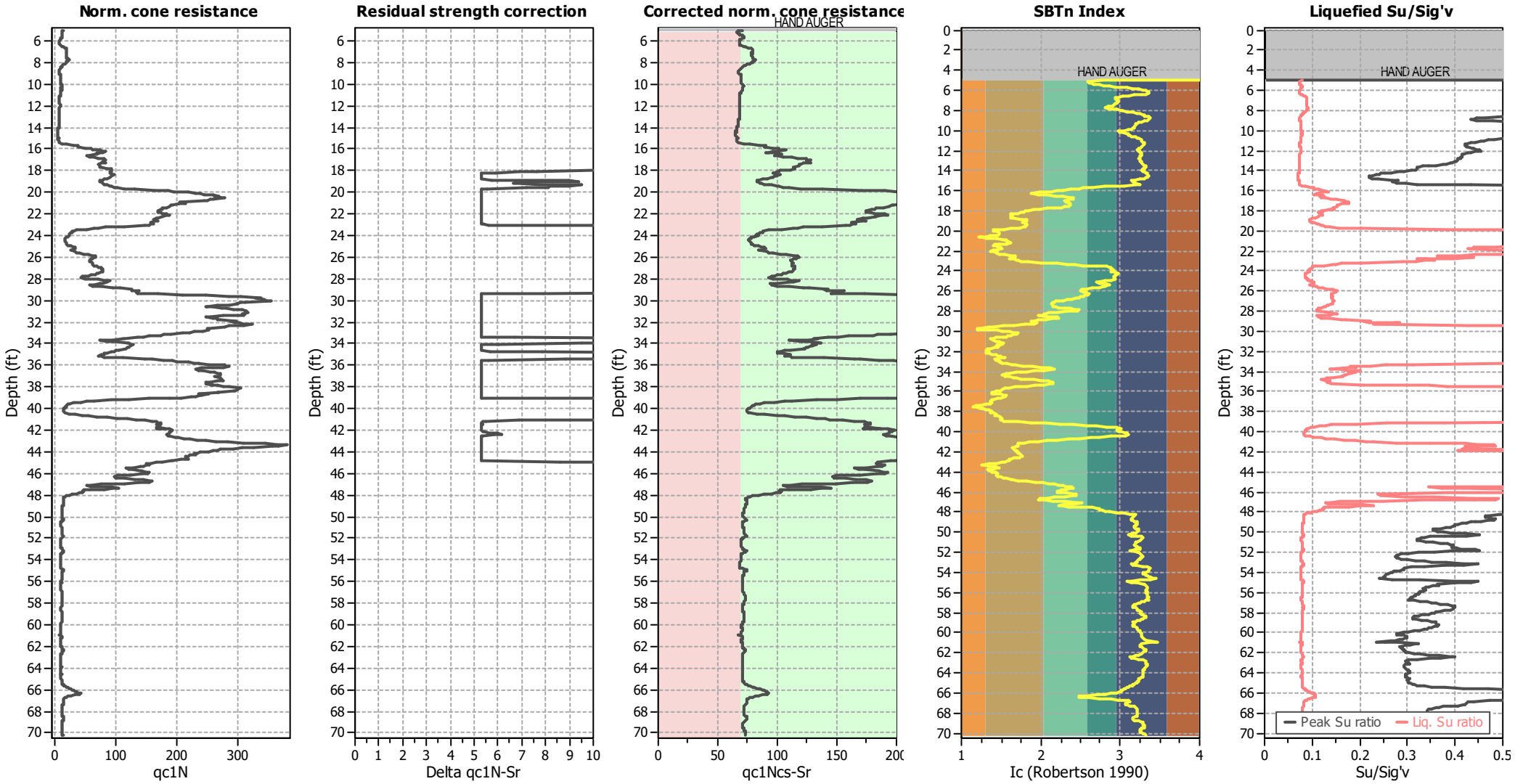
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

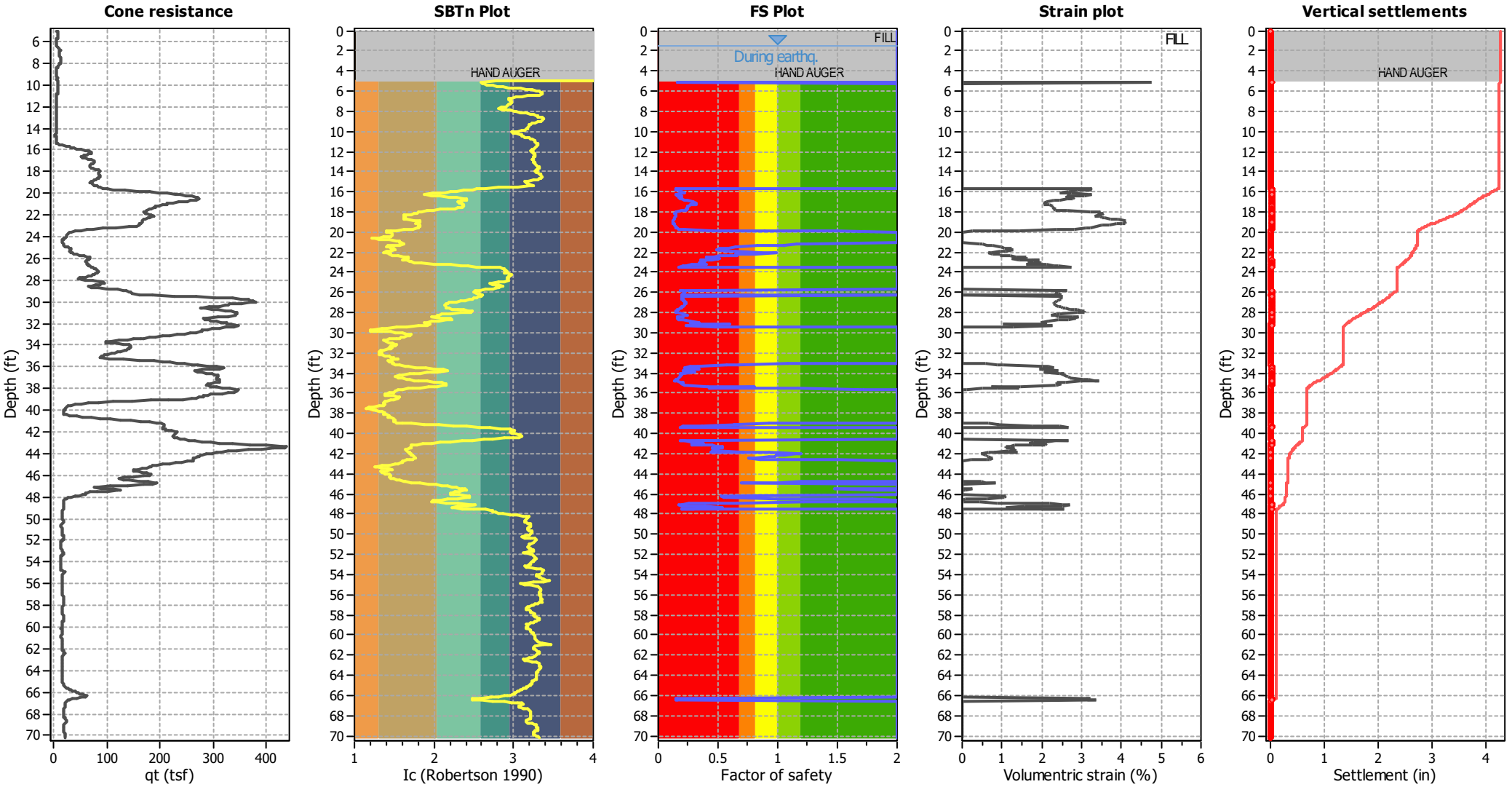
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	11.71 ft	Fill height:	0.70 ft	Limit depth:	N/A

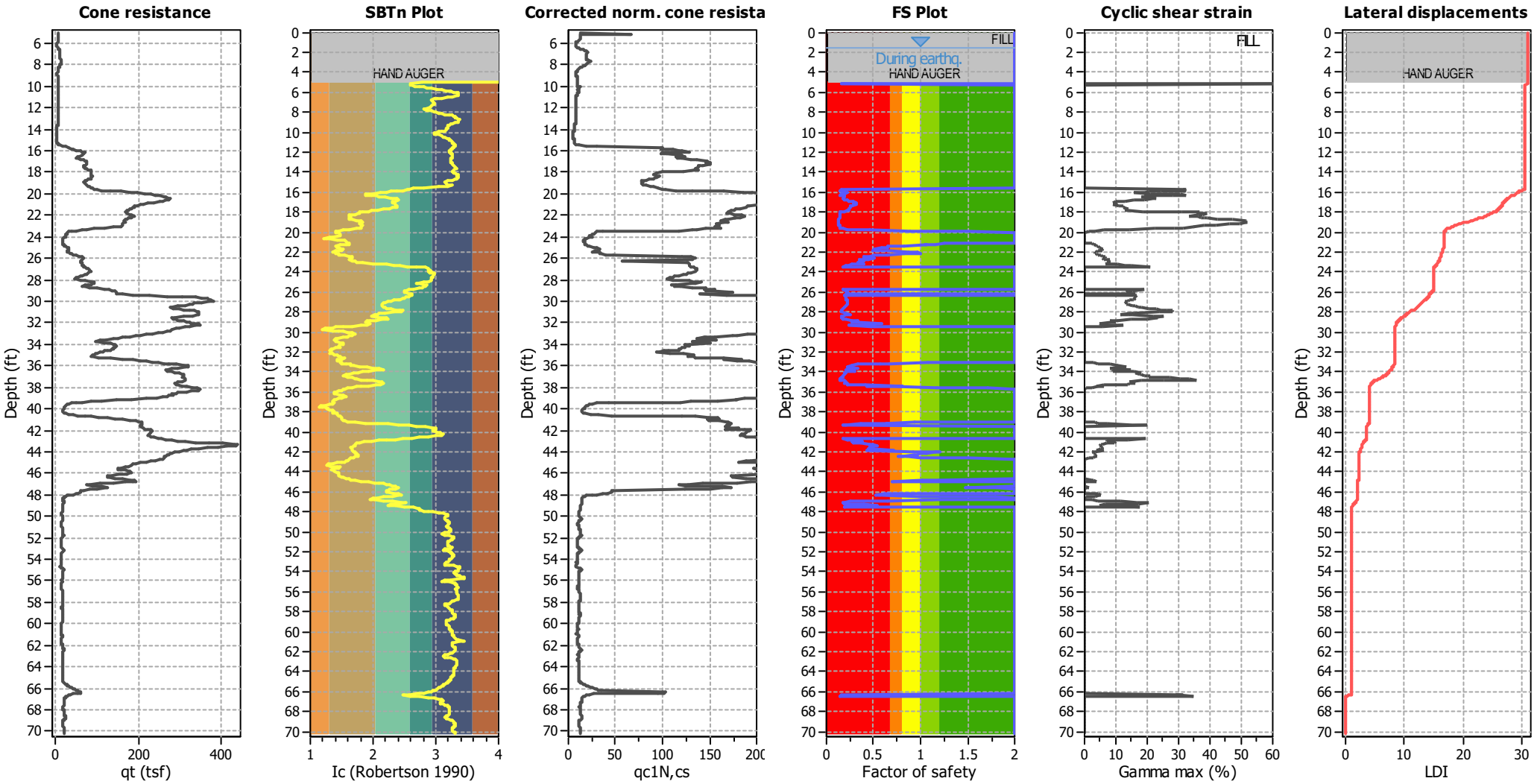
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

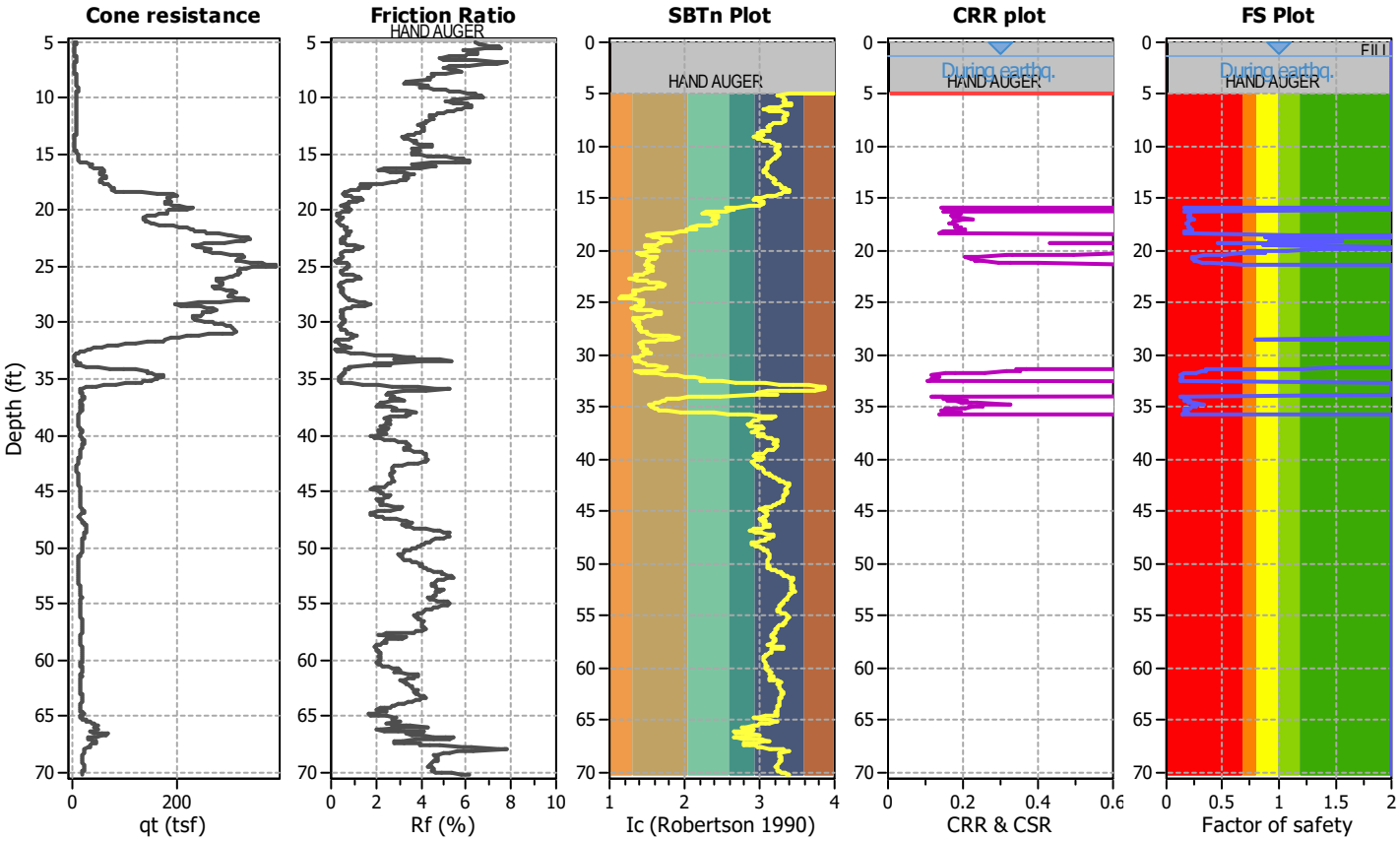
LIQUEFACTION ANALYSIS REPORT

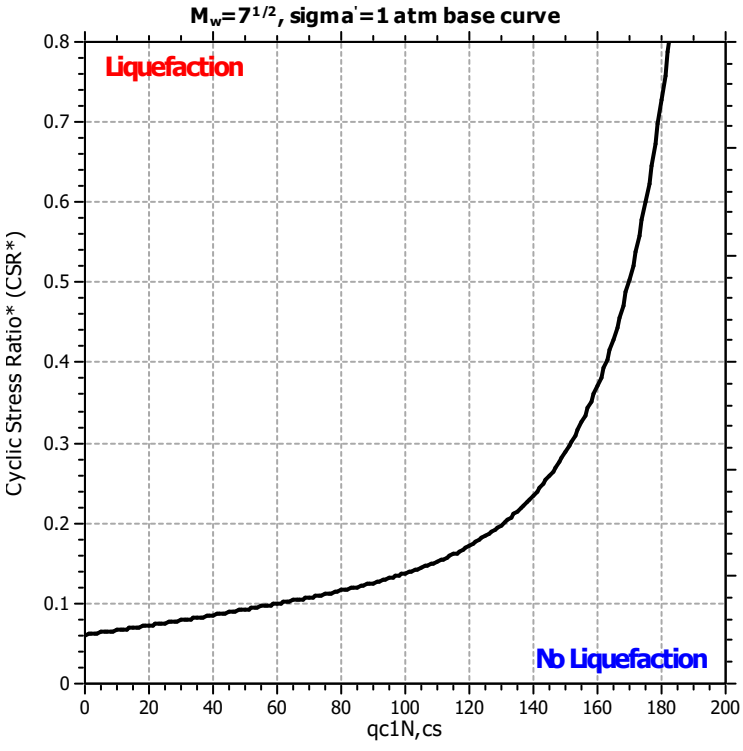
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

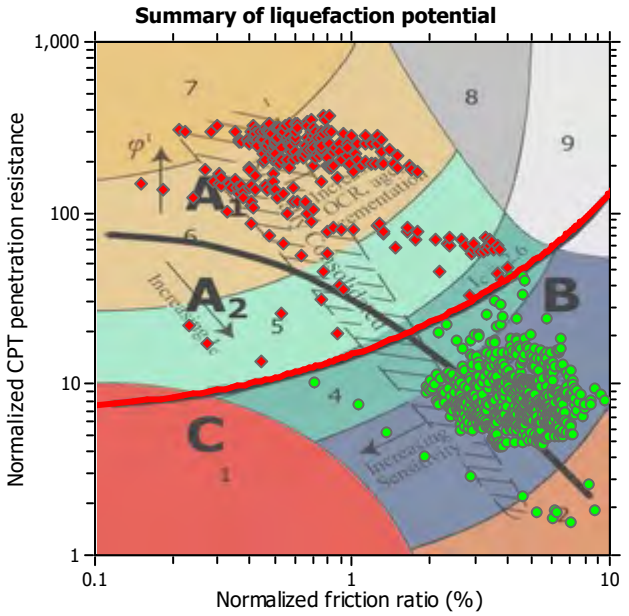
CPT file : GS26-CPT-03

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	12.68 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	0.80 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

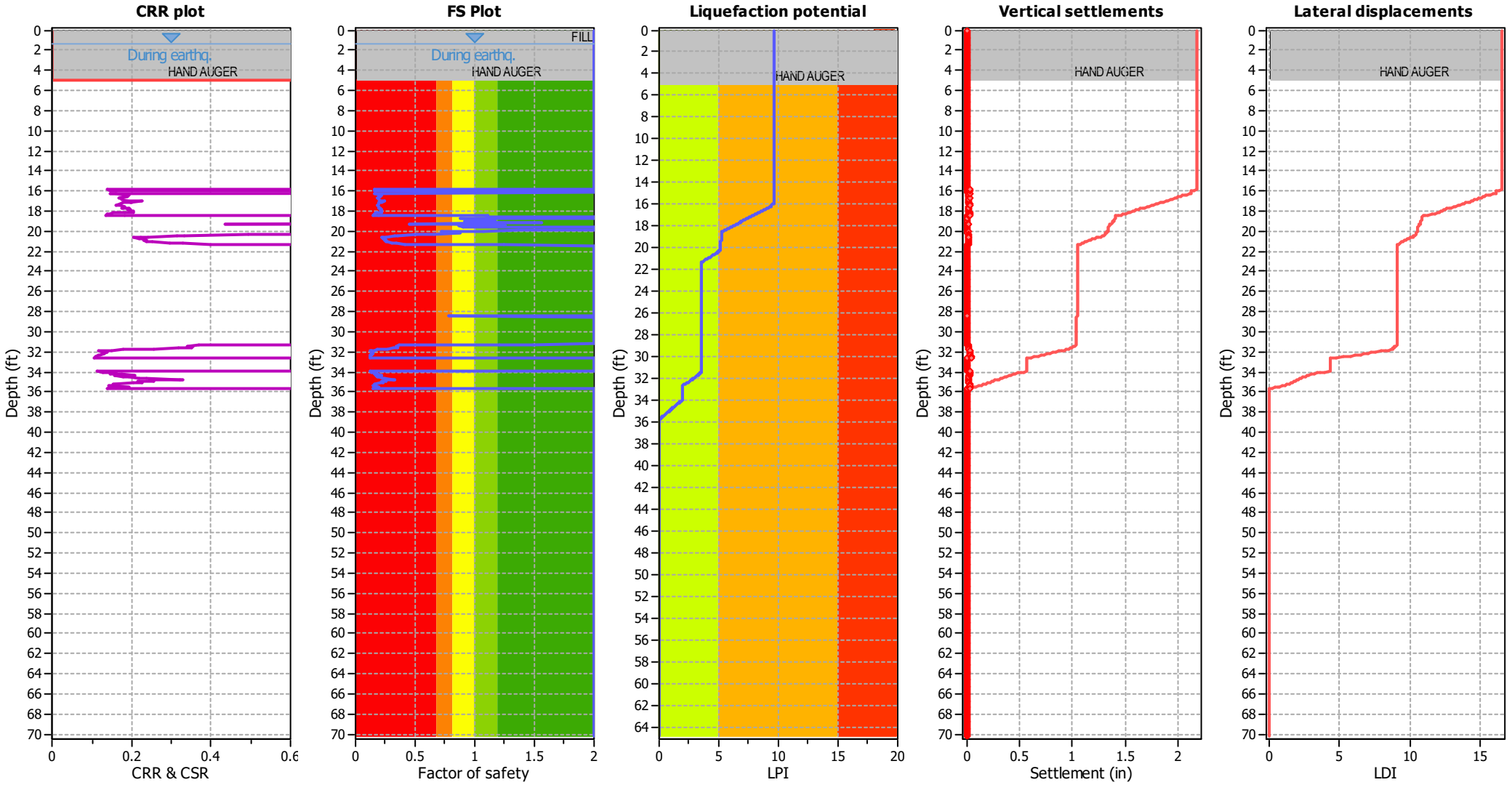






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	12.68 ft	Fill height:	0.80 ft	Limit depth:	N/A

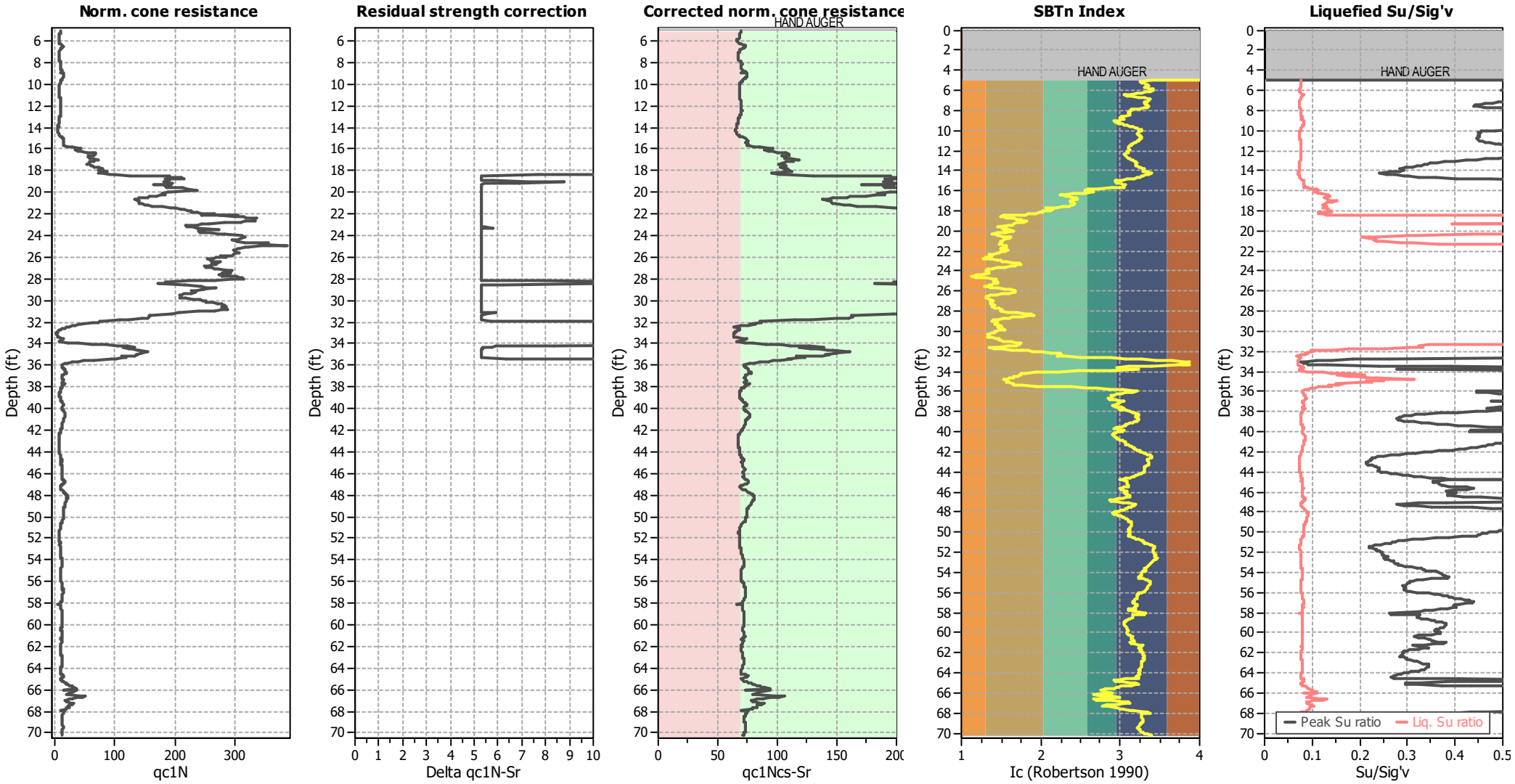
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

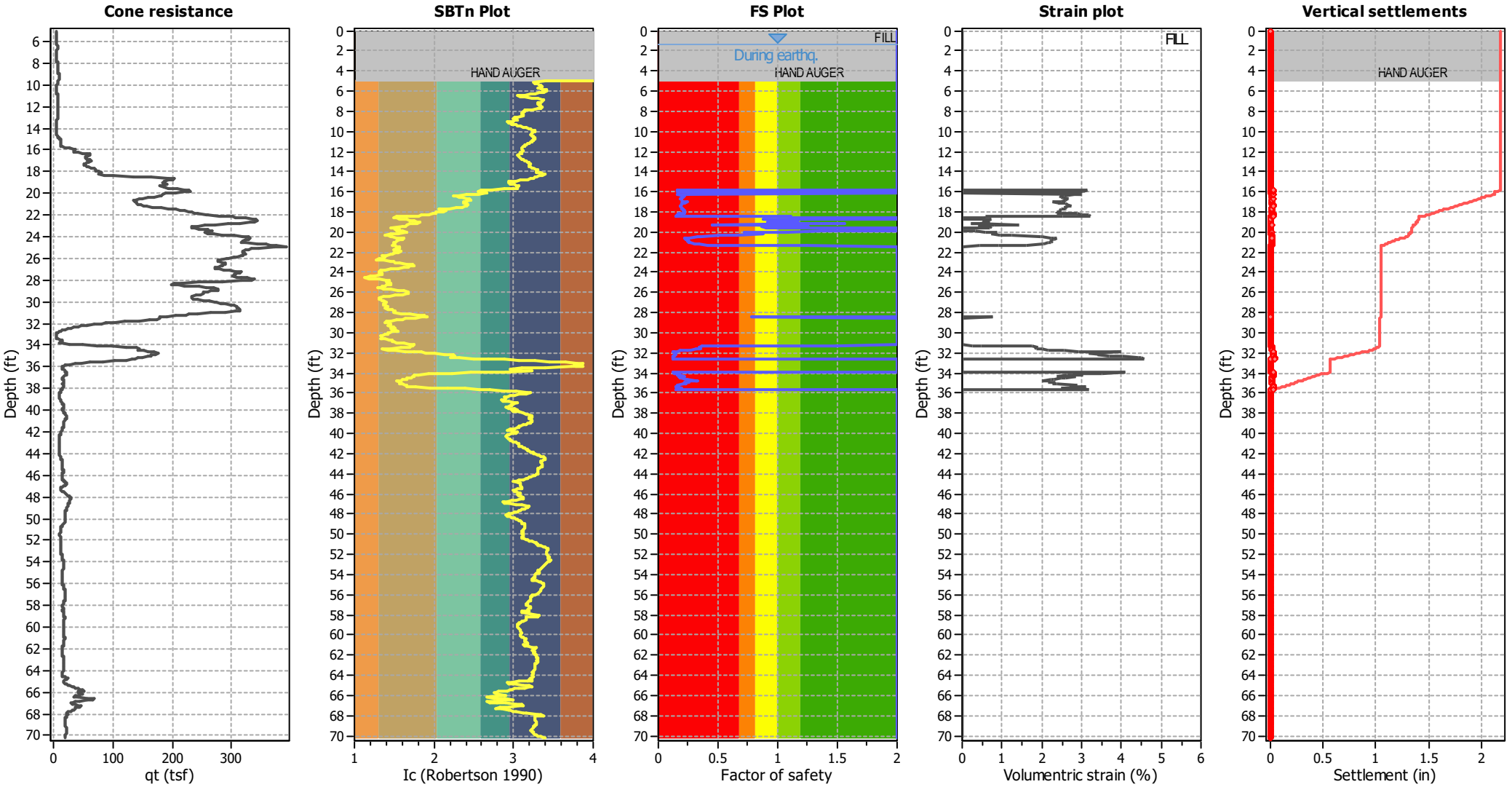
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	12.68 ft	Fill height:	0.80 ft	Limit depth:	N/A

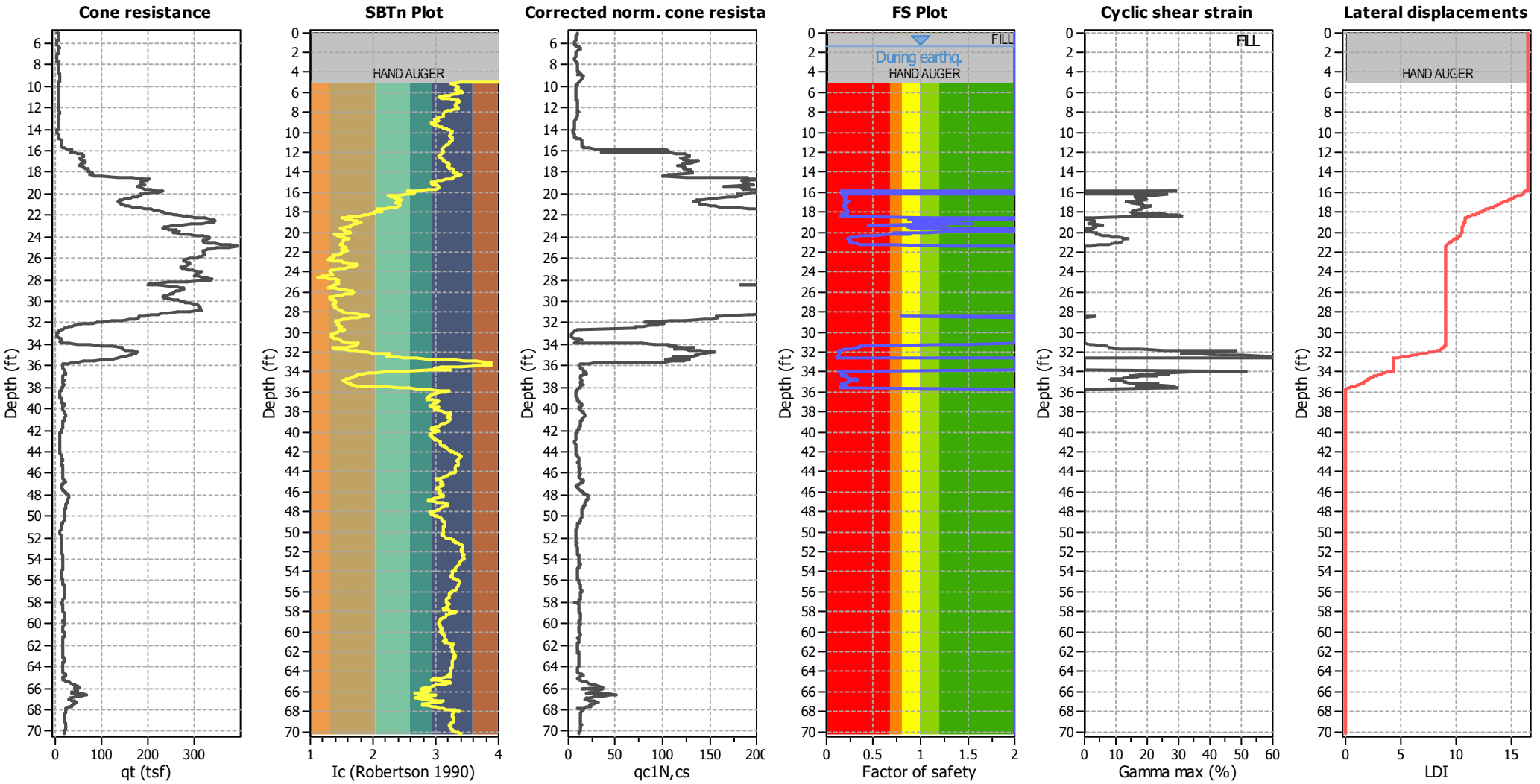
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
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F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Geosyntec consultants
Geosyntec Consultants, Inc.
SF Bay Area Branch

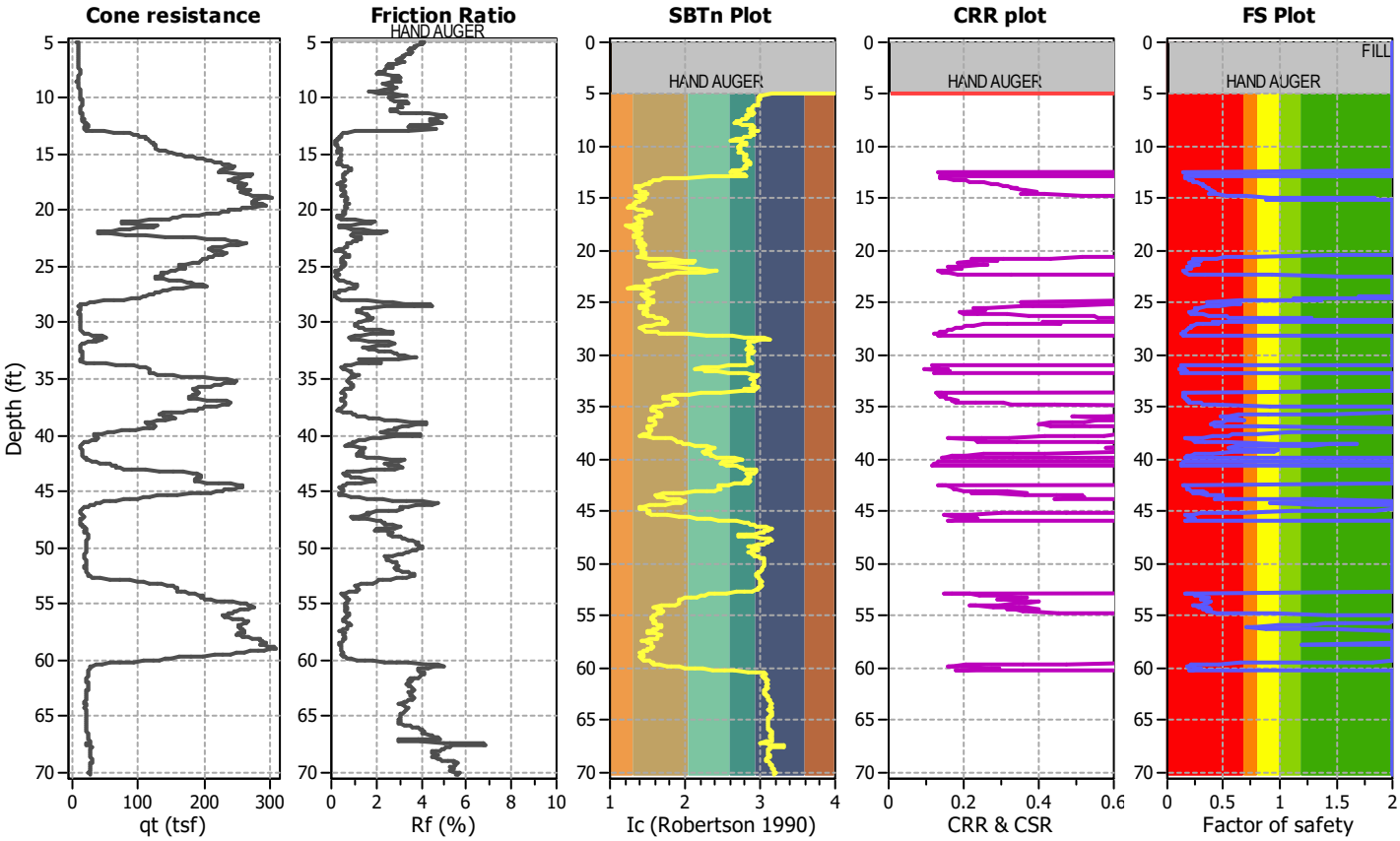
LIQUEFACTION ANALYSIS REPORT

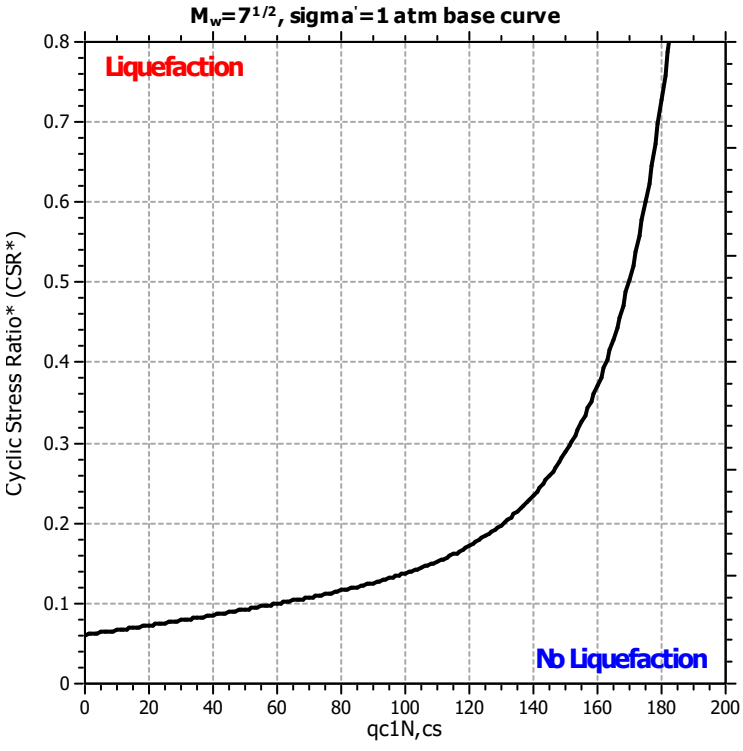
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

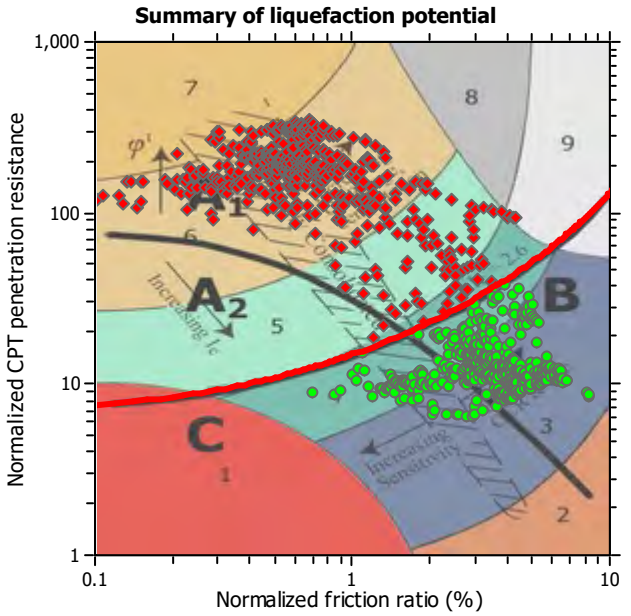
CPT file : GS26-CPT-04

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.23 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	8.10 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

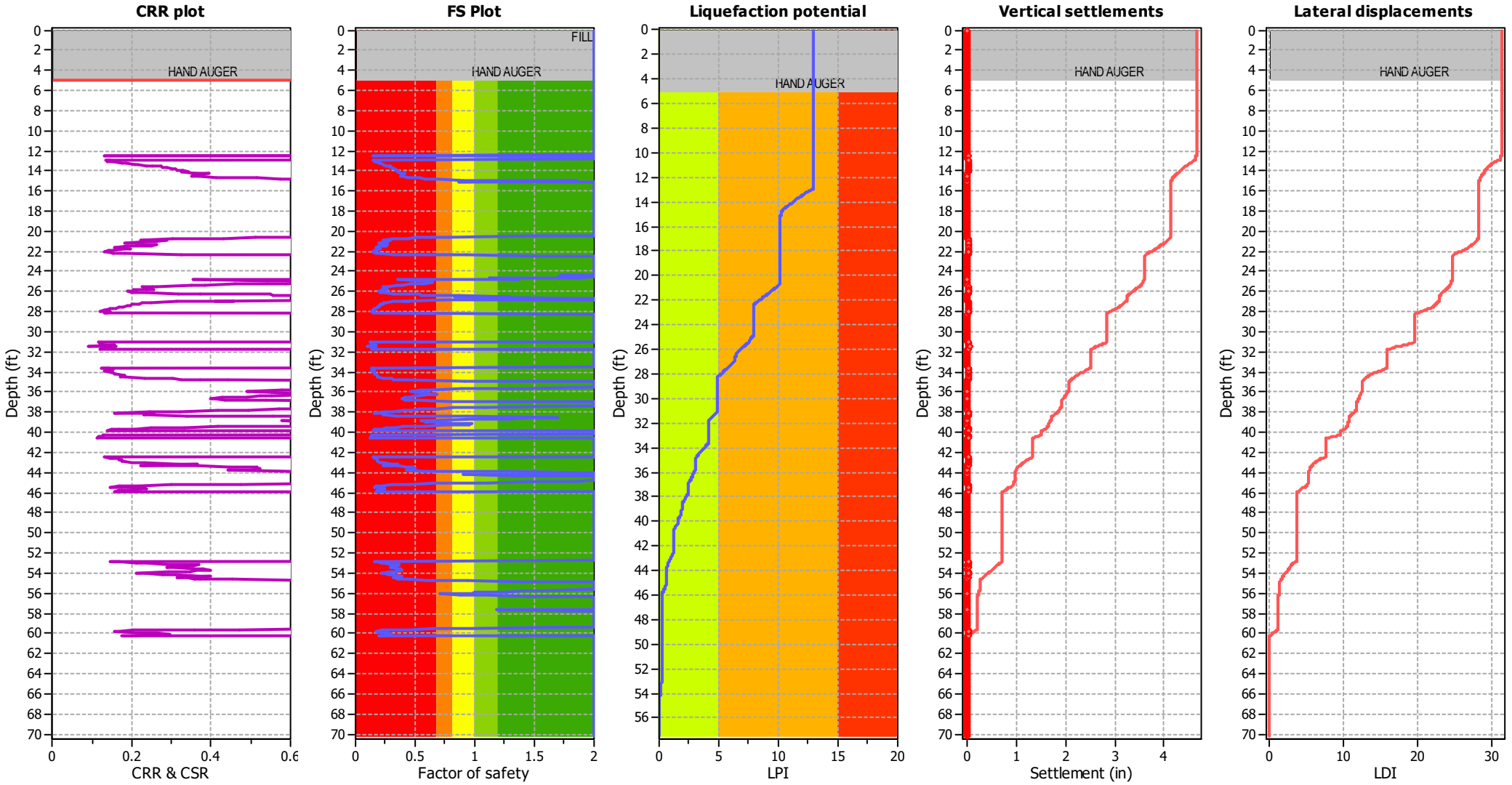






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.23 ft	Fill height:	8.10 ft	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
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LPI color scheme

- Very high risk
- High risk
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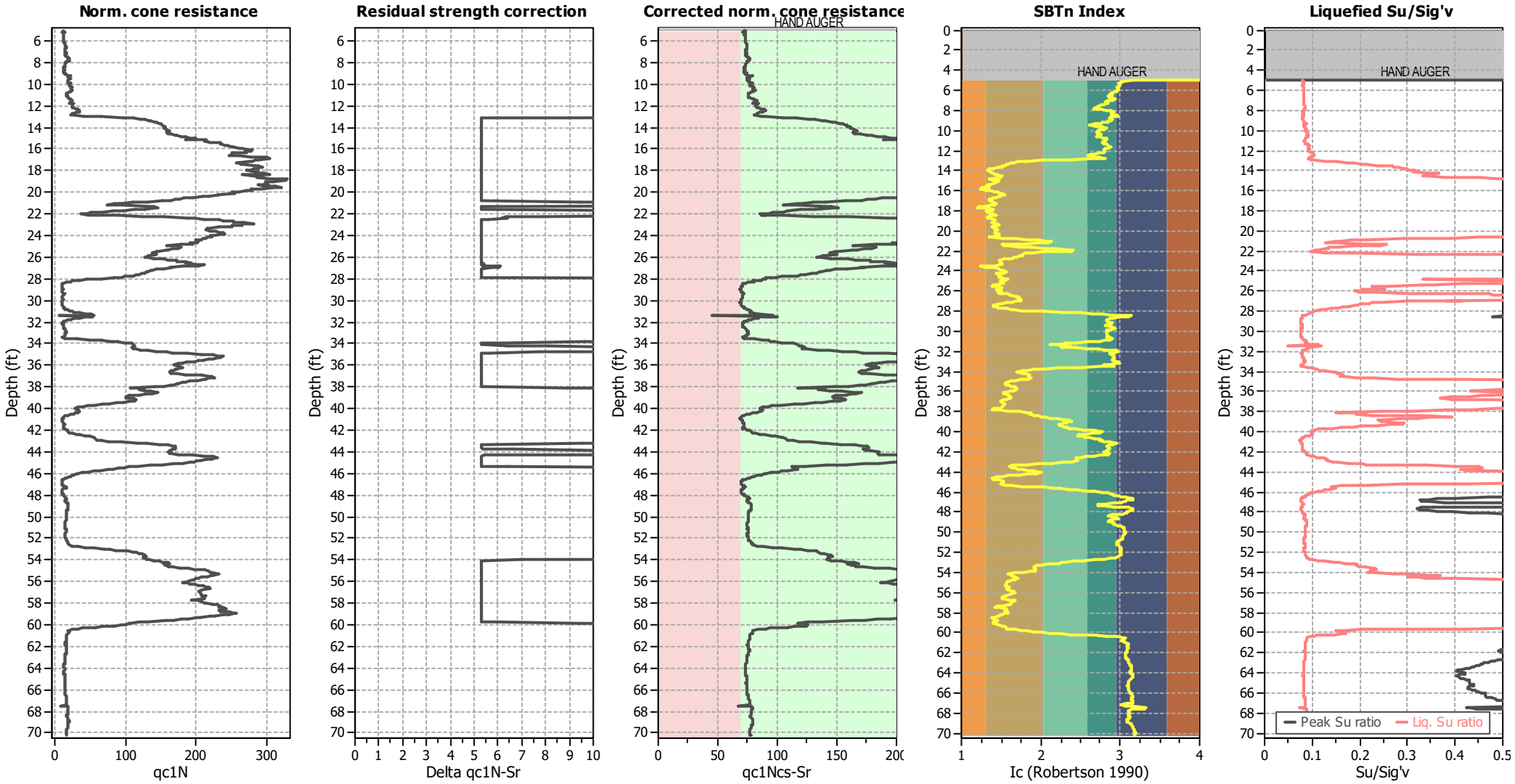
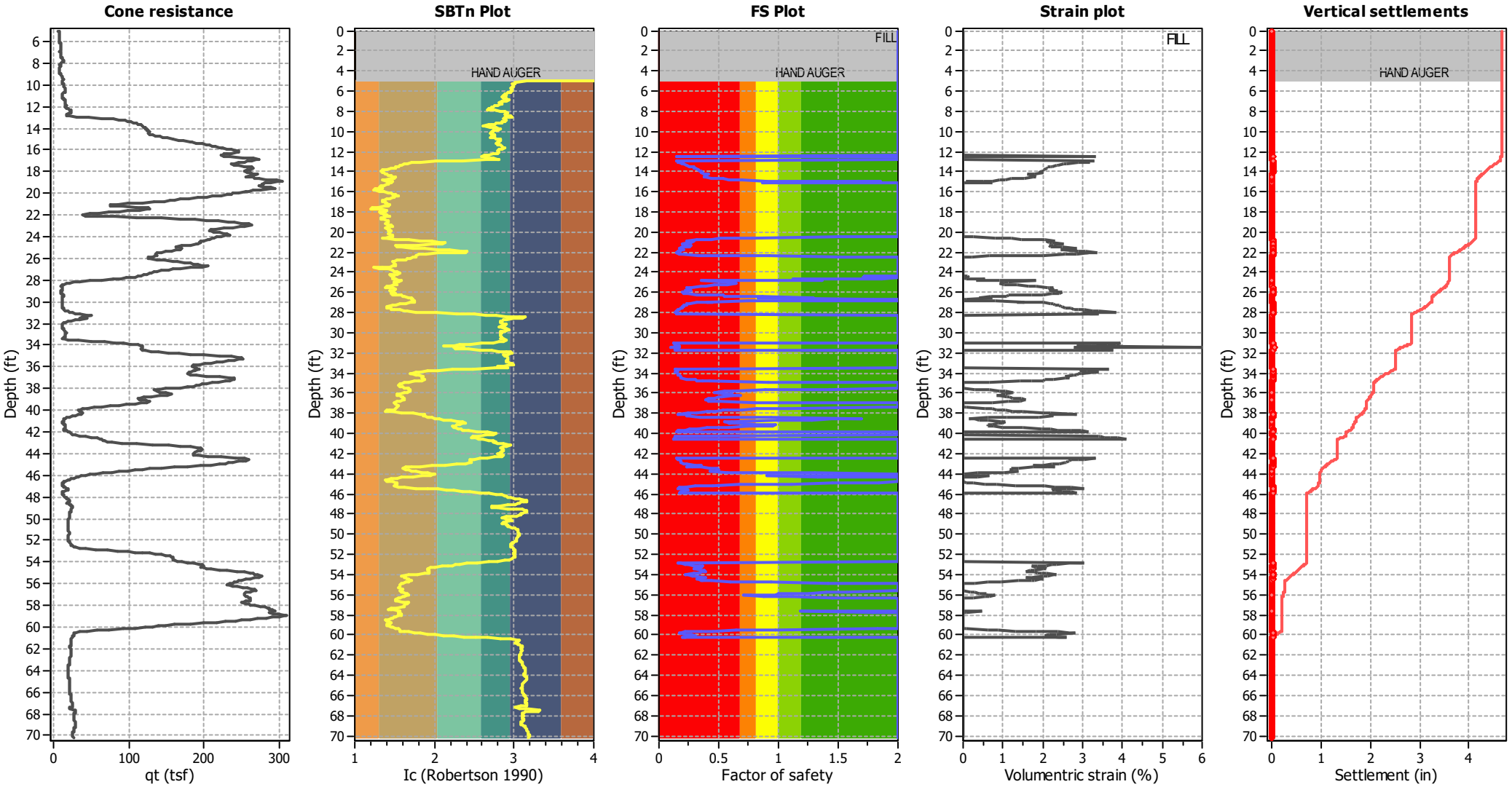
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.23 ft	Fill height:	8.10 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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I_c:	Soil Behaviour Type Index
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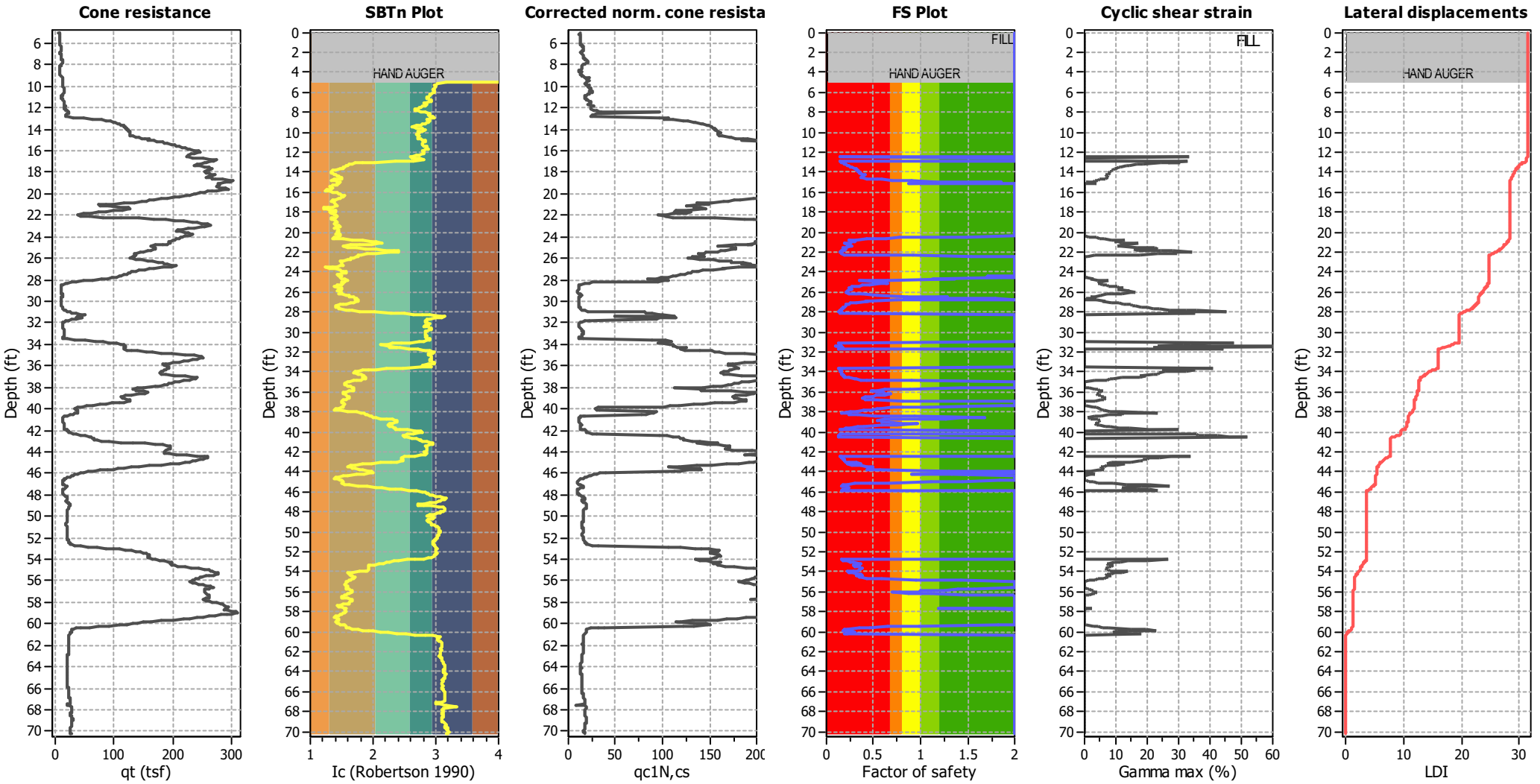
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

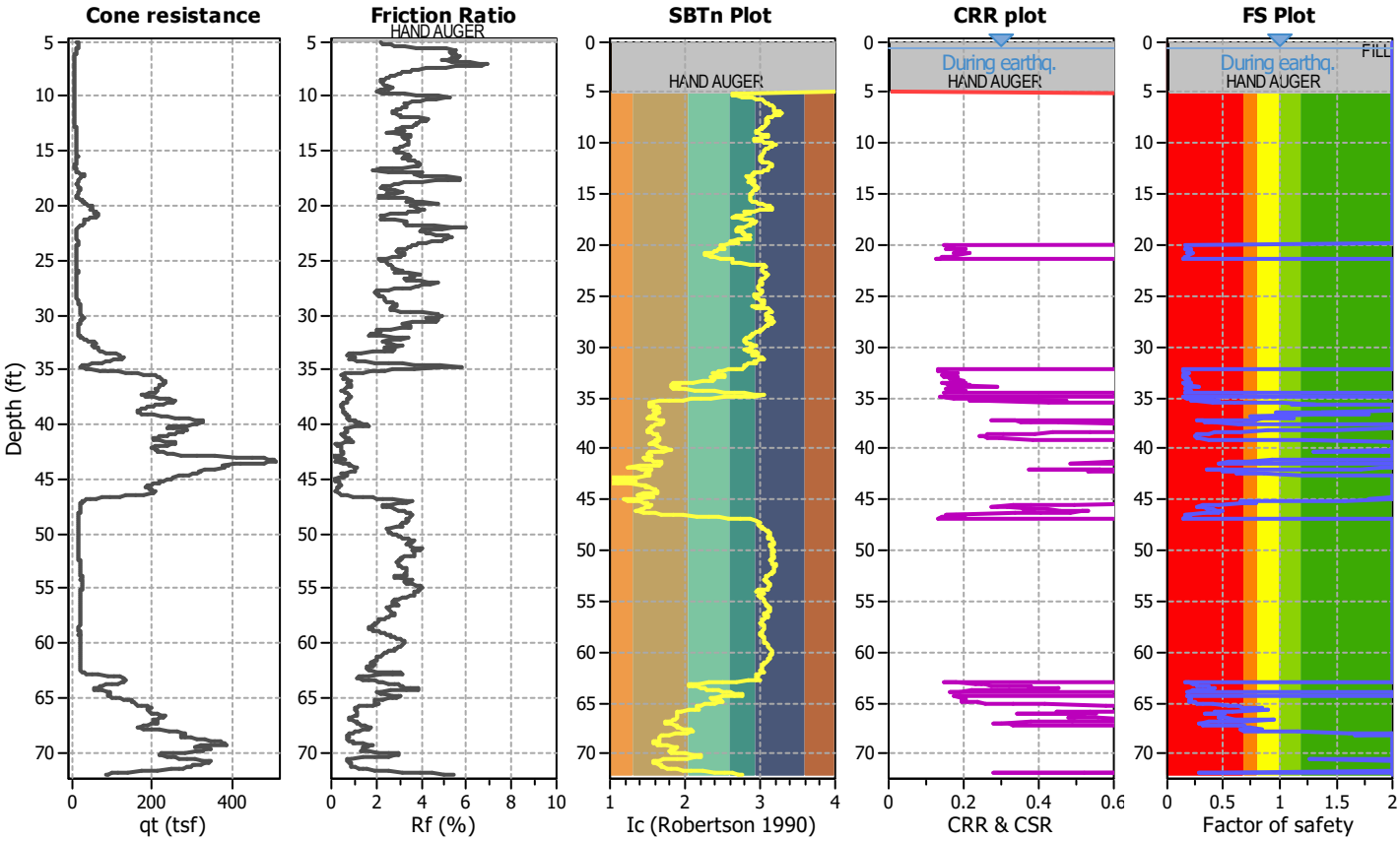
LIQUEFACTION ANALYSIS REPORT

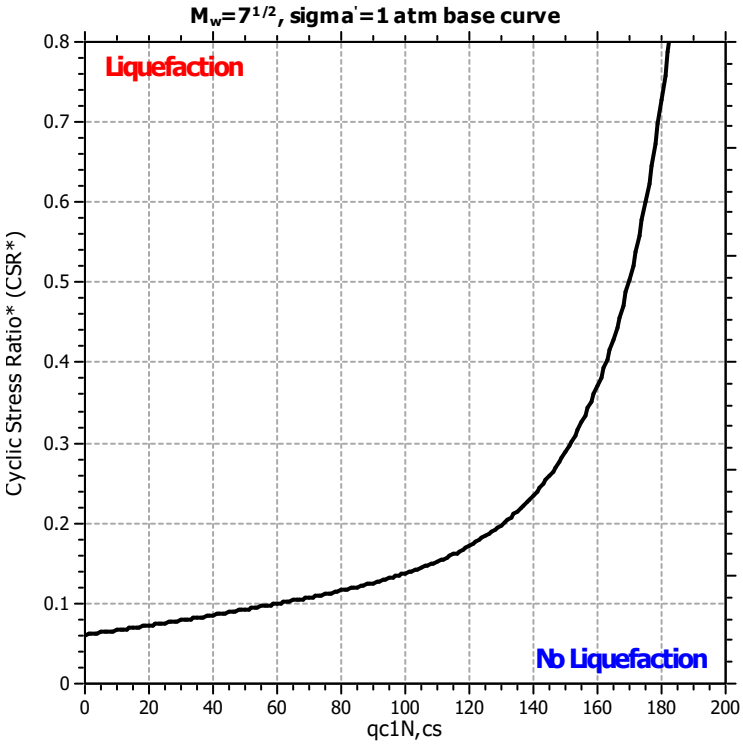
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

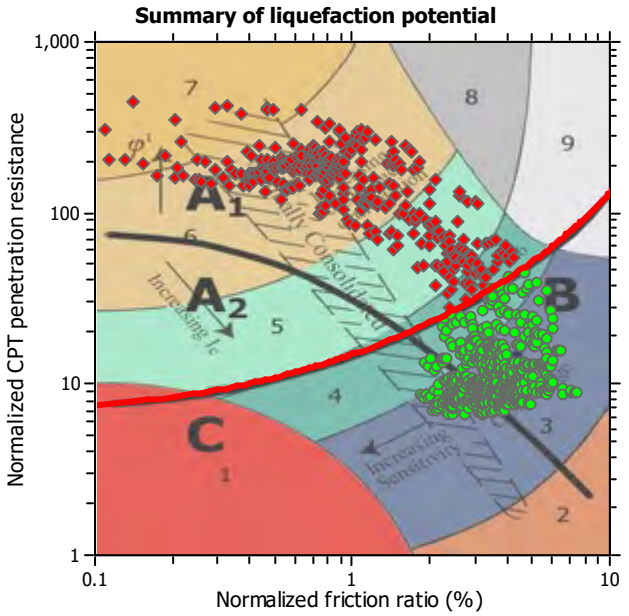
CPT file : GS26-CPT-05

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	9.67 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.60 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

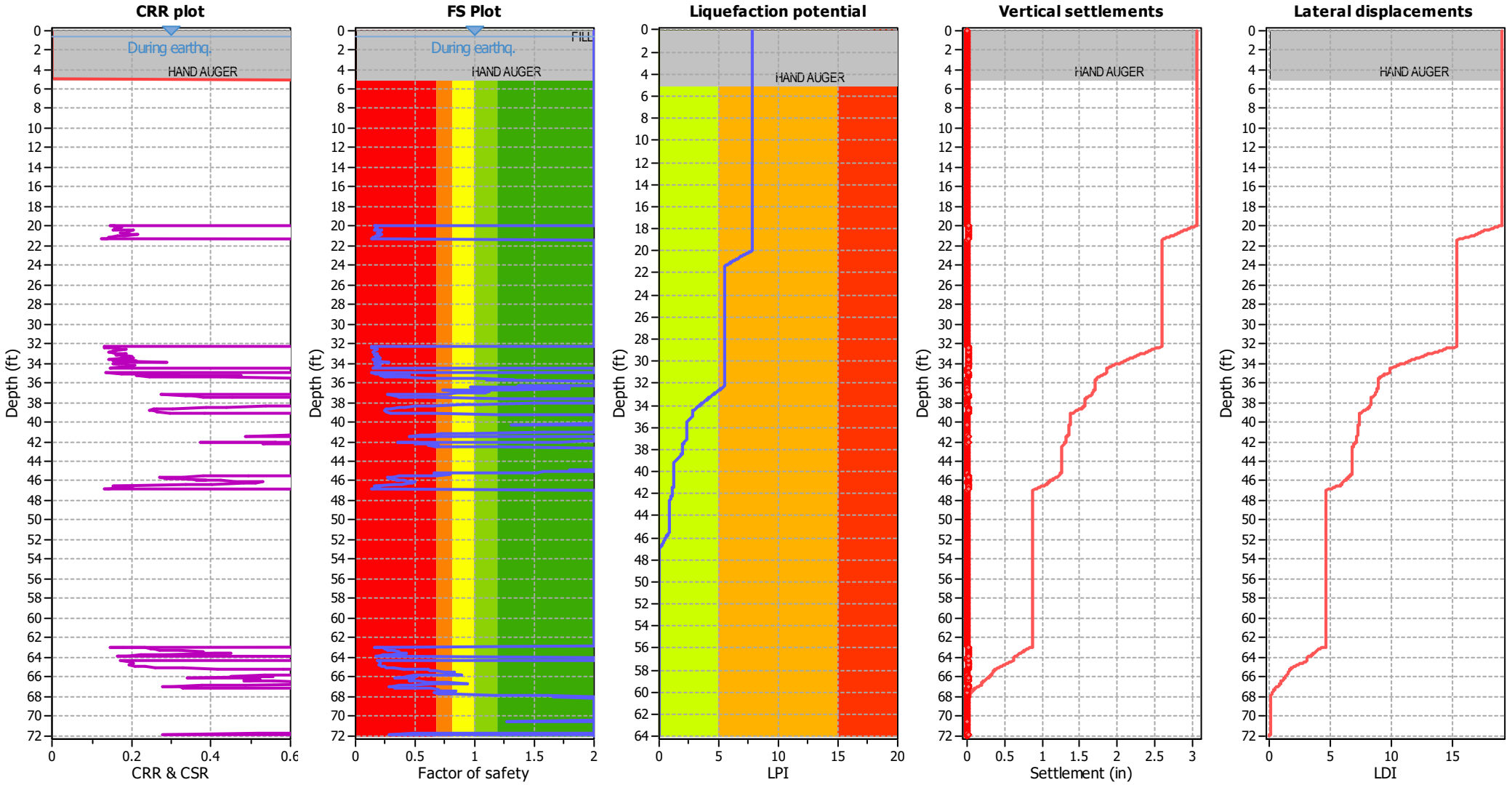






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	9.67 ft	Fill height:	1.60 ft	Limit depth:	N/A

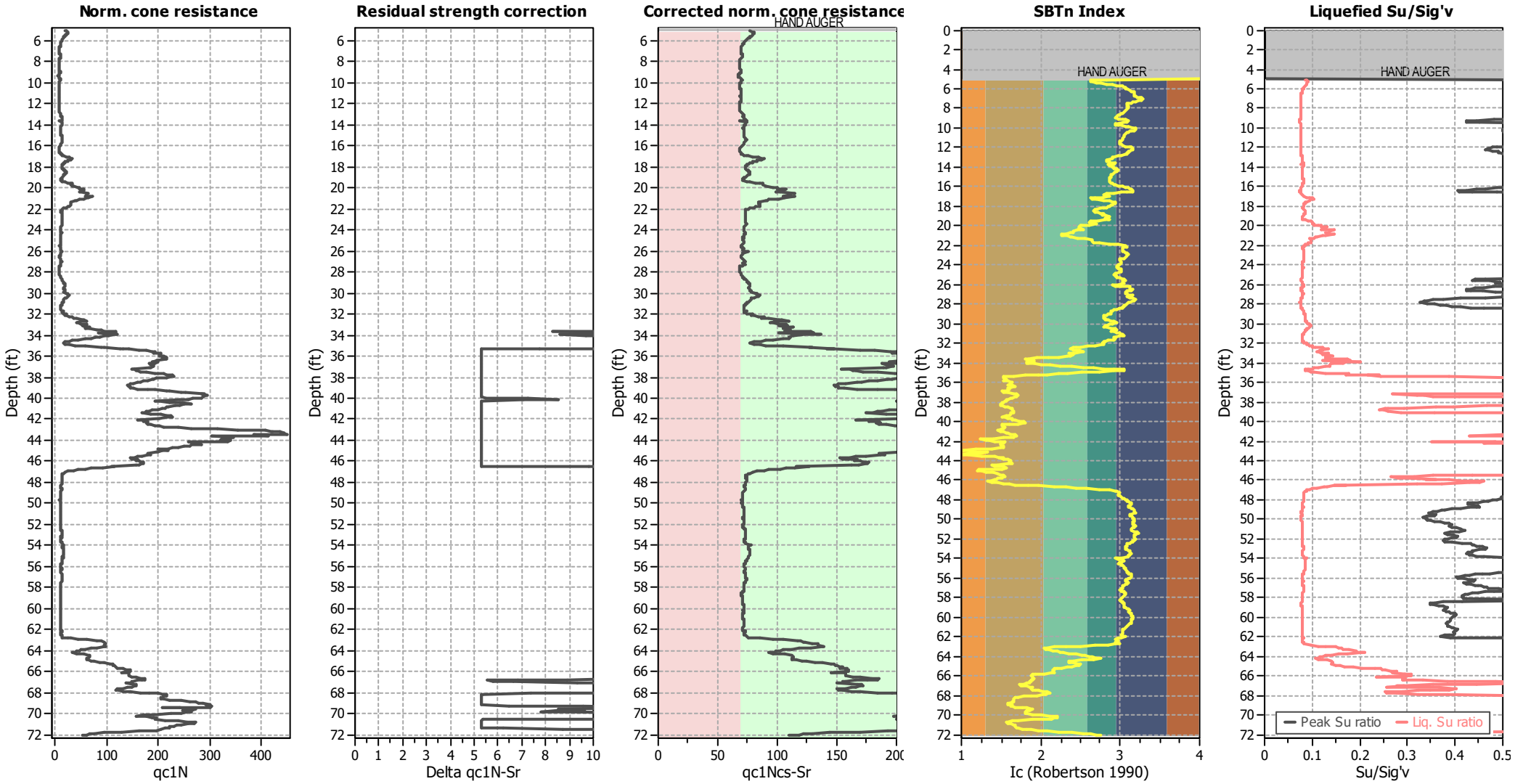
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

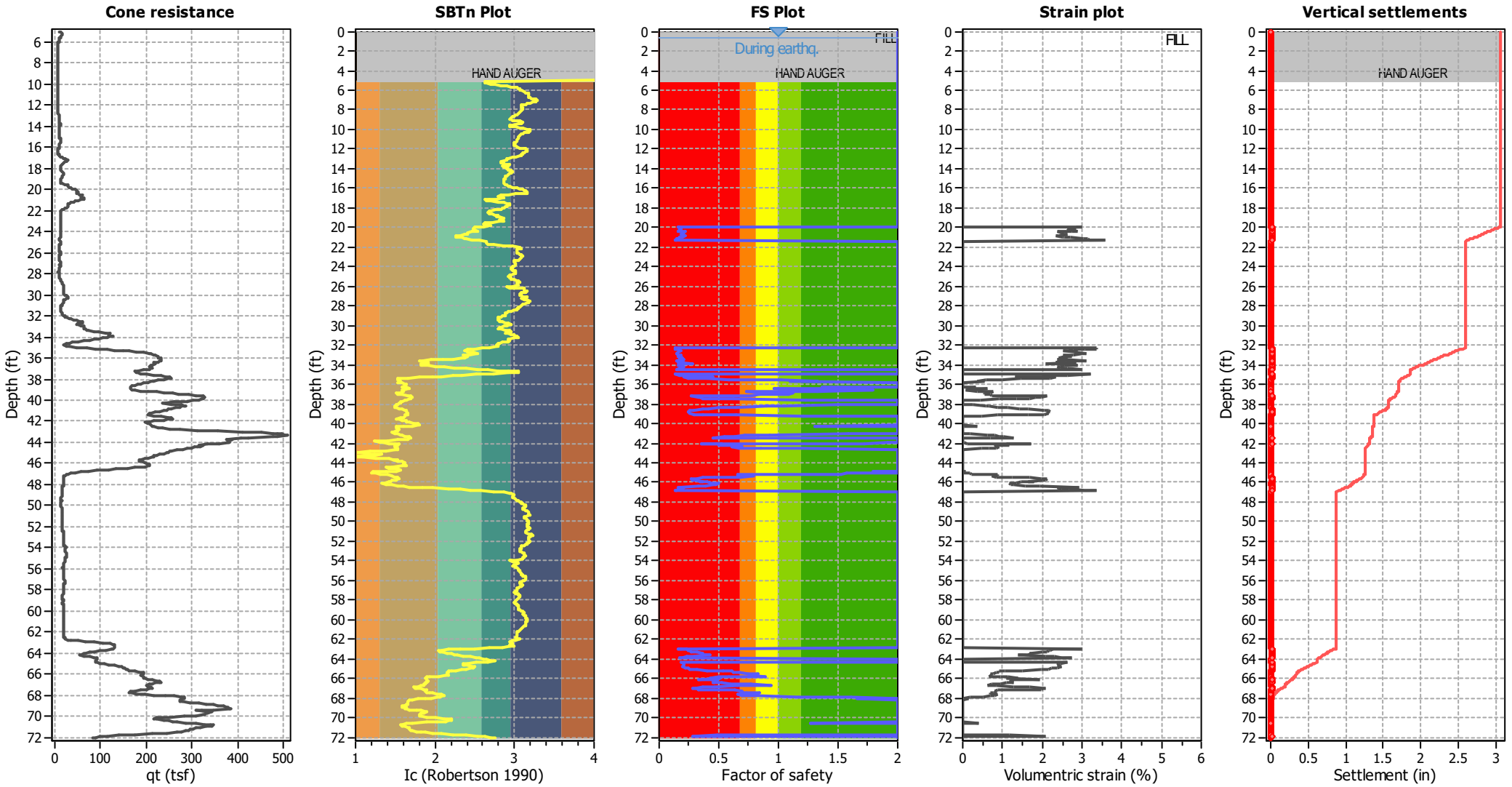
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	9.67 ft	Fill height:	1.60 ft	Limit depth:	N/A

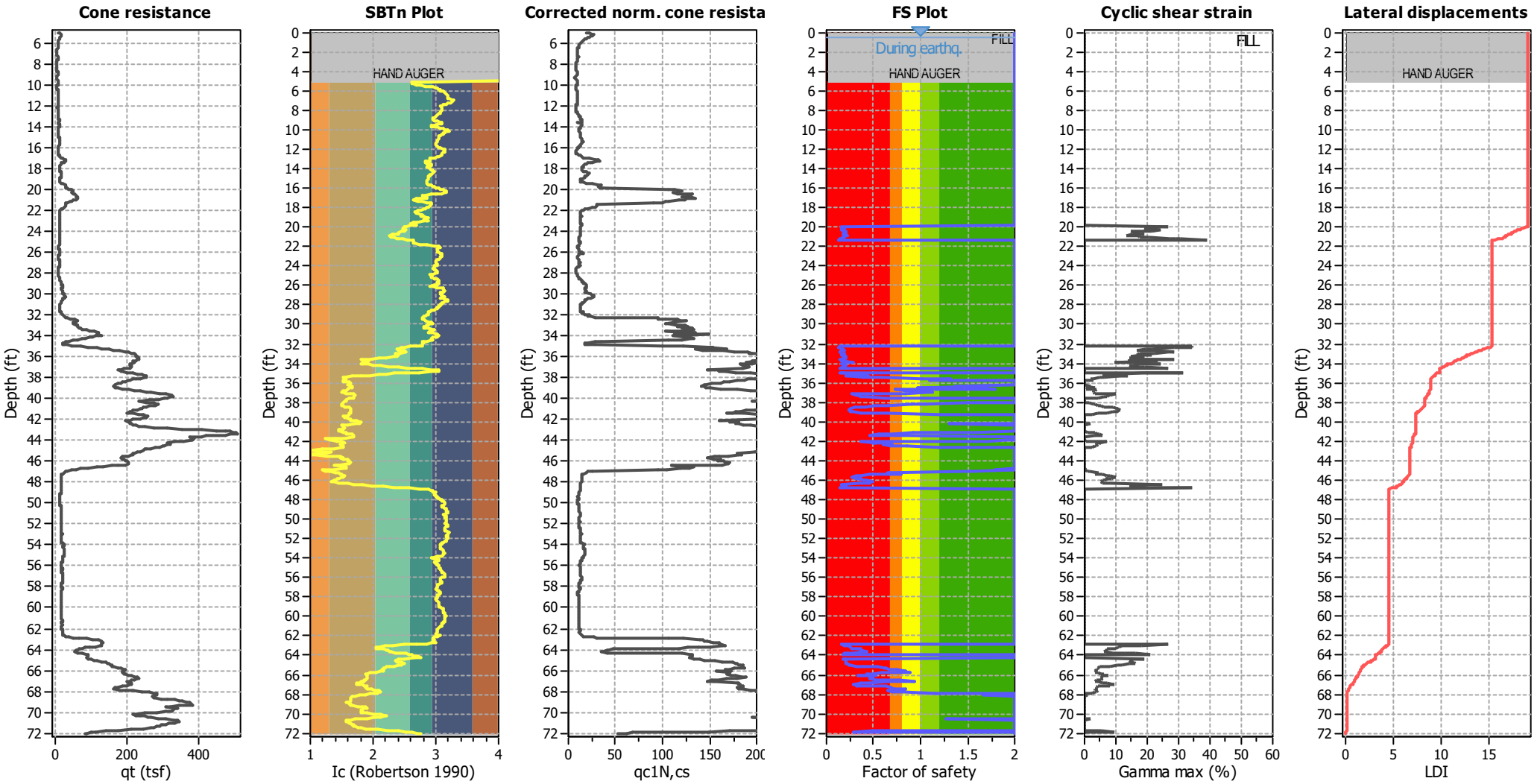
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

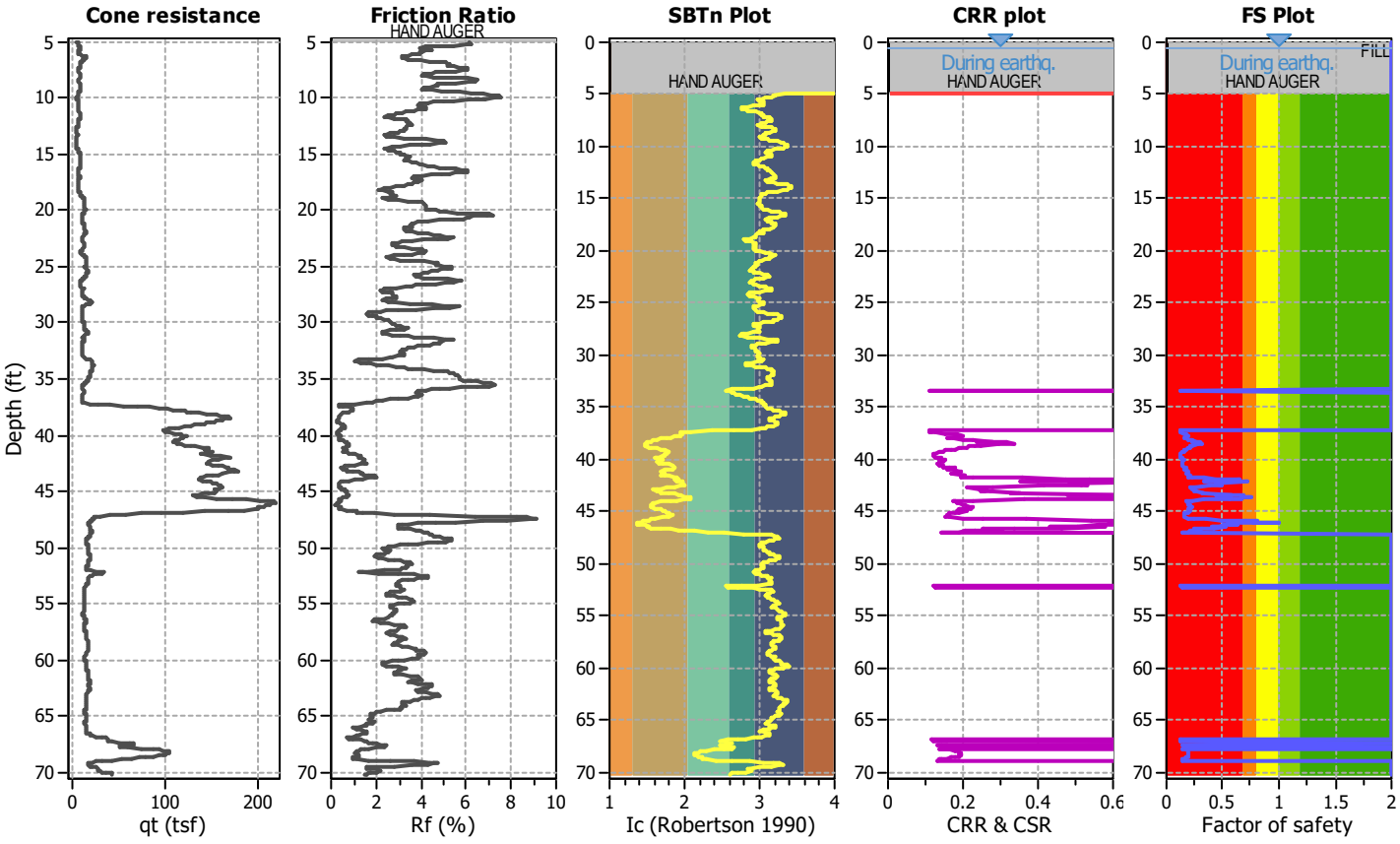
LIQUEFACTION ANALYSIS REPORT

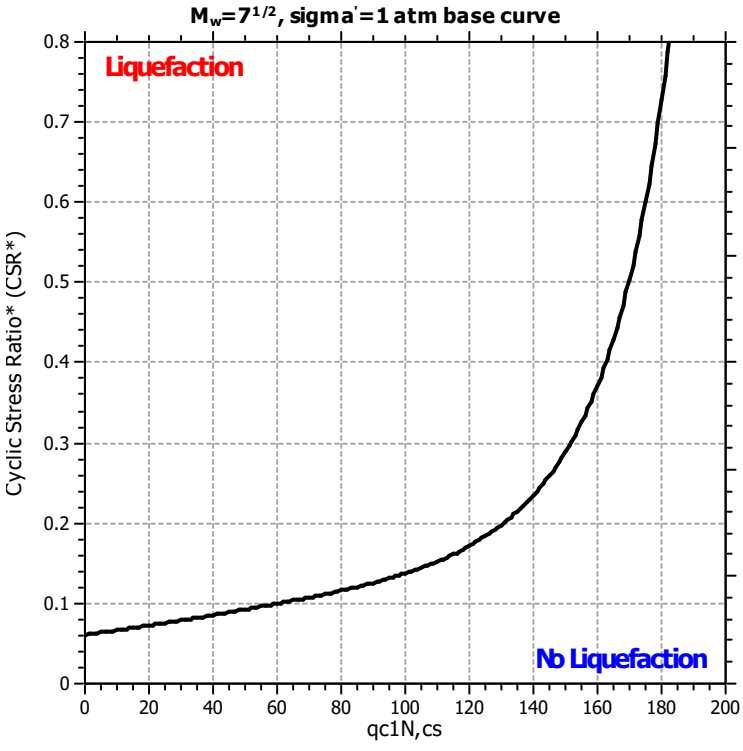
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

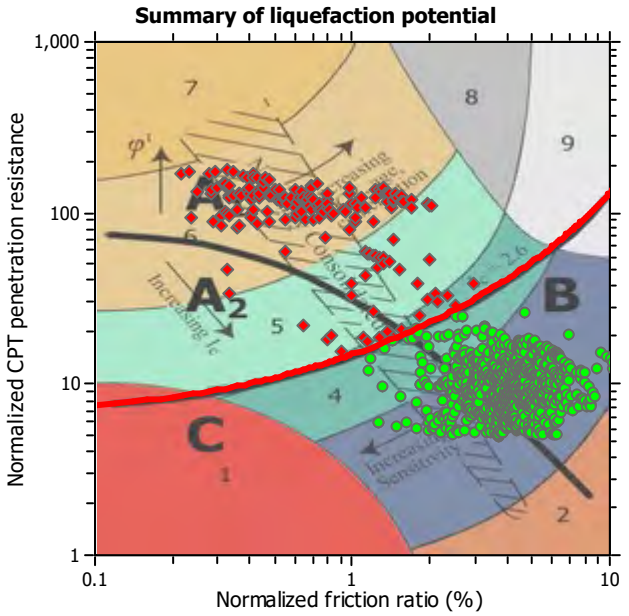
CPT file : GS26-CPT-06

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.75 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.60 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

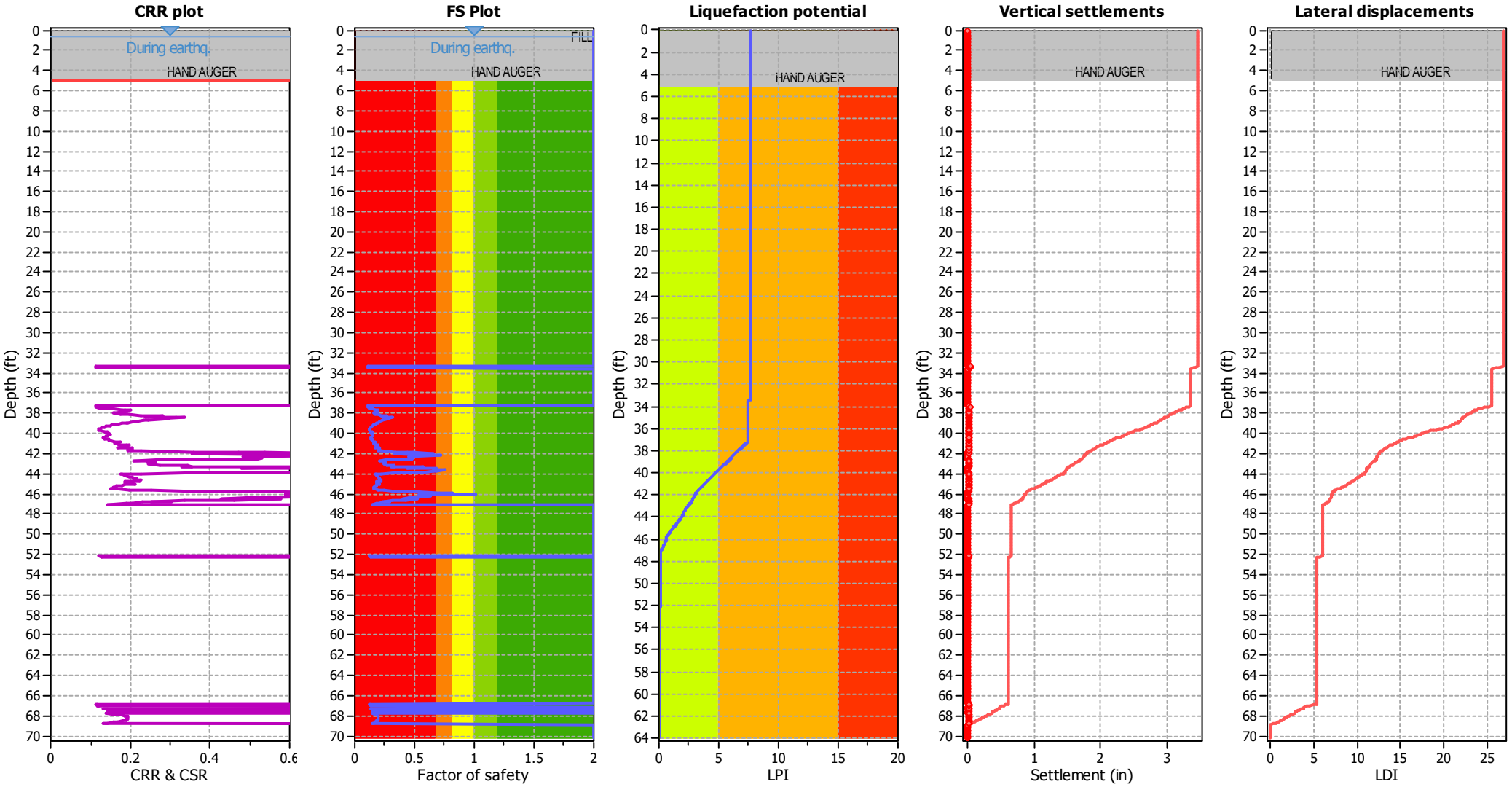






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	8.75 ft	Fill height:	1.60 ft	Limit depth:	N/A

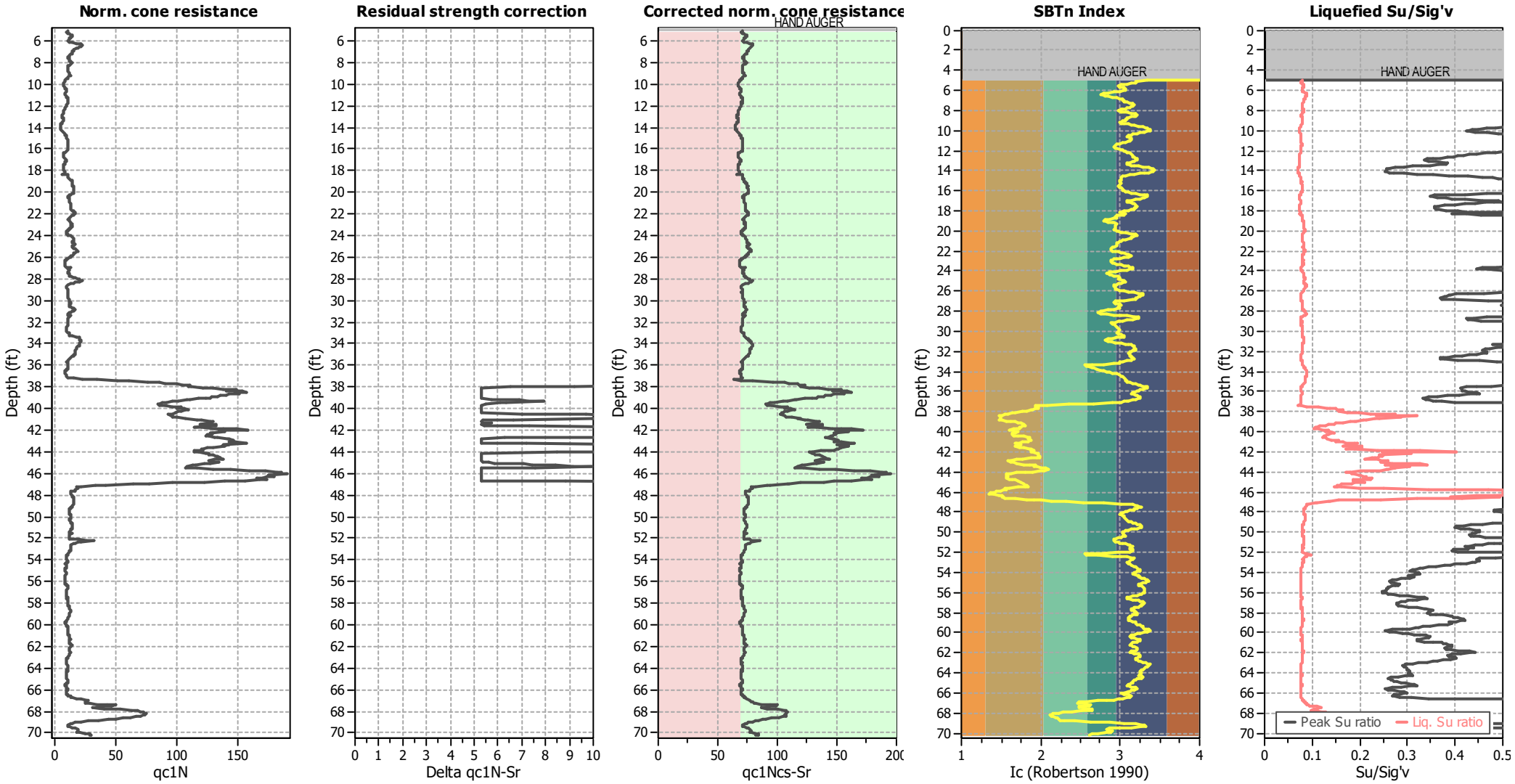
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

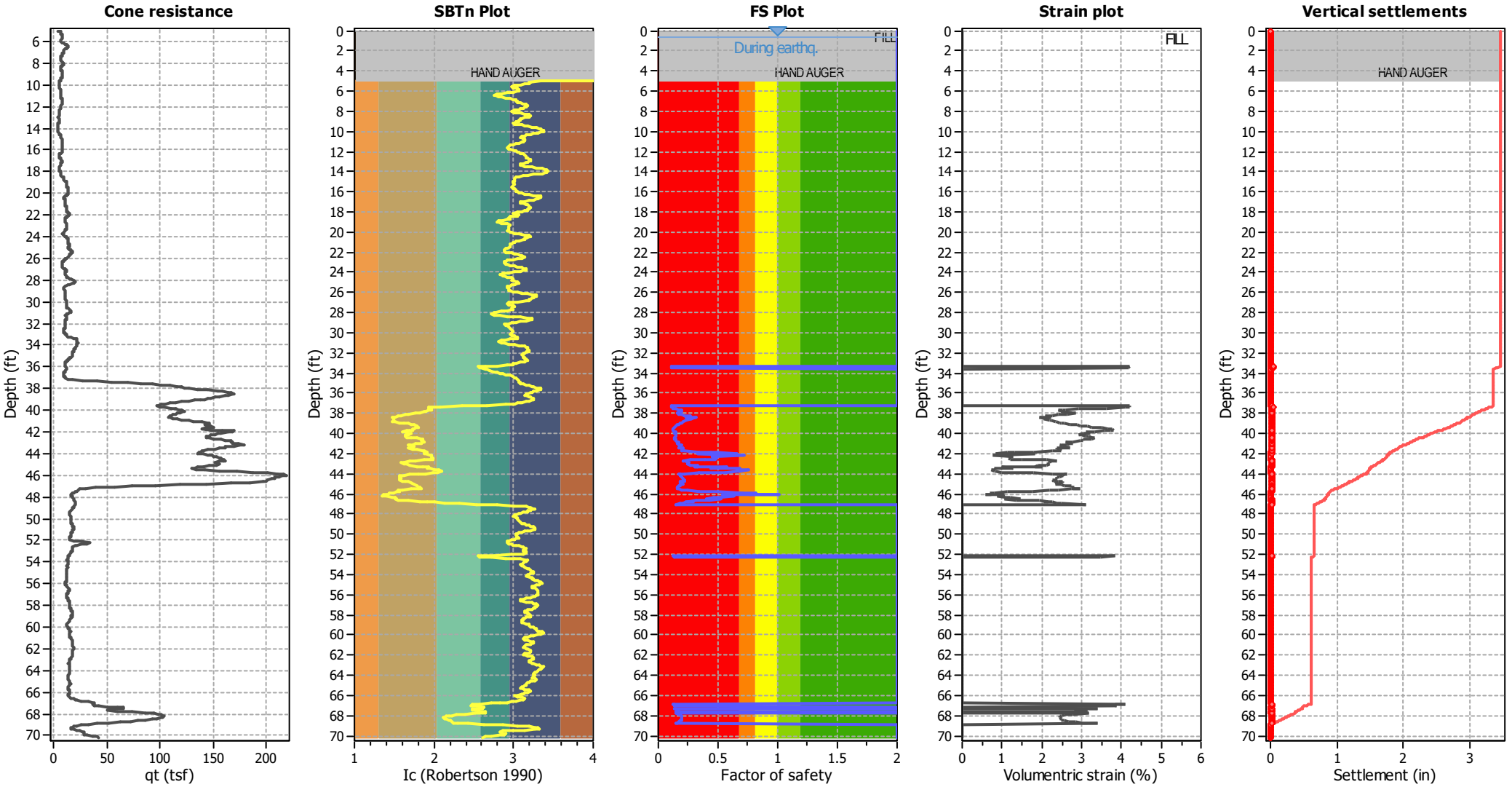
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	8.75 ft	Fill height:	1.60 ft	Limit depth:	N/A

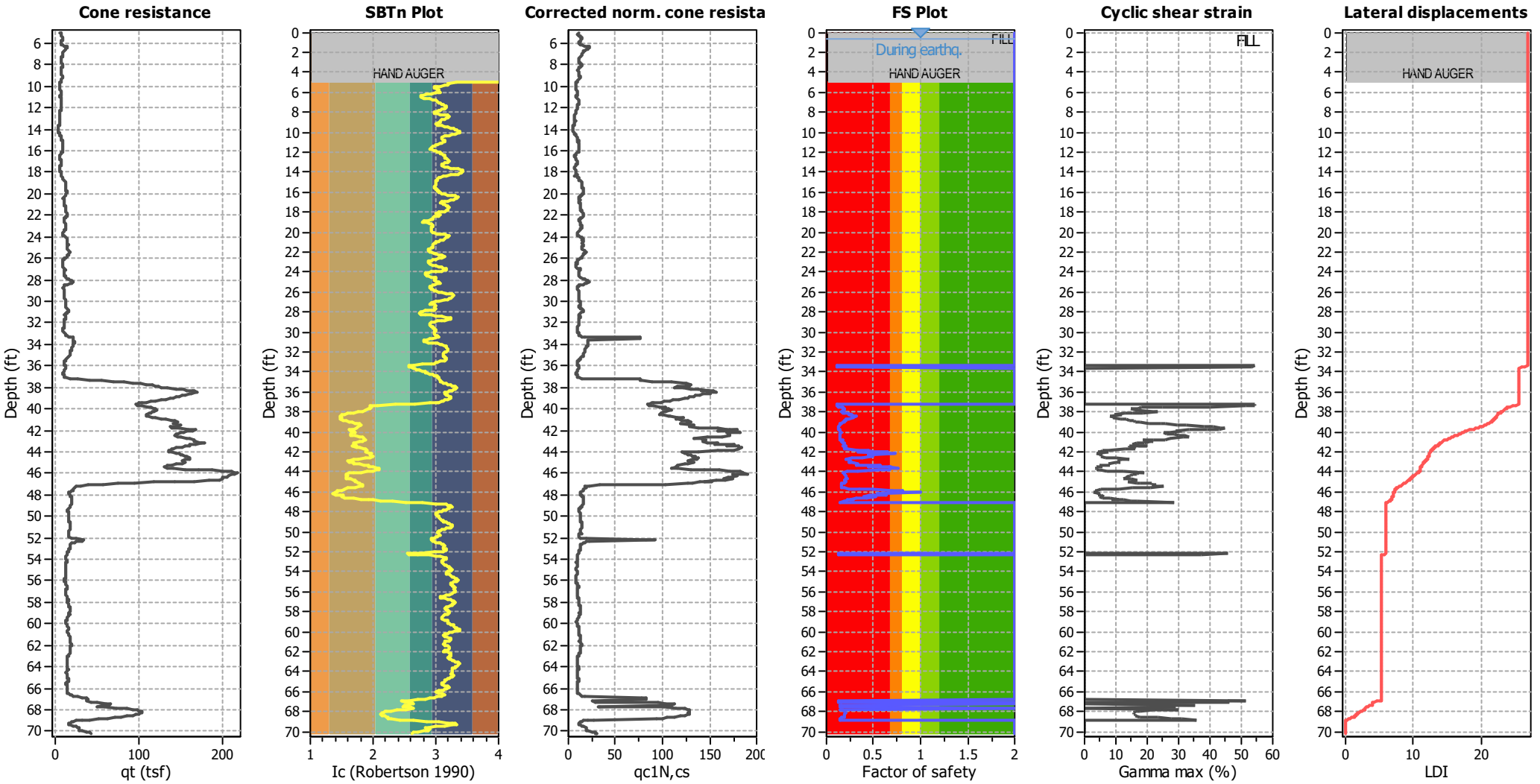
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

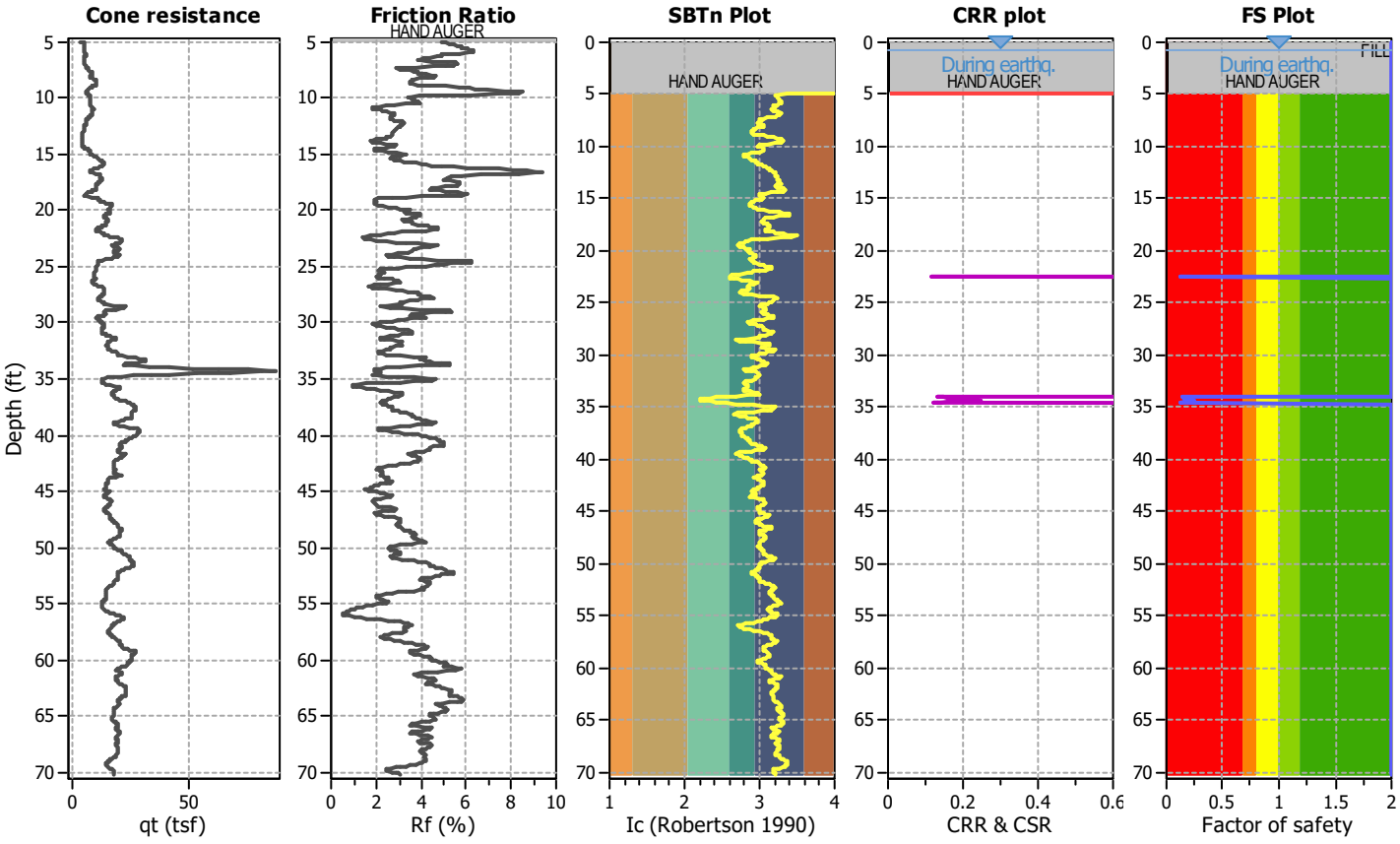
LIQUEFACTION ANALYSIS REPORT

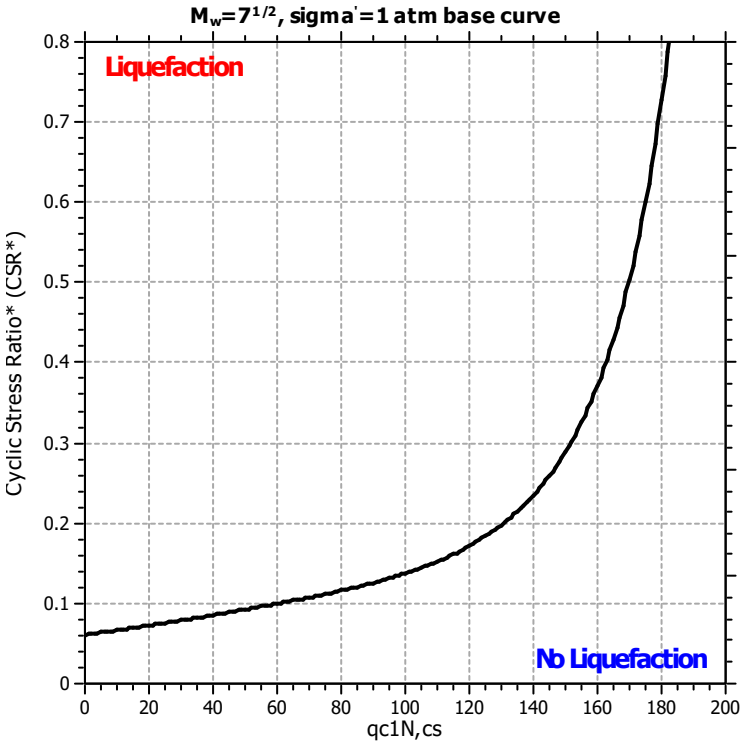
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

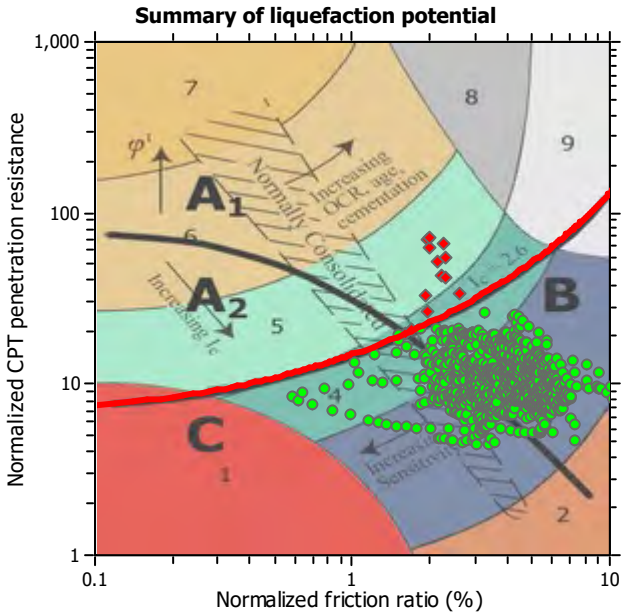
CPT file : GS26-CPT-07

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	12.98 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.30 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

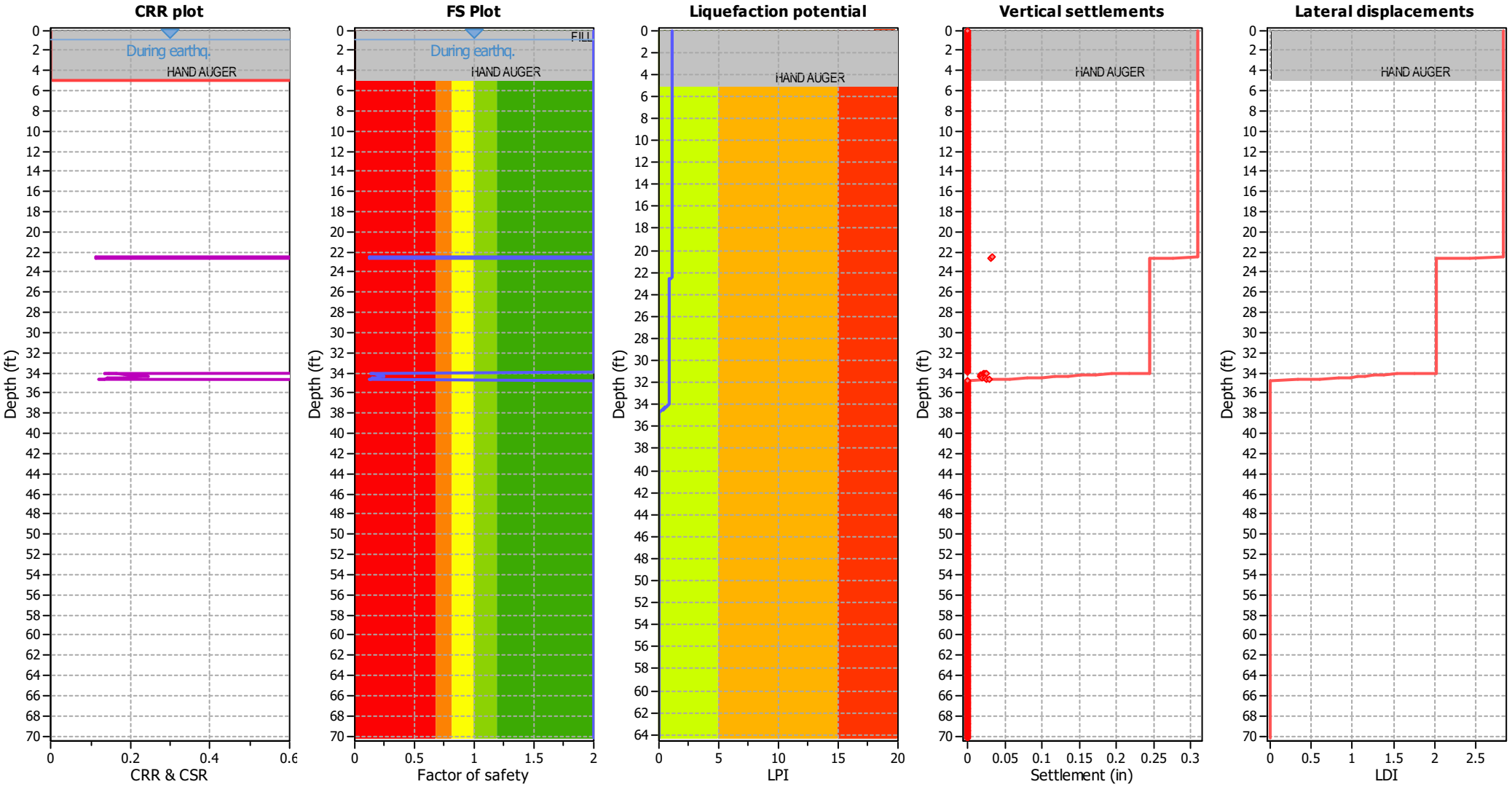






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	12.98 ft	Fill height:	1.30 ft	Limit depth:	N/A

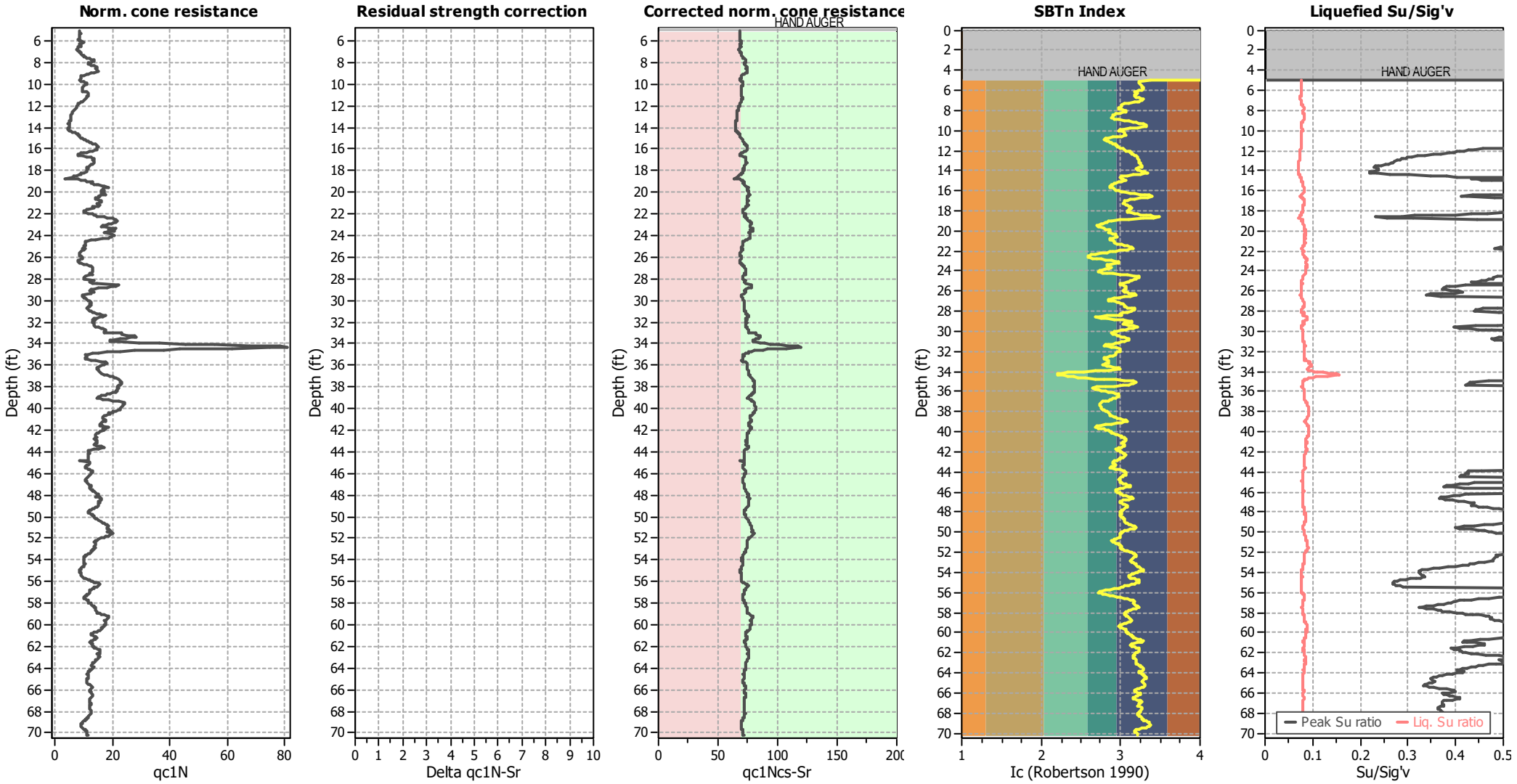
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

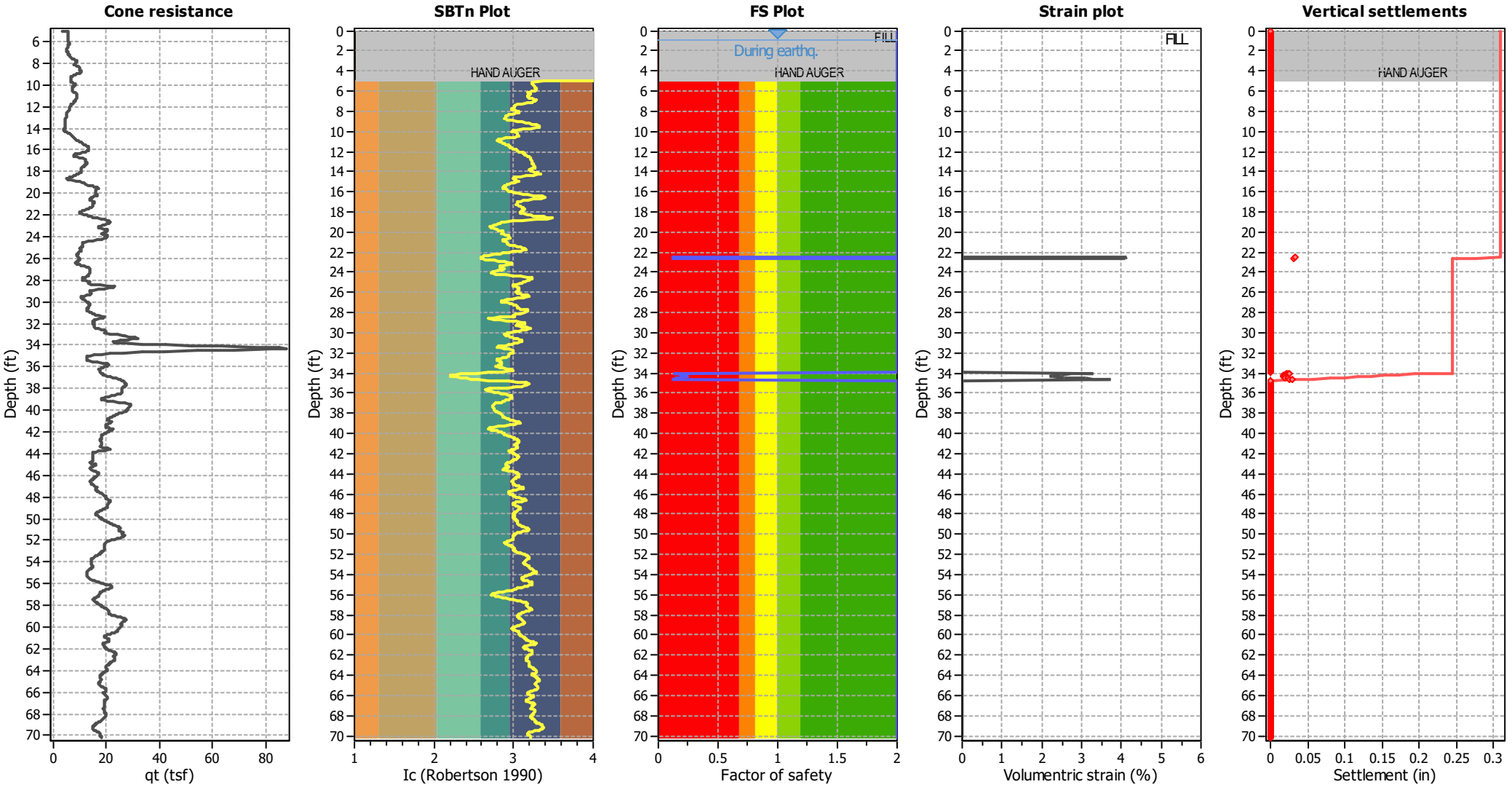
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	12.98 ft	Fill height:	1.30 ft	Limit depth:	N/A

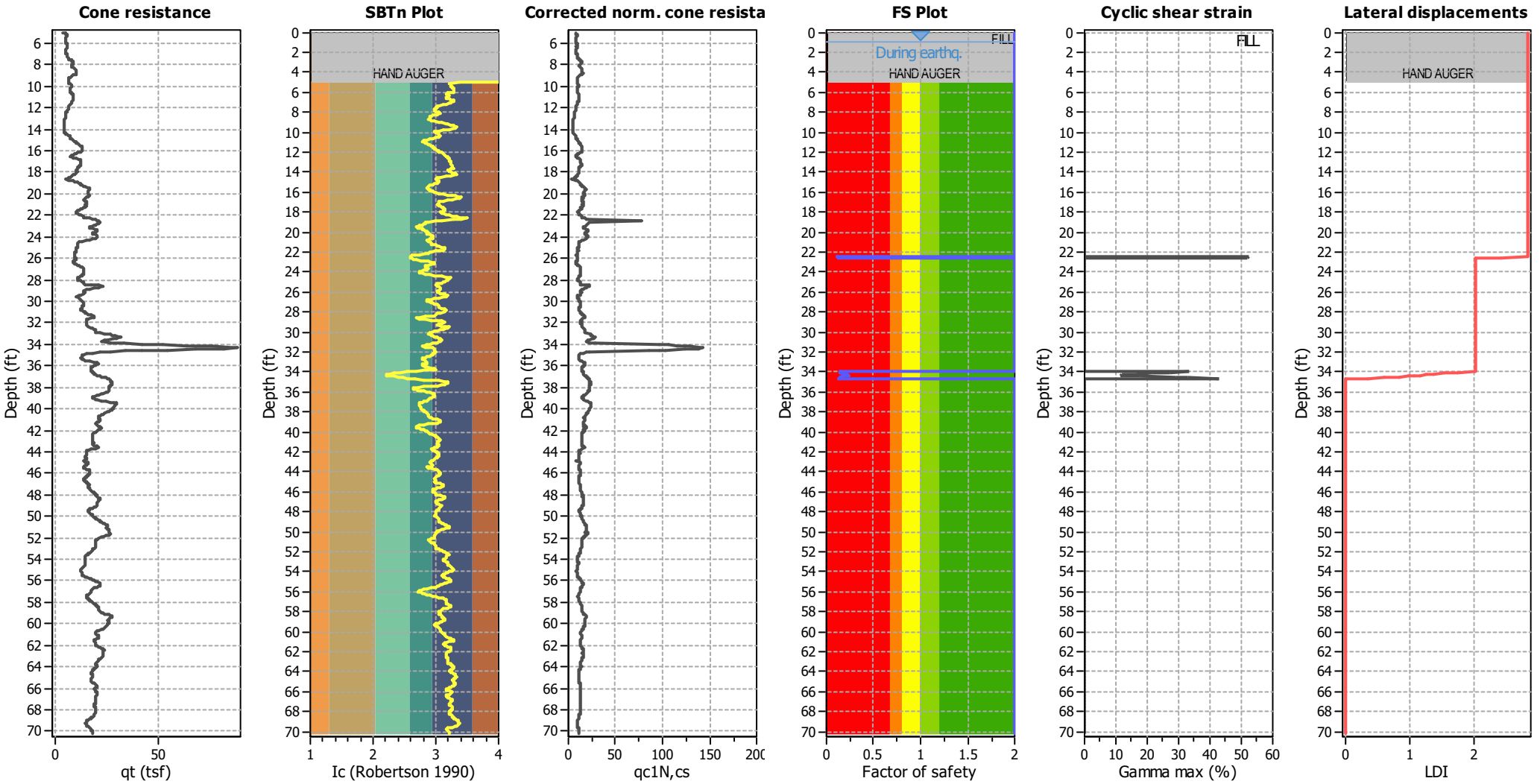
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

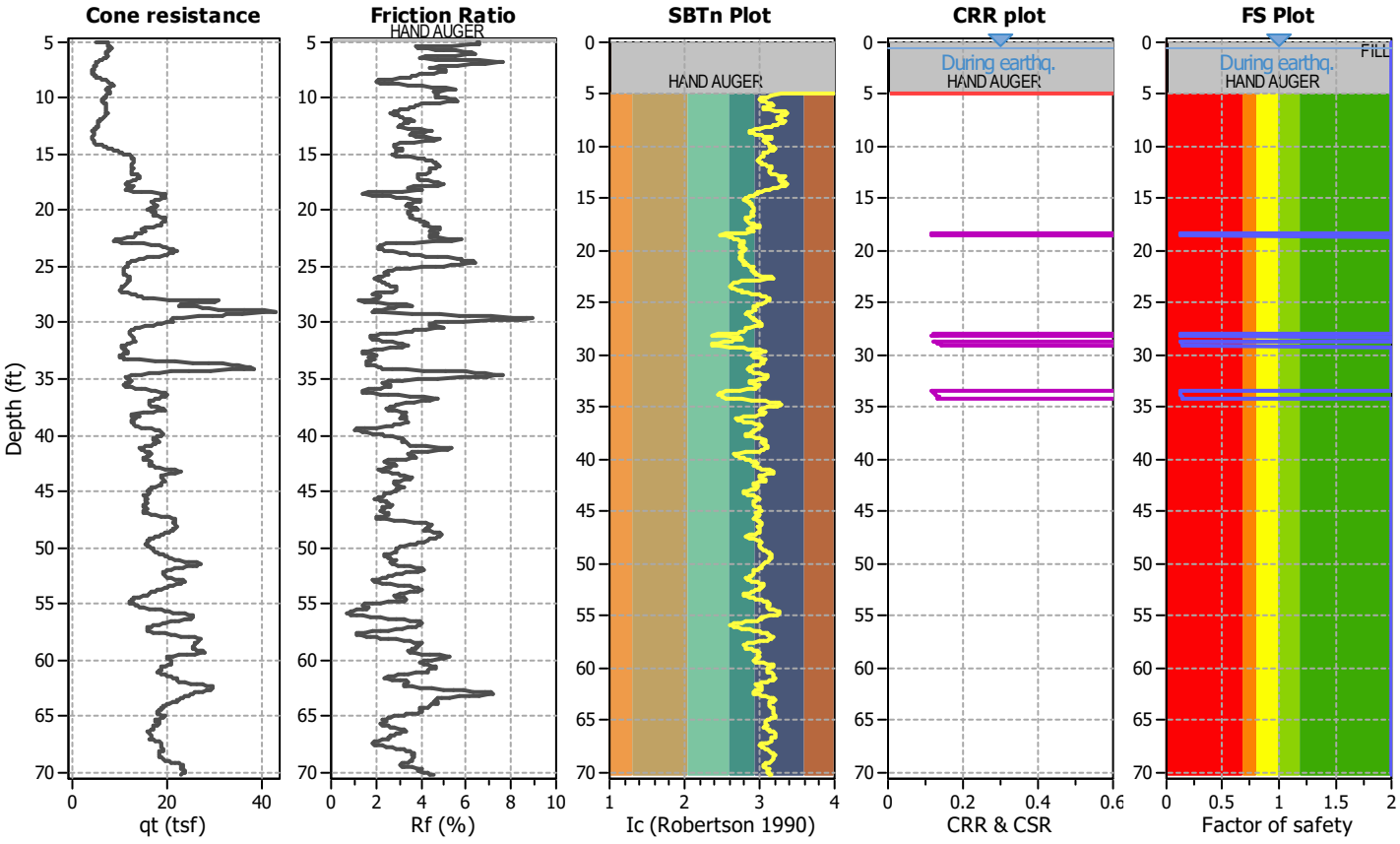
LIQUEFACTION ANALYSIS REPORT

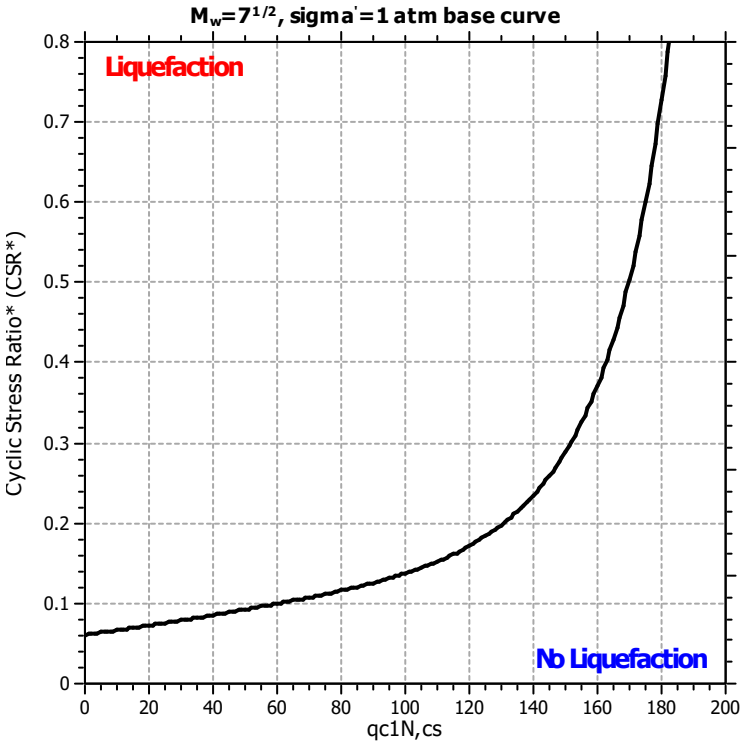
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

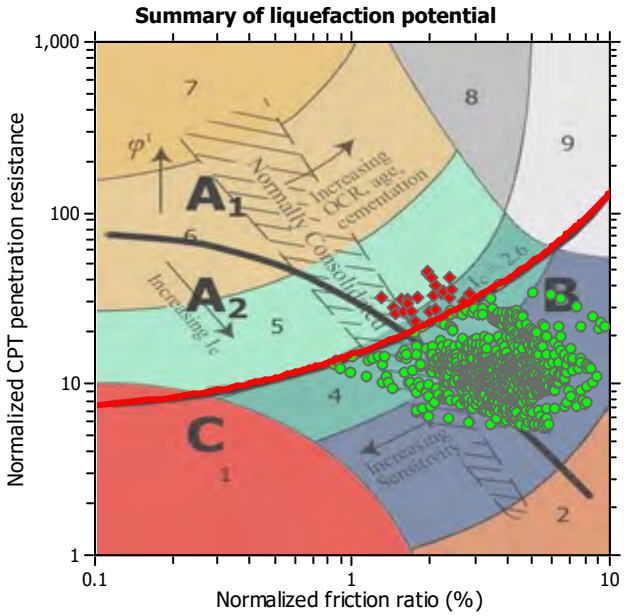
CPT file : GS26-CPT-08

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.99 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.60 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

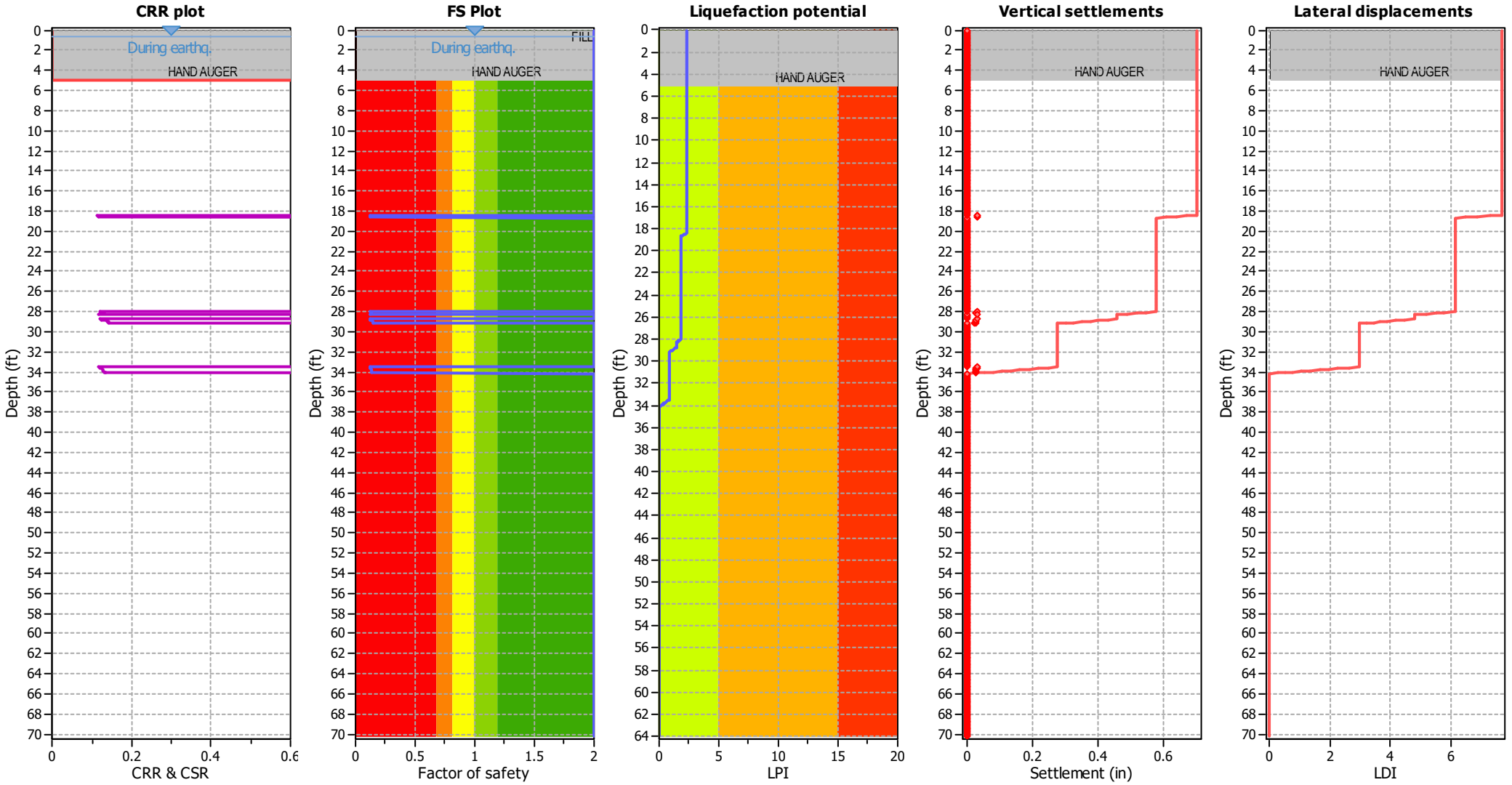






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.99 ft	Fill height:	1.60 ft	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
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LPI color scheme

- Very high risk
- High risk
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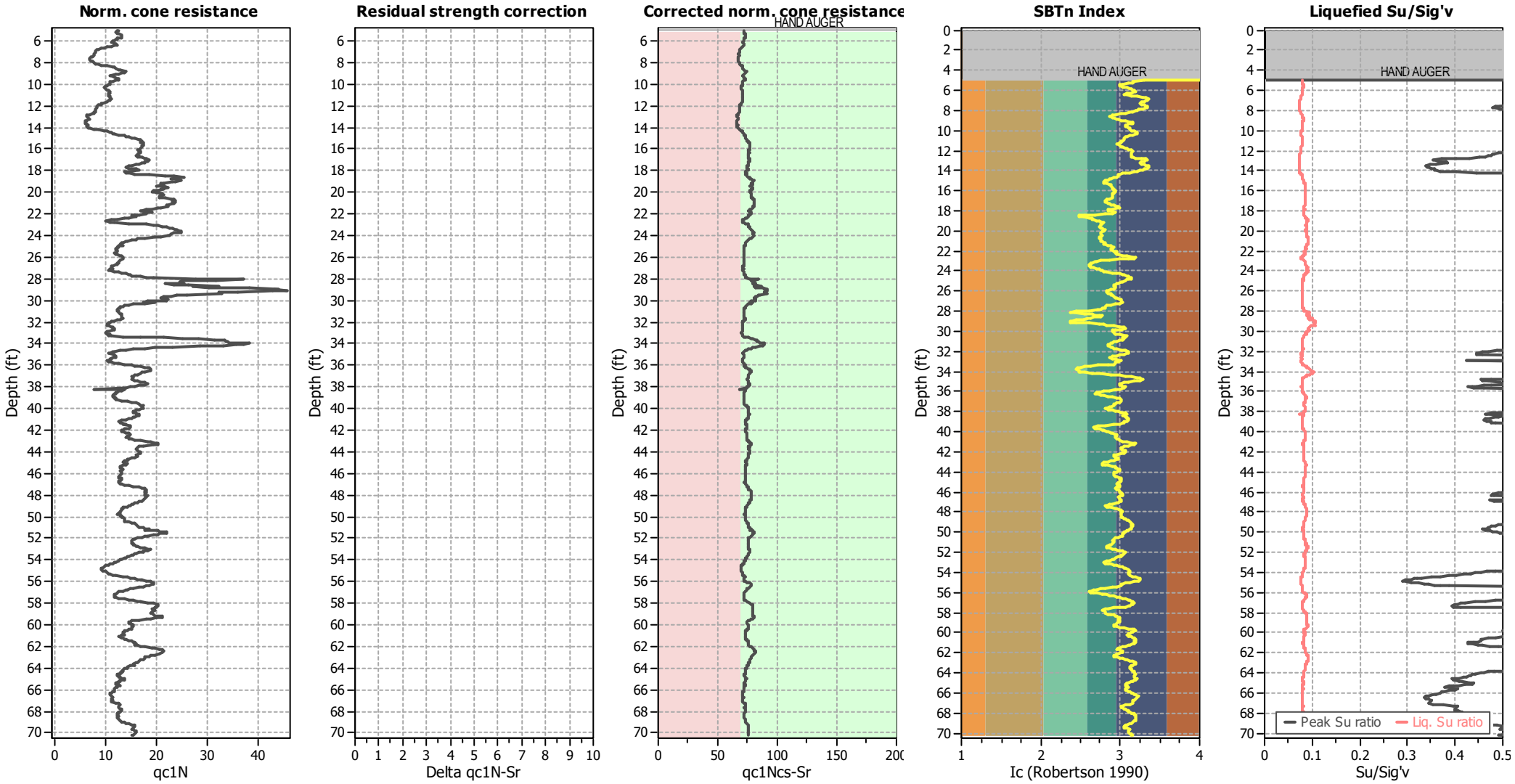
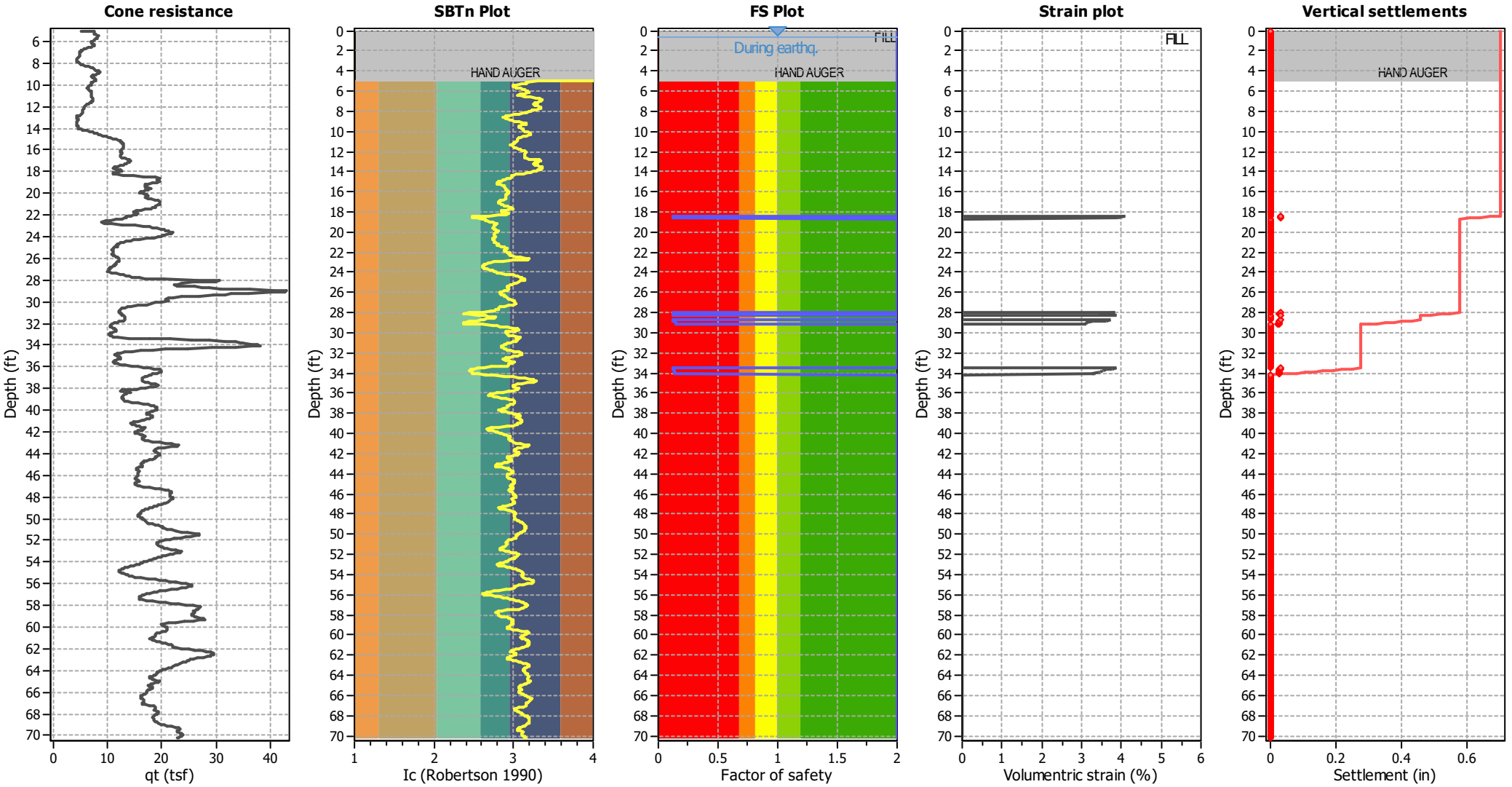
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.99 ft	Fill height:	1.60 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



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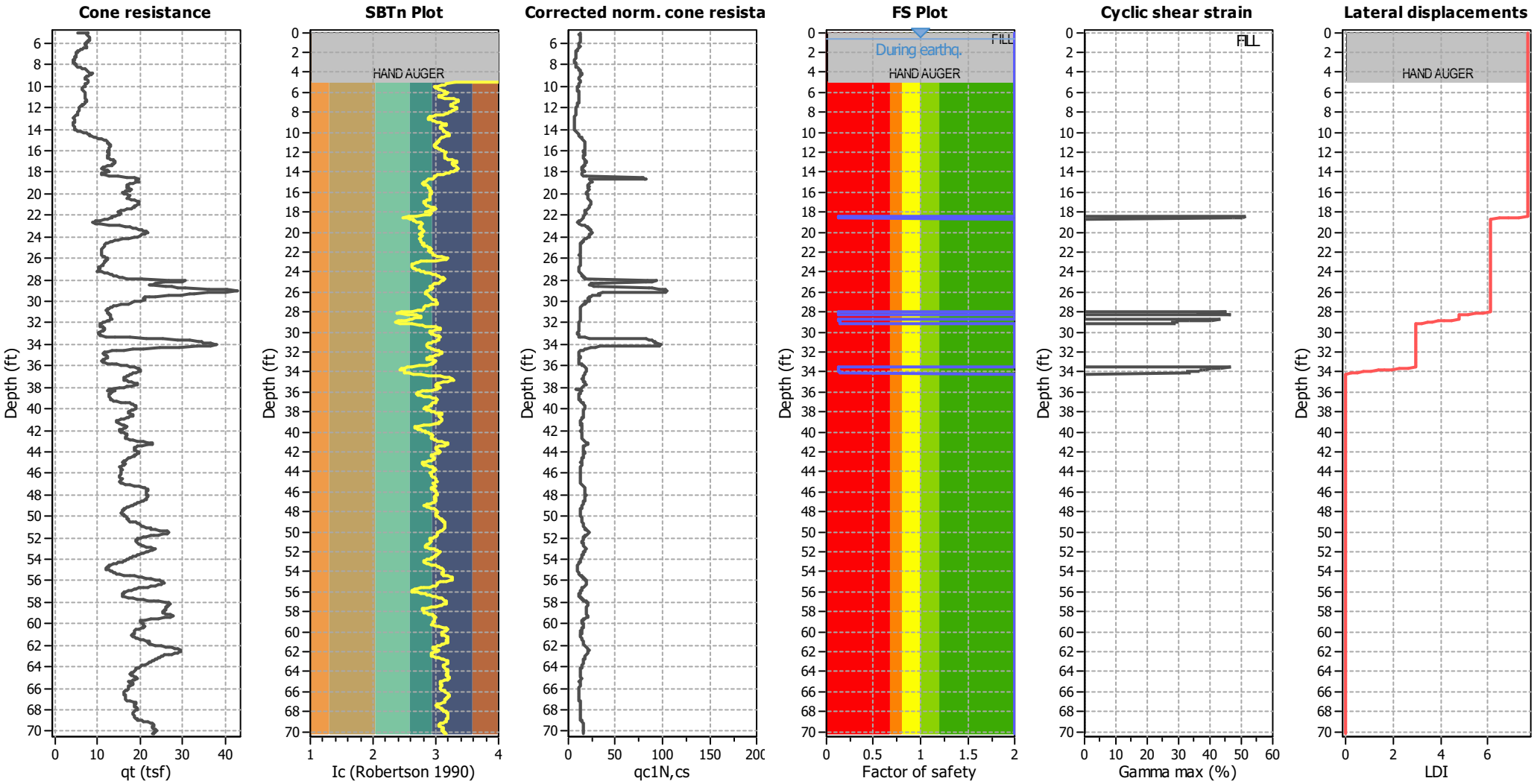
Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
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$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

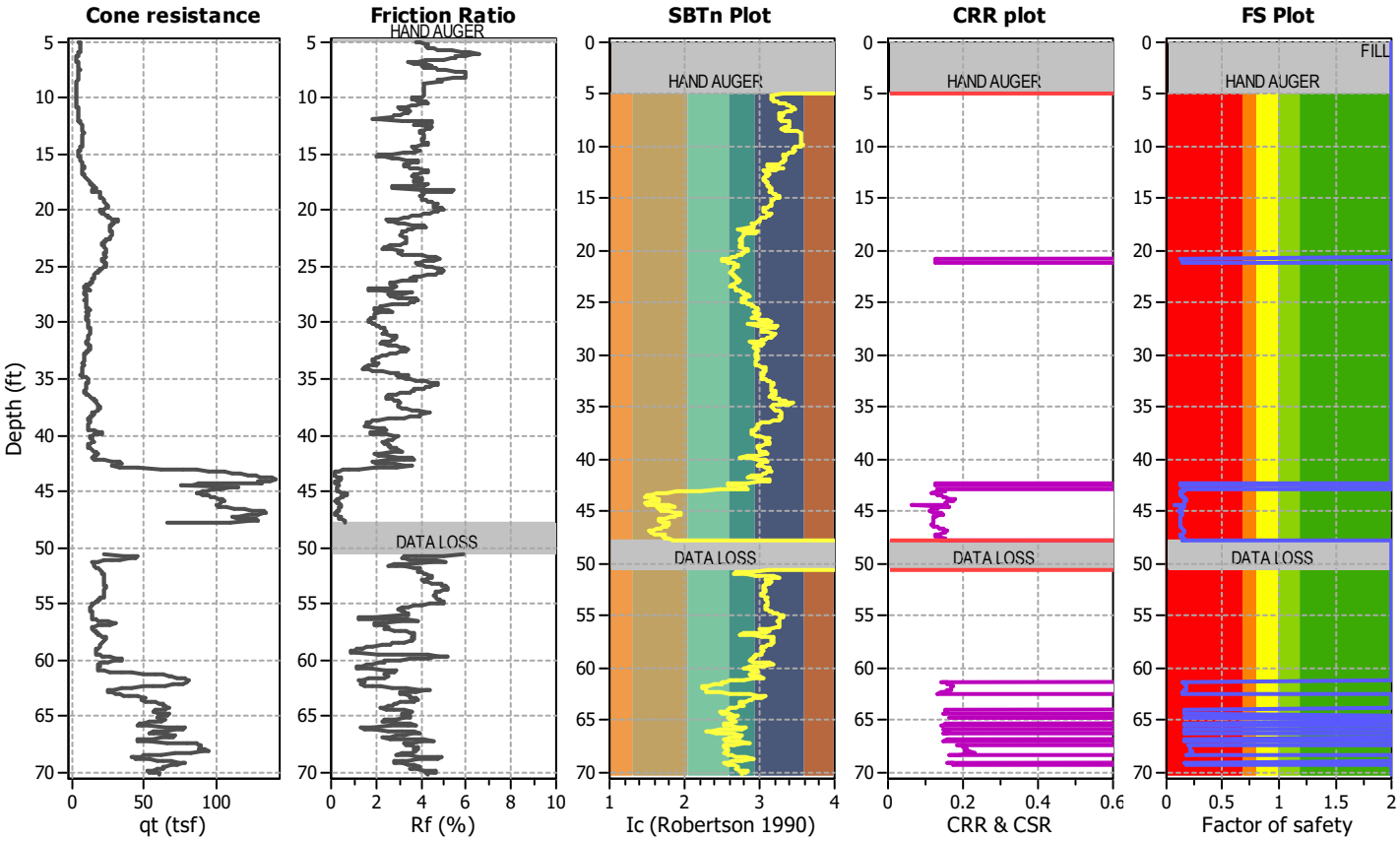
LIQUEFACTION ANALYSIS REPORT

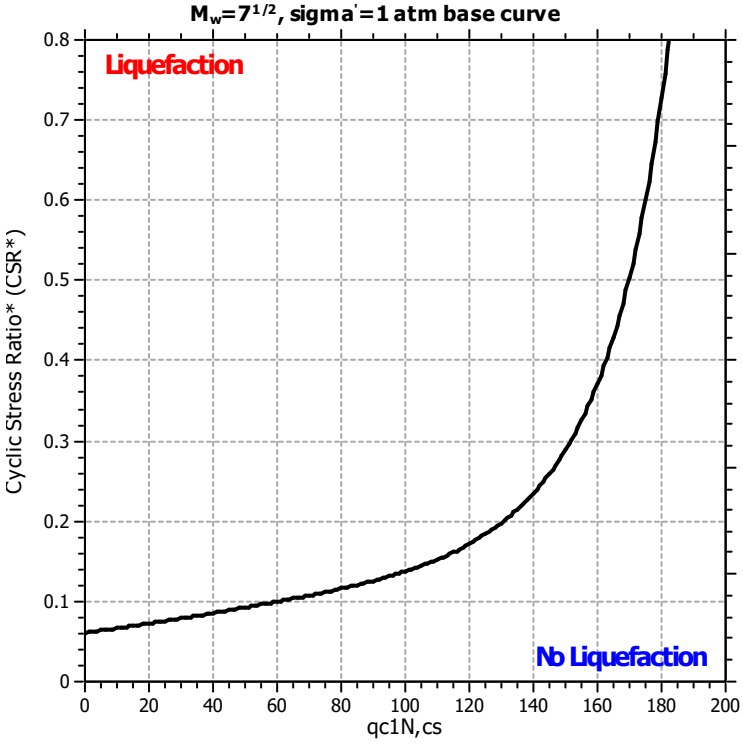
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

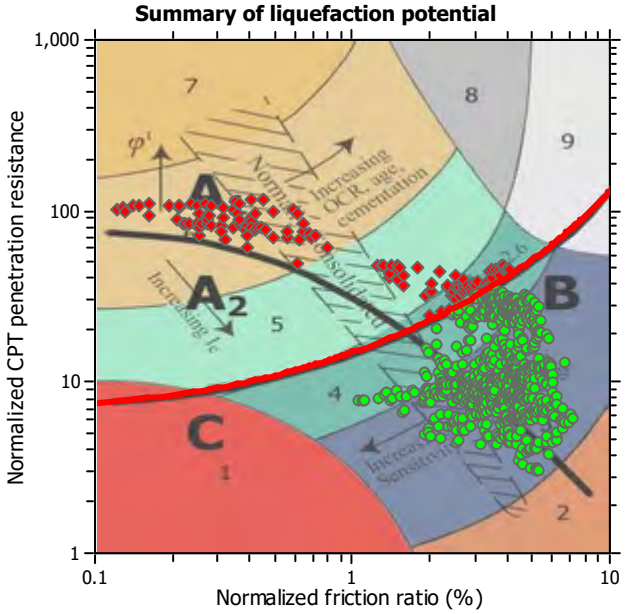
CPT file : GS26-CPT-09

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	8.94 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	2.30 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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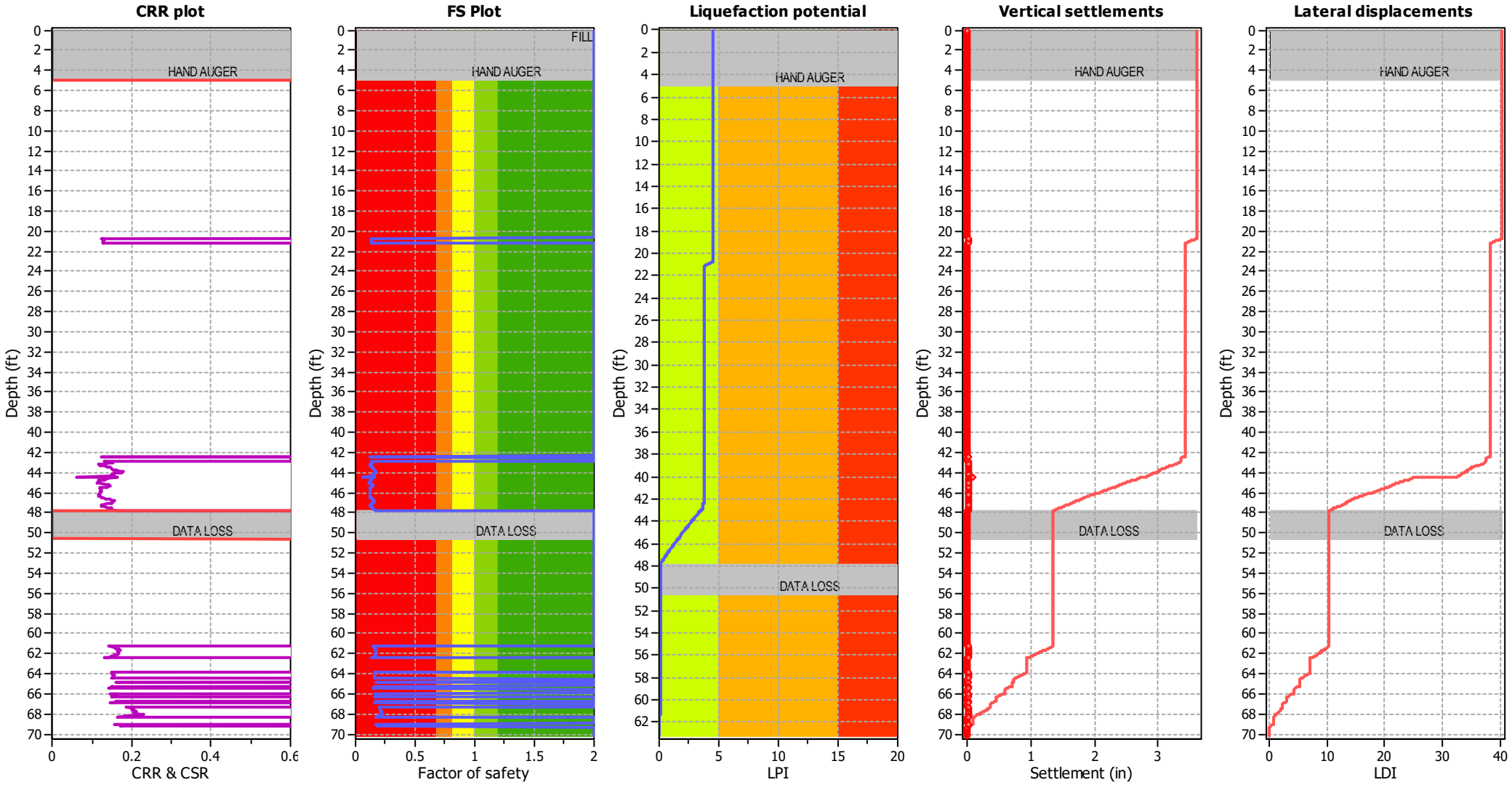
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	8.94 ft	Fill height:	2.30 ft	Limit depth:	N/A

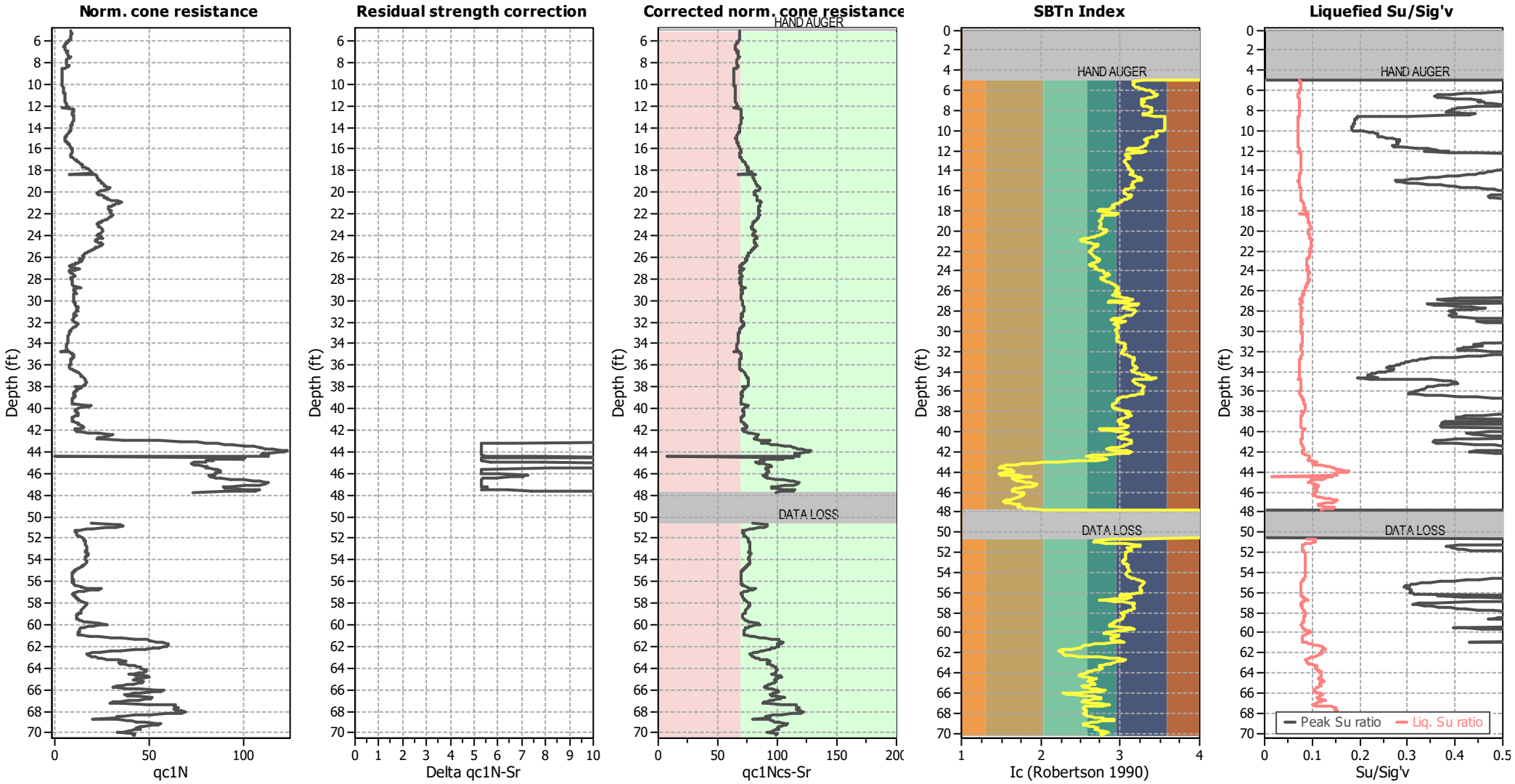
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

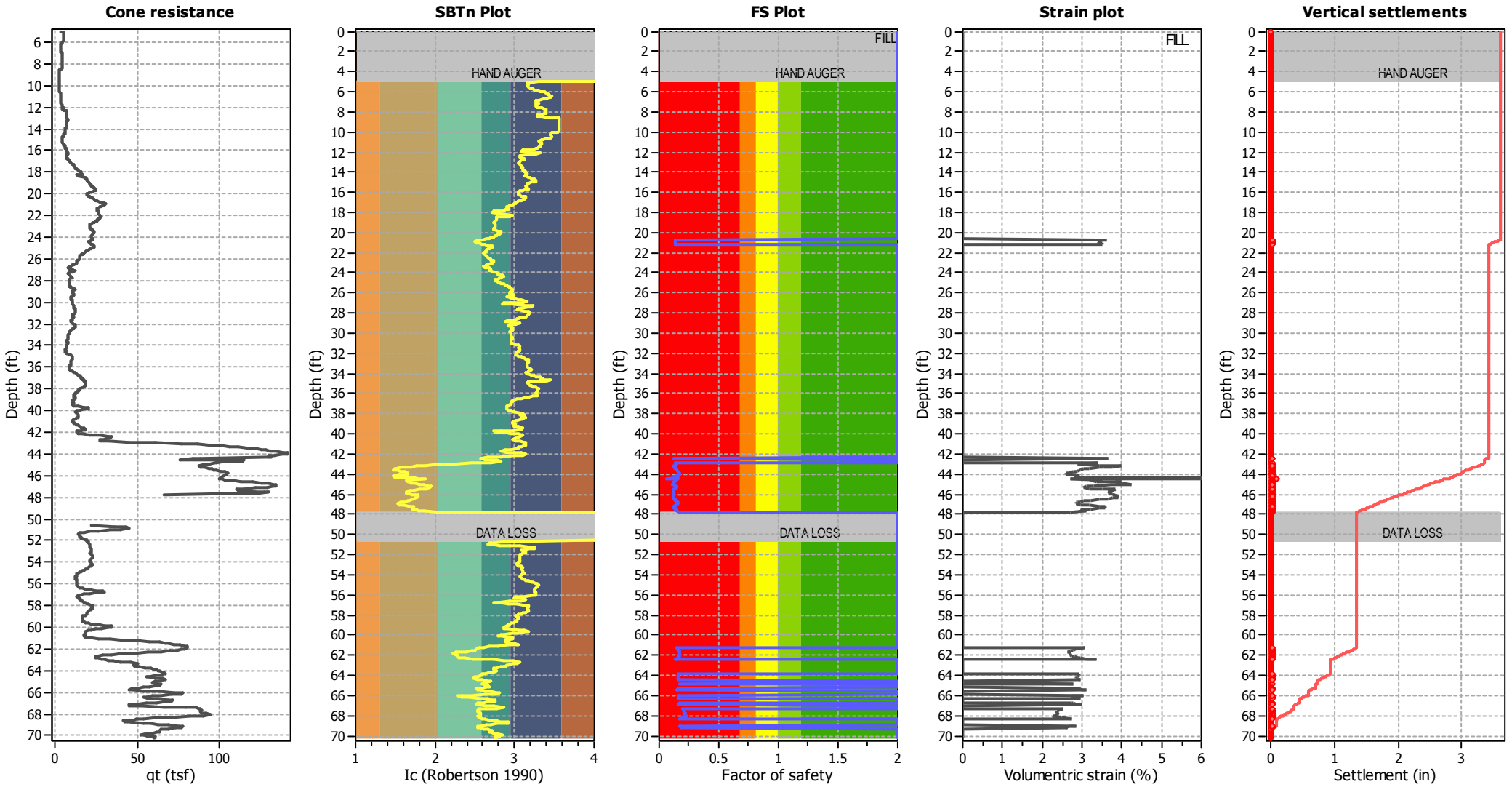
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	8.94 ft	Fill height:	2.30 ft	Limit depth:	N/A

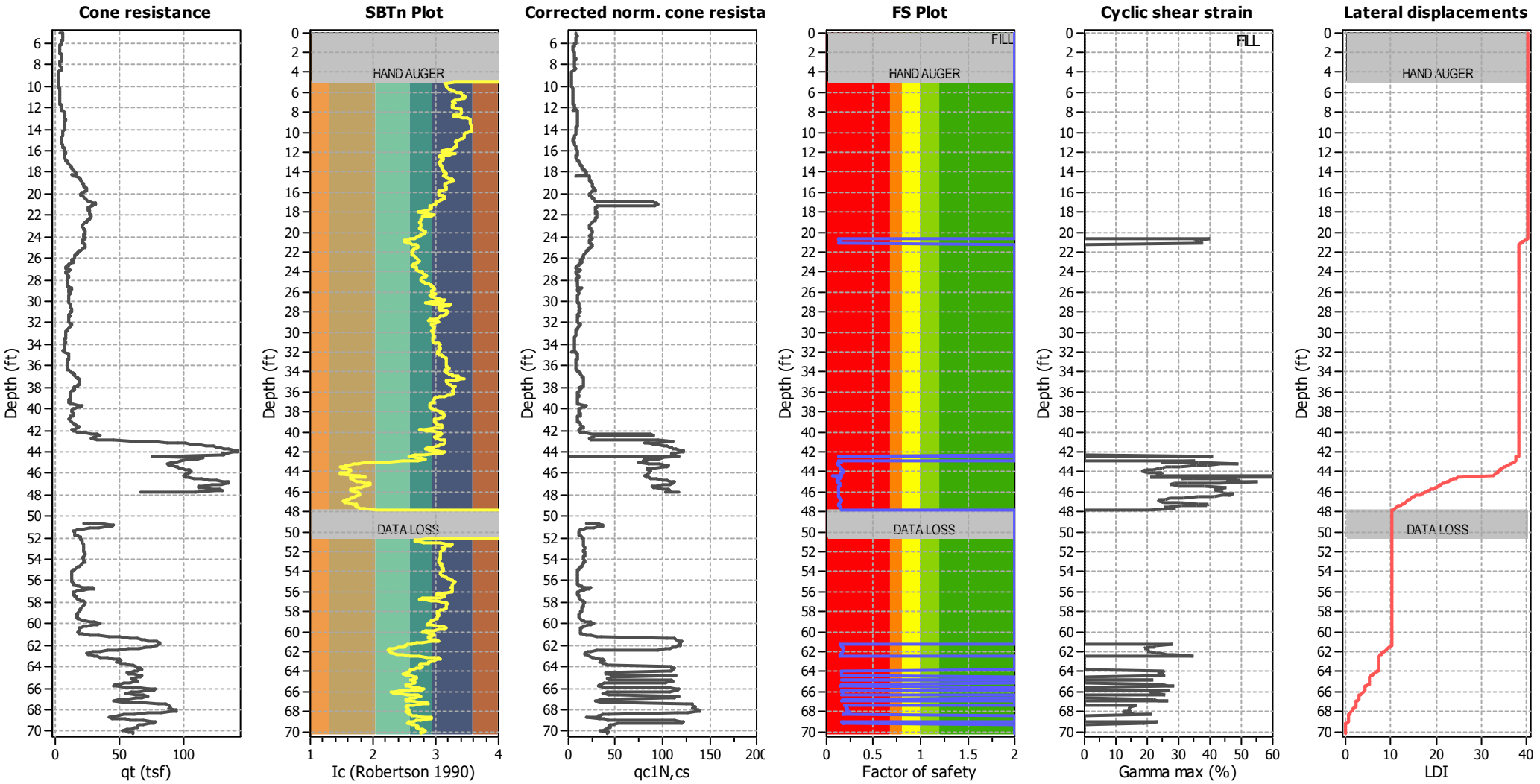
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

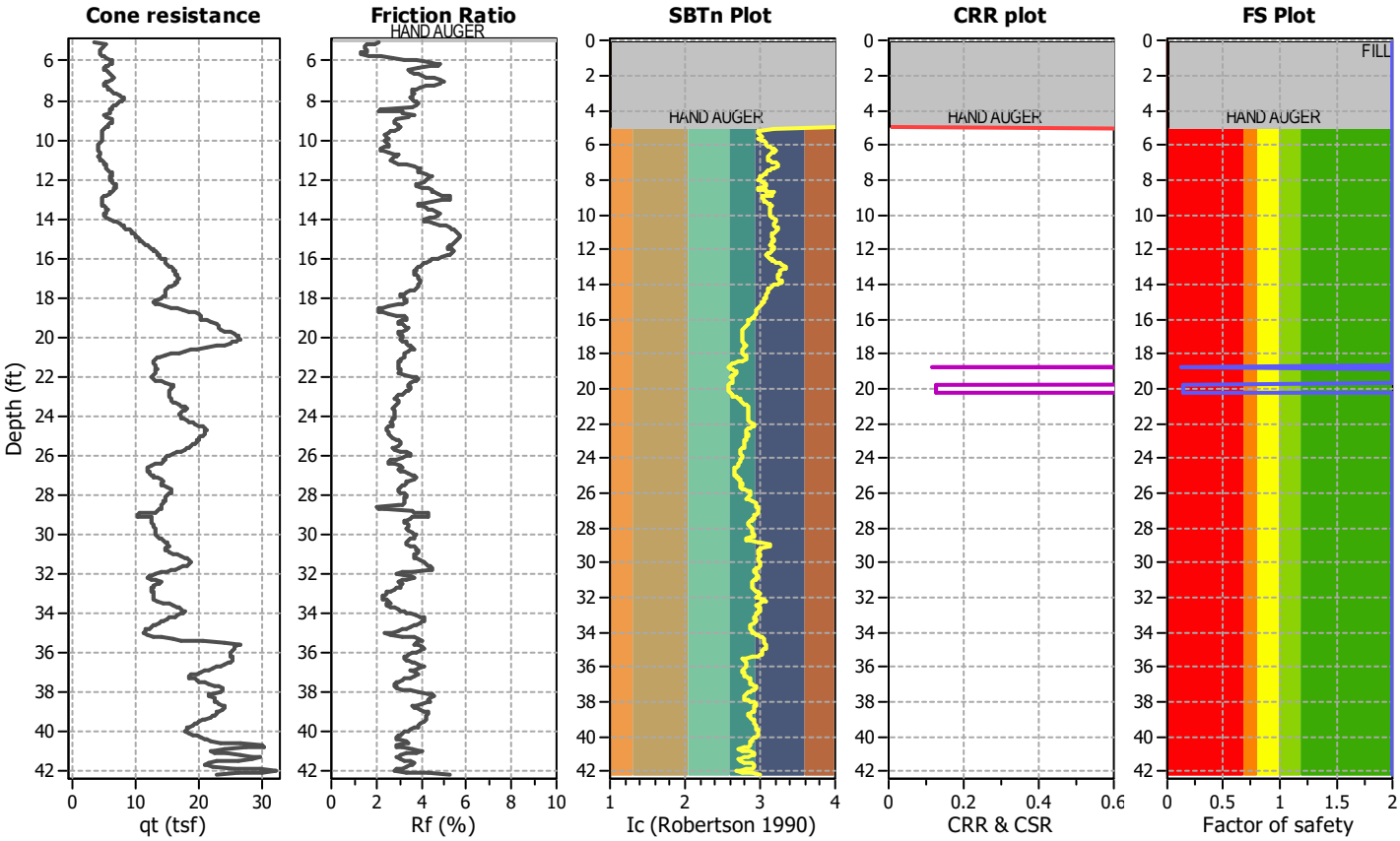
LIQUEFACTION ANALYSIS REPORT

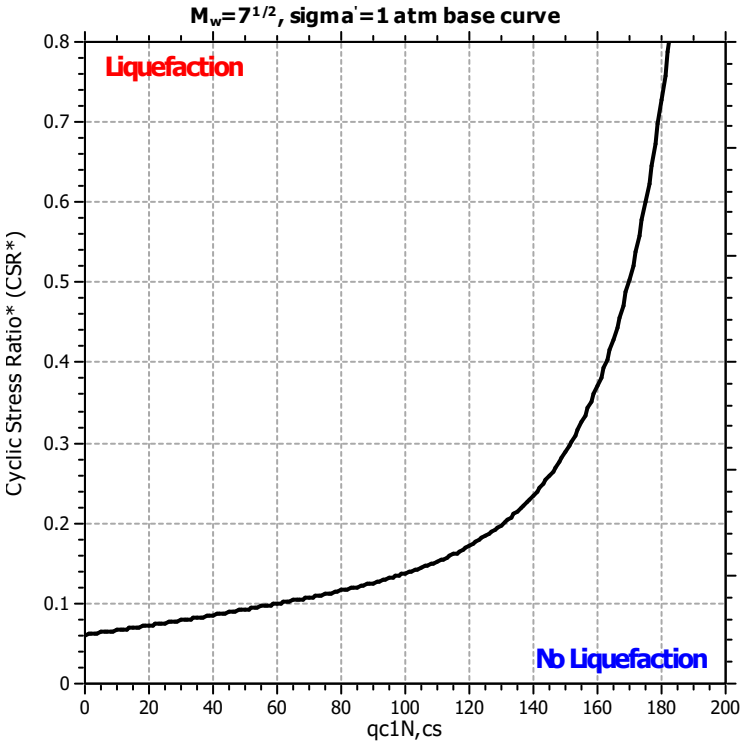
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

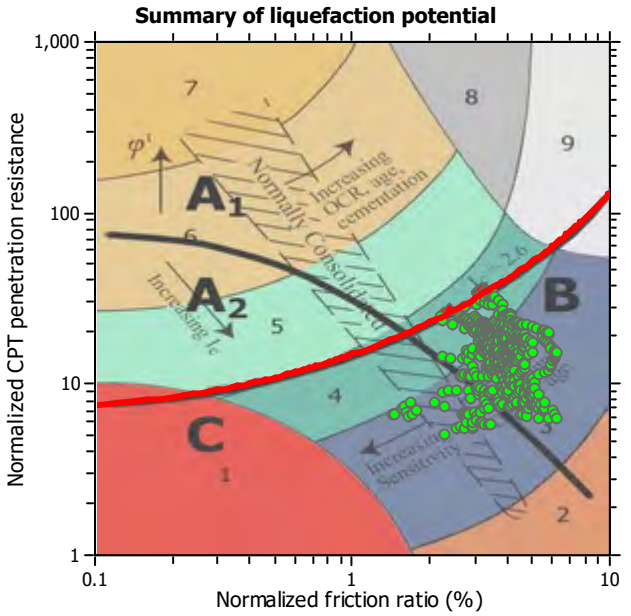
CPT file : GS26-CPT-10

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.01 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	4.10 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

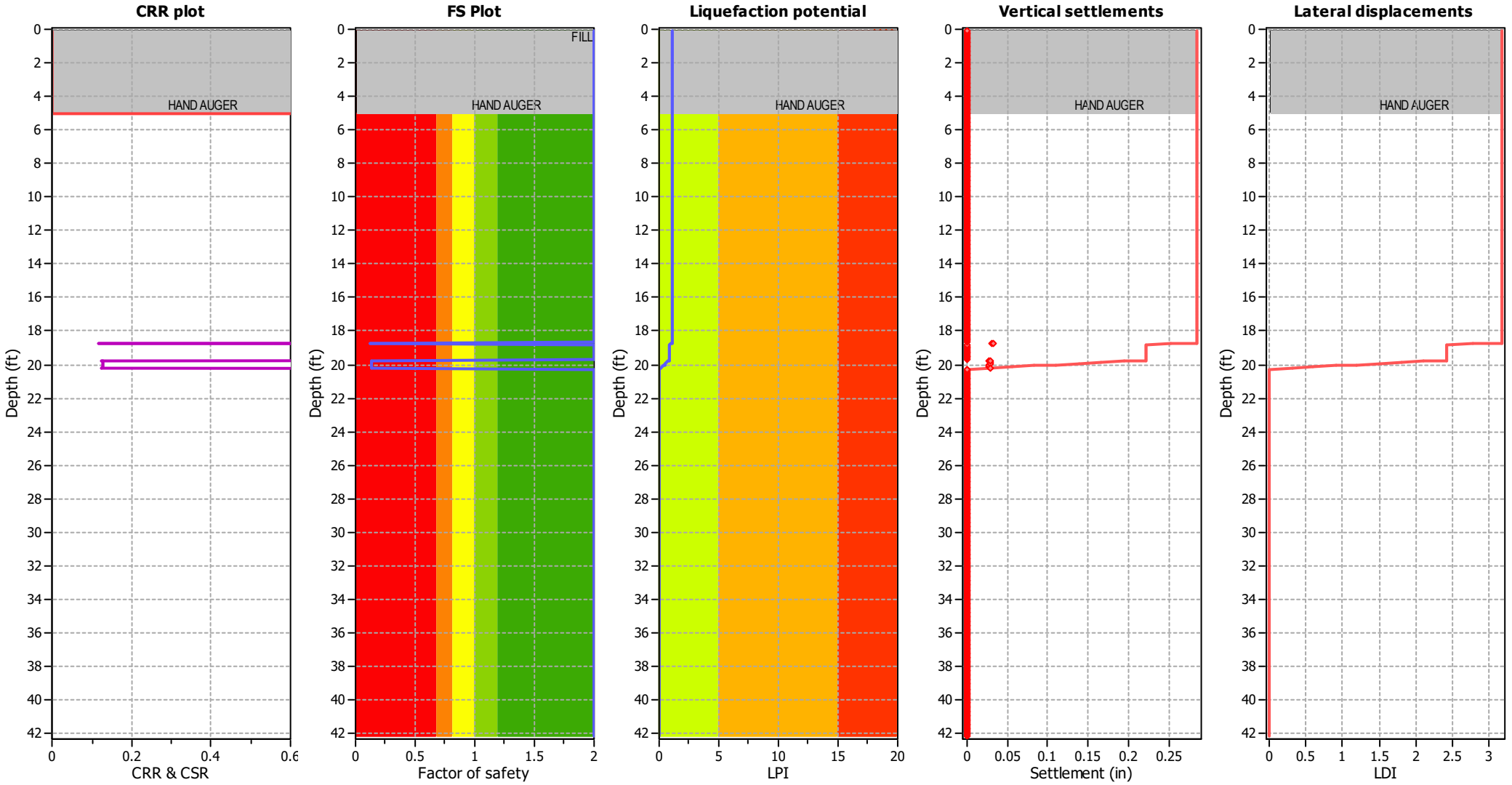






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.01 ft	Fill height:	4.10 ft	Limit depth:	N/A

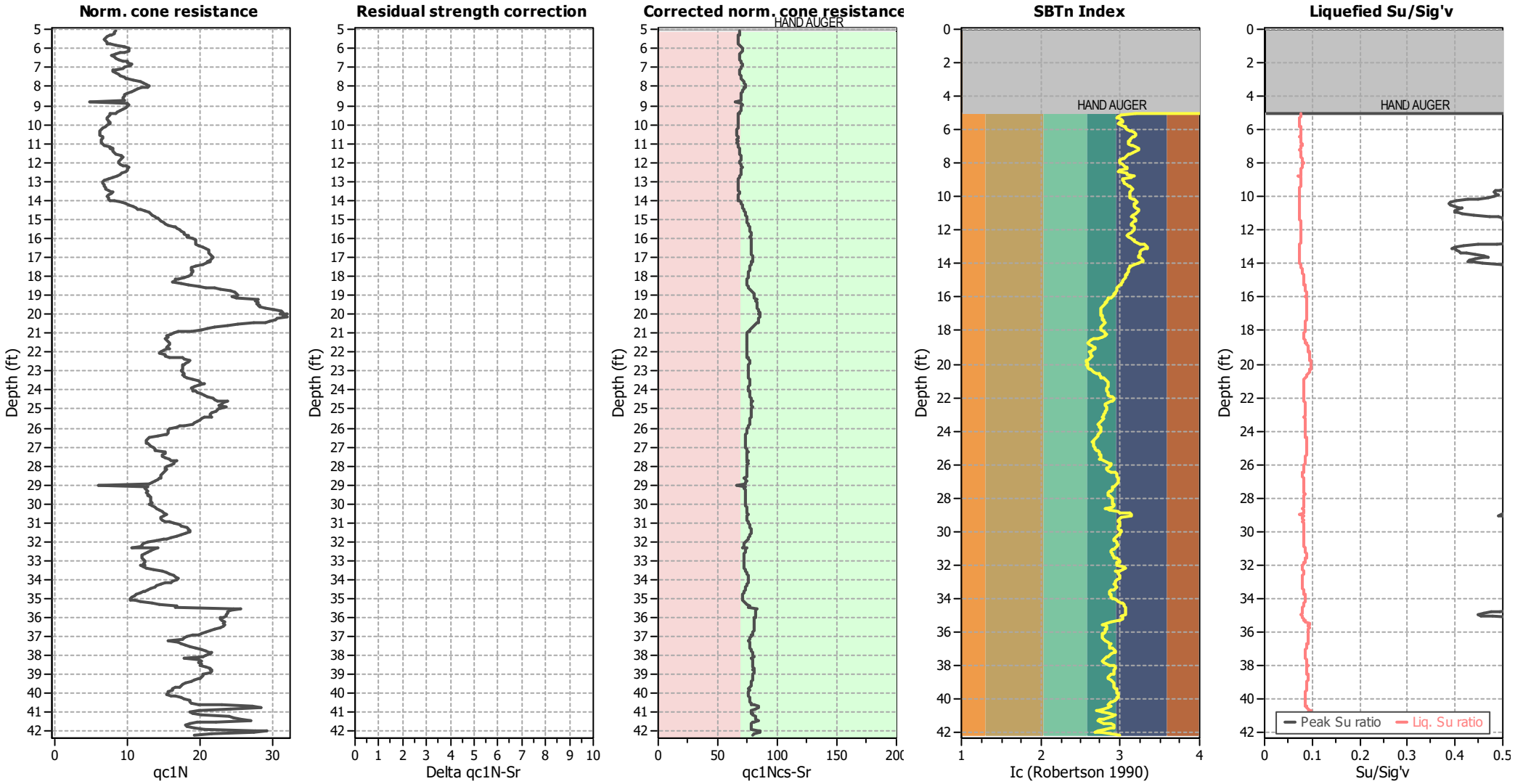
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

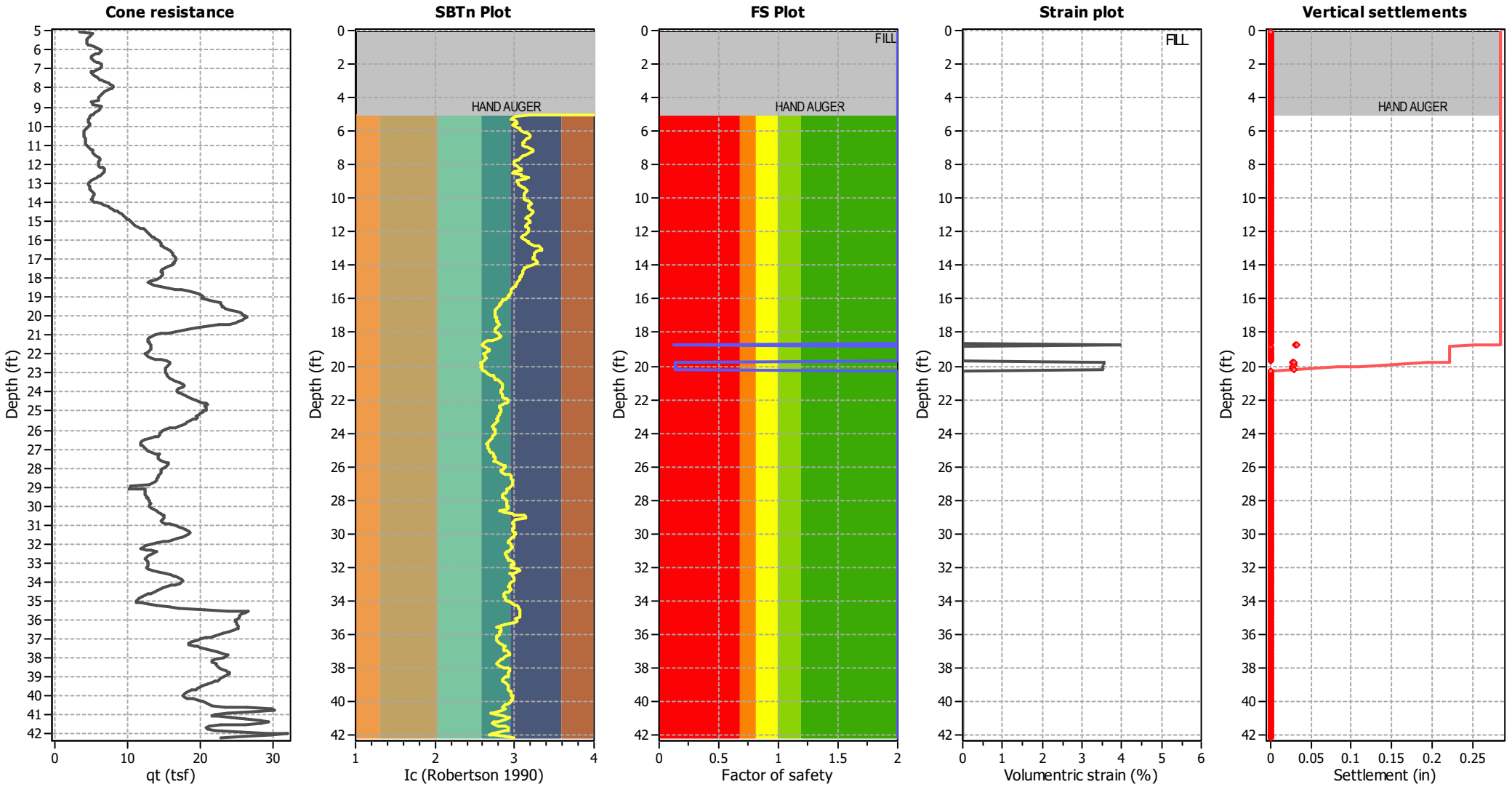
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.01 ft	Fill height:	4.10 ft	Limit depth:	N/A

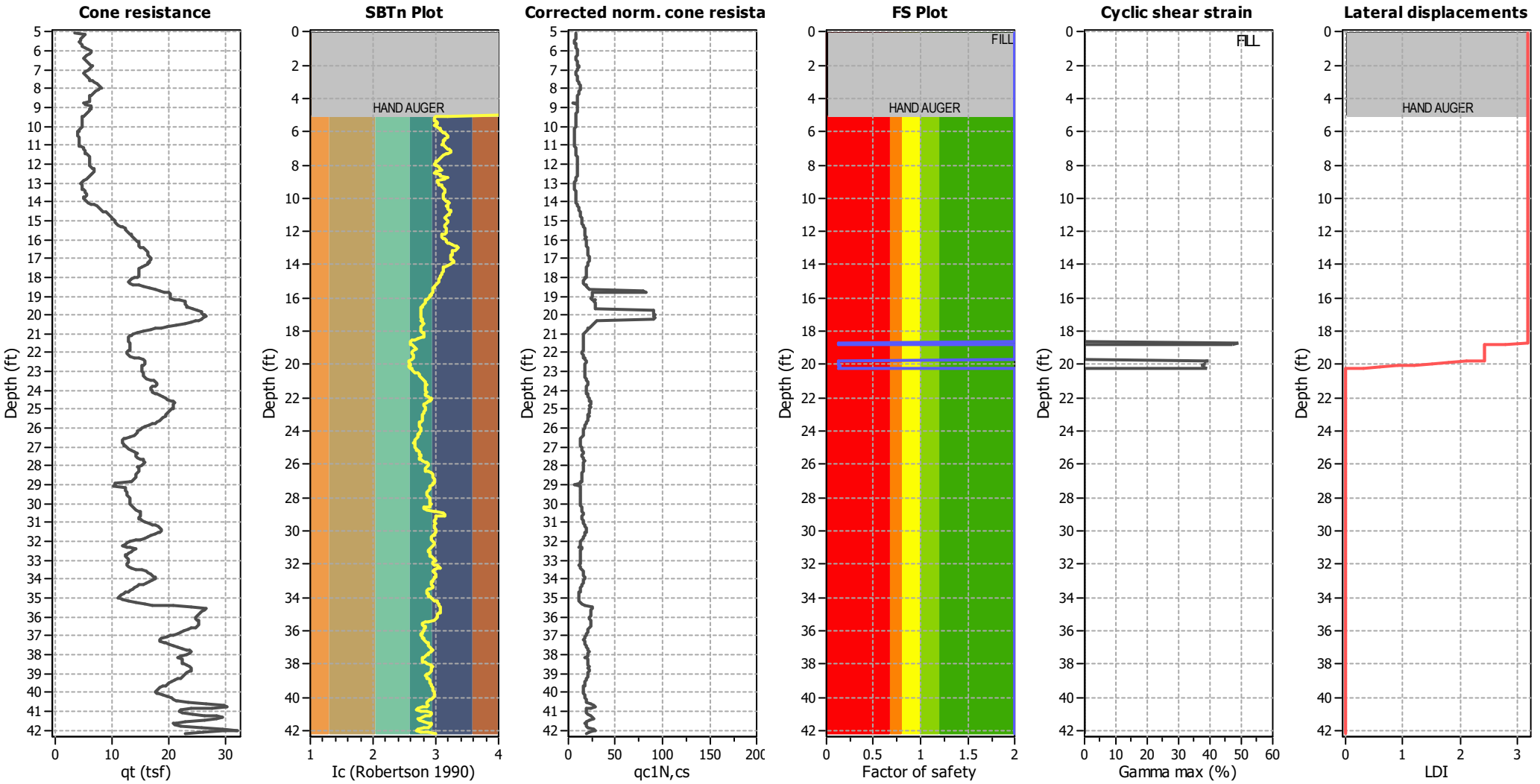
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

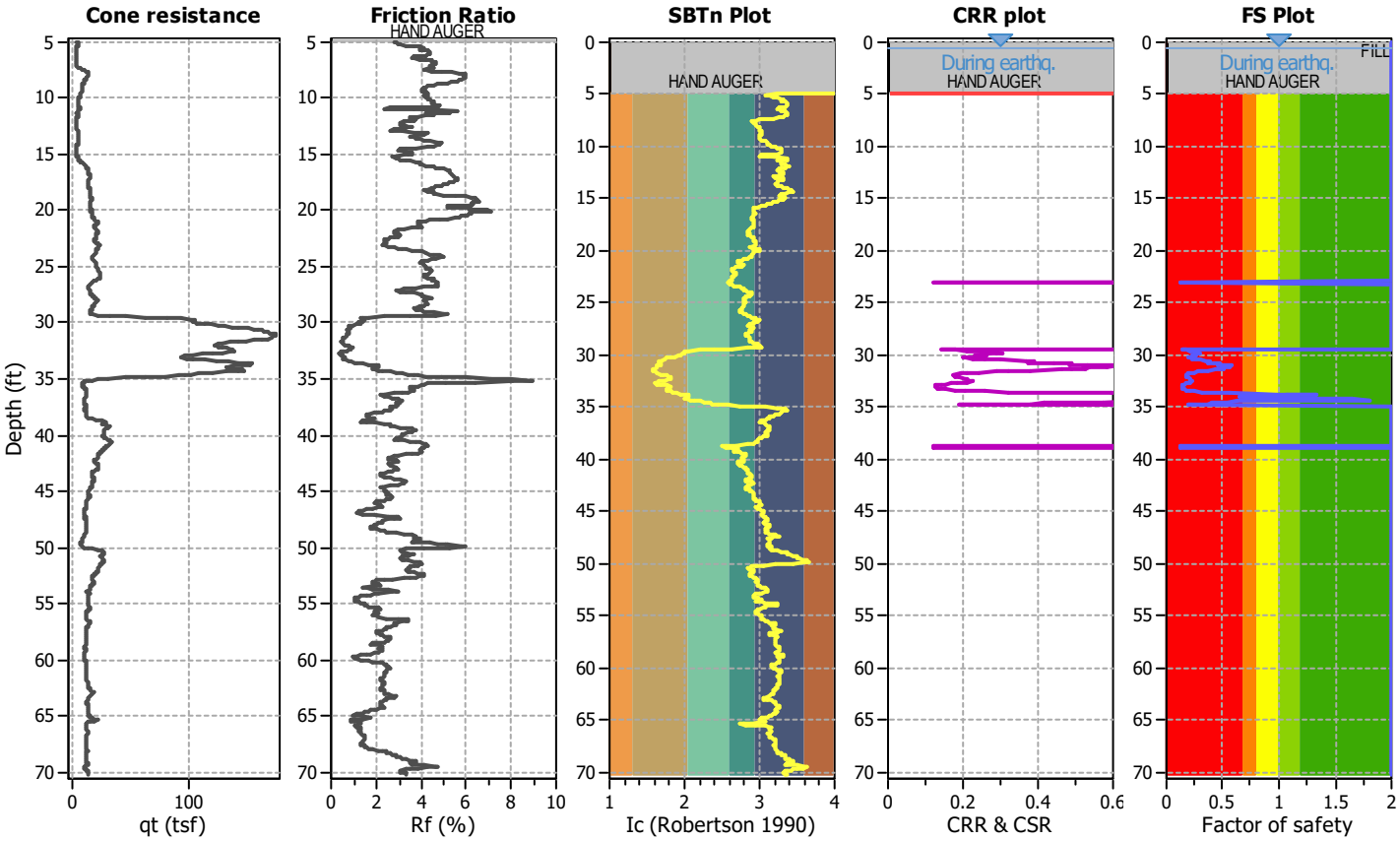
LIQUEFACTION ANALYSIS REPORT

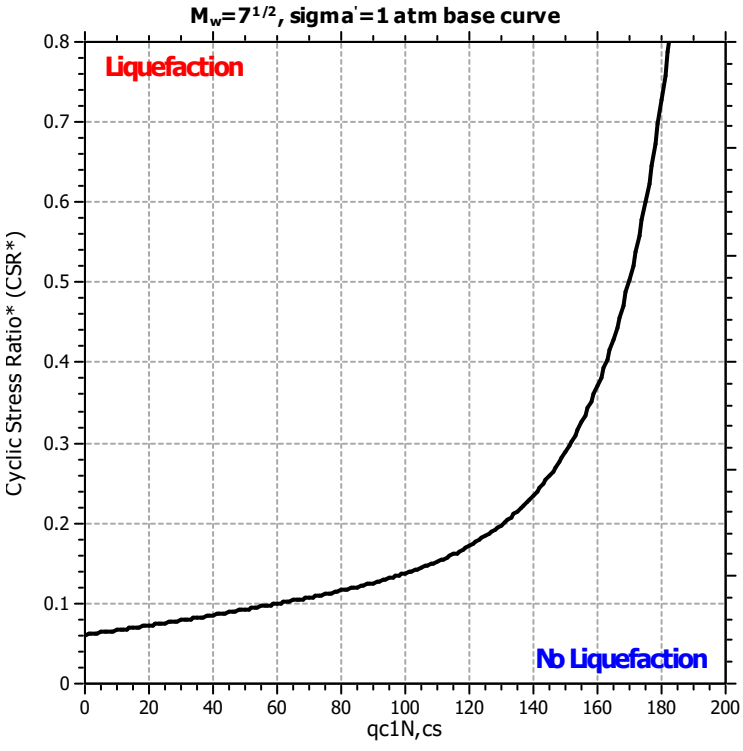
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

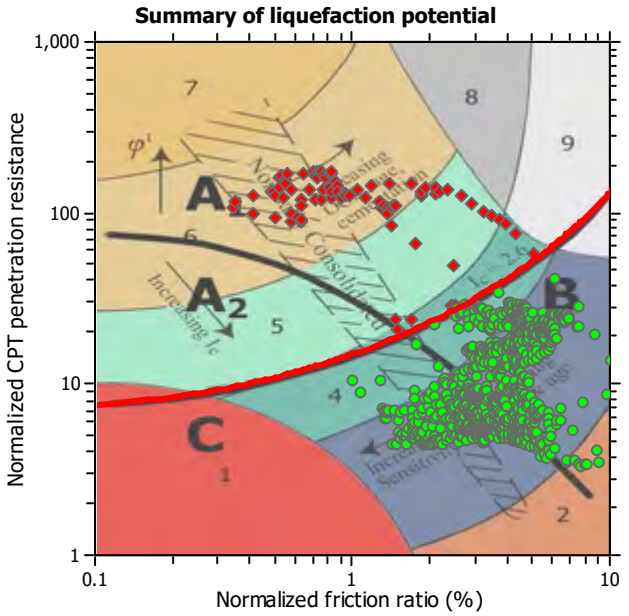
CPT file : GS26-CPT-11

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.05 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.50 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

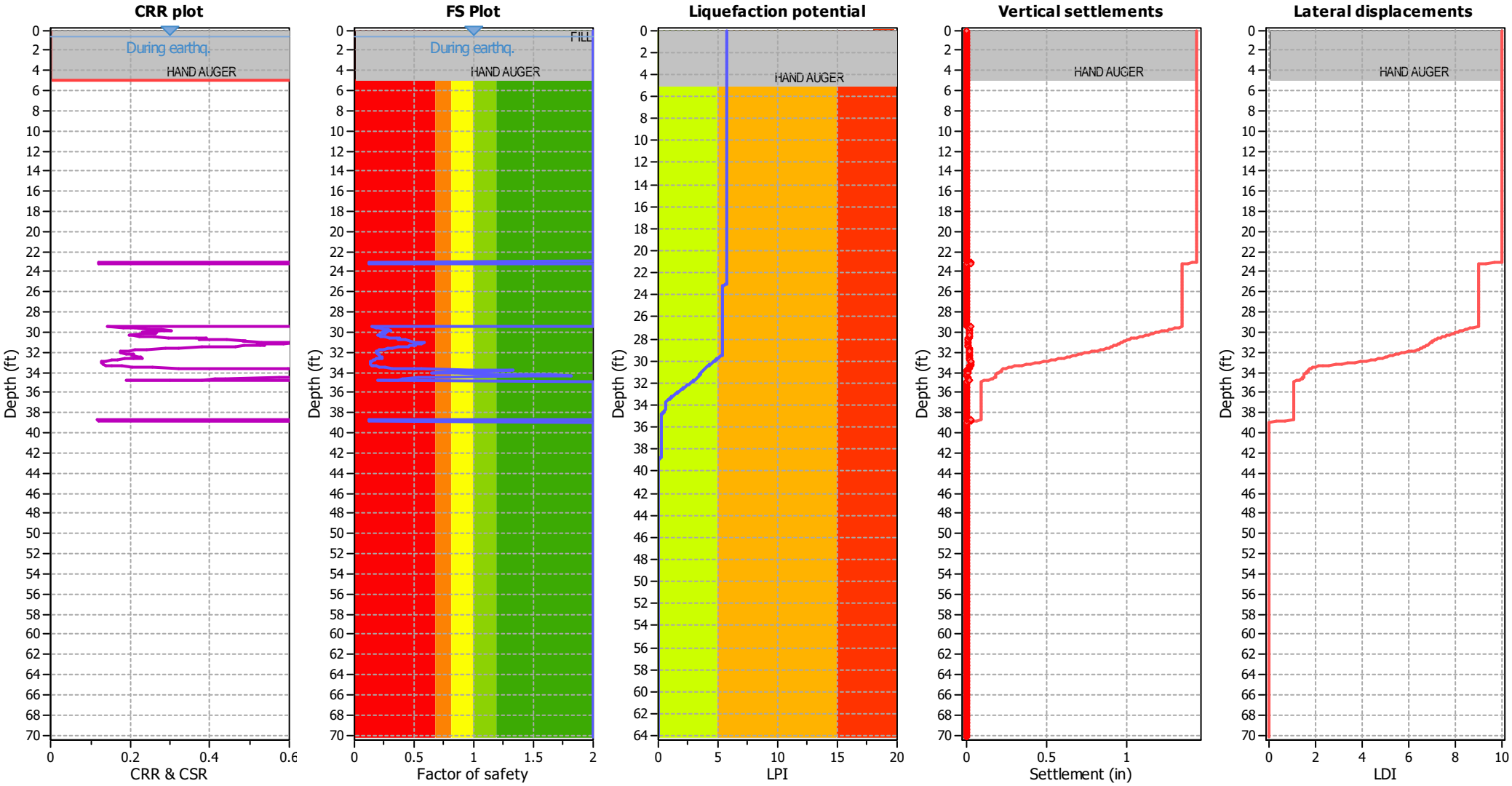






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.05 ft	Fill height:	1.50 ft	Limit depth:	N/A

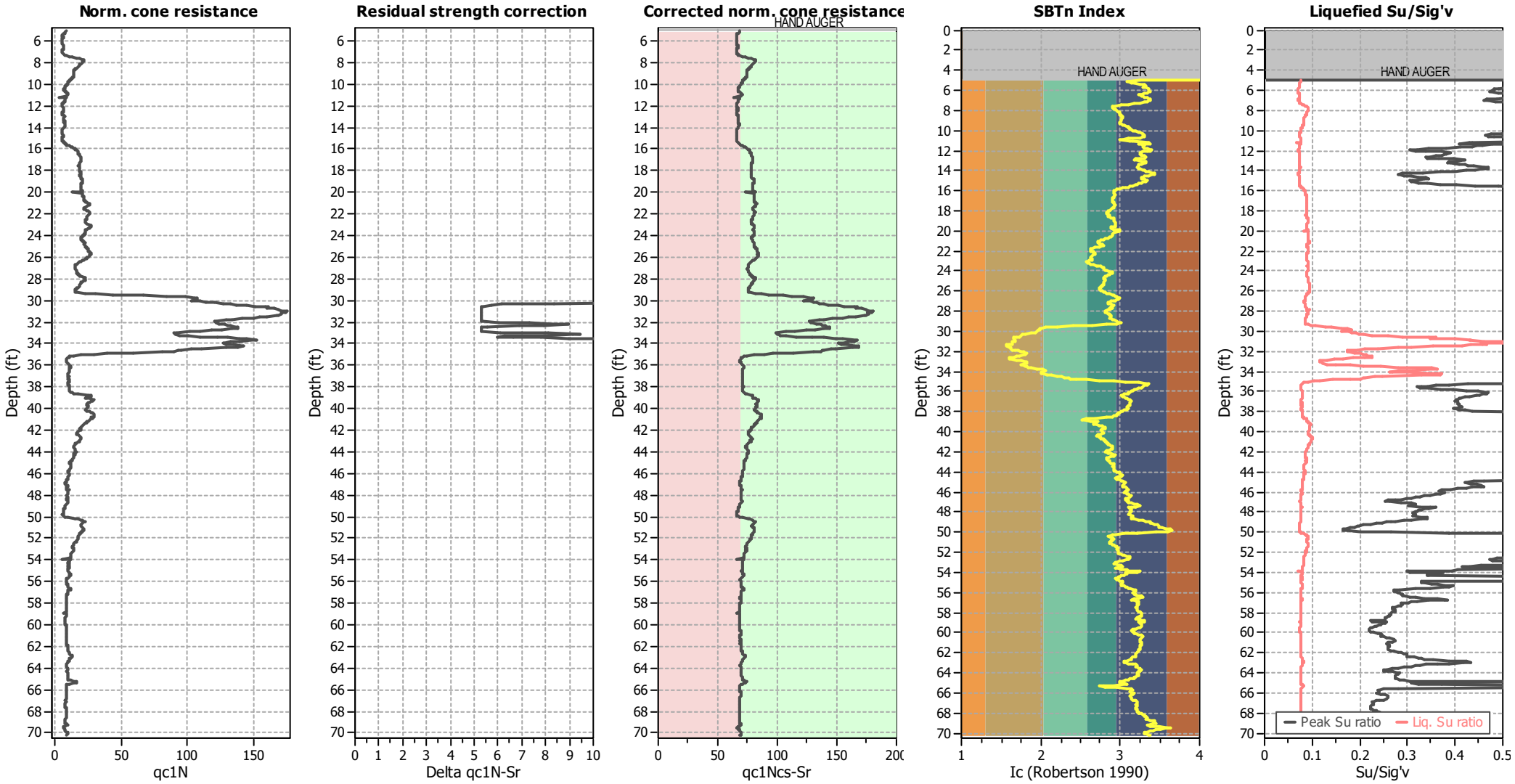
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

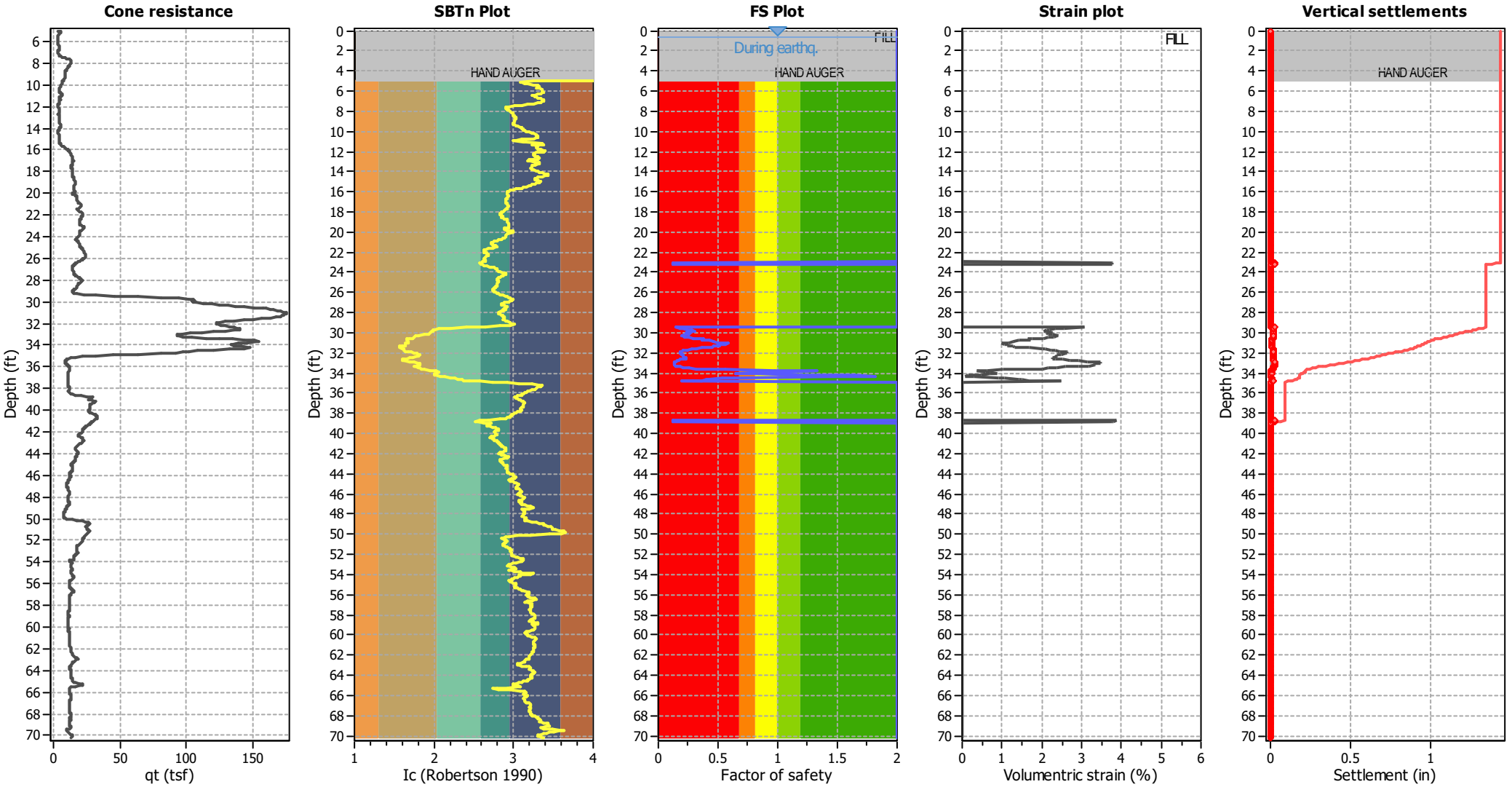
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.05 ft	Fill height:	1.50 ft	Limit depth:	N/A

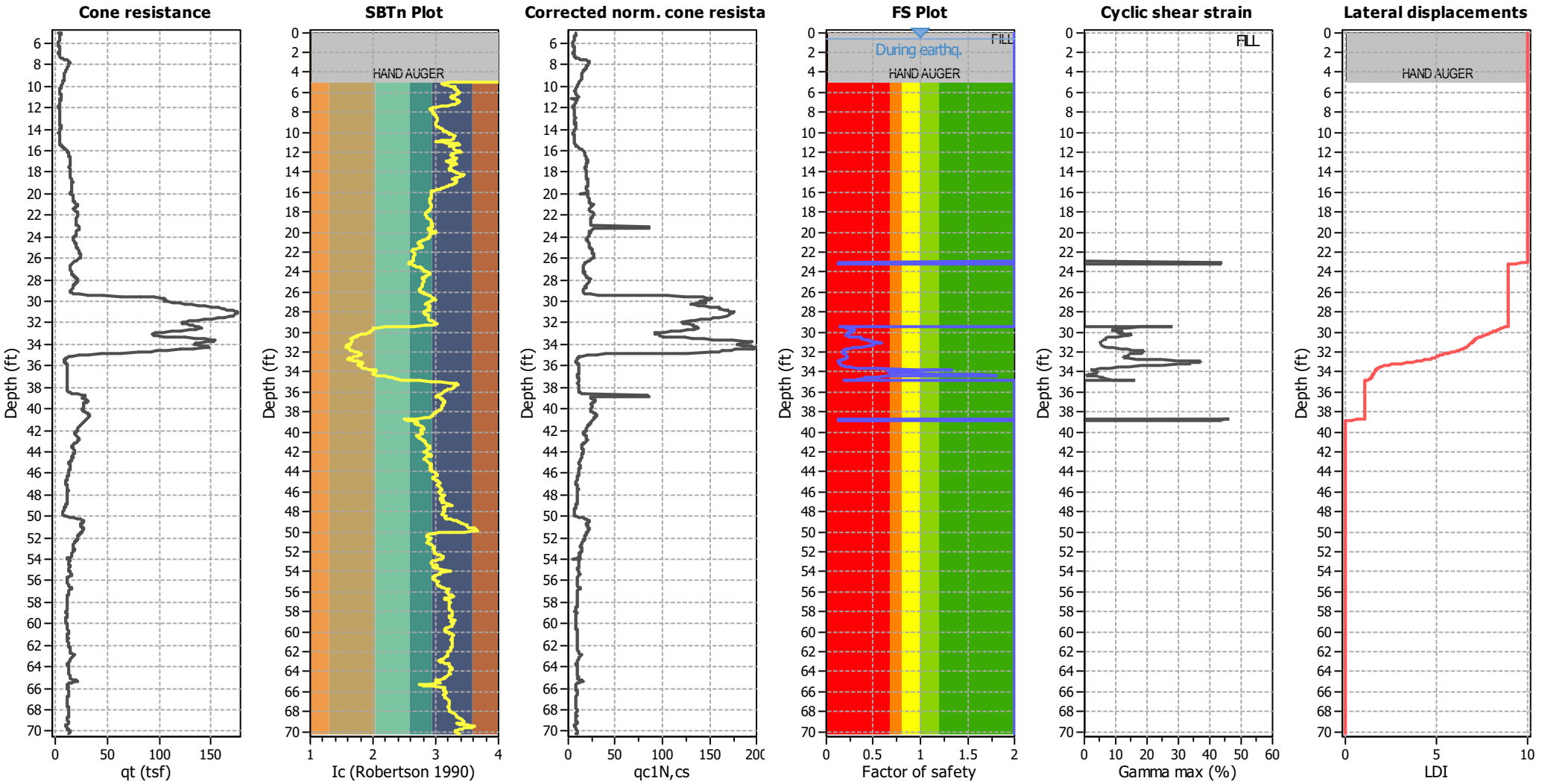
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

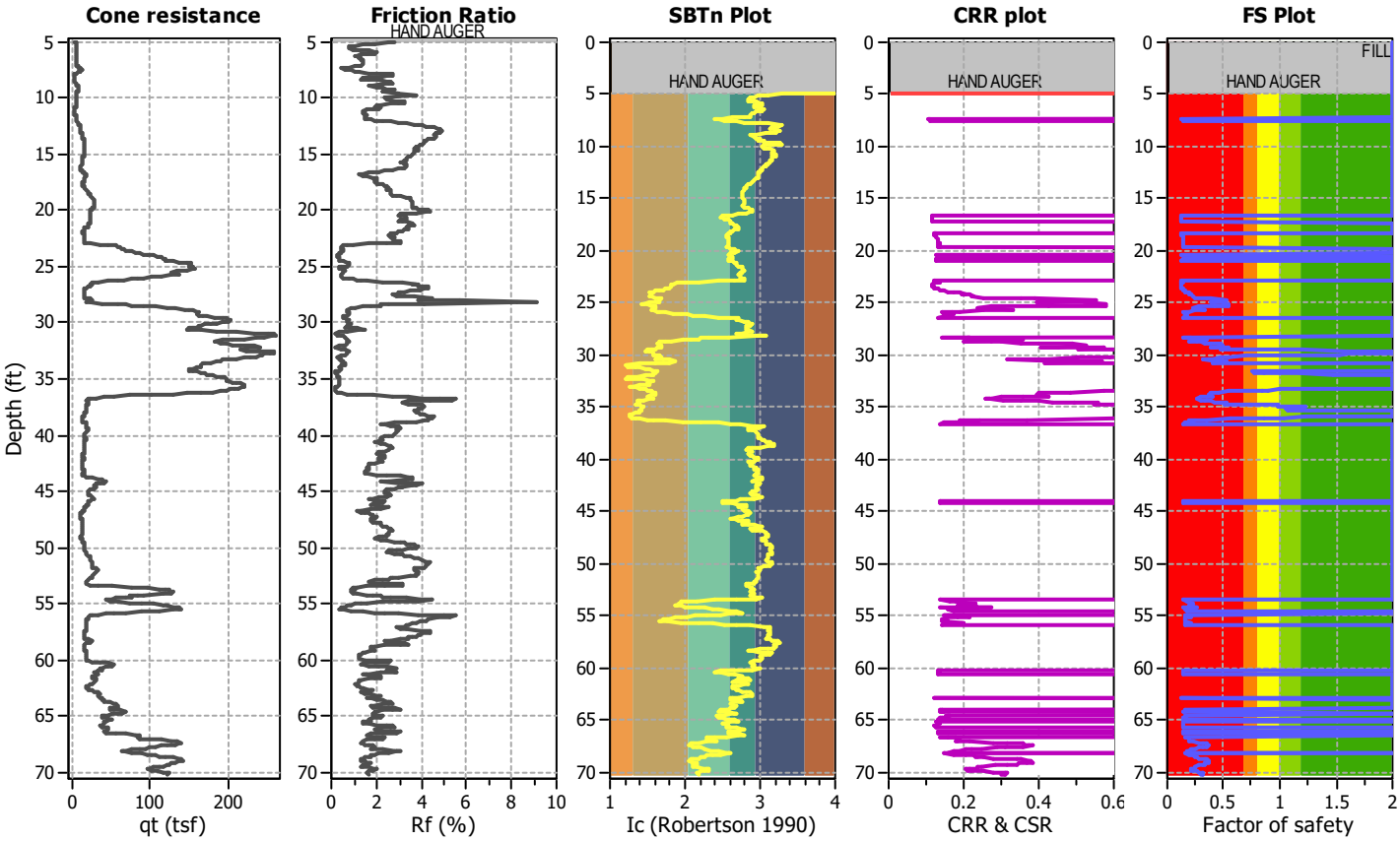
LIQUEFACTION ANALYSIS REPORT

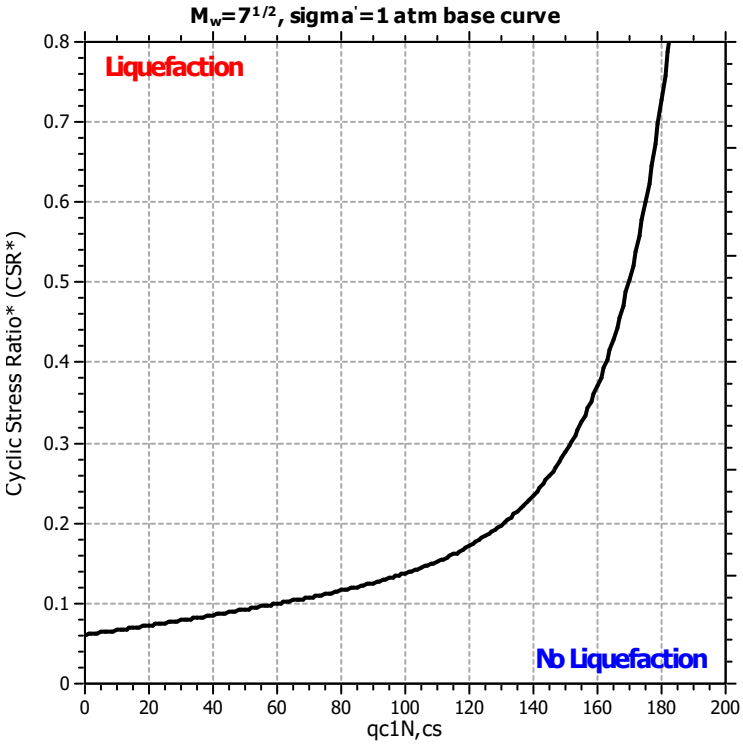
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

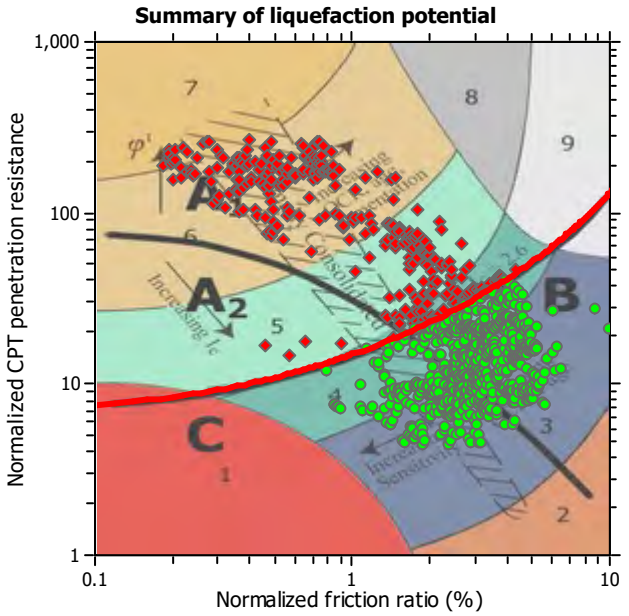
CPT file : GS26-CPT-12

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	3.02 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	5.40 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

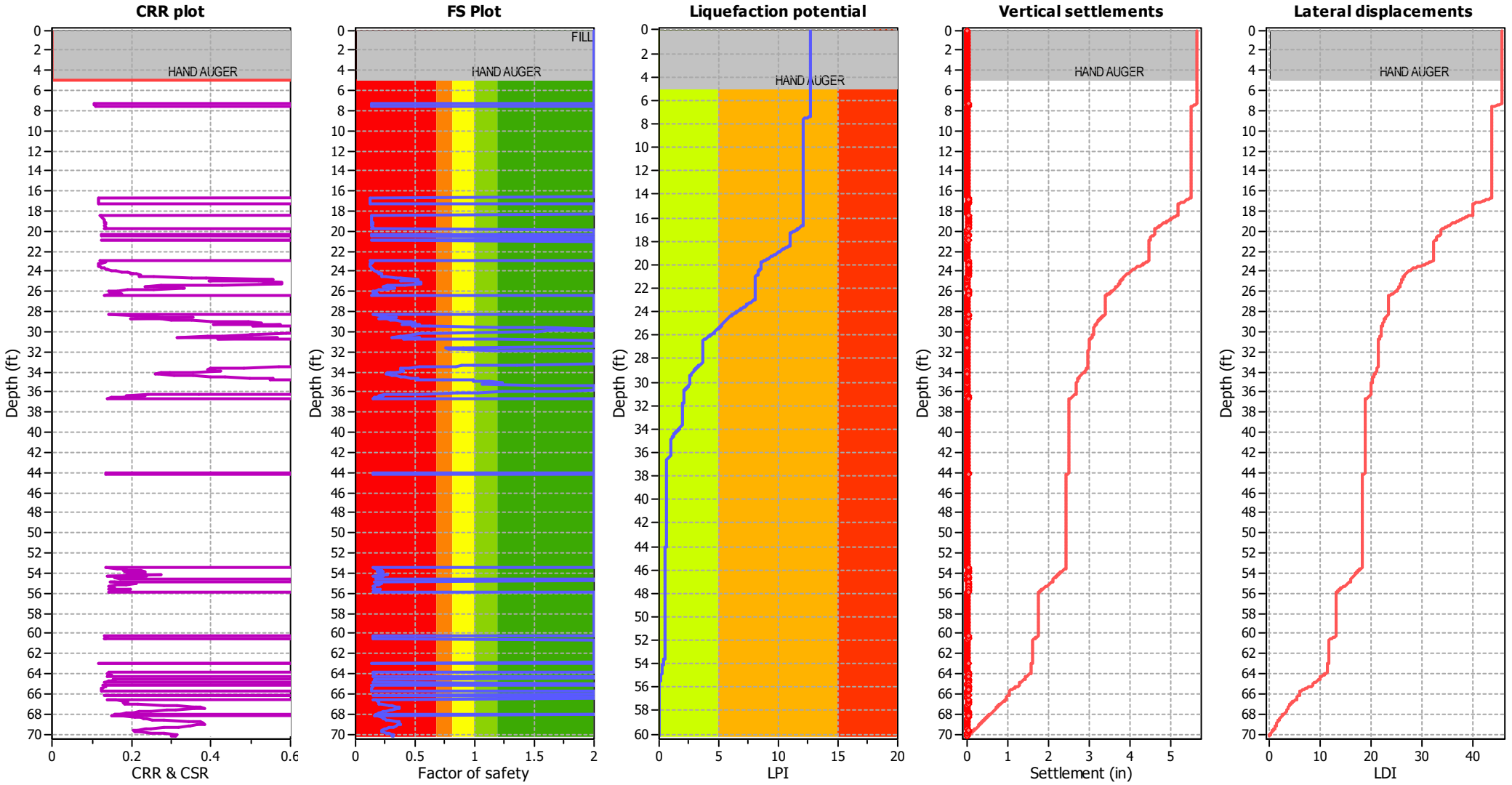






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	3.02 ft	Fill height:	5.40 ft	Limit depth:	N/A

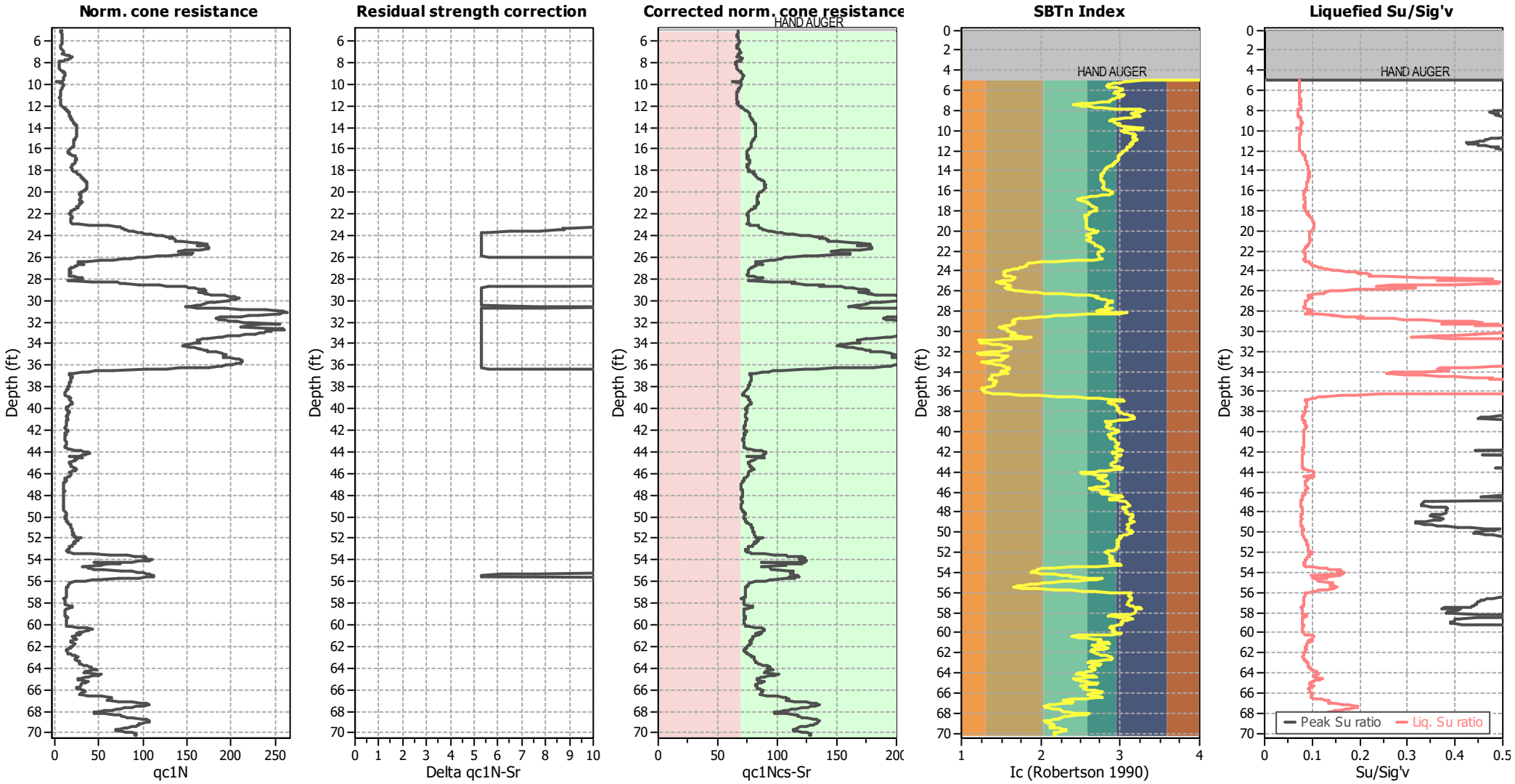
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

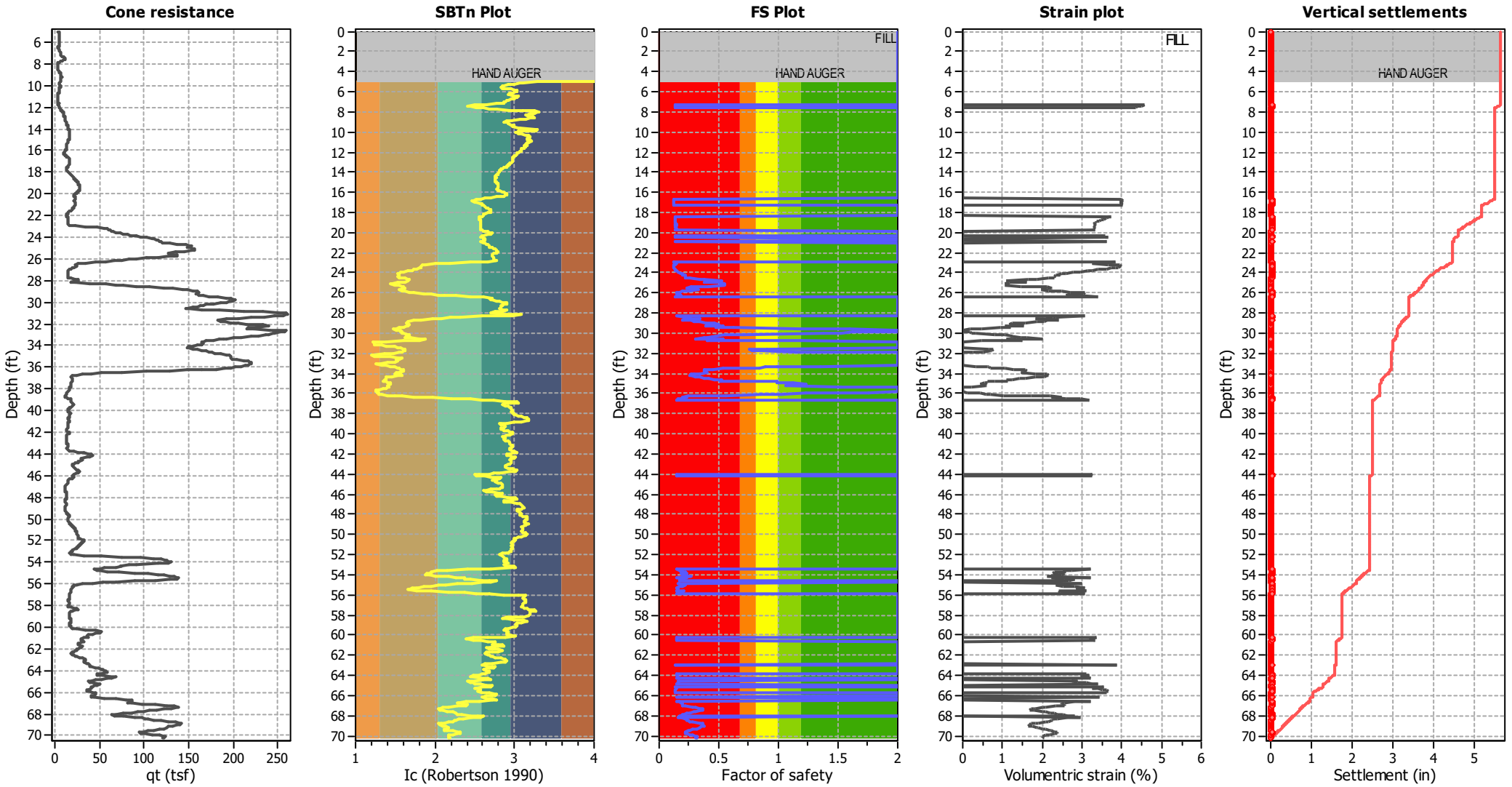
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	3.02 ft	Fill height:	5.40 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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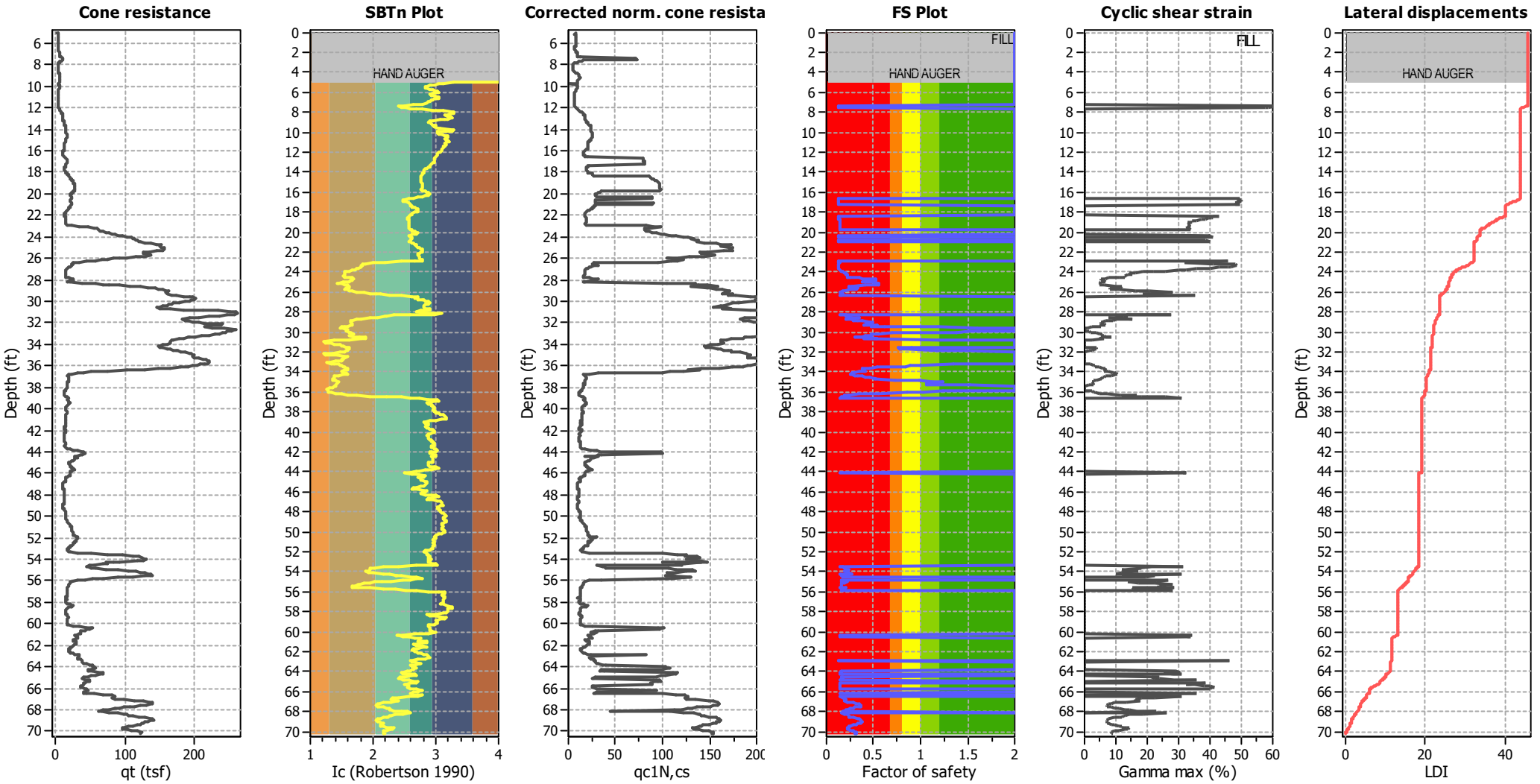
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
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$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

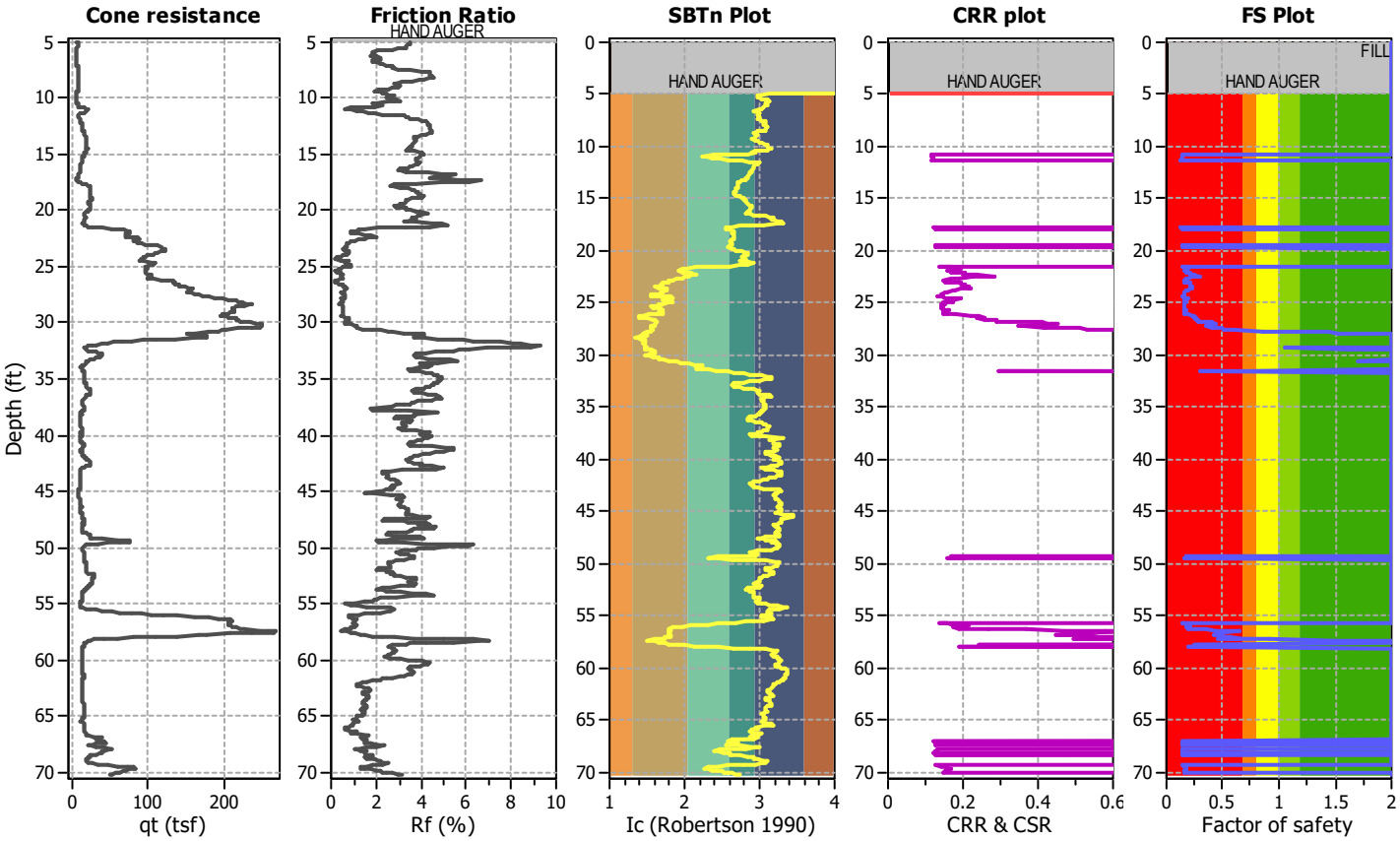
LIQUEFACTION ANALYSIS REPORT

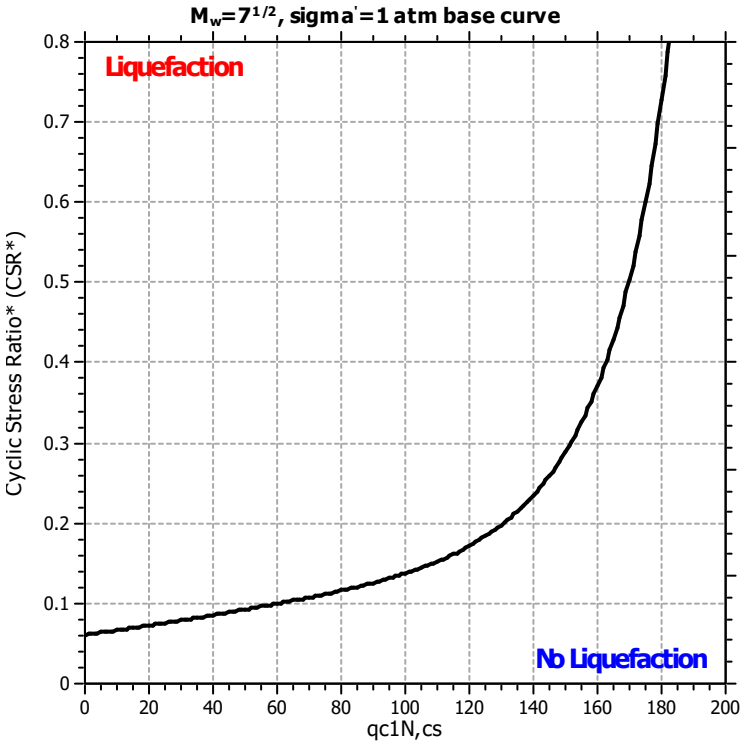
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

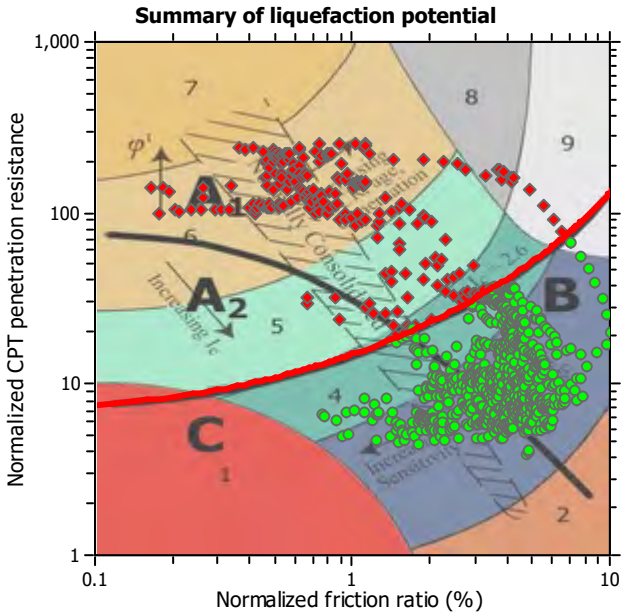
CPT file : GS26-CPT-13

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.51 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	5.80 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

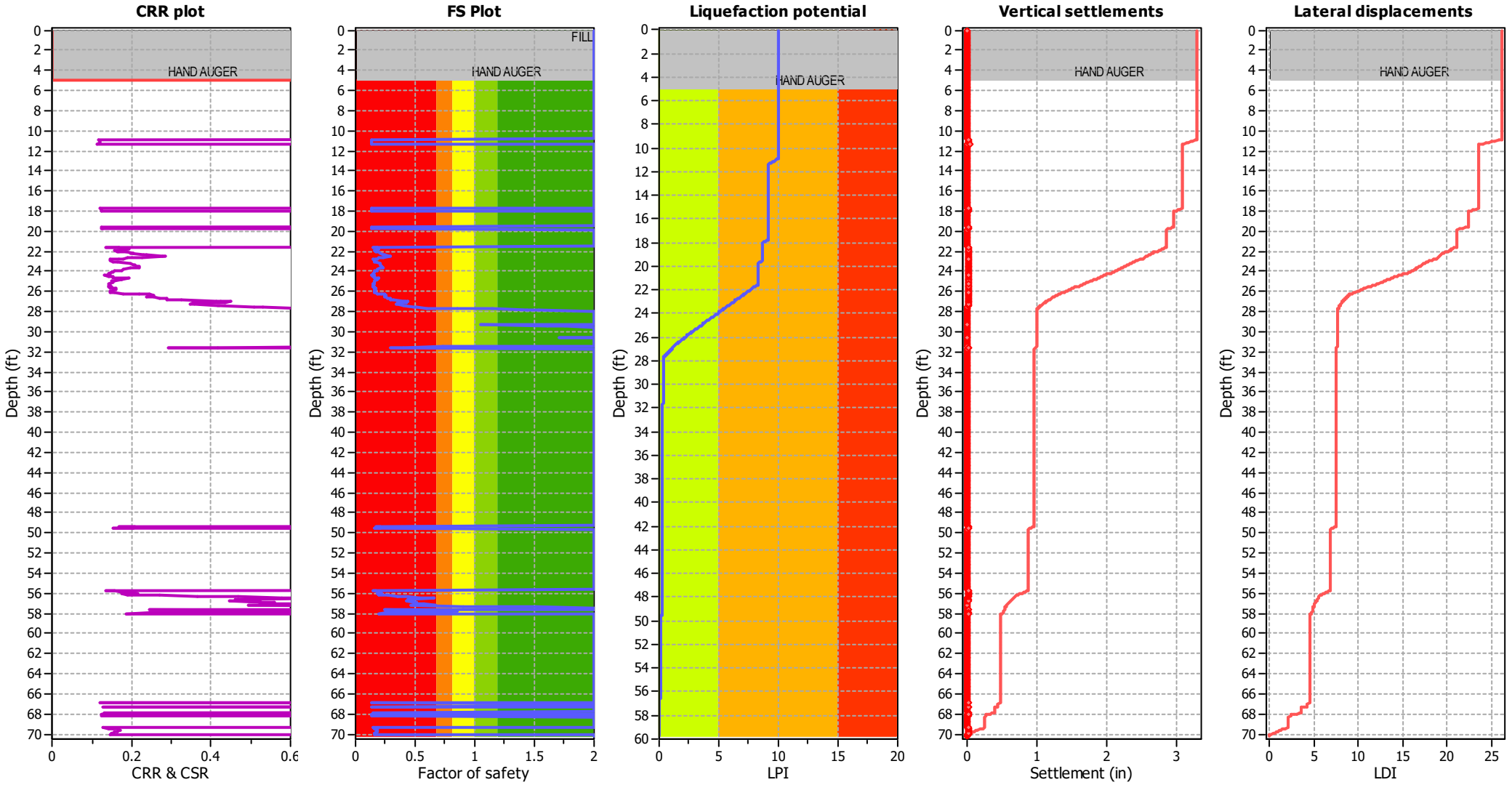






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.51 ft	Fill height:	5.80 ft	Limit depth:	N/A

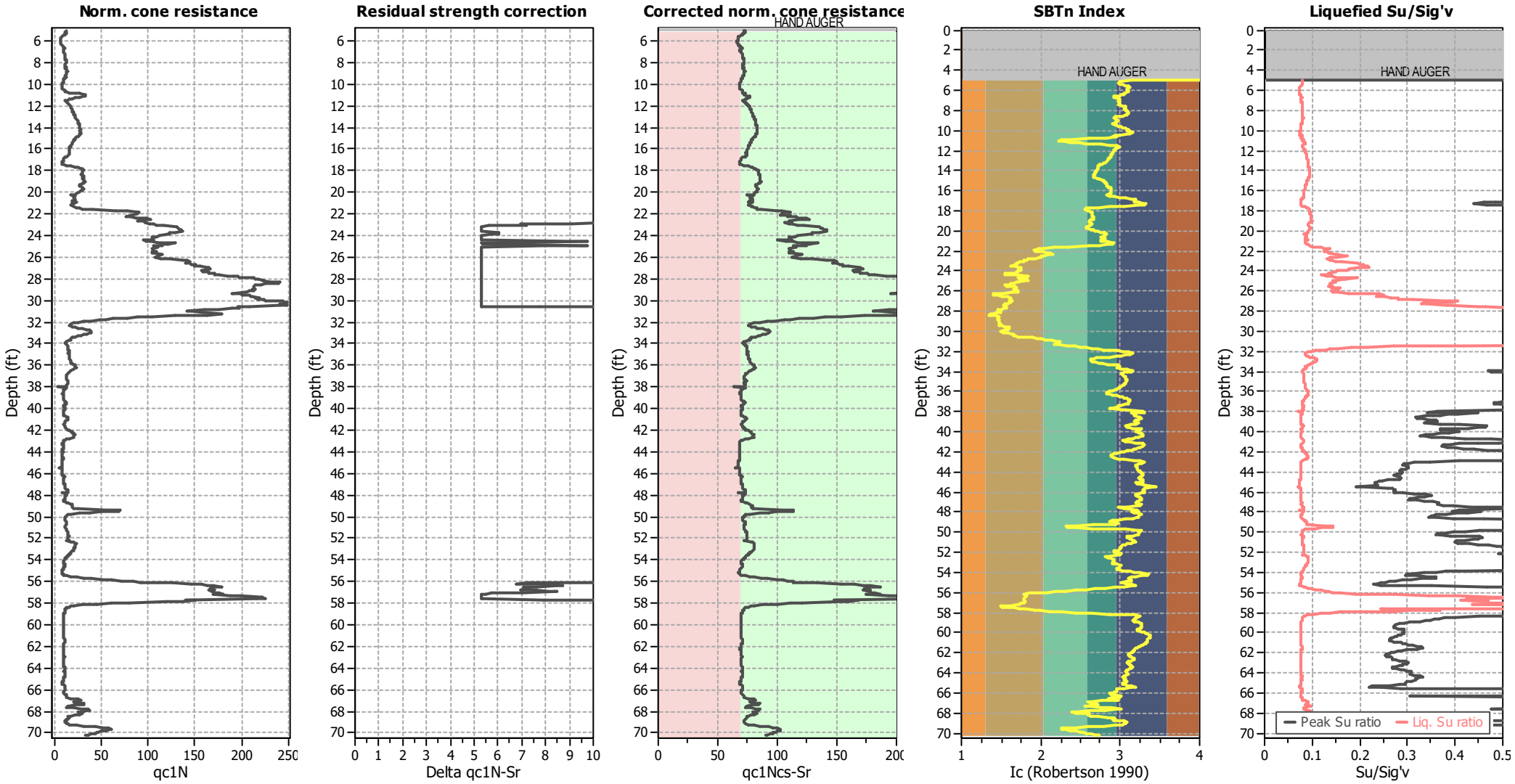
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

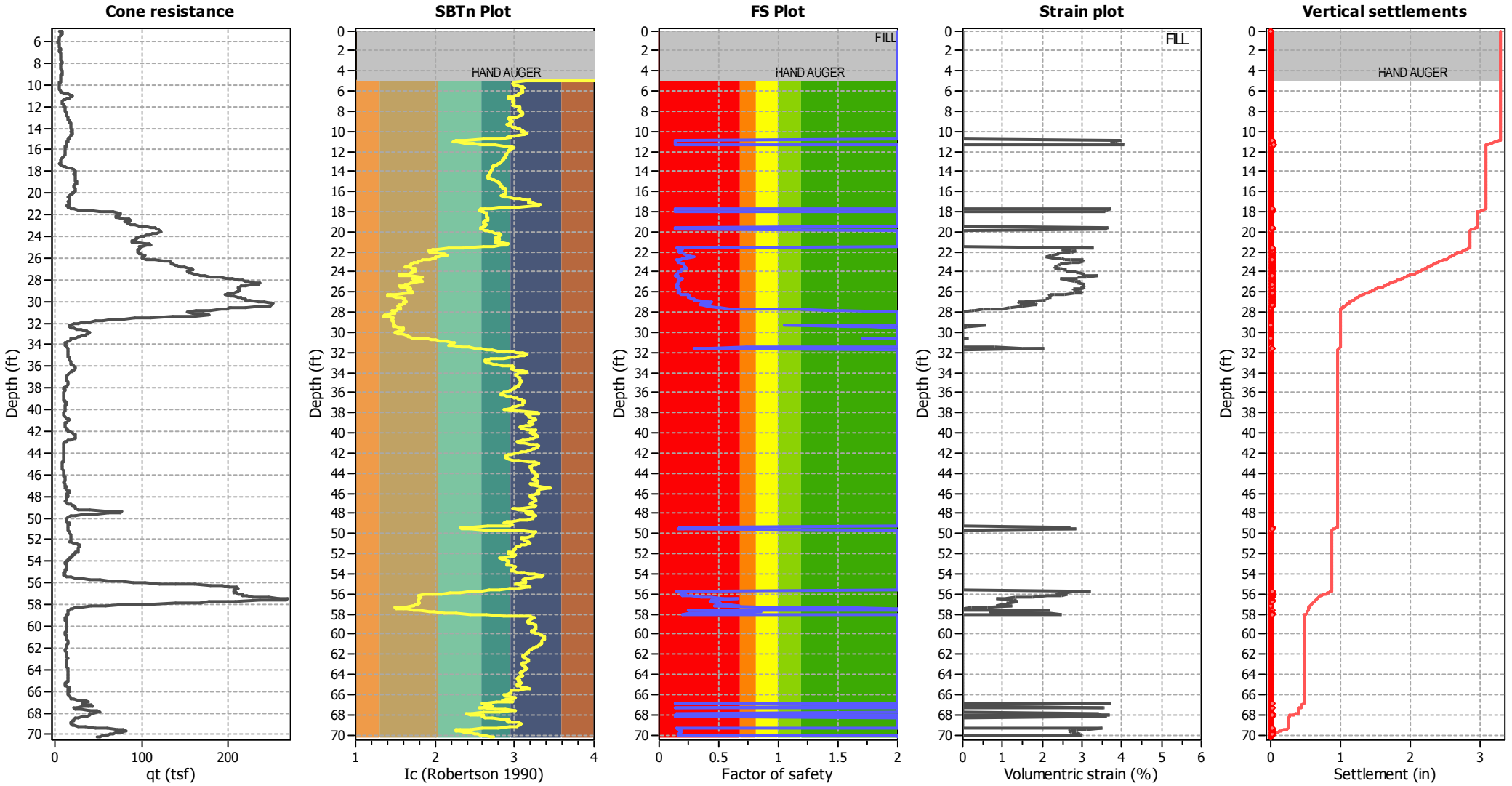
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.51 ft	Fill height:	5.80 ft	Limit depth:	N/A

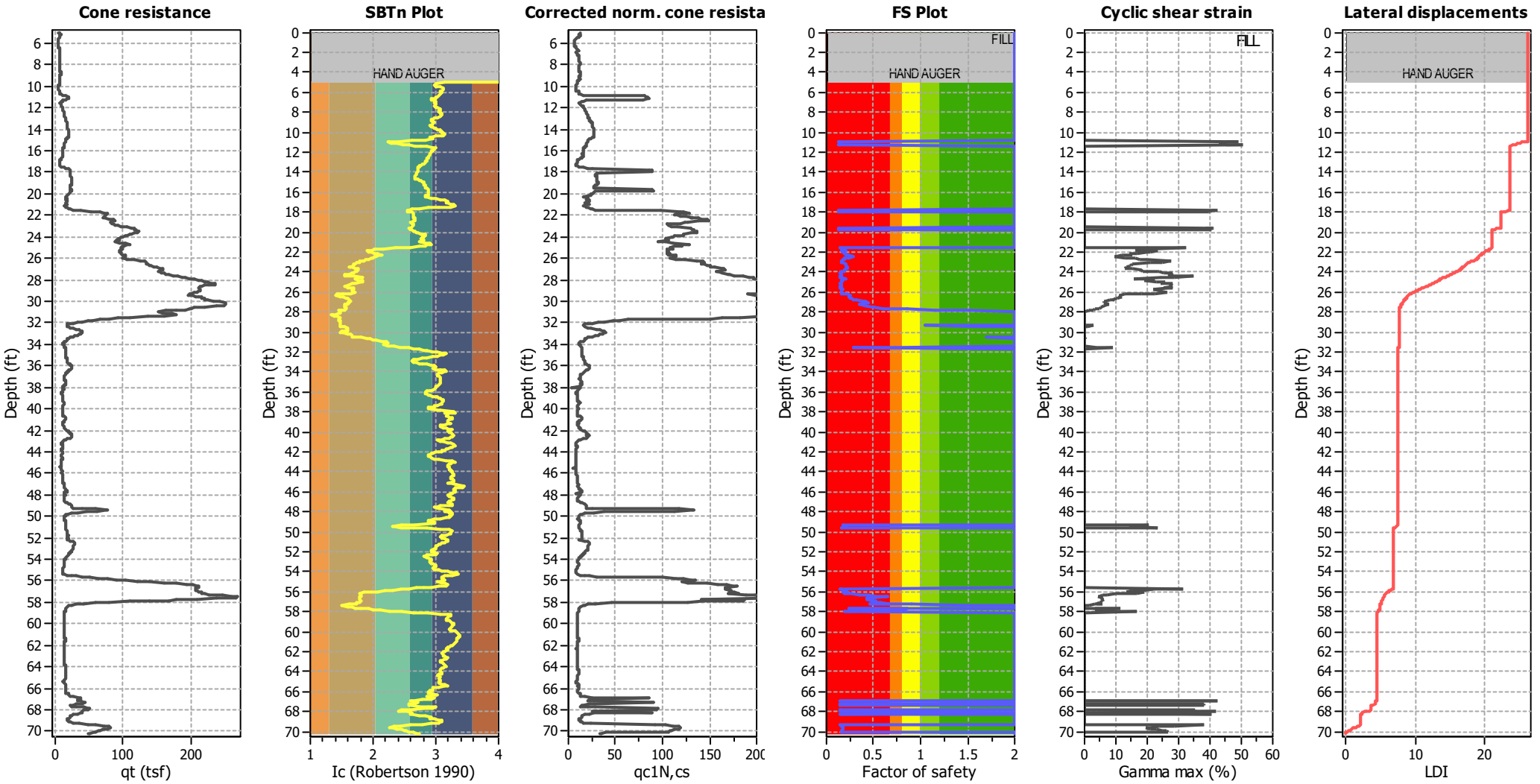
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

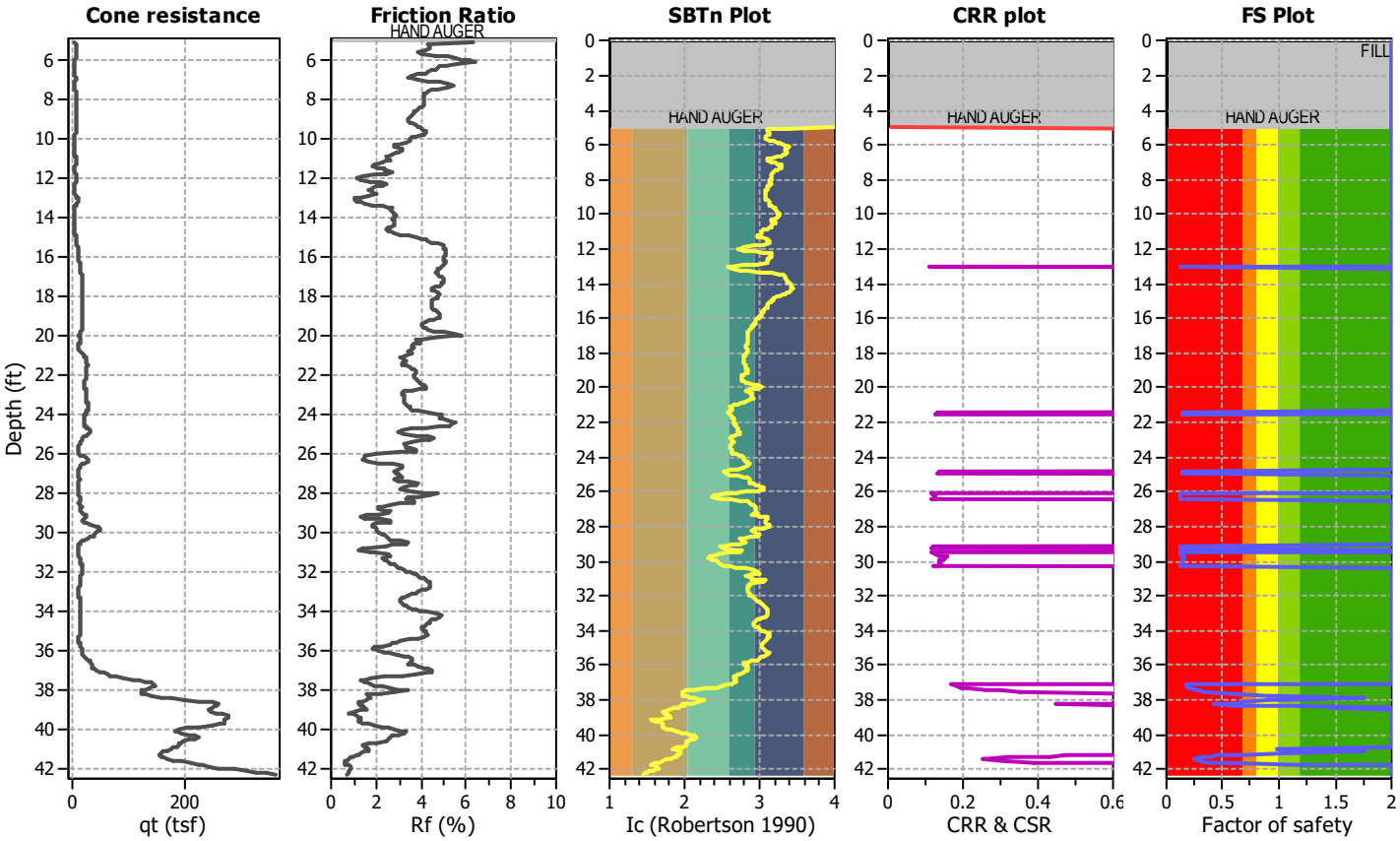
LIQUEFACTION ANALYSIS REPORT

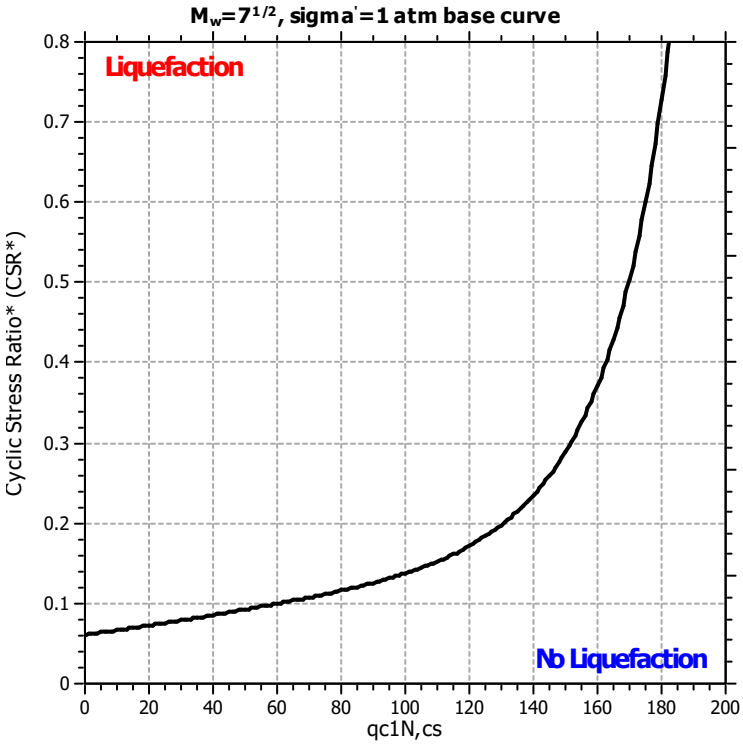
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

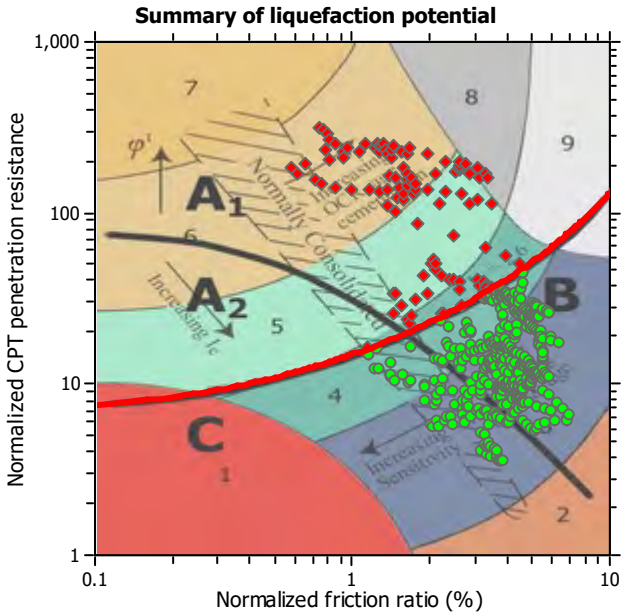
CPT file : GS26-CPT-14

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	4.72 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	3.30 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

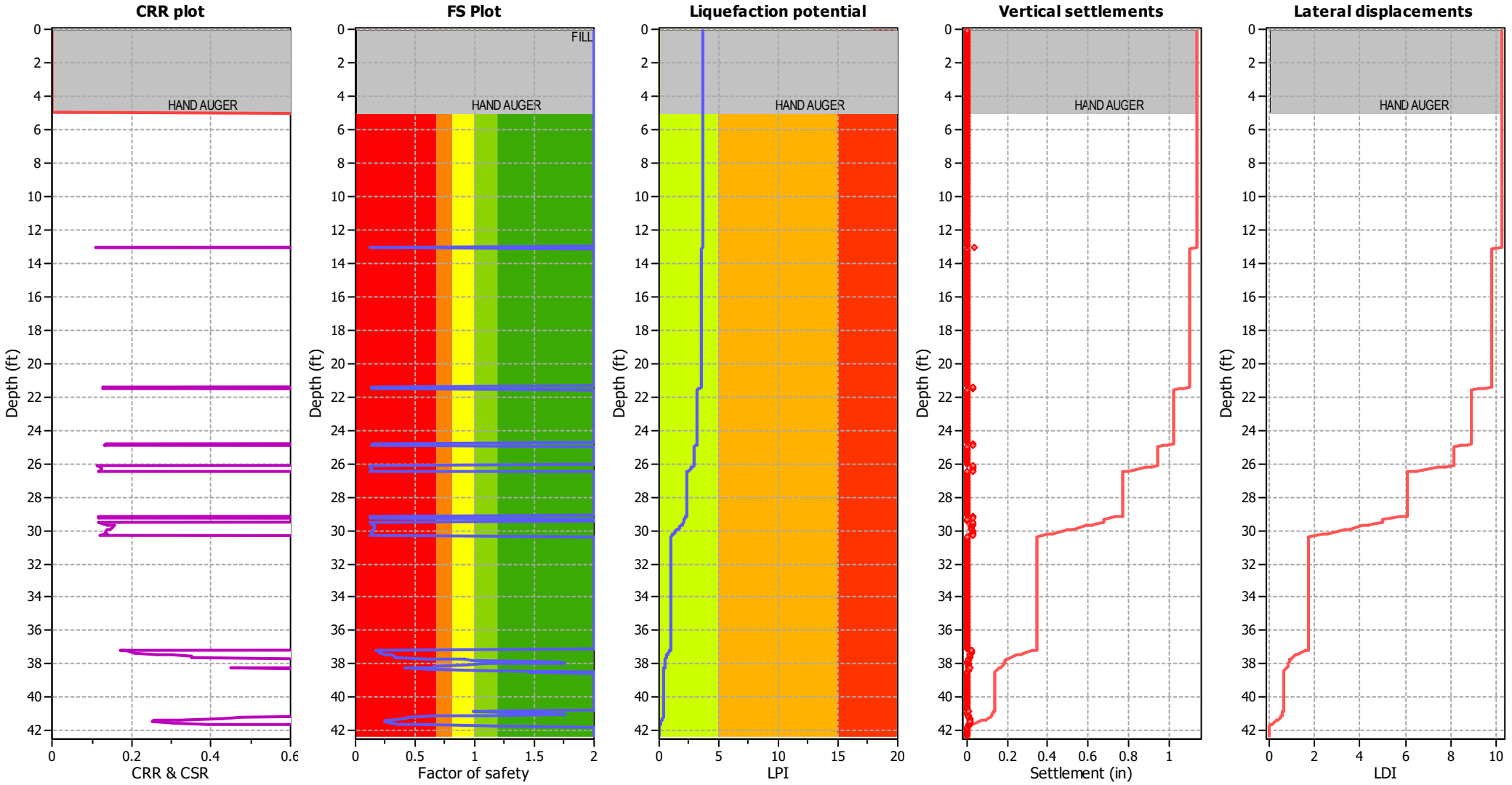






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.72 ft	Fill height:	3.30 ft	Limit depth:	N/A

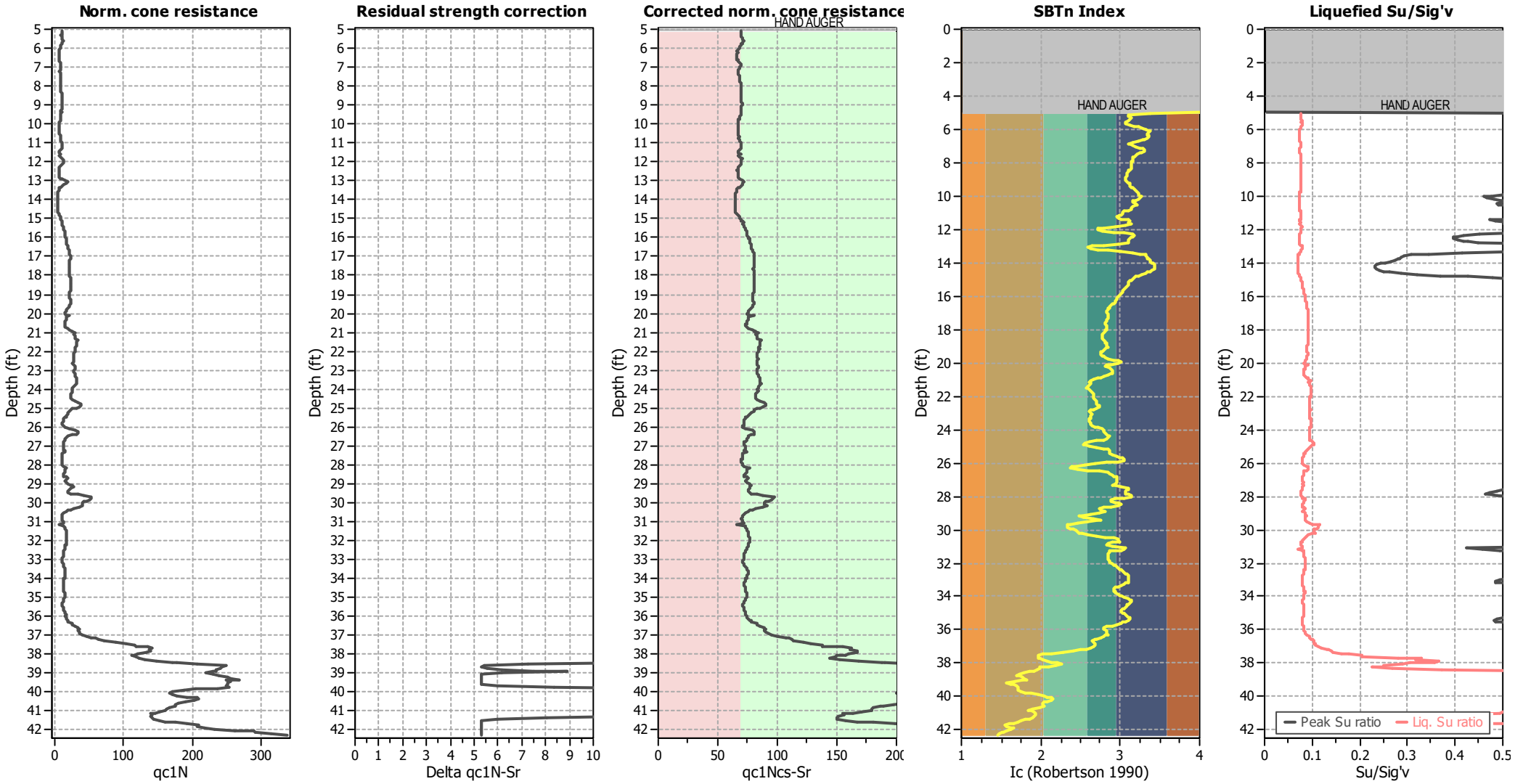
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

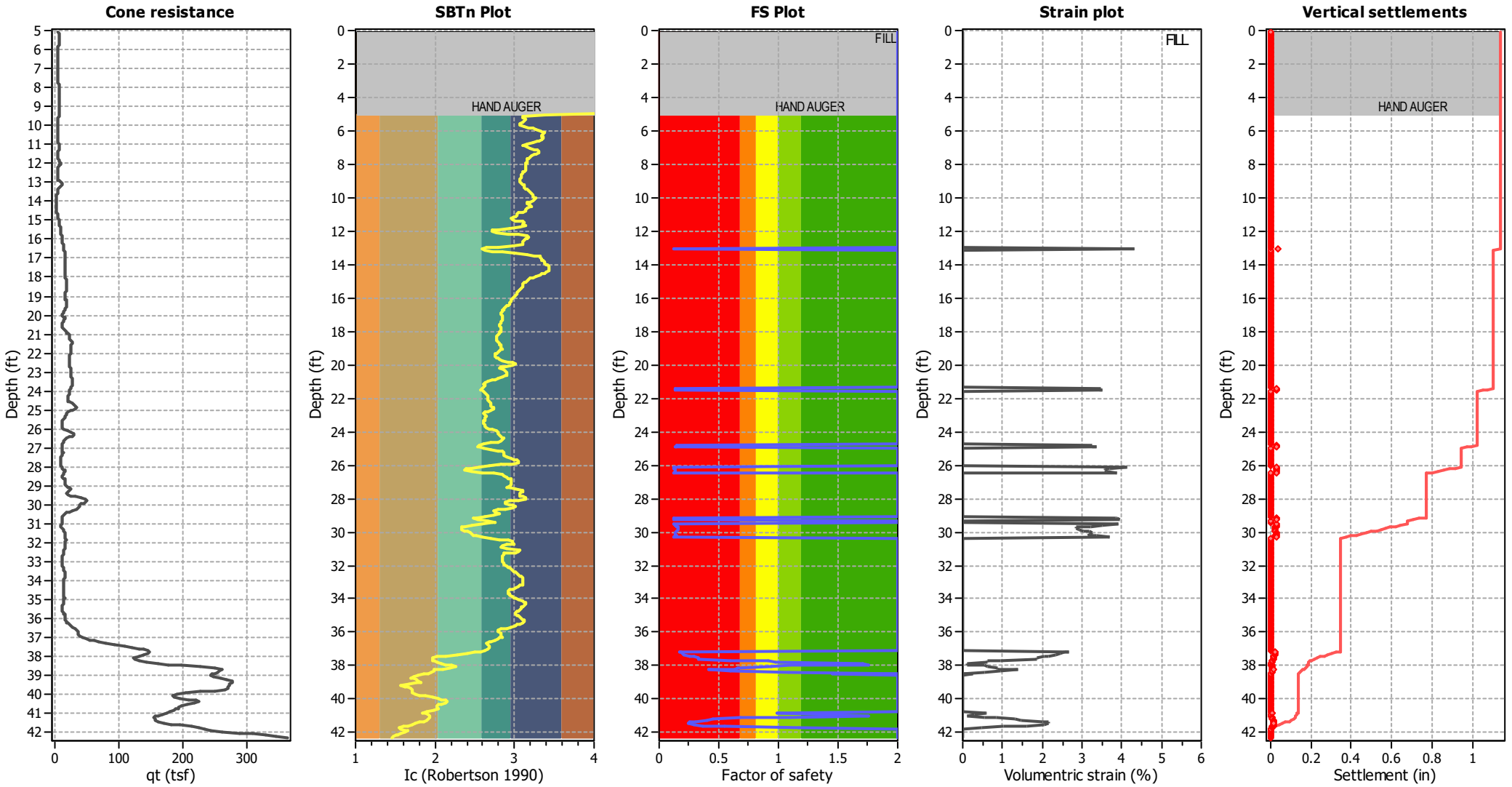
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	4.72 ft	Fill height:	3.30 ft	Limit depth:	N/A

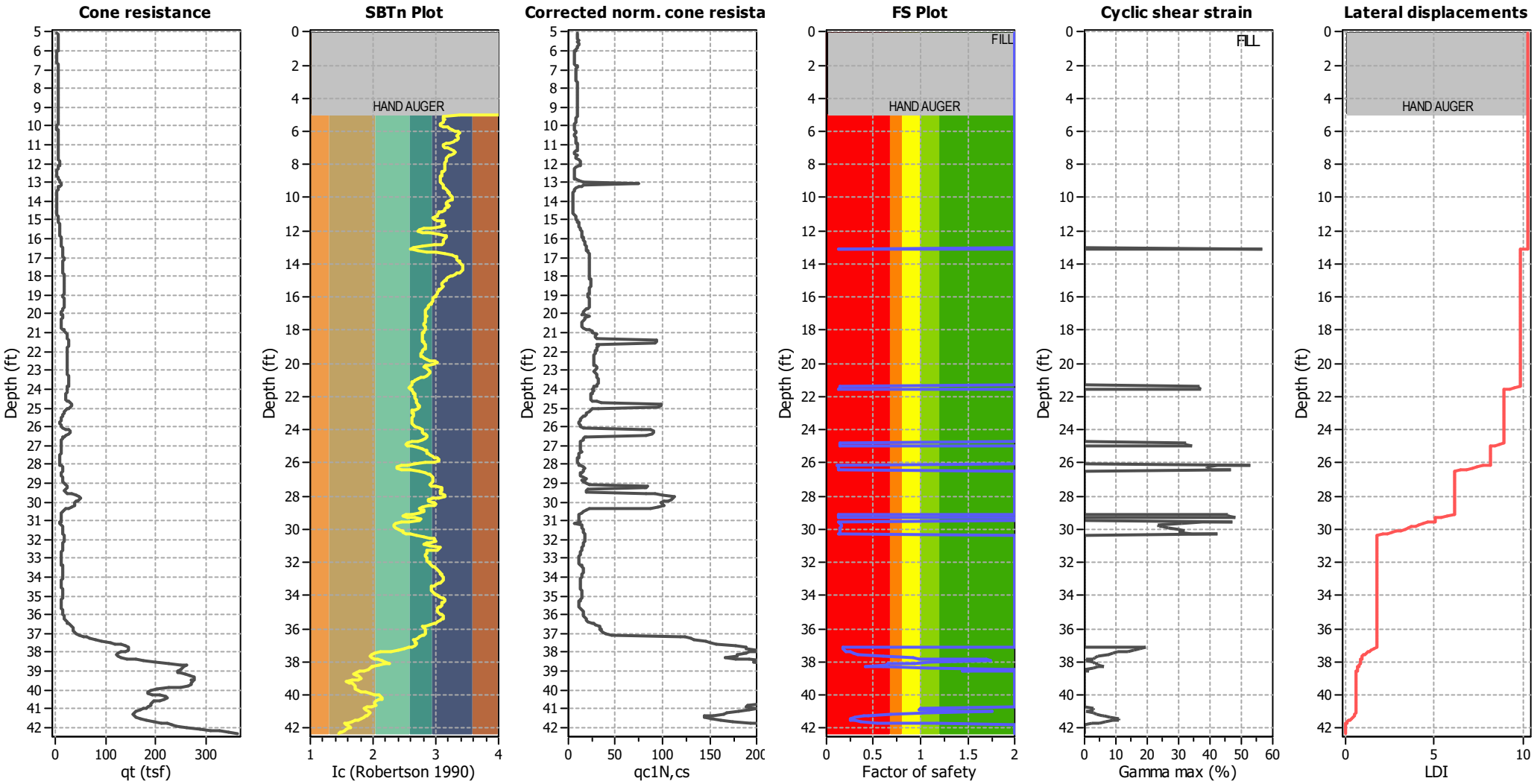
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

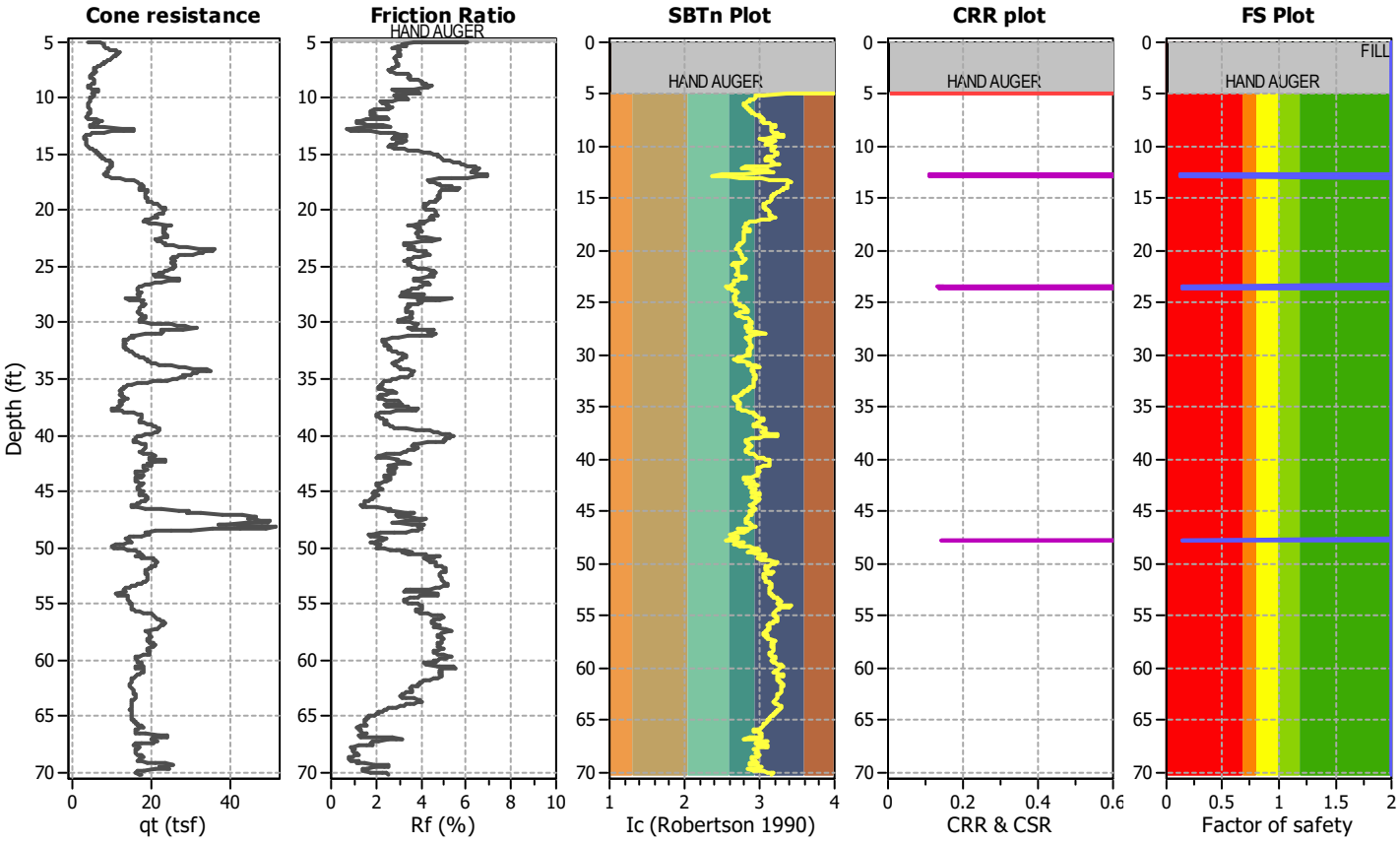
LIQUEFACTION ANALYSIS REPORT

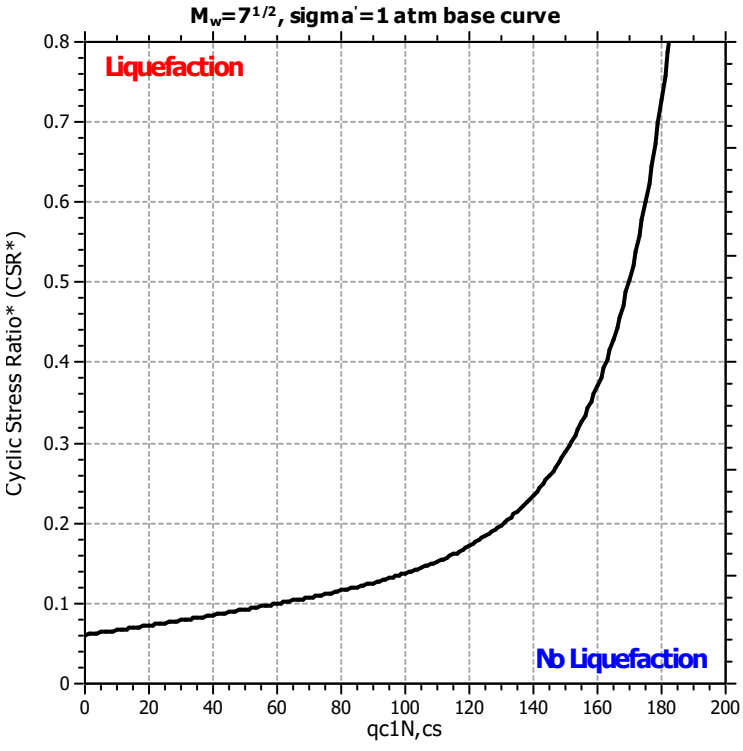
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

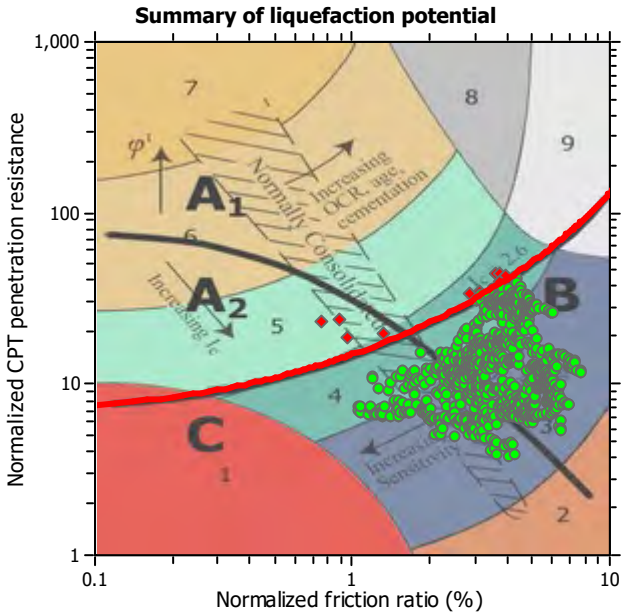
CPT file : GS26-CPT-15

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	5.96 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	3.20 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

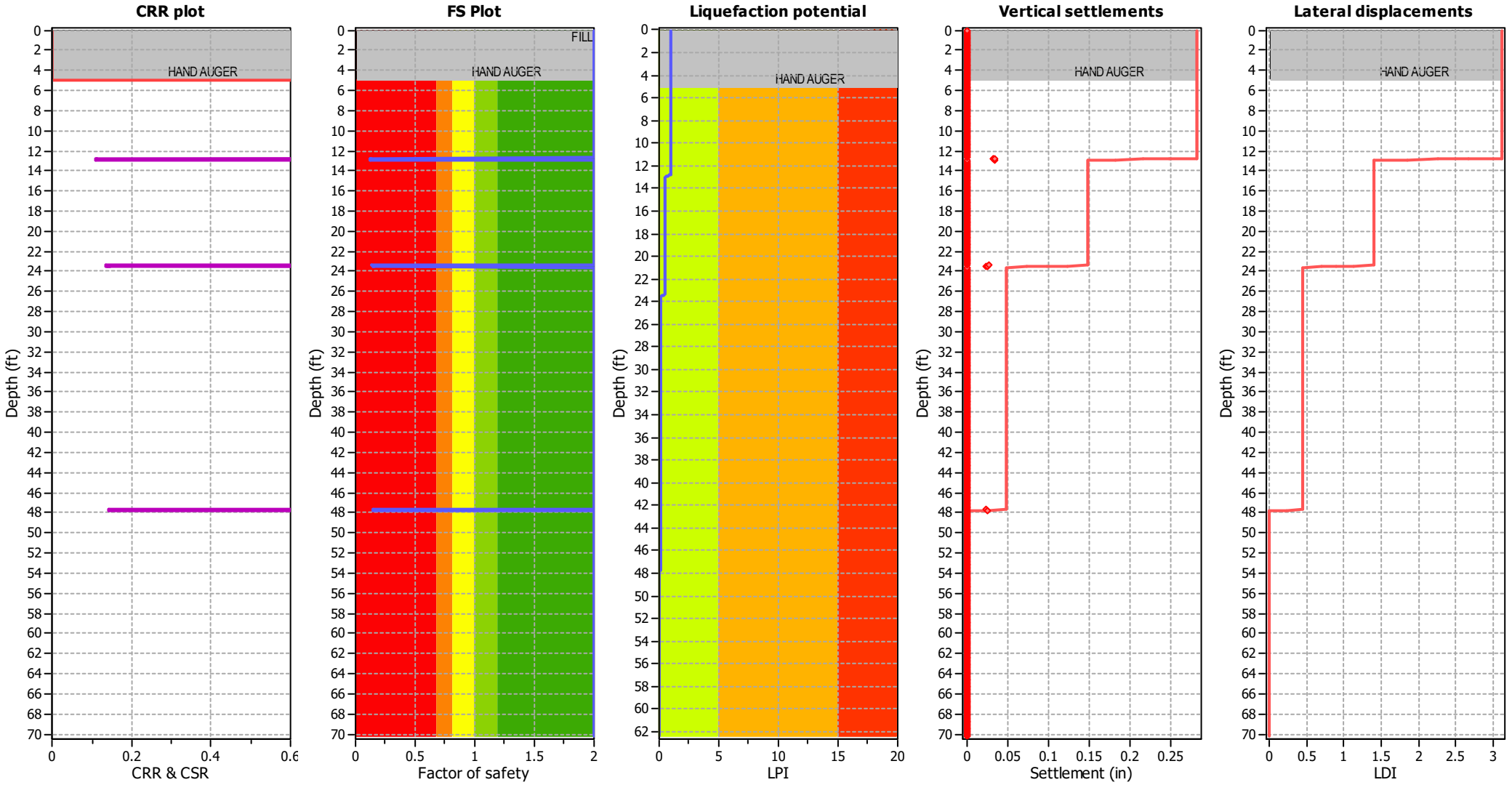






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.96 ft	Fill height:	3.20 ft	Limit depth:	N/A

F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
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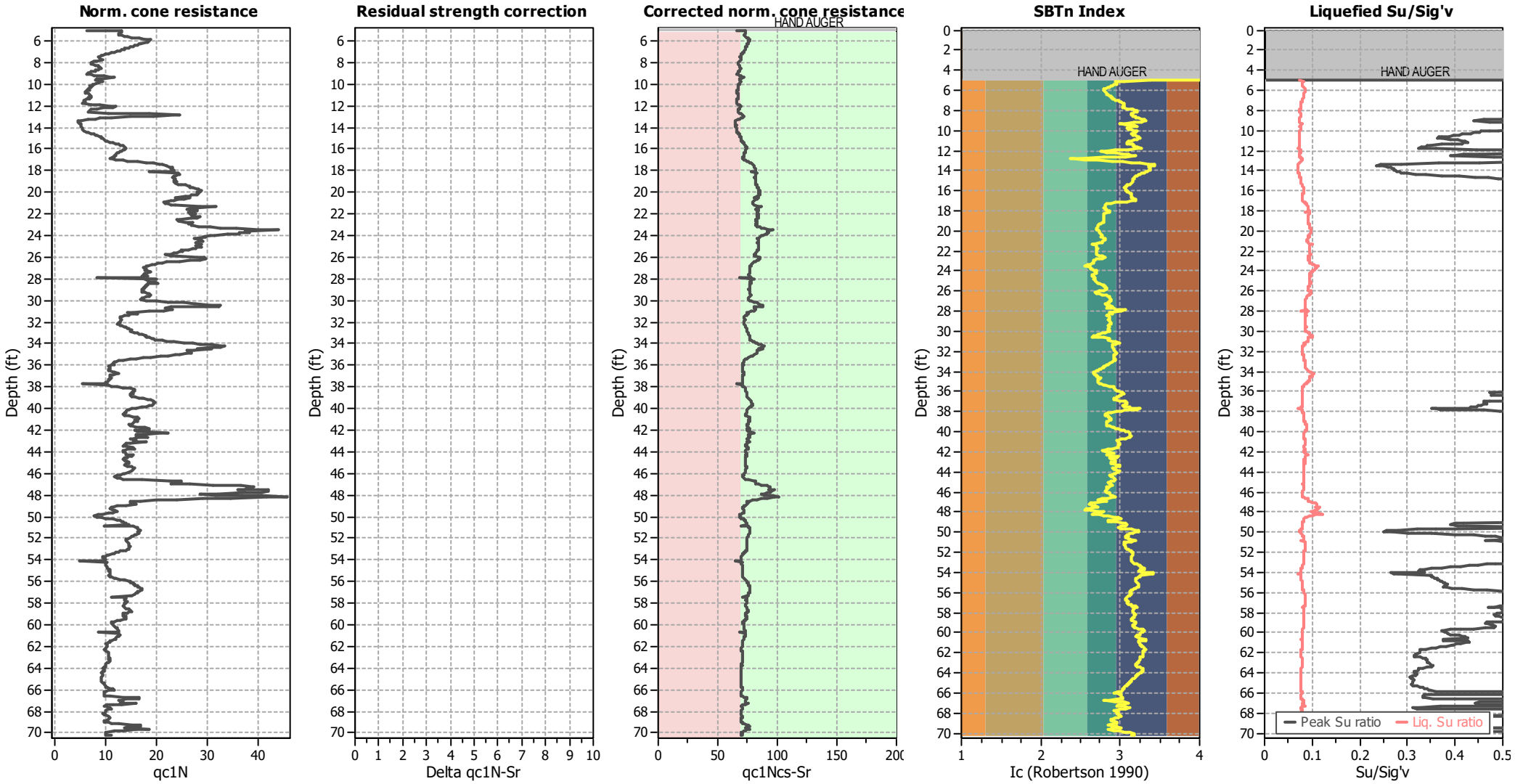
LPI color scheme

- Very high risk
- High risk
- Low risk

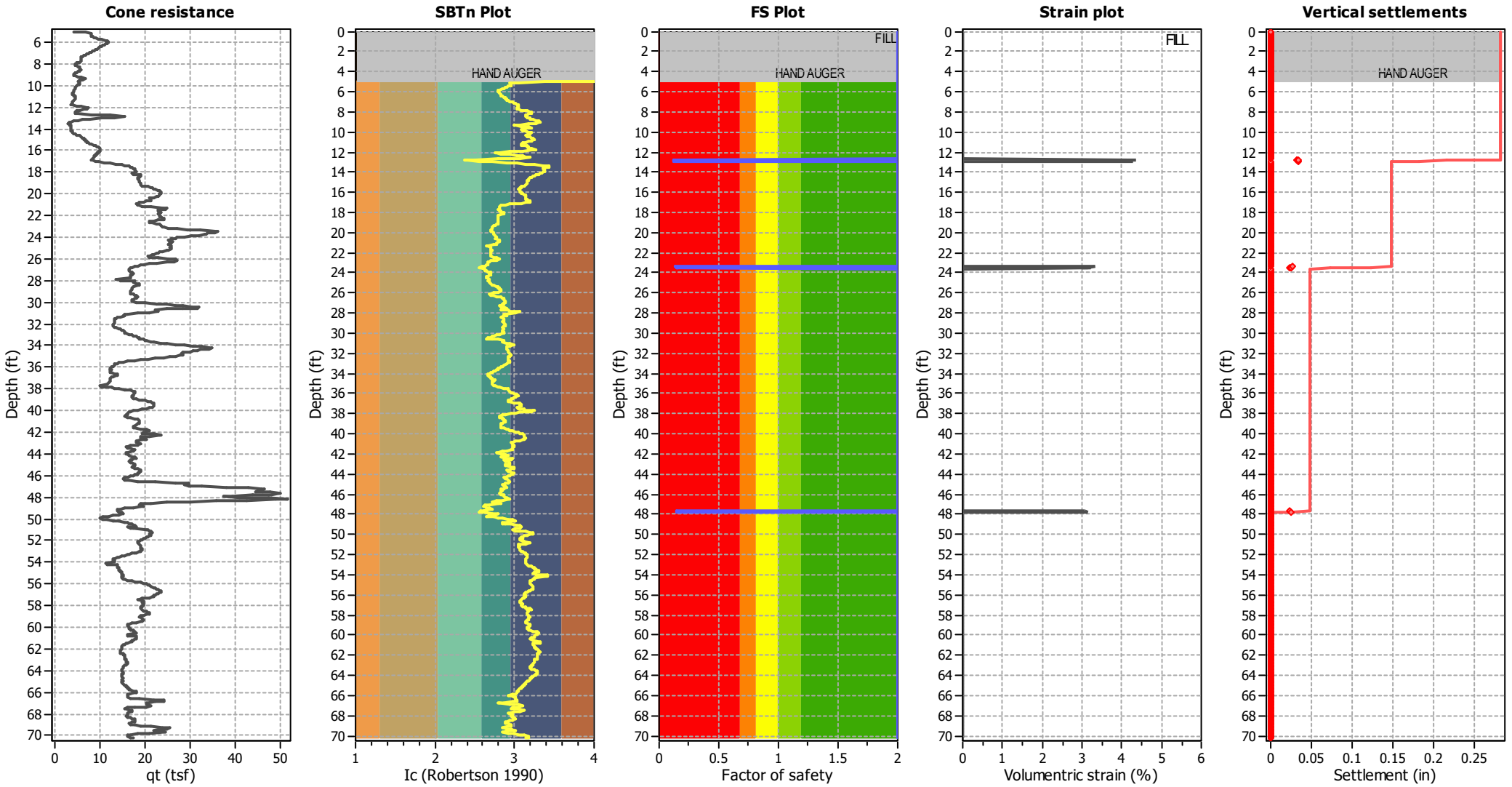
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	5.96 ft	Fill height:	3.20 ft	Limit depth:	N/A

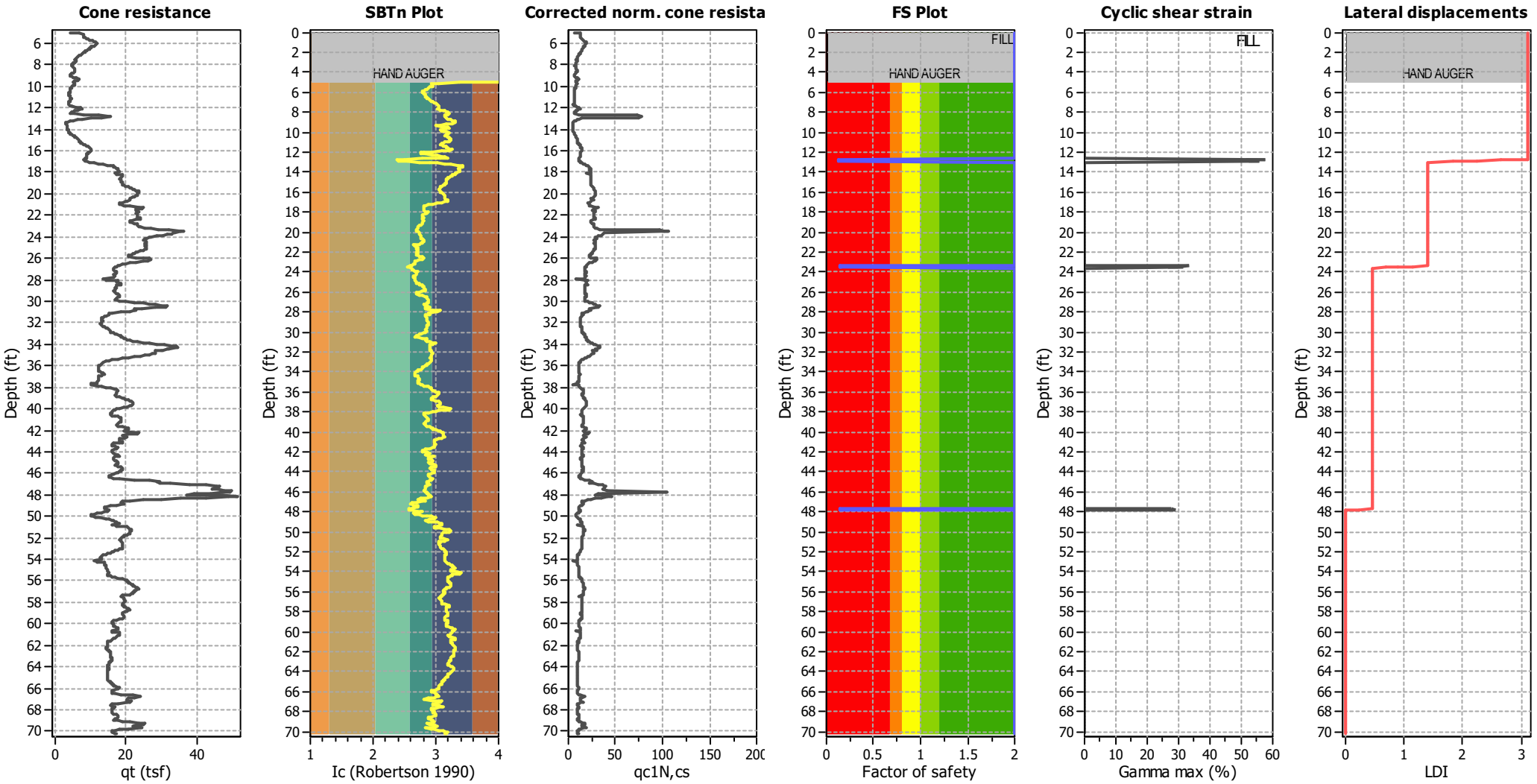
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
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$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

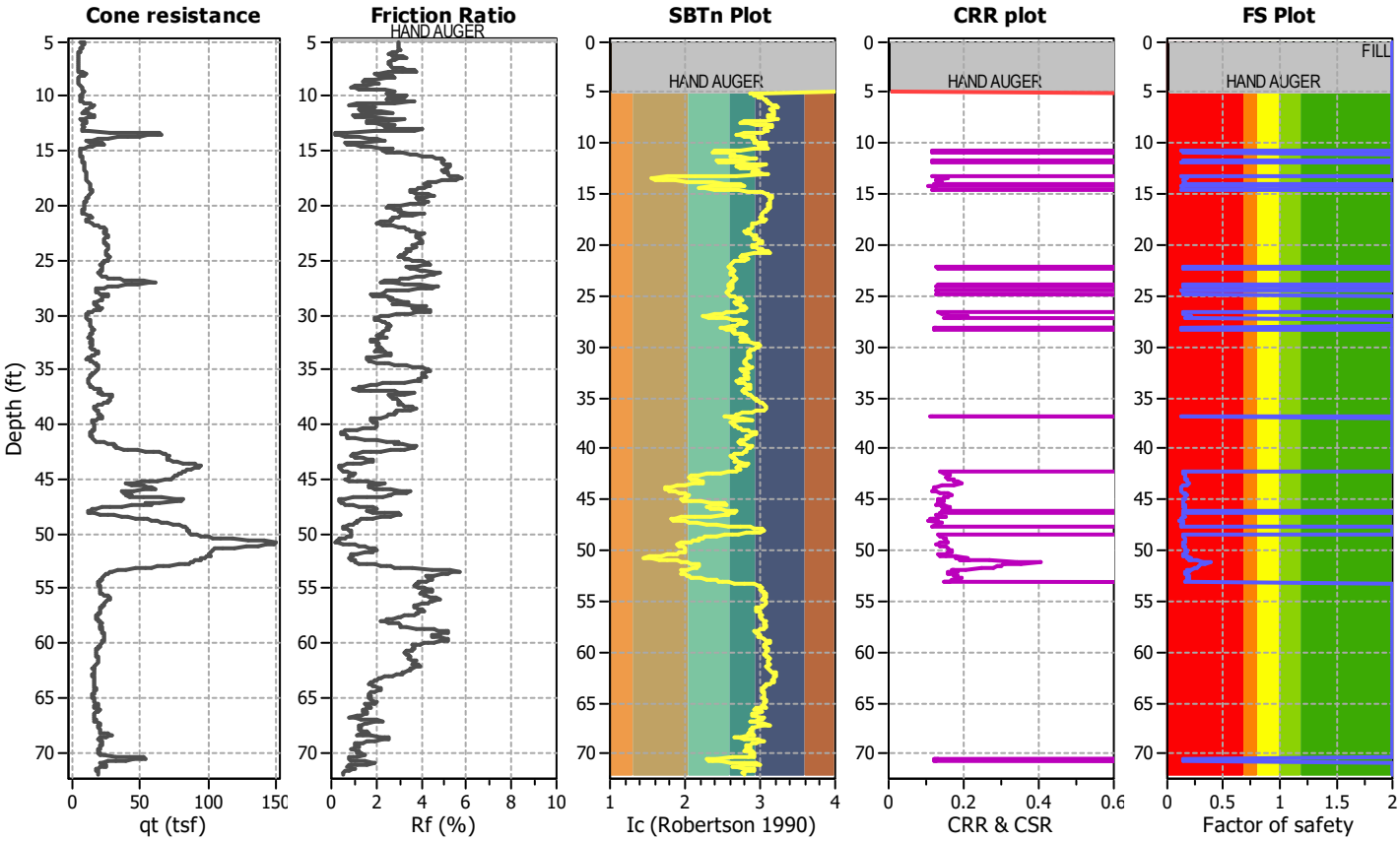
LIQUEFACTION ANALYSIS REPORT

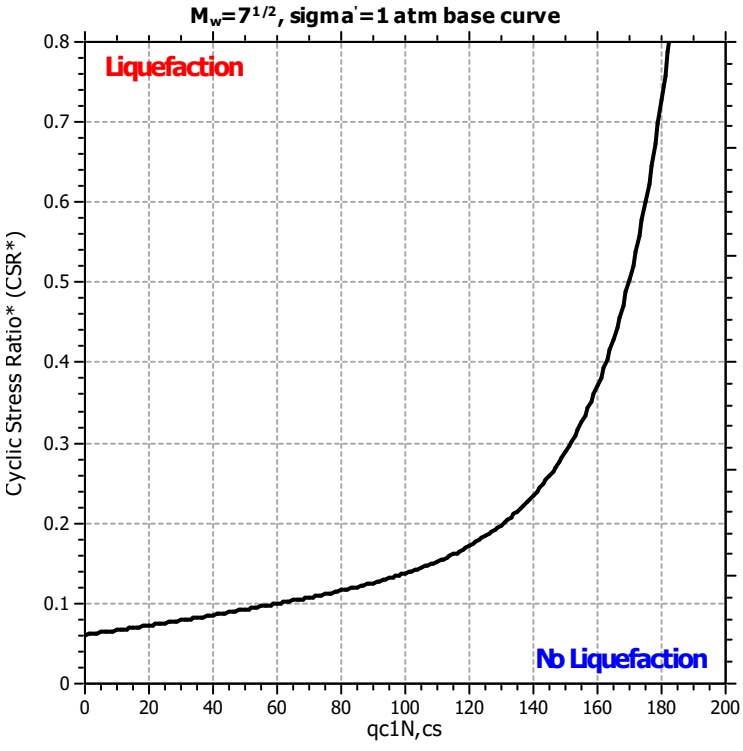
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

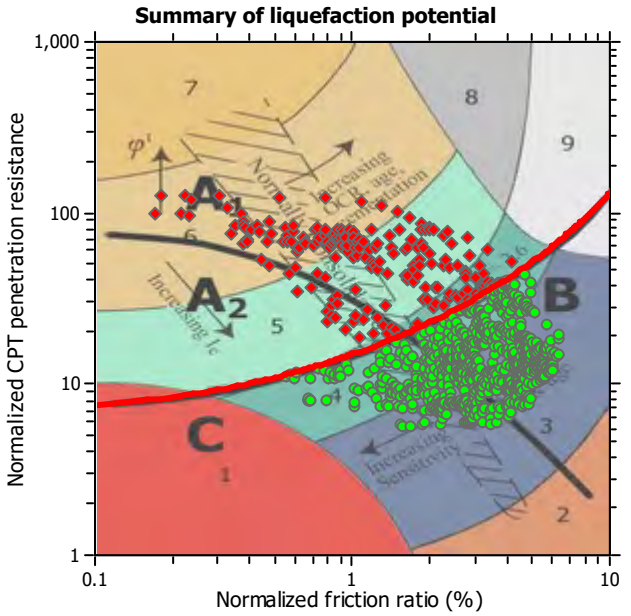
CPT file : GS26-CPT-16

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	1.70 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	4.30 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

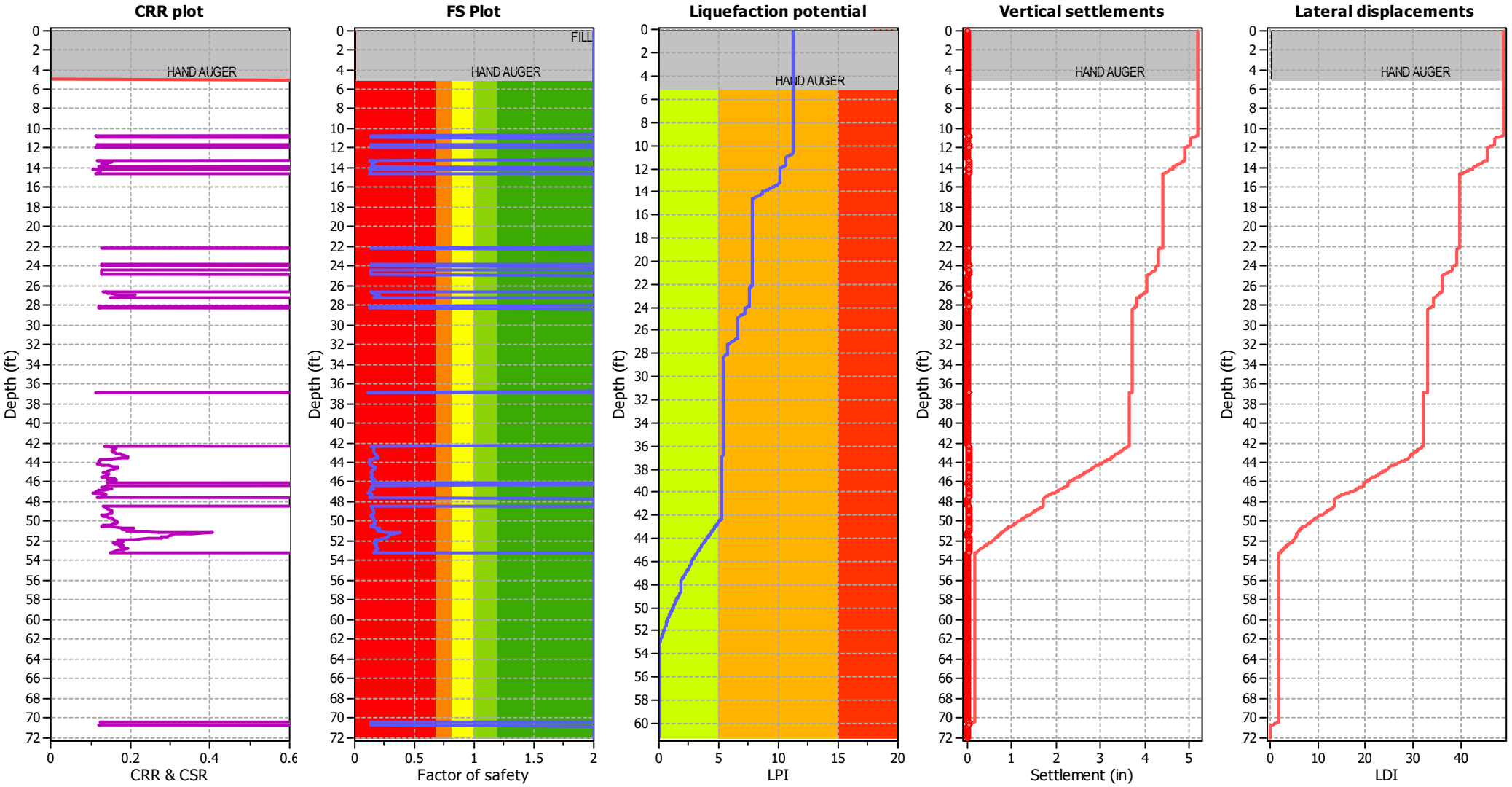






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	1.70 ft	Fill height:	4.30 ft	Limit depth:	N/A

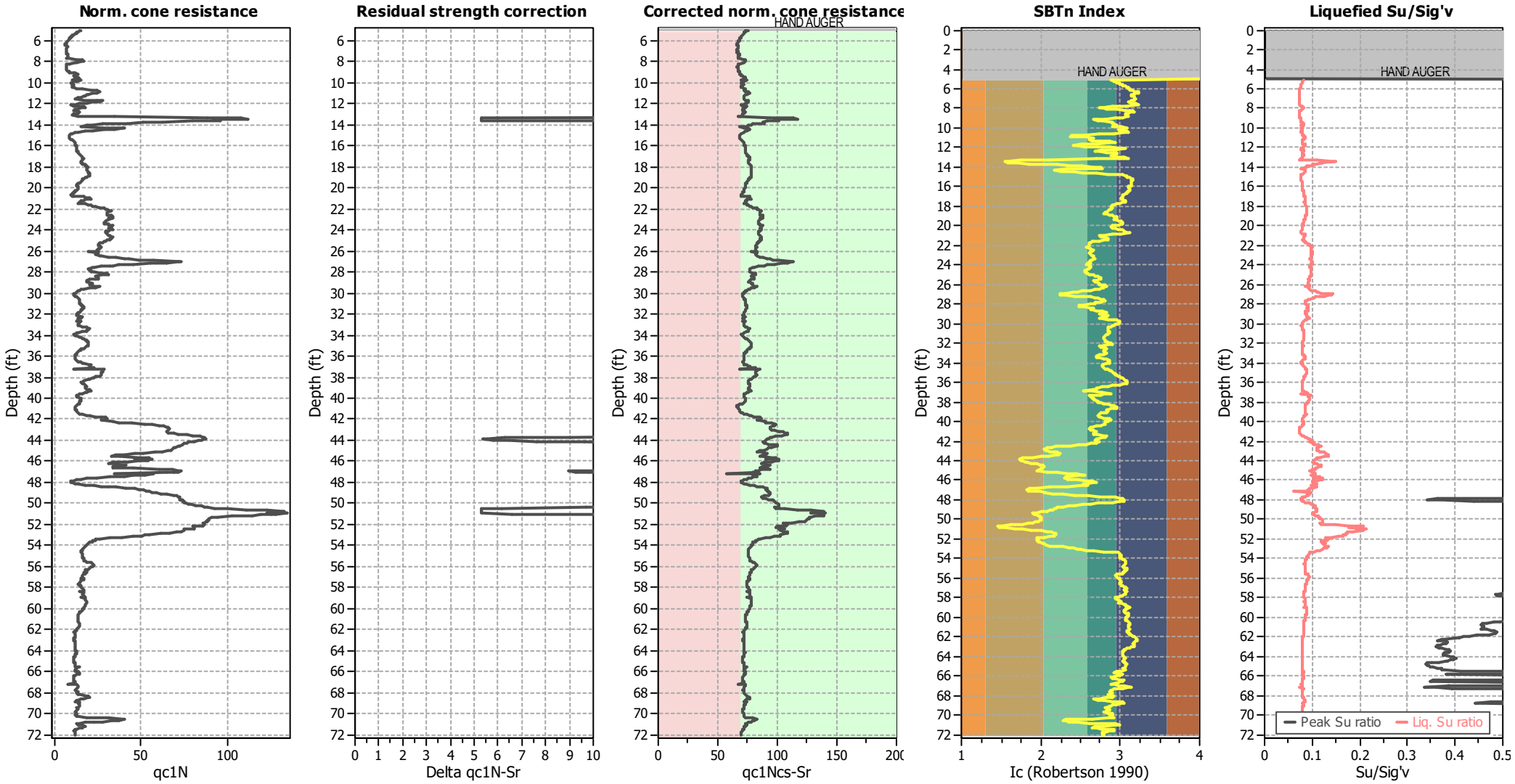
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

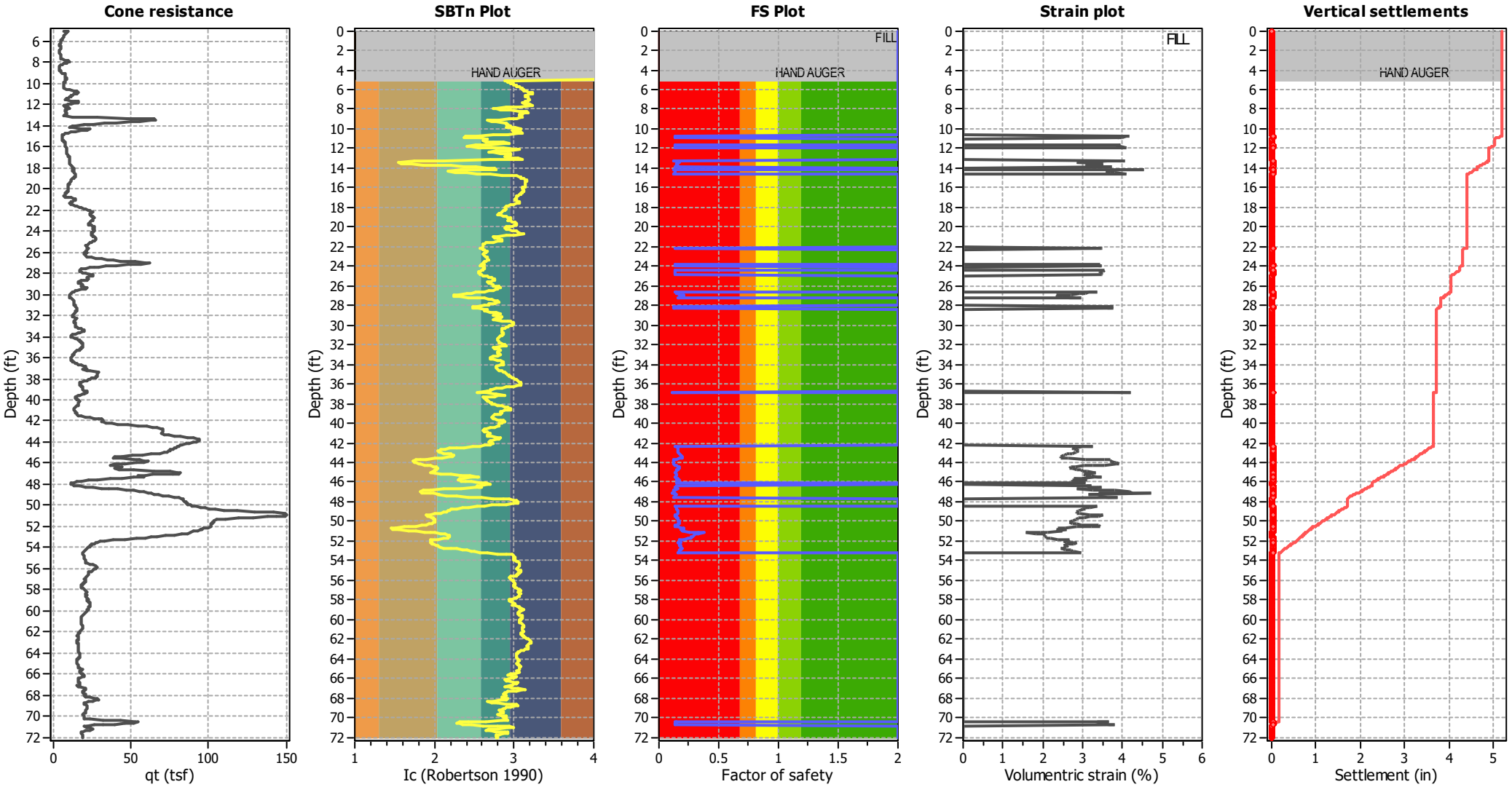
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	1.70 ft	Fill height:	4.30 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



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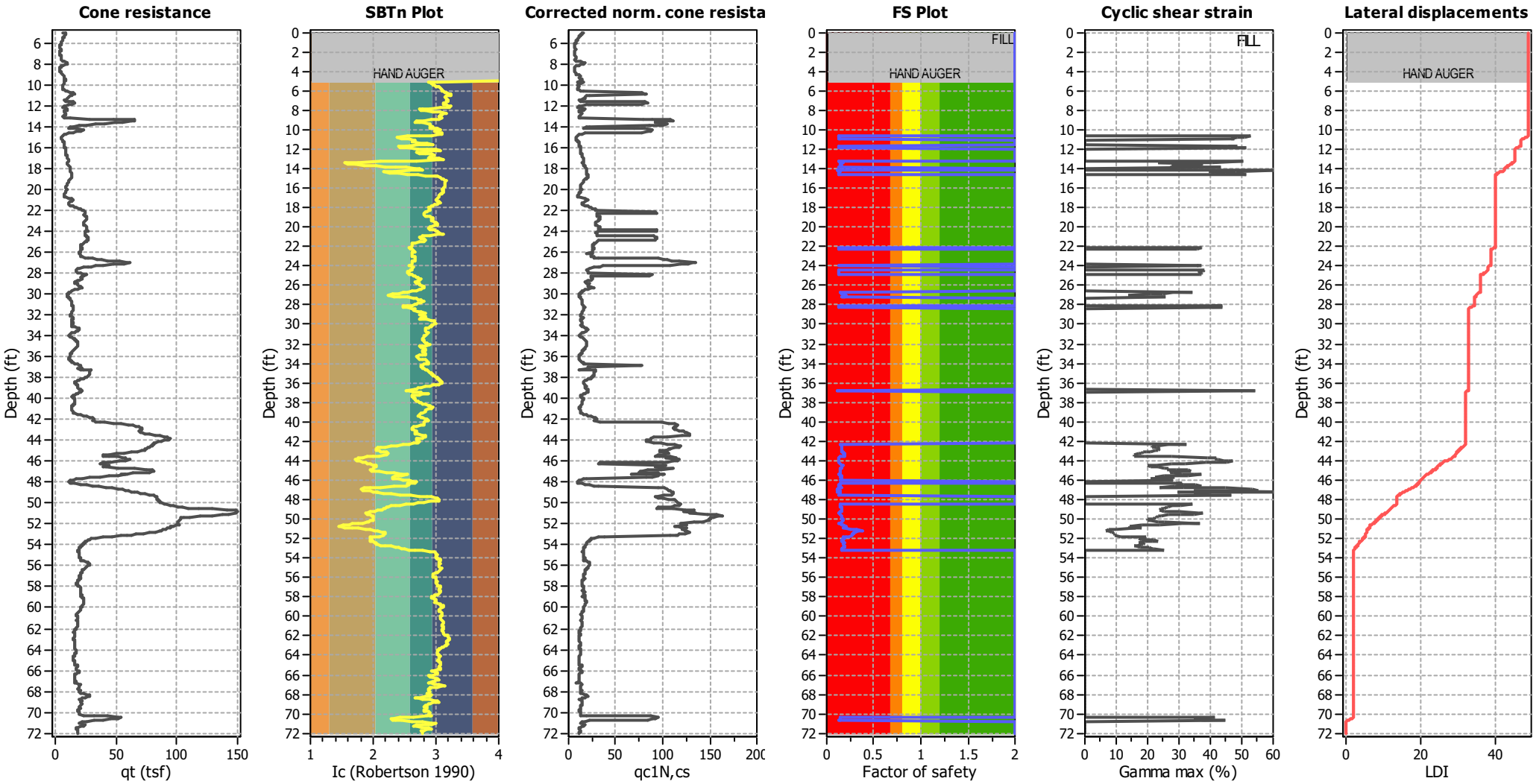
Estimation of post-earthquake lateral Displacements



Abbreviations

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LDI: Lateral displacement index

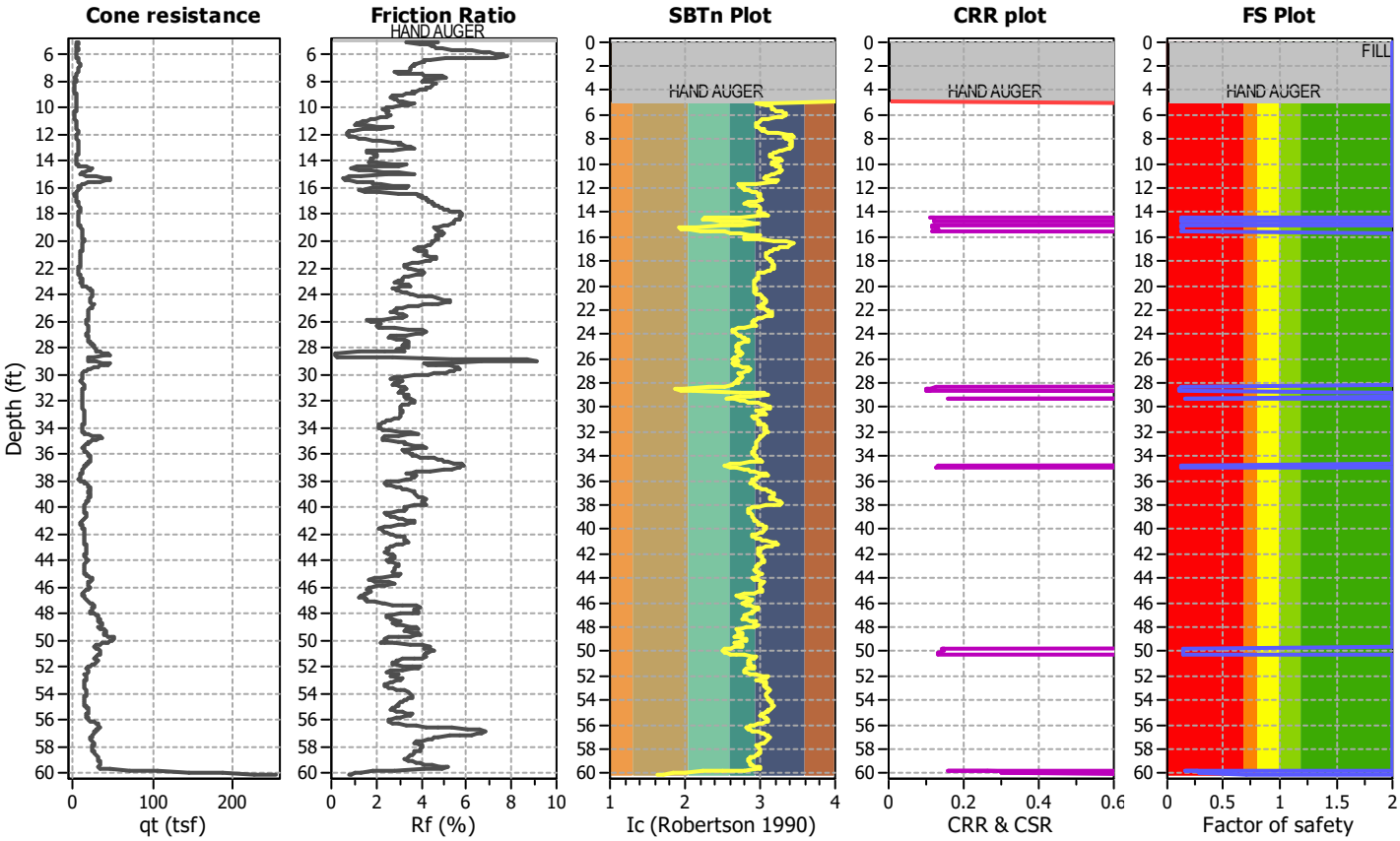
LIQUEFACTION ANALYSIS REPORT

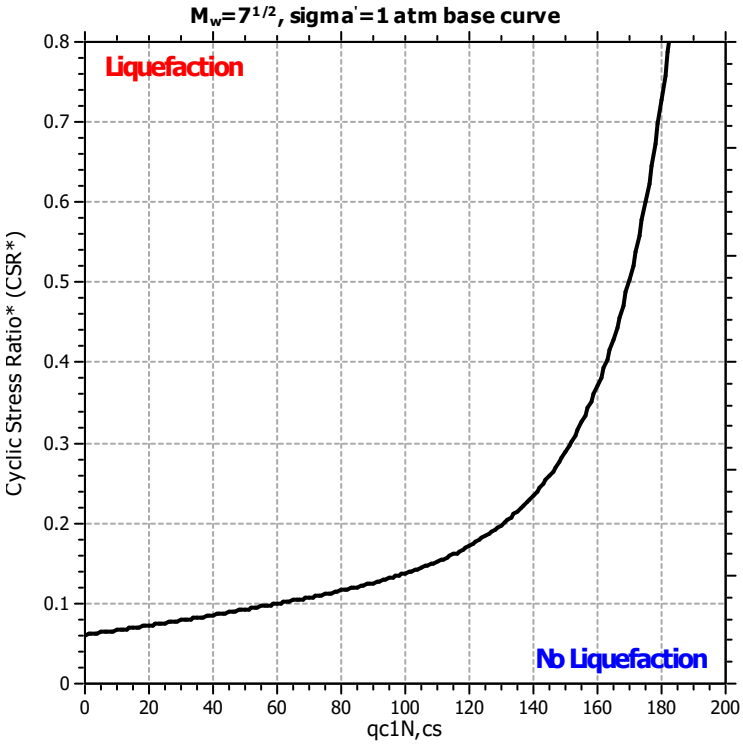
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

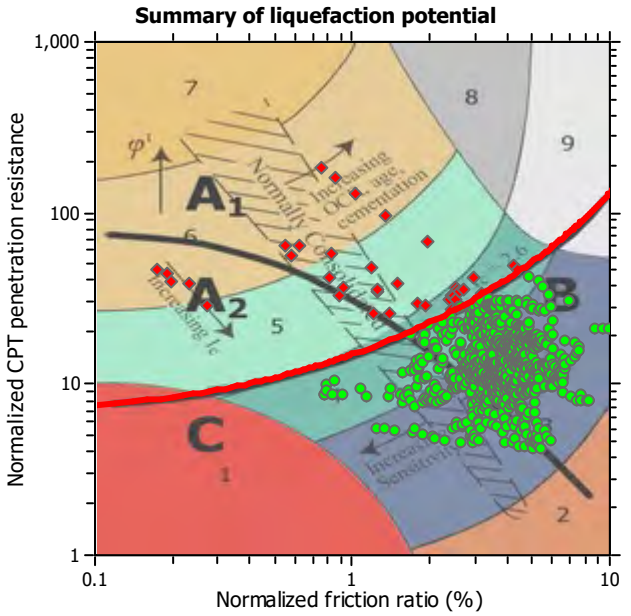
CPT file : GS26-CPT-17

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.98 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	3.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

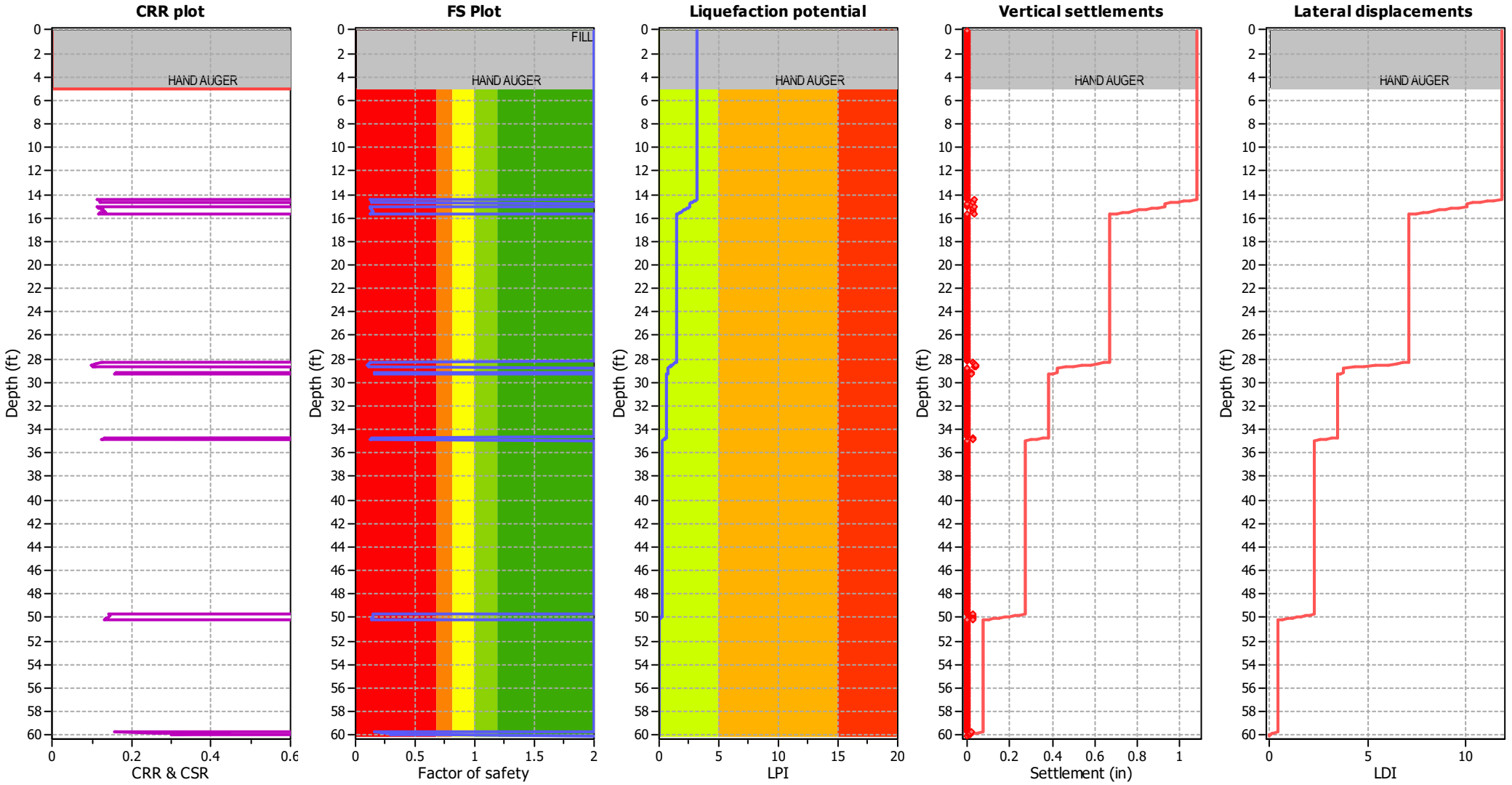






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.98 ft	Fill height:	3.00 ft	Limit depth:	N/A

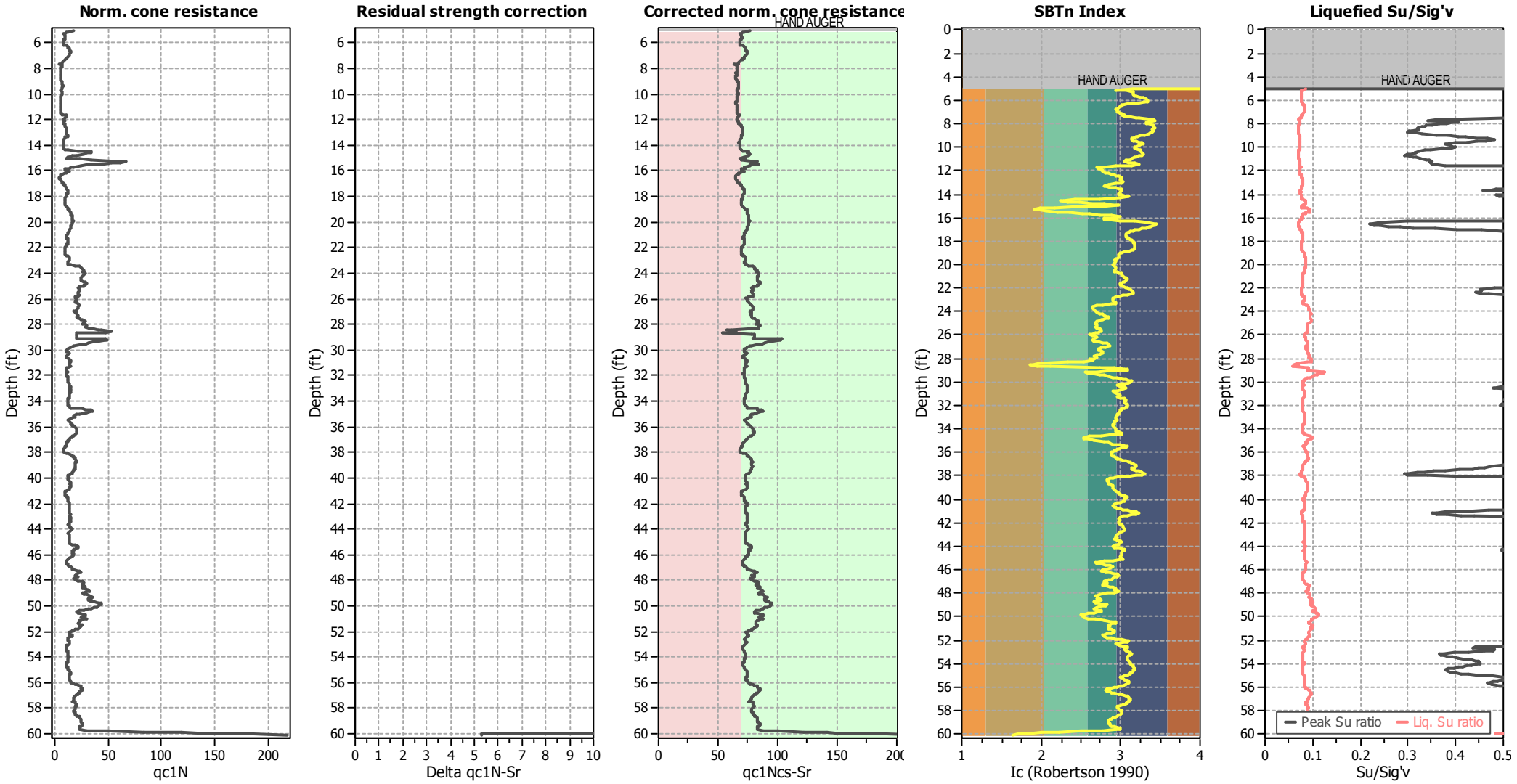
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

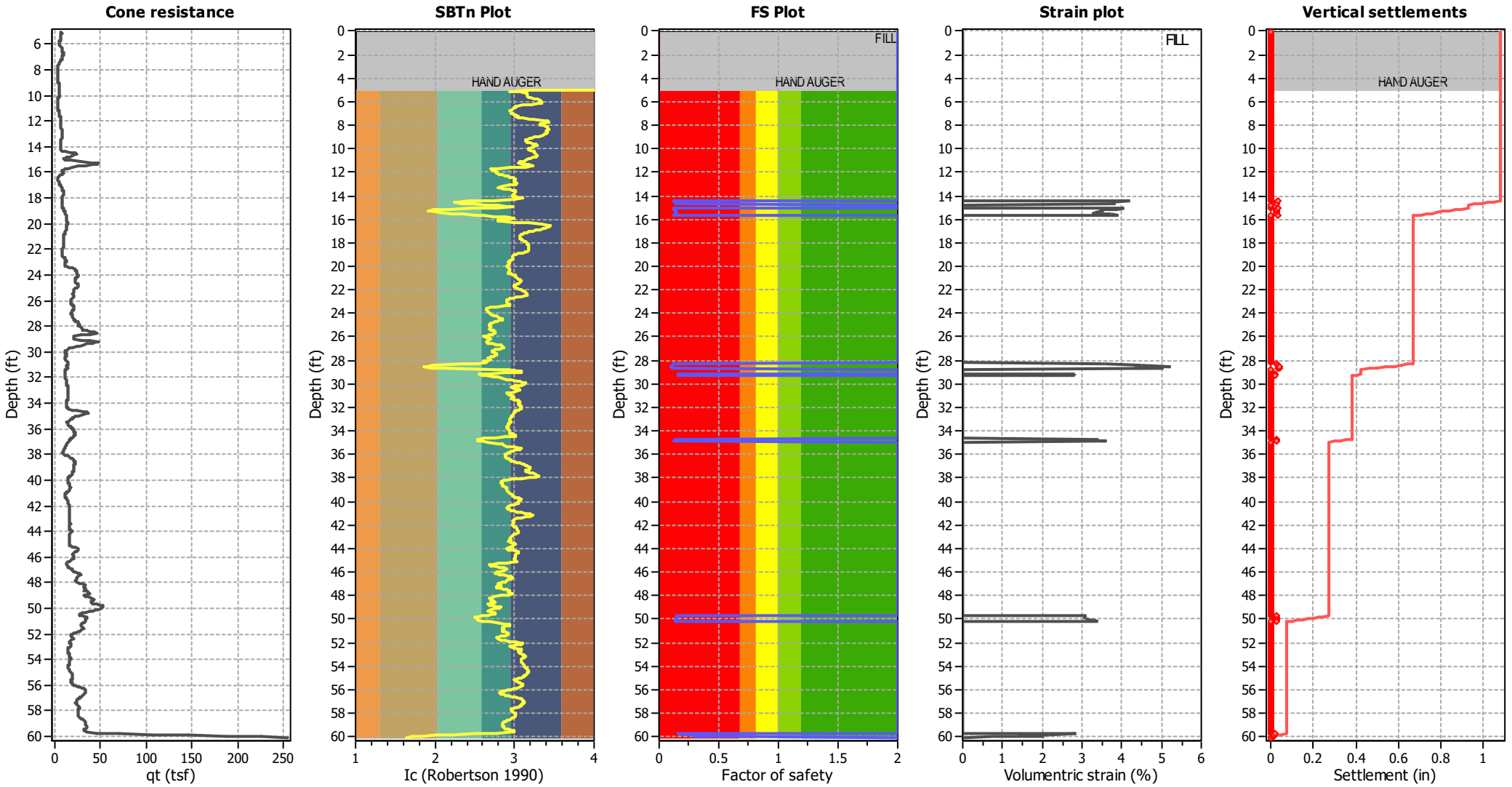
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.98 ft	Fill height:	3.00 ft	Limit depth:	N/A

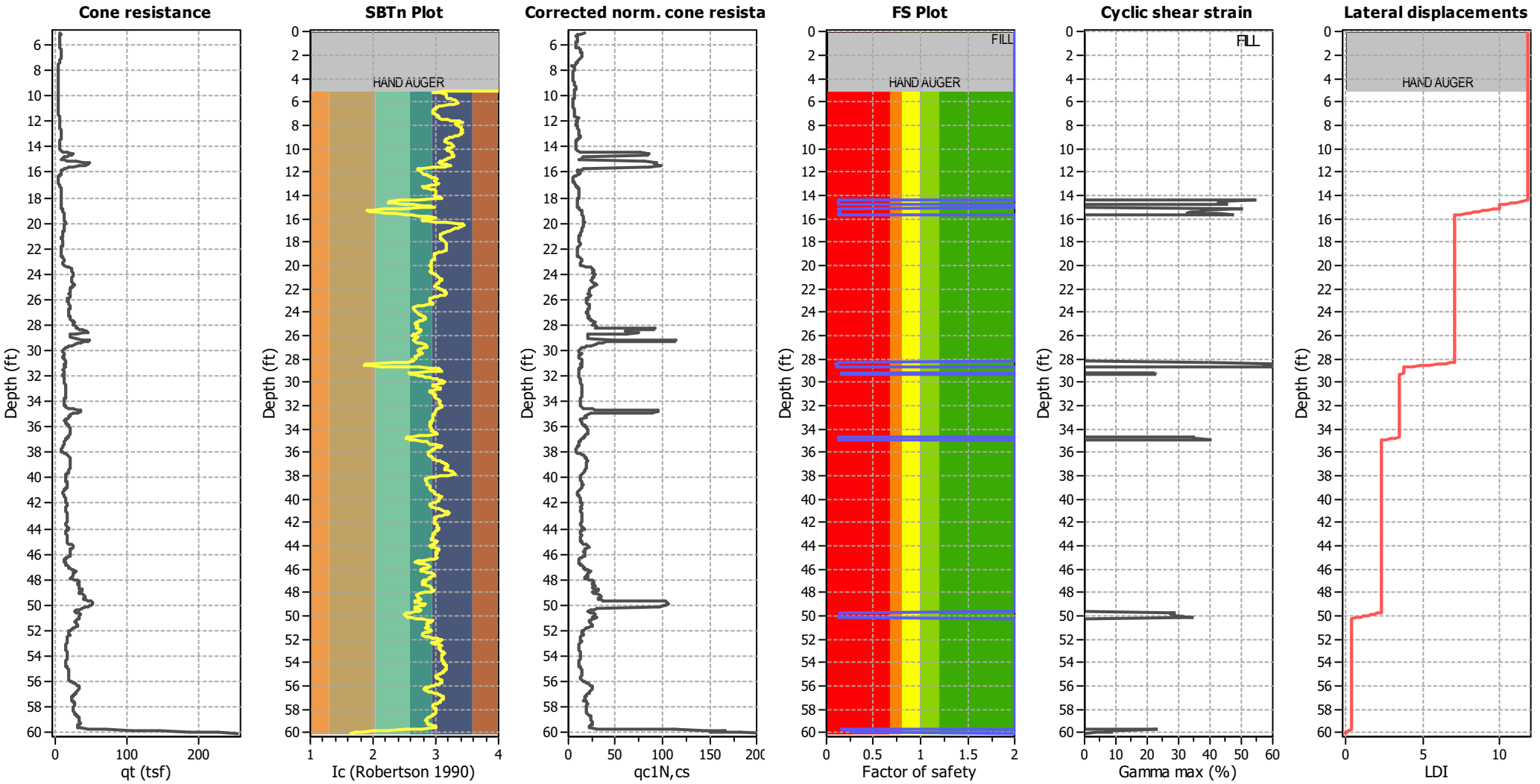
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

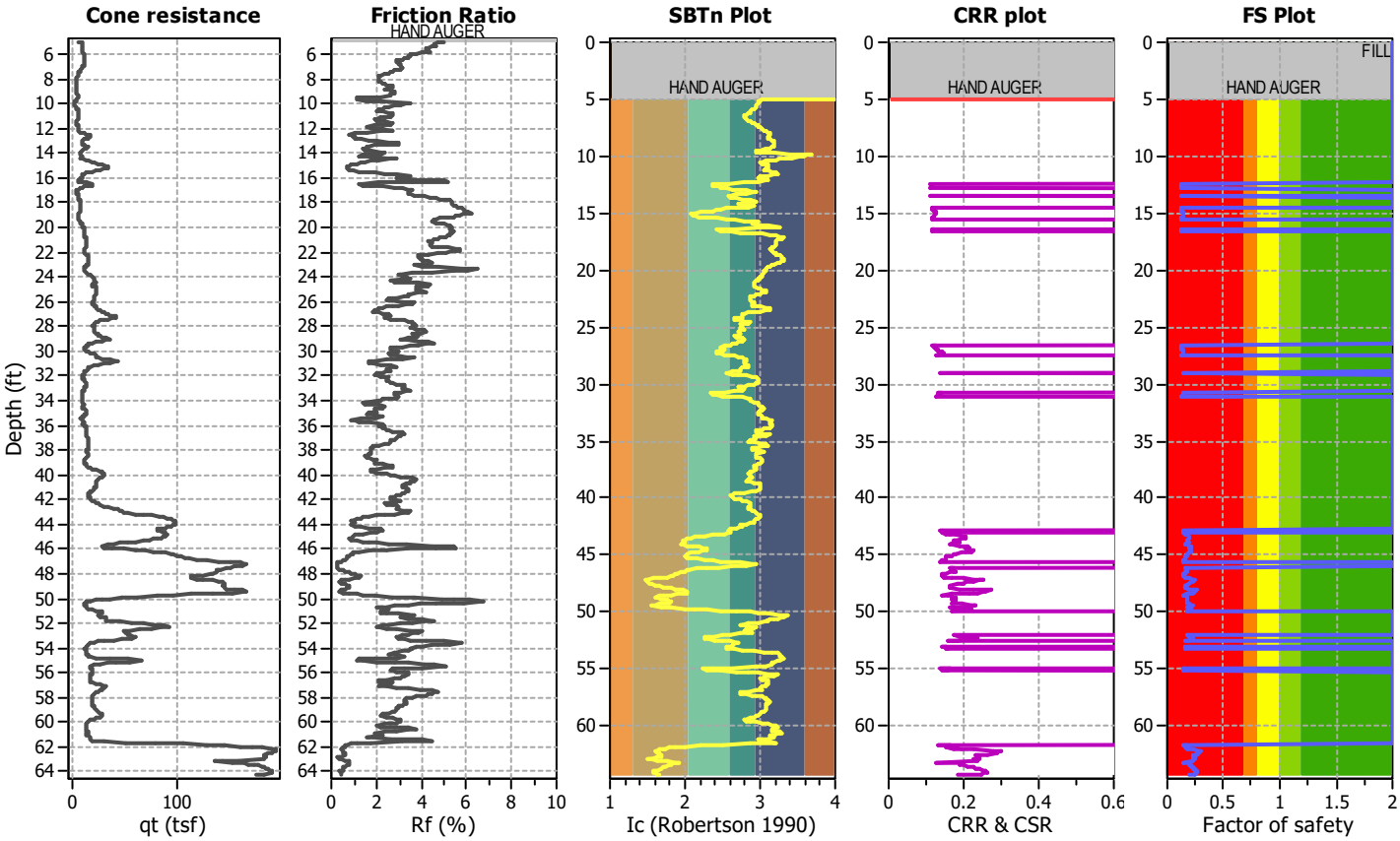
LIQUEFACTION ANALYSIS REPORT

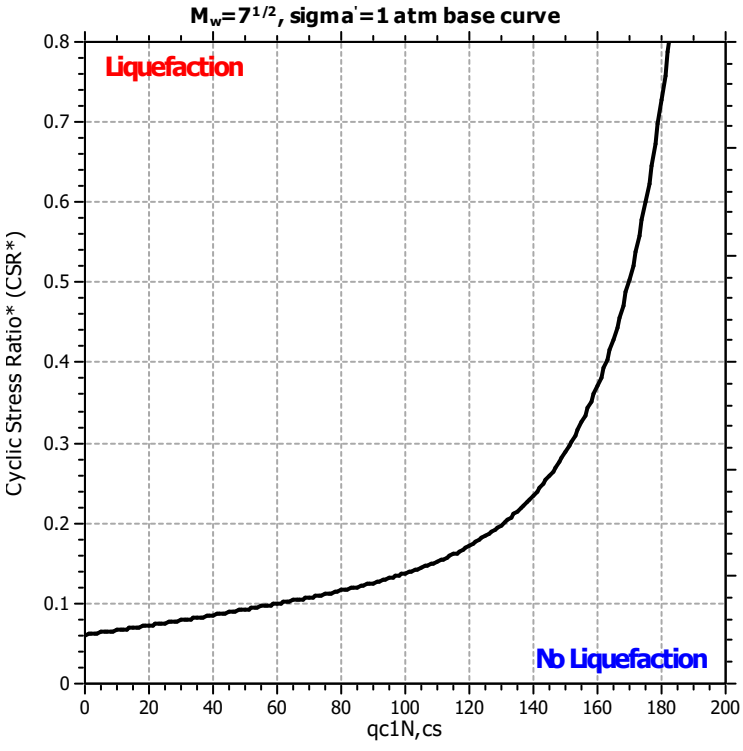
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

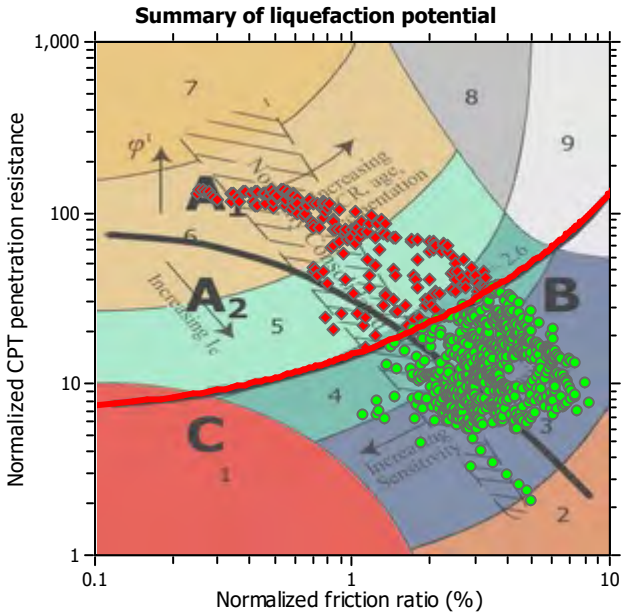
CPT file : GS26-CPT-18

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	6.64 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	3.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.90	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.77	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

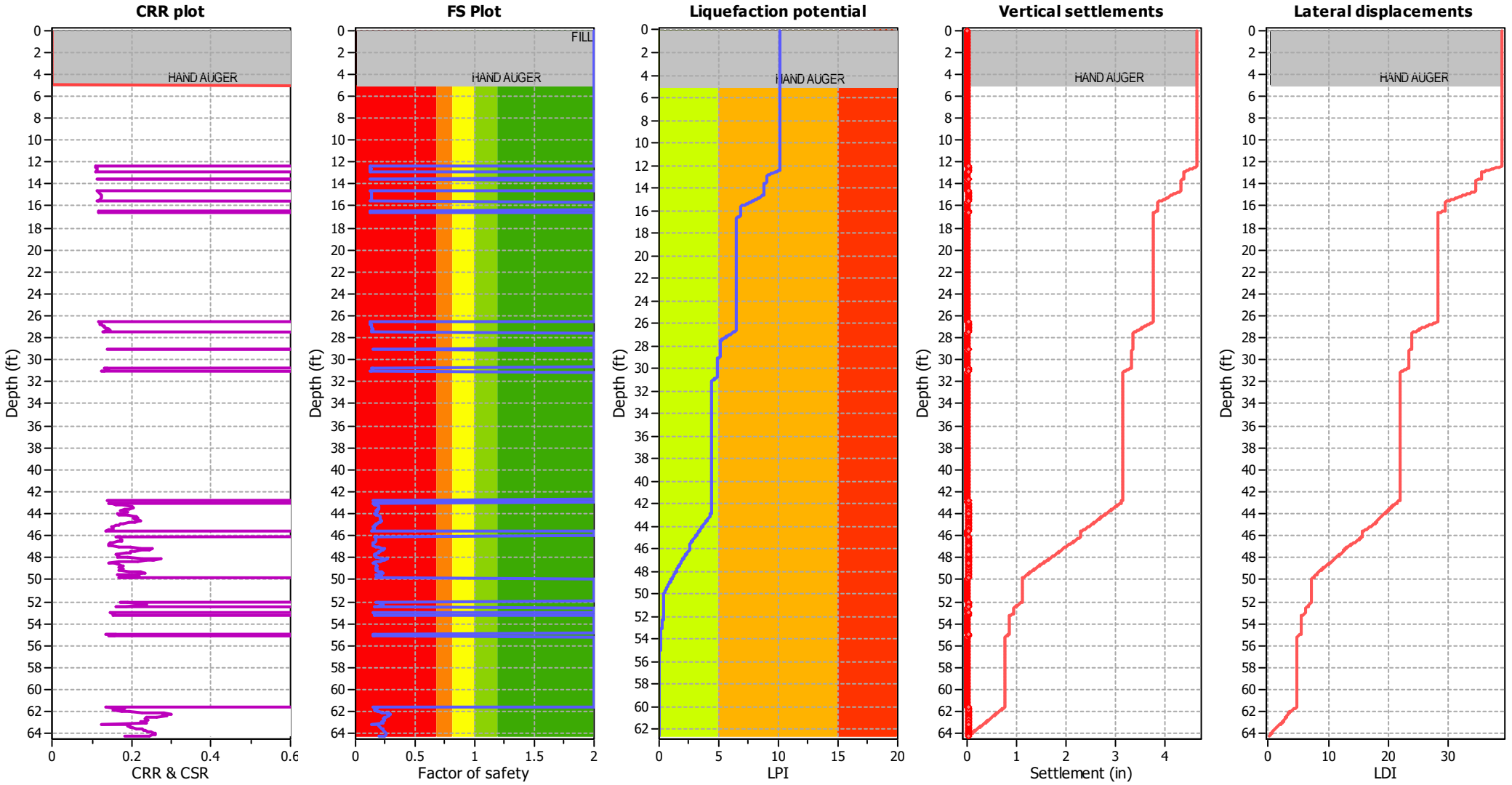






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.64 ft	Fill height:	3.00 ft	Limit depth:	N/A

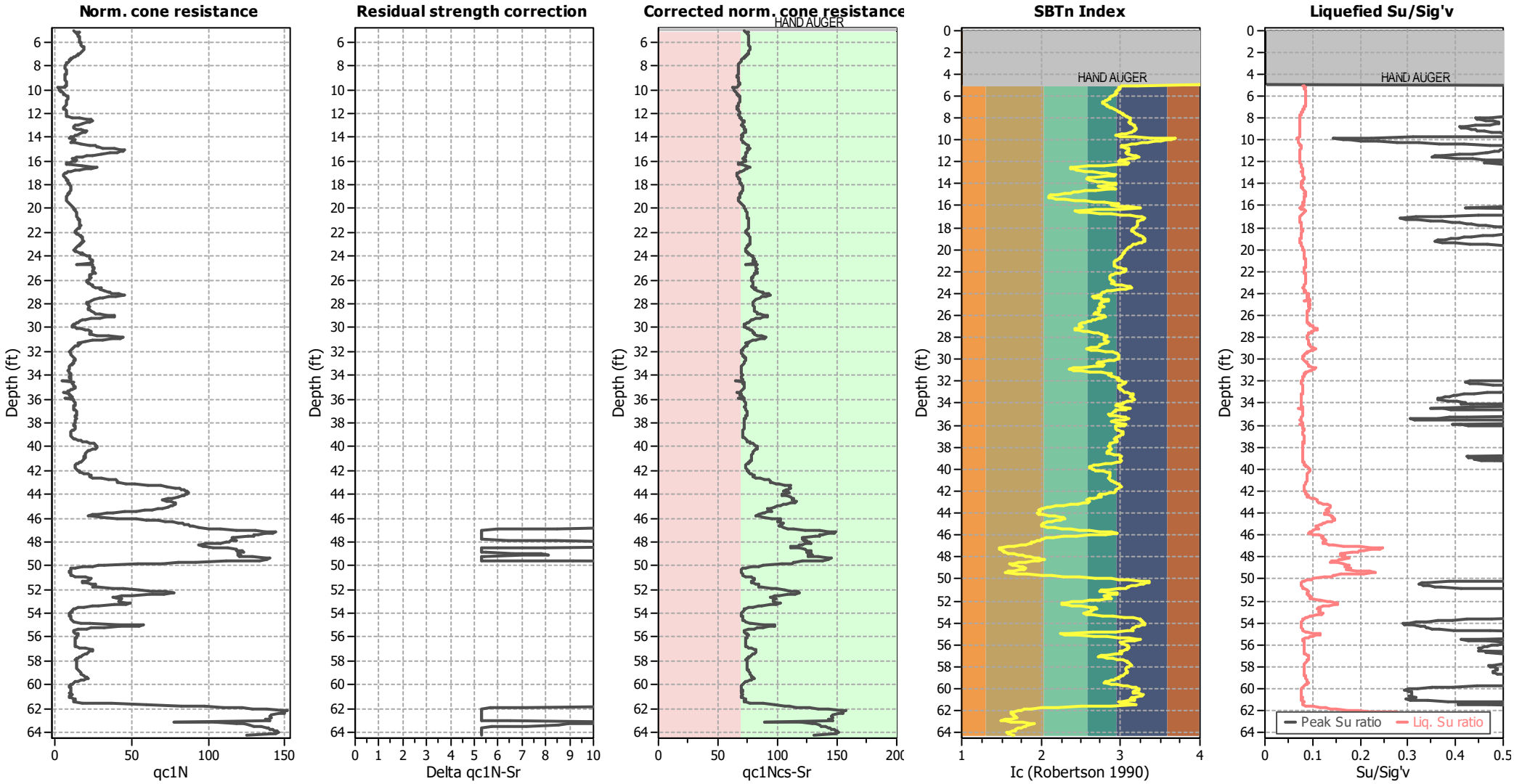
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

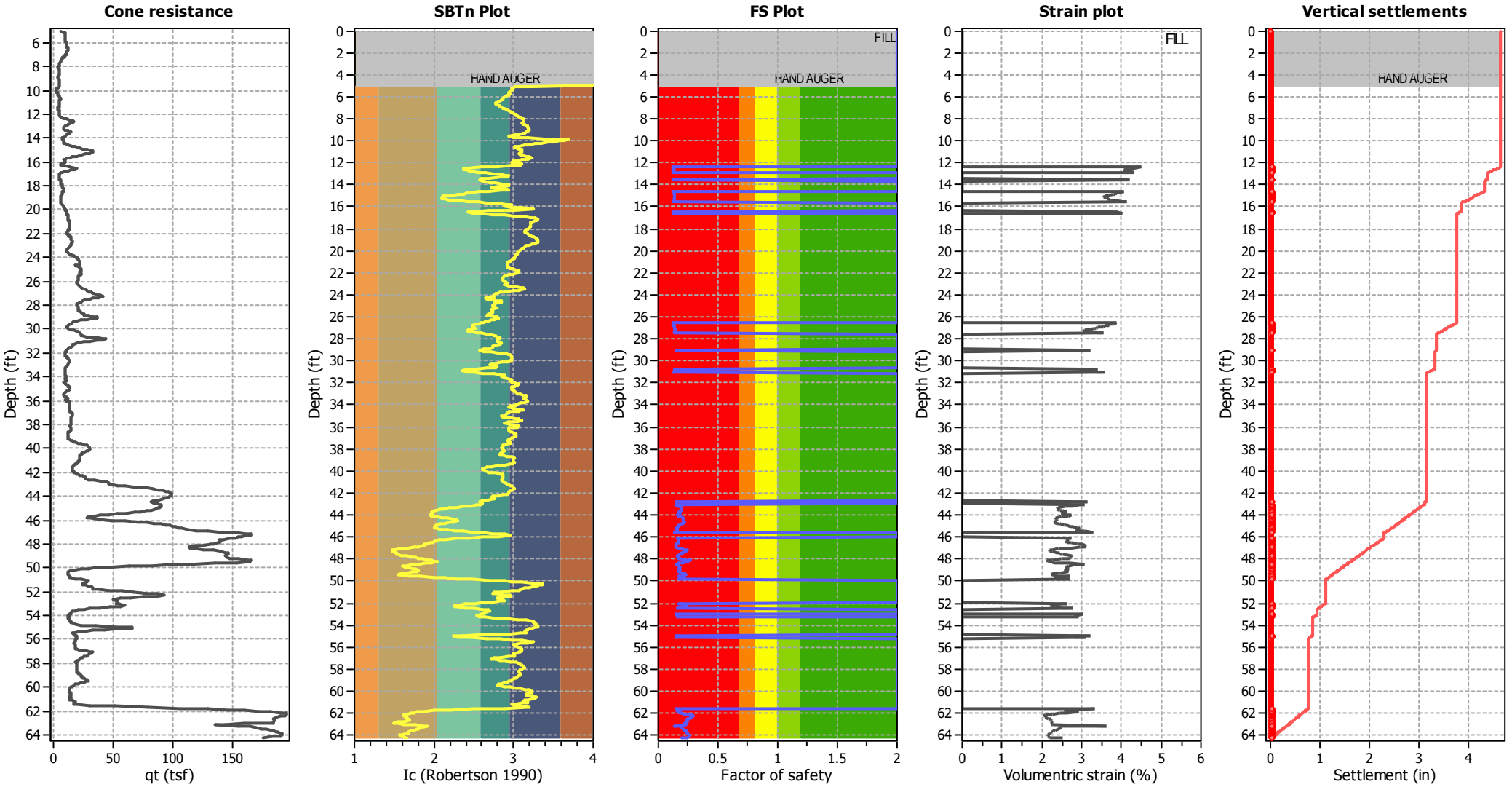
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.90	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.77	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	6.64 ft	Fill height:	3.00 ft	Limit depth:	N/A

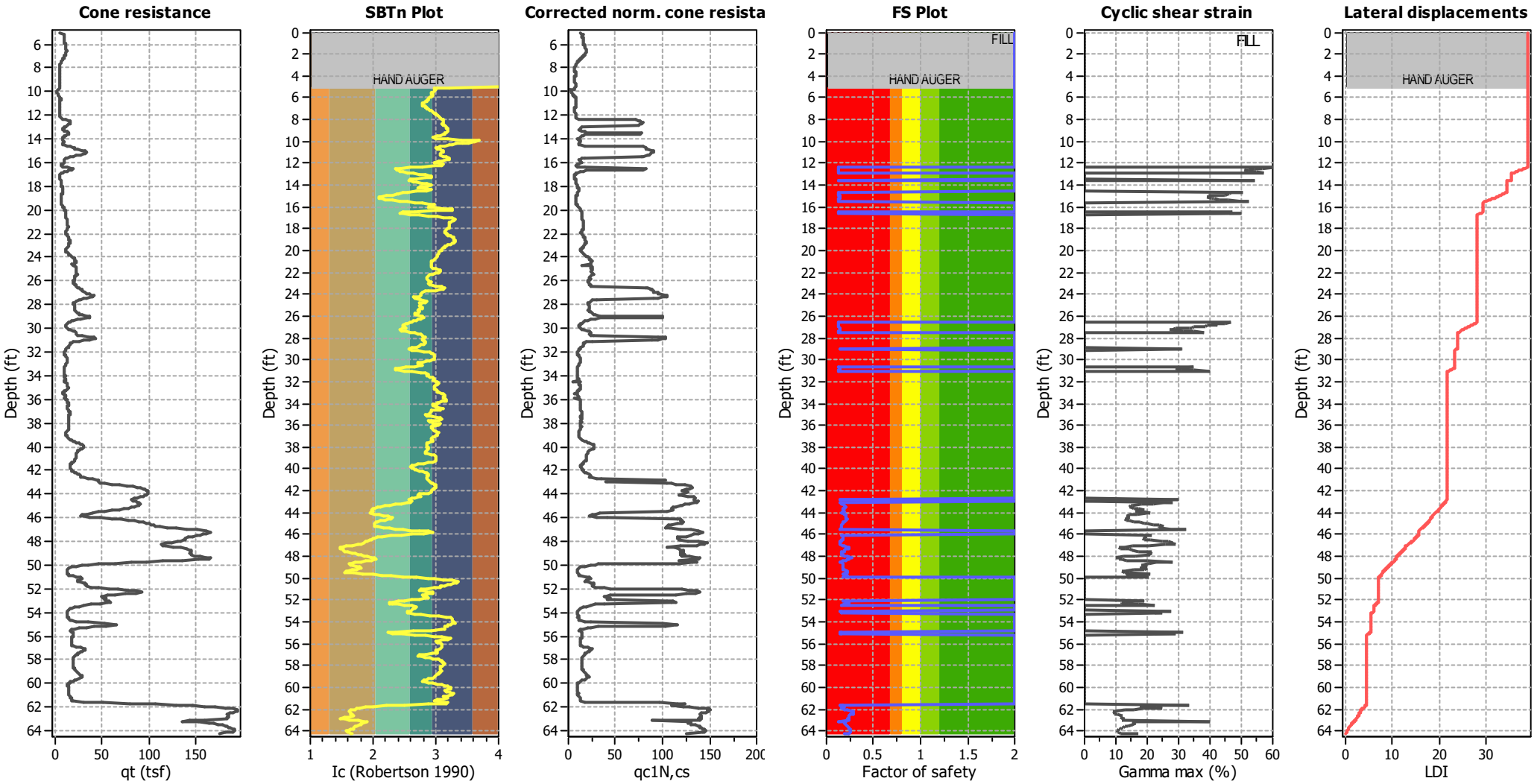
Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

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I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

Maximum Design Earthquake Ground Motion (MDE-GM) CPT-based Liquefaction Outputs with Fines Content Adjustment

GS26-CPT-1

GS26-CPT-5

Geosyntec Consultants, Inc.
SF Bay Area Branch

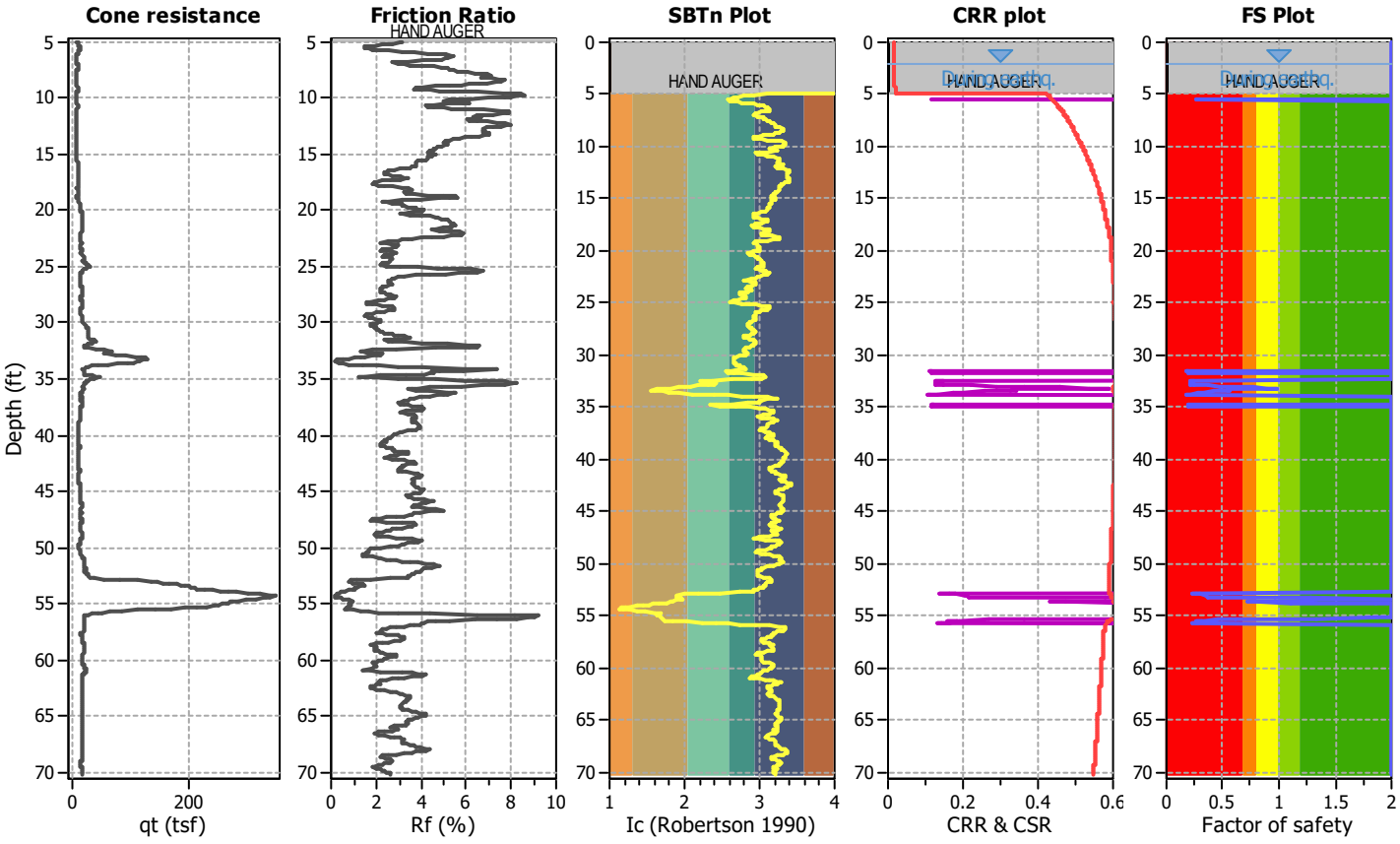
LIQUEFACTION ANALYSIS REPORT

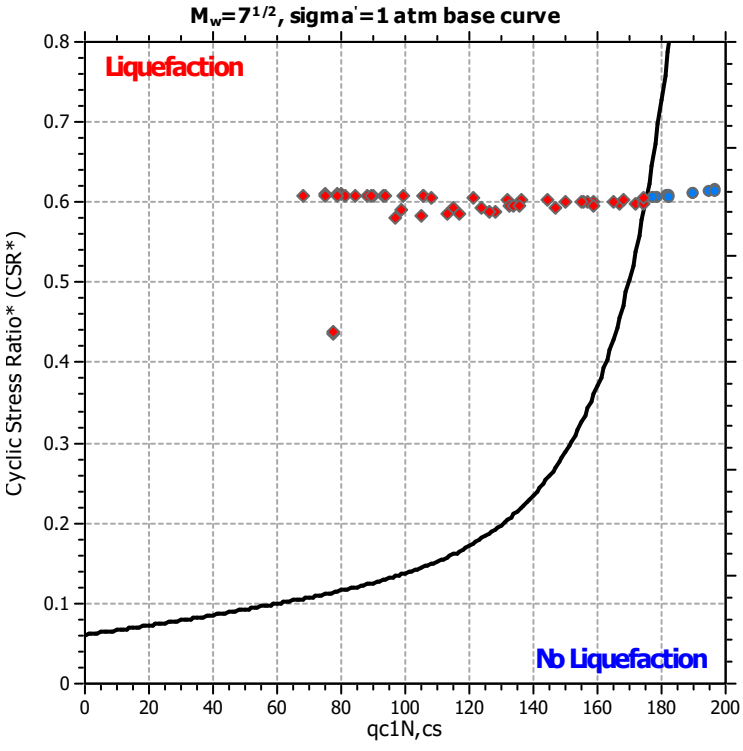
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

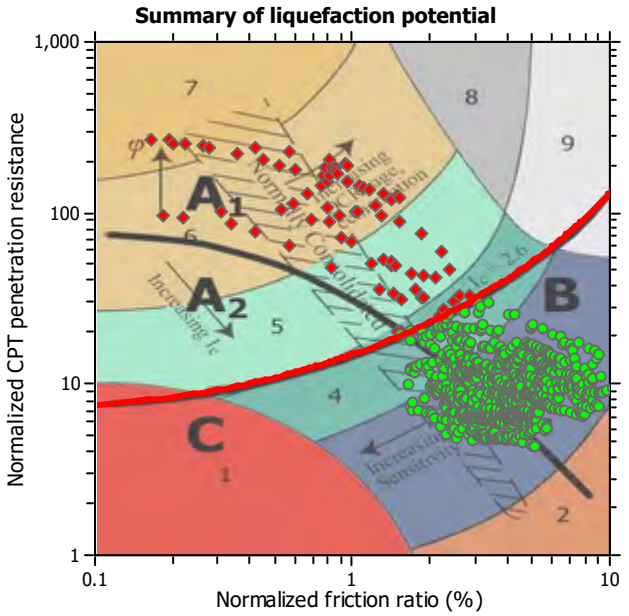
CPT file : GS26-CPT-01

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	14.59 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	0.00 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

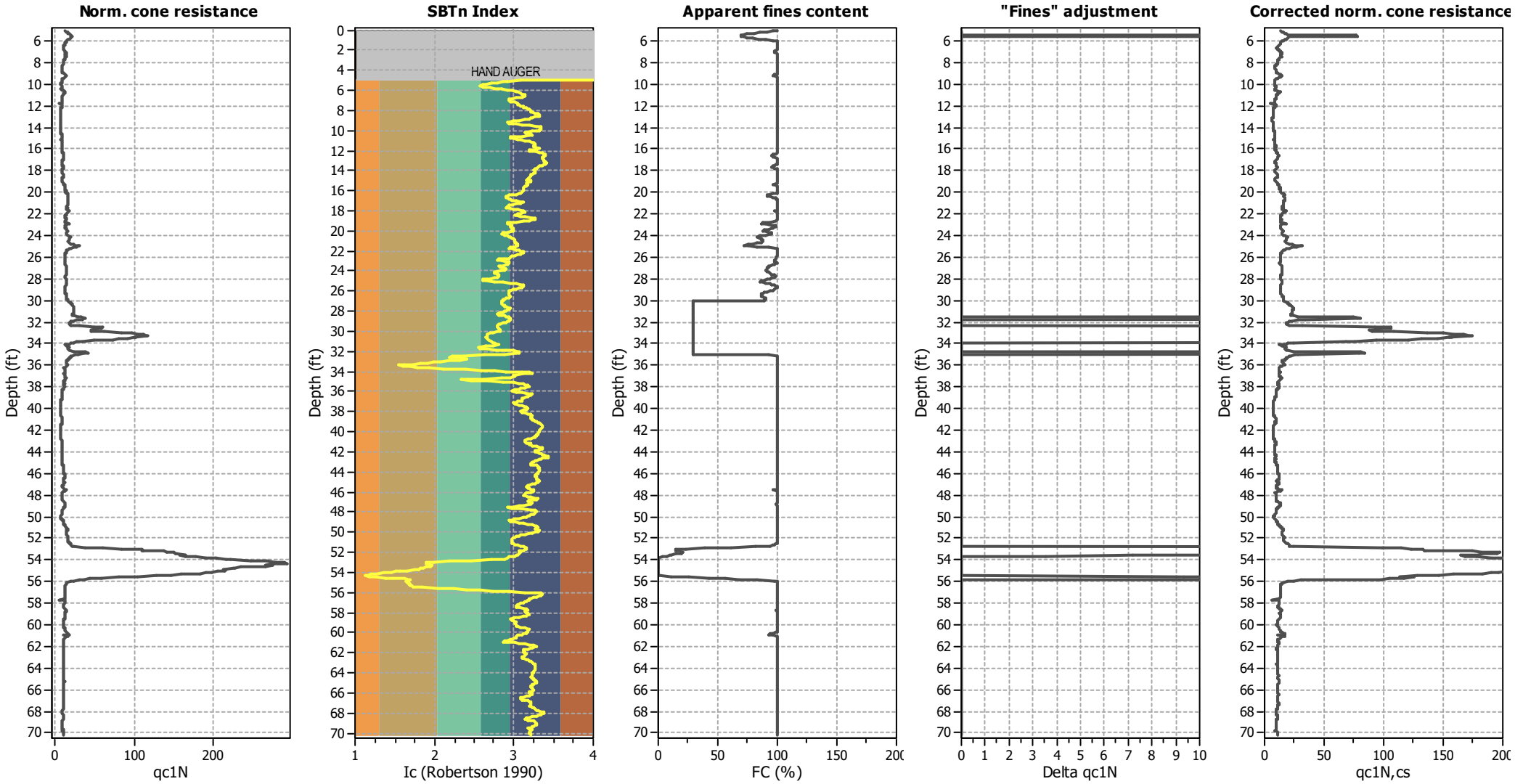






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

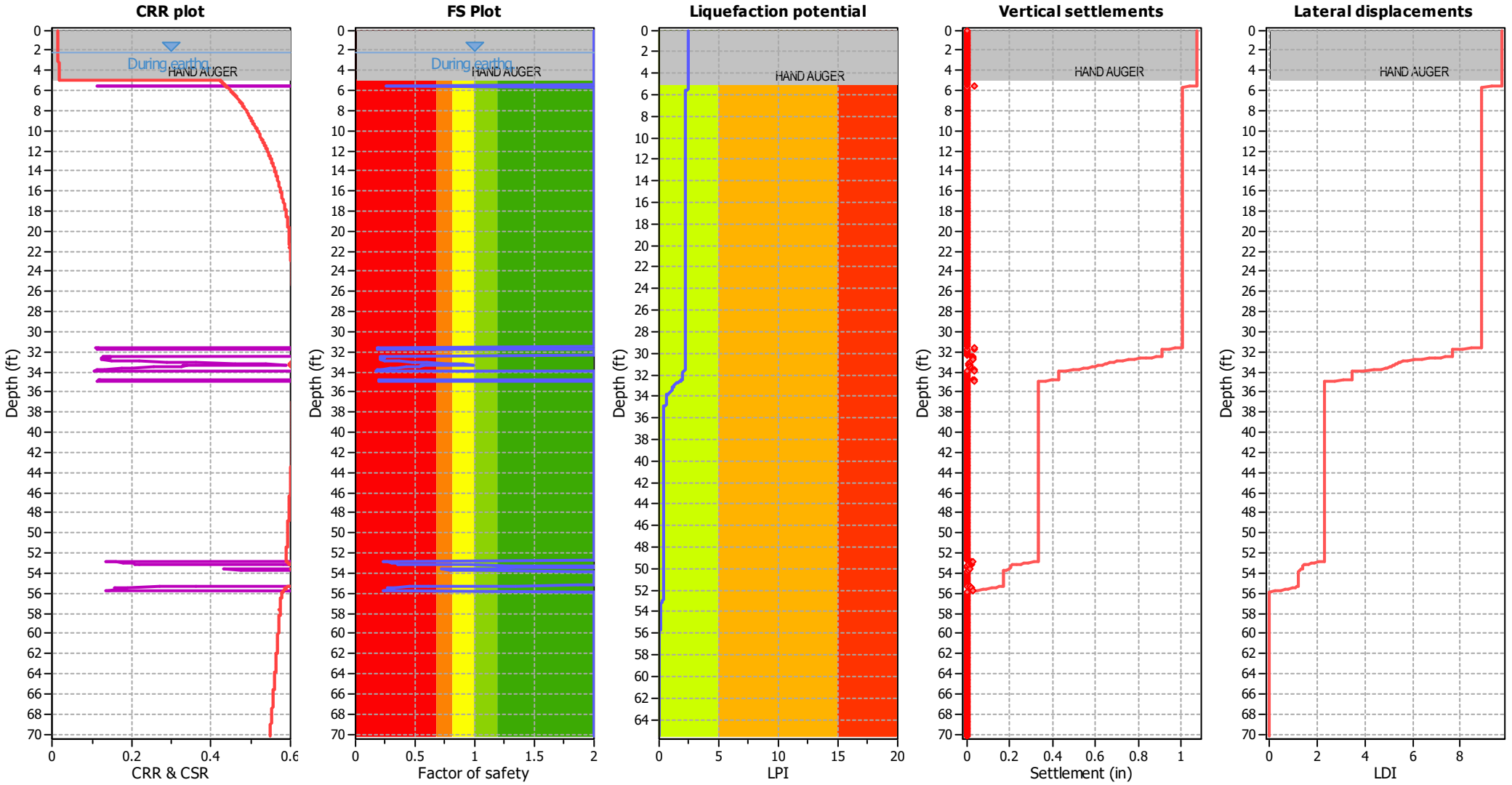
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

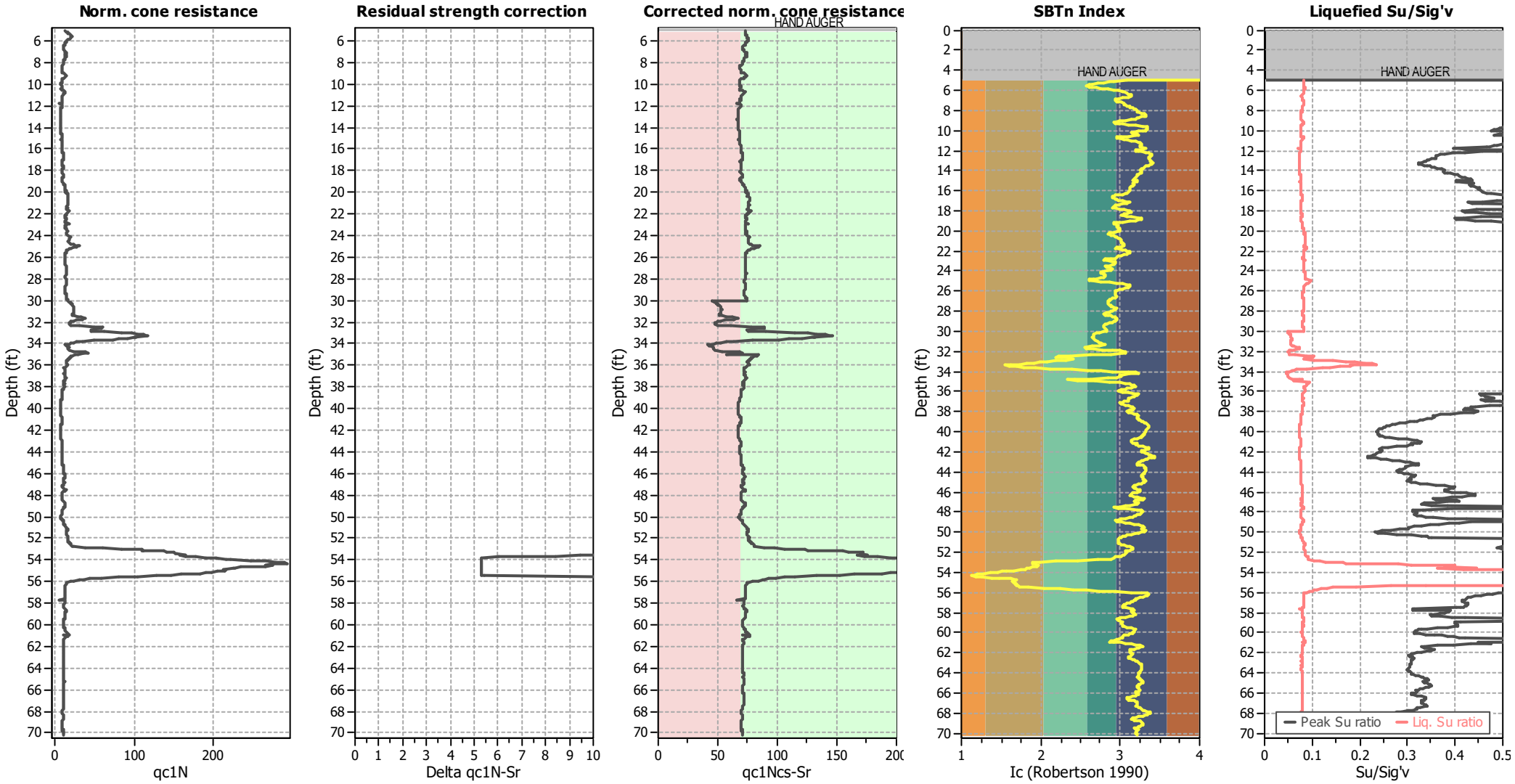
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

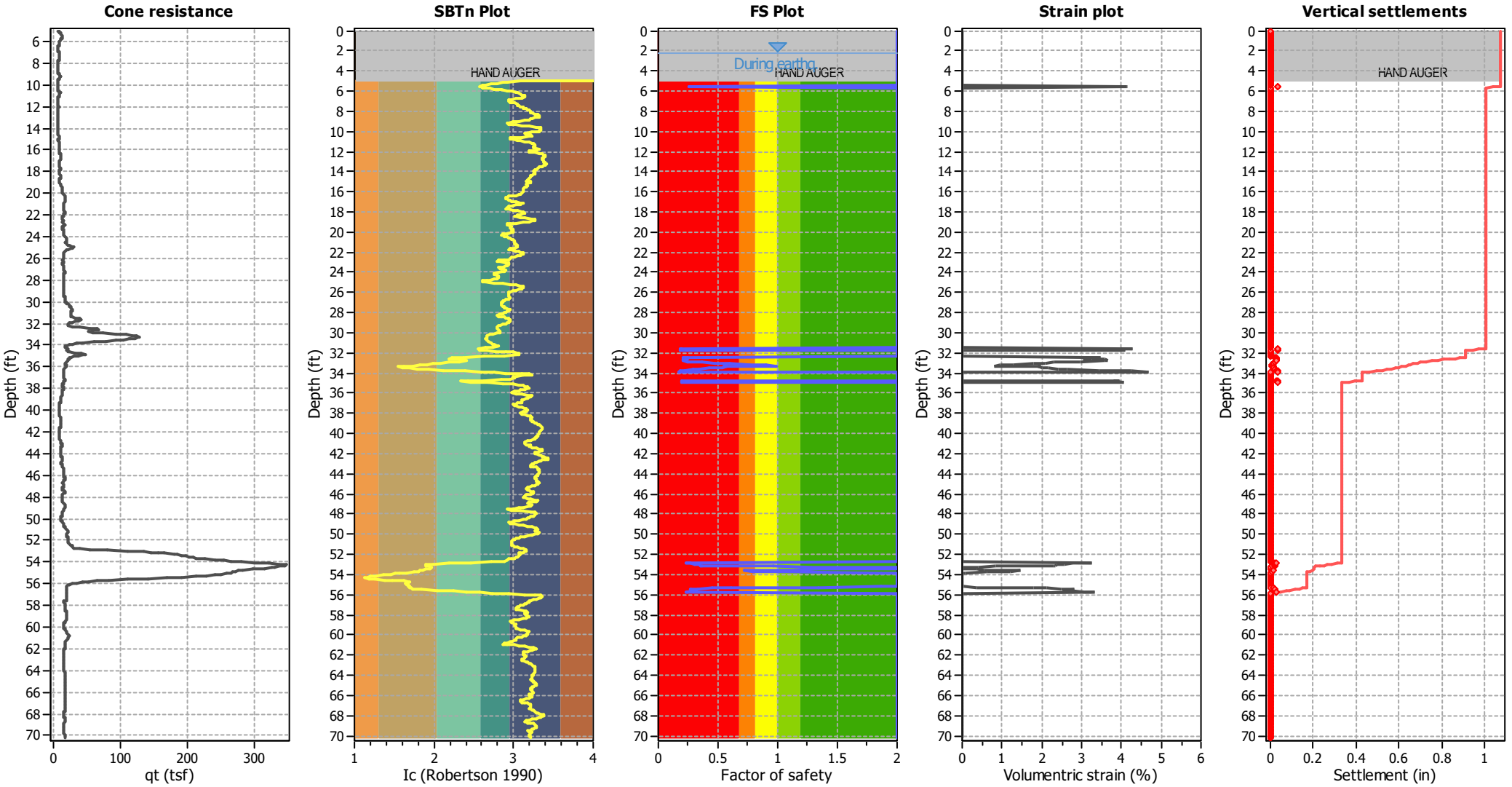
Check for strength loss plots (Idriss & Boulanger (2008))



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	14.59 ft	Fill height:	0.00 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
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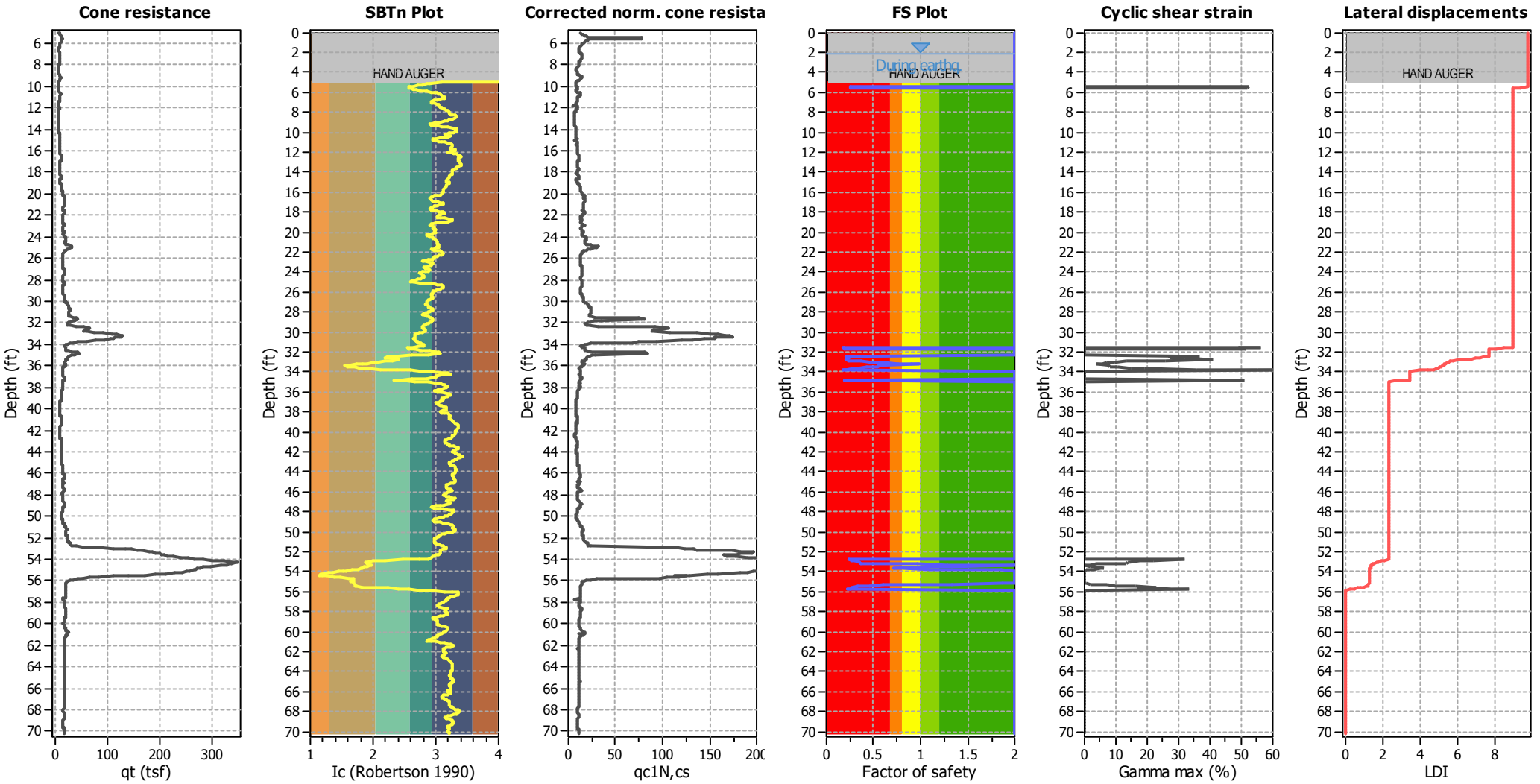
Estimation of post-earthquake lateral Displacements



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q_t: Total cone resistance (cone resistance q_c corrected for pore water effects)
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F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

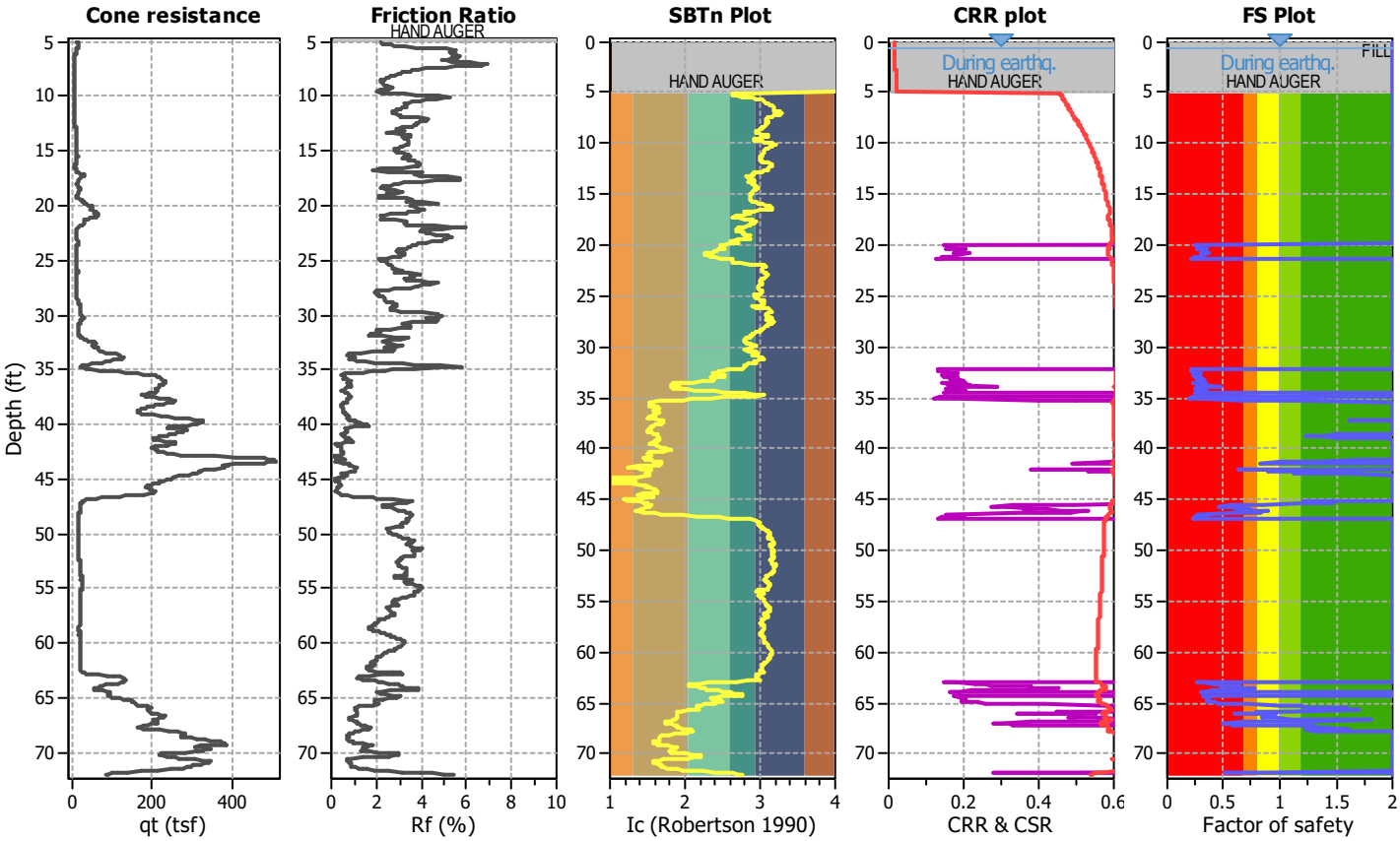
LIQUEFACTION ANALYSIS REPORT

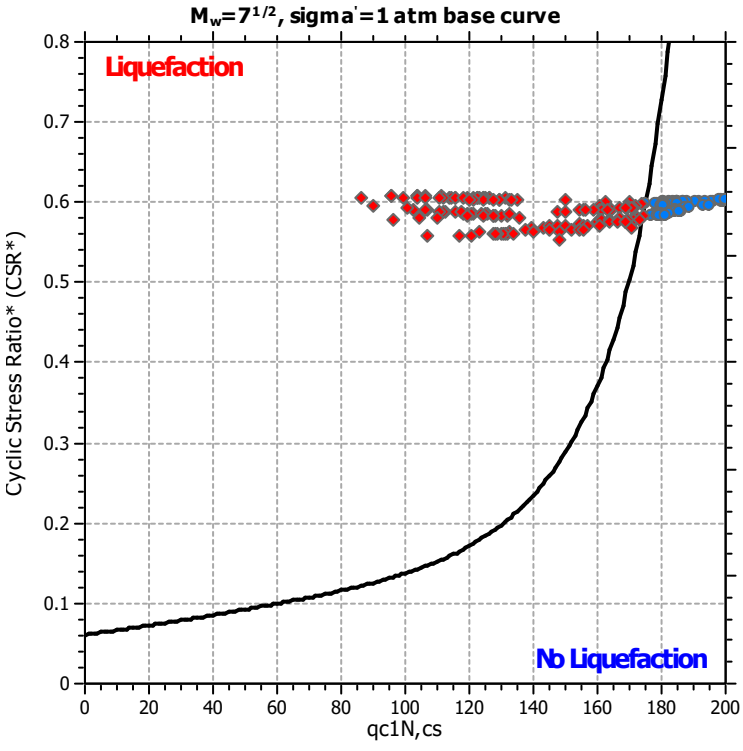
Project title : SAFER Bay Project (South of Bay Road) **Location : East Palo Alto, San Mateo County, California**

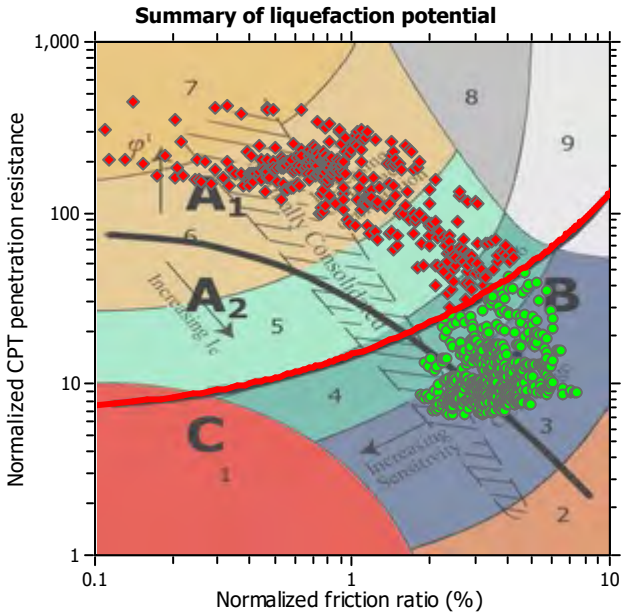
CPT file : GS26-CPT-05

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	9.67 ft	Use fill:	Yes	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	2.20 ft	Fill height:	1.60 ft	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	125.00 lb/ft³	Limit depth:	N/A
Earthquake magnitude M_w:	7.50	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.51	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		







Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
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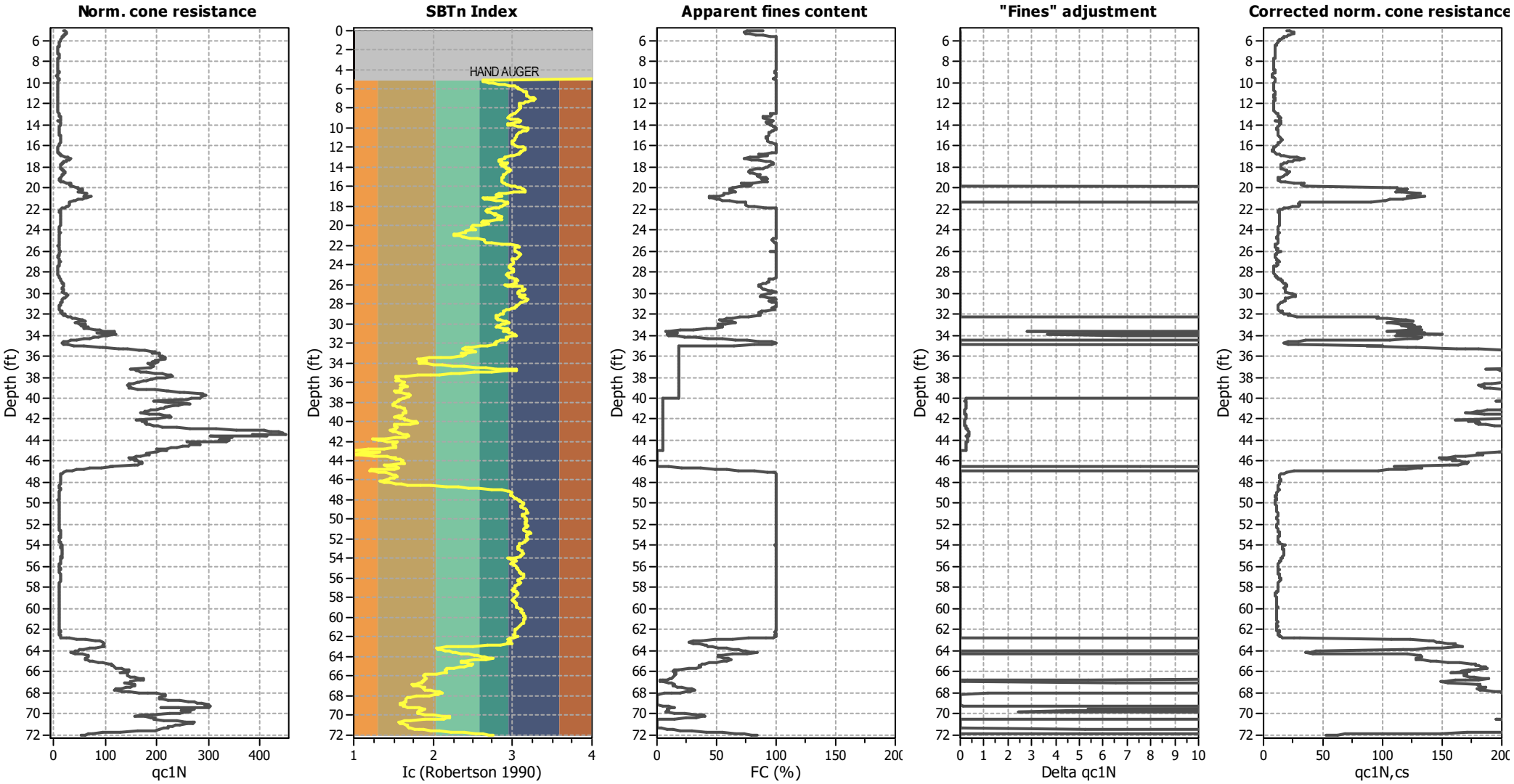
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

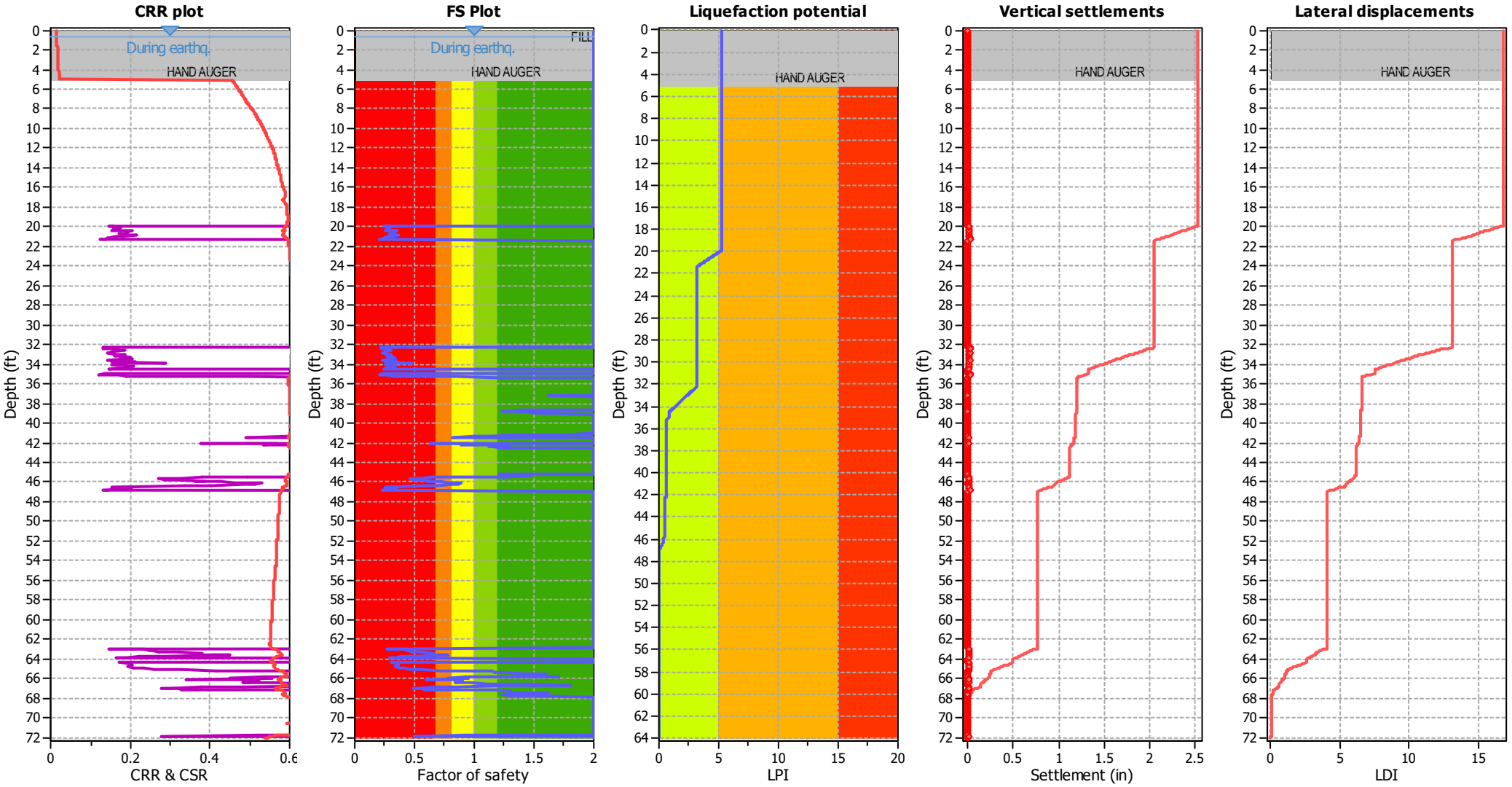
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	9.67 ft	Fill height:	1.60 ft	Limit depth:	N/A

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	9.67 ft	Fill height:	1.60 ft	Limit depth:	N/A

F.S. color scheme

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LPI color scheme

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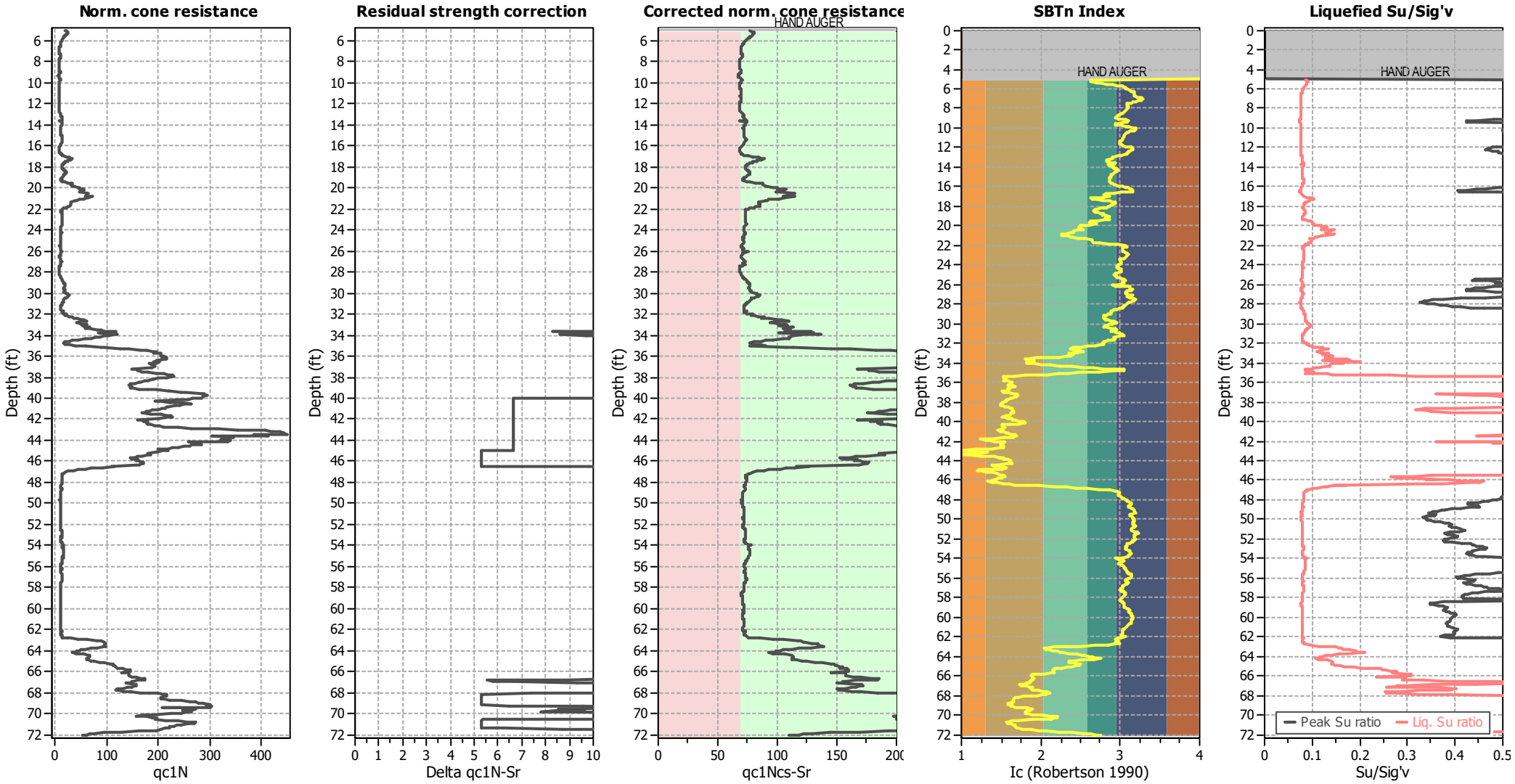
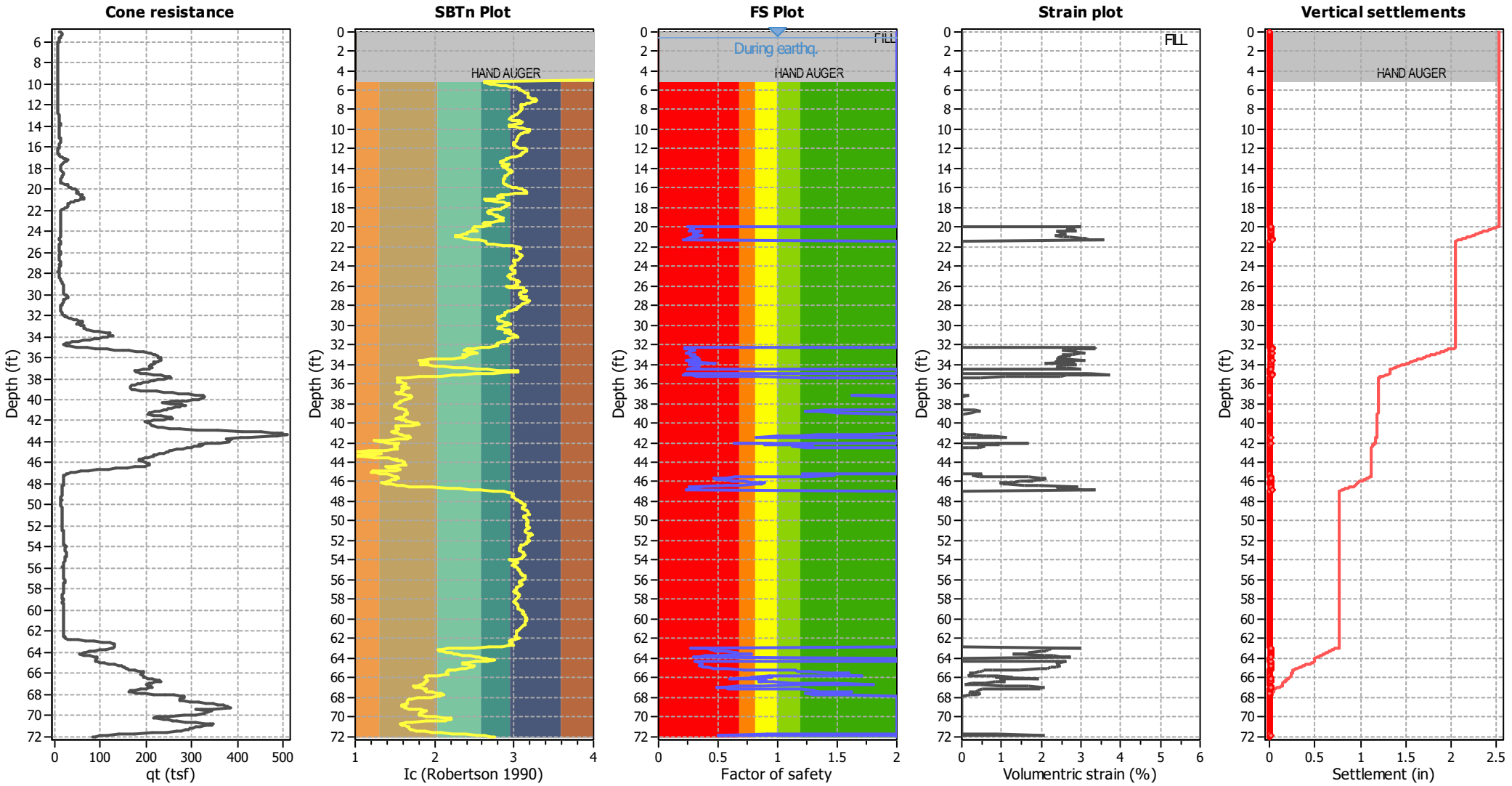


Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	2.20 ft	Fill weight:	125.00 lb/ft³
Fines correction method:	B&I (2014)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.50	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.51	Use fill:	Yes	Limit depth applied:	No
Depth to water table (insitu):	9.67 ft	Fill height:	1.60 ft	Limit depth:	N/A

Estimation of post-earthquake settlements



Abbreviations

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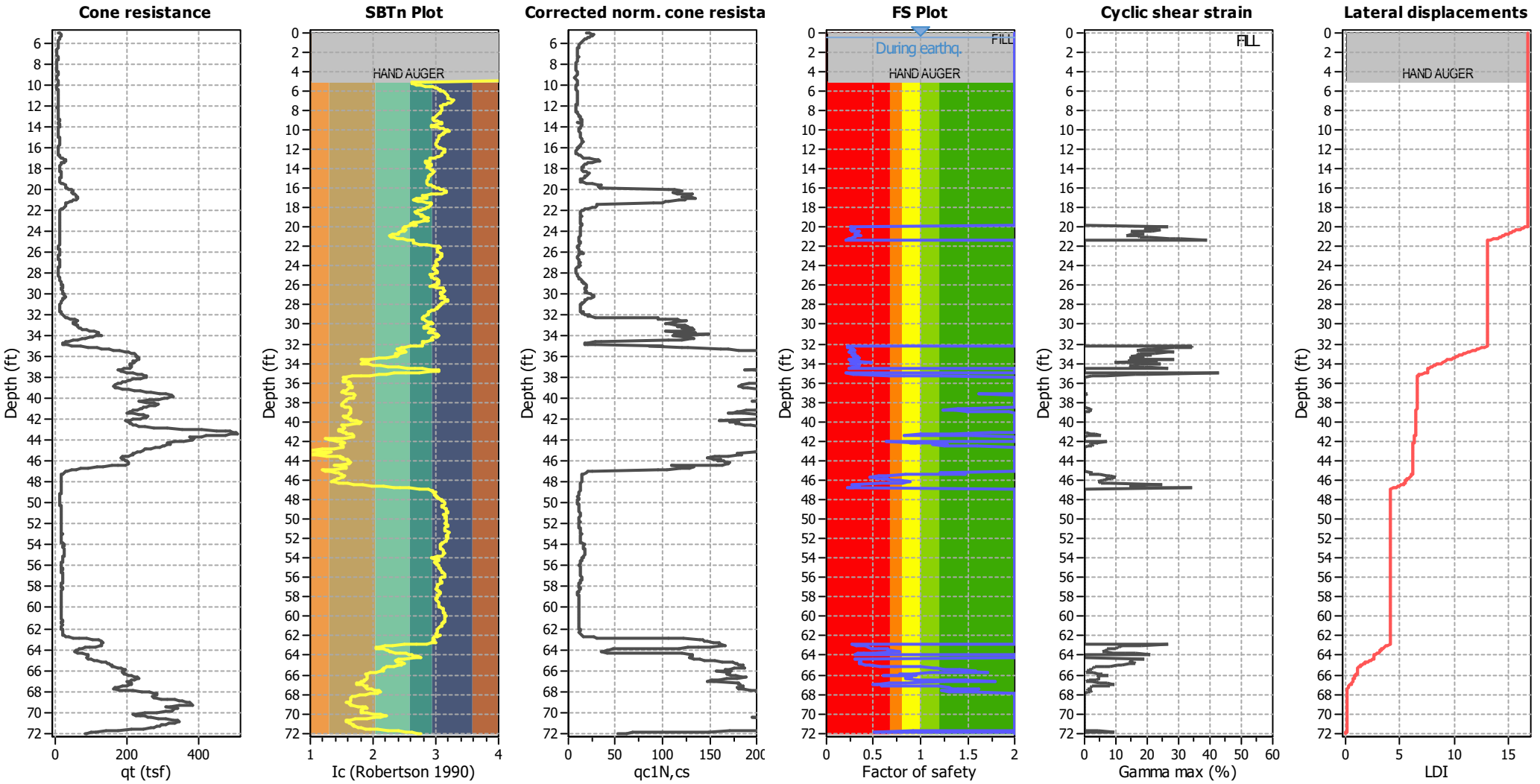
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Estimation of post-earthquake lateral Displacements



Abbreviations

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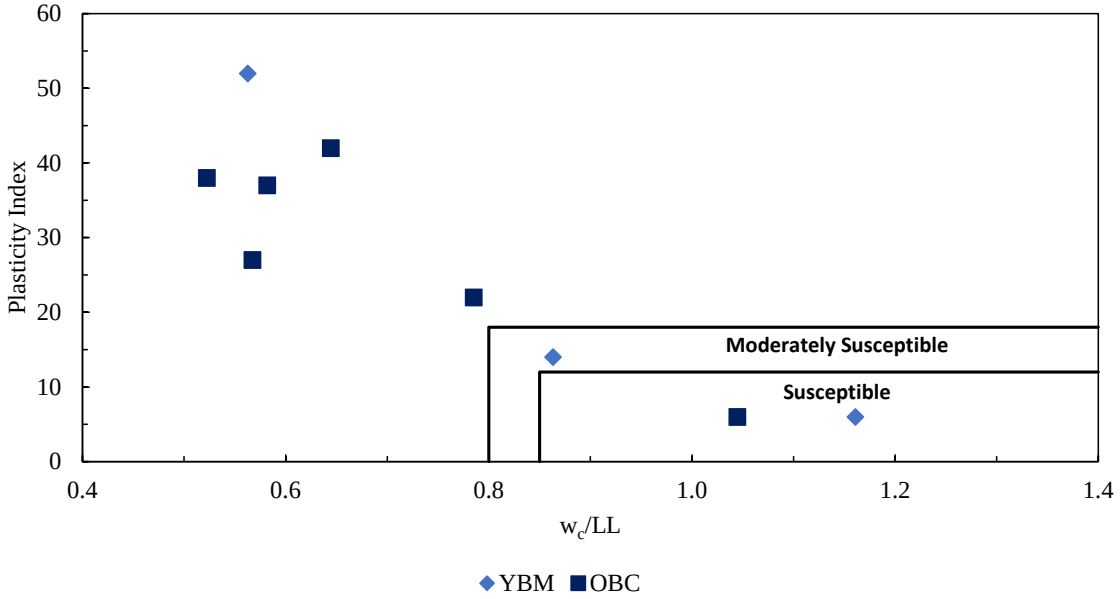
I_c: Soil Behaviour Type Index
$q_{c1N,cs}$: Equivalent clean sand normalized CPT total cone resistance
F.S.: Factor of safety
γ_{max}: Maximum cyclic shear strain
LDI: Lateral displacement index

ATTACHMENT B

Bray & Sancio (2006) Liquefaction Susceptibility Method Evaluation

Bray and Sancio (2006) Liquefaction Susceptibility Method
SAFER Bay Project (South of Bay Road)
East Palo Alto, California

Boring	Sample	Material Description	Soil Unit[1]	Depth (ft)	Liquid Limit LL	Plastic Limit PL	Plasticity Index PI	Water Content w_c (%)	w_c/LL	Susceptibility
GS26-B1	B1-2	Dark Gray Fat CLAY w/ Sand	YBM	7.5	69	22	47	-	-	Not[2]
GS26-B1	B1-3	Gray Fat CLAY	YBM	10	86	34	52	48.4	0.6	Not
GS26-B2	B2-3	Olive Brown Lean Clayey SAND w/ Gravel	YBM	15	30	16	14	25.9	0.9	Moderate
GS26-B2	B2-6	Yellowish Brown Lean CLAY w/ Sand, trace Gravel	OBC	25	47	25	22	36.9	0.8	Not
GS26-B3	B3-6	Olive Sandy Lean CLAY	OBC	25	38	16	22	-	-	Not[2]
GS26-B3	B3-12	Olive Gray Fat CLAY	OBC	55	61	24	37	35.5	0.6	Not
GS26-B3	B3-13	Olive Gray SILT w/ Sand	OBC	60	29	23	6	30.3	1.0	Susceptible
GS26-B4	B4-3	Olive Gray Sandy Silty CLAY	YBM	10	23	17	6	26.7	1.2	Susceptible
GS26-B4	B4-4	Dark Gray Fat CLAY w/ Sand	OBC	15.5	65	23	42	41.9	0.6	Not
GS26-B4	B4-9B	Greenish Gray Fat CLAY w/ Sand	OBC	36	58	20	38	30.3	0.5	Not
GS26-B4	B4-13	Olive Gray Lean CLAY	OBC	55	49	22	27	27.8	0.6	Not

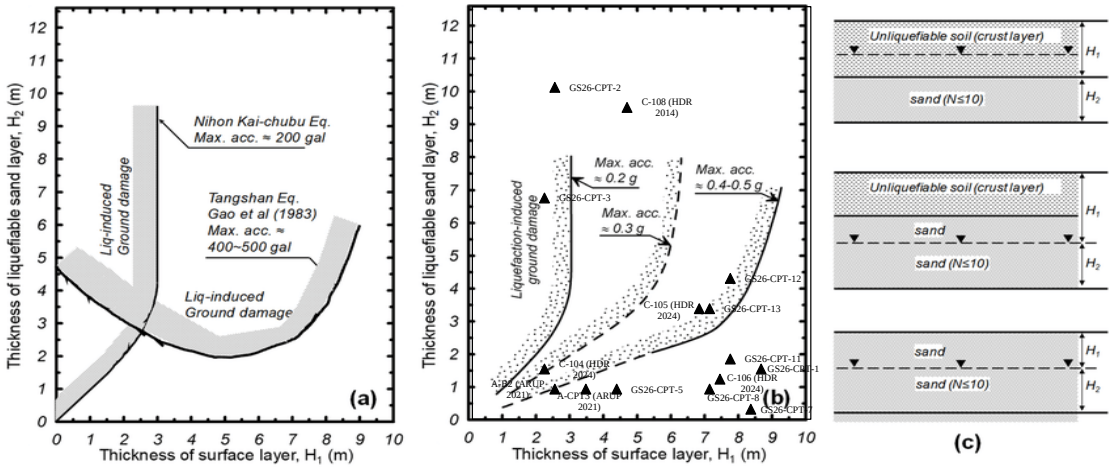


Notes:
1) Young Bay Mud = YBM, Old Bay Clay = OBC
2) Laboratory results of water content (w_c) was not available, however, soil would not be susceptible to liquefaction based on it's Plasticity Index regardless of w_c.
3) Granular/coarse-grained soil deposits (Units 4 and 5) are considered potentially susceptible to liquefaction based on plasticity and water content characteristics and are not considered as part of this Bray and Sancio (2006) liquefaction susceptibility review.

ATTACHMENT C
Ishihara (1985) Surface Manifestation Method Evaluation

Ishihara (1985) Surface Manifestation Method Evaluation
SAFER Bay Project (South of Bay Road)
East Palo Alto, California

CPT[1]	Station	Thickness of Surface Layer, H_1		Thickness of Liquefiable Sand Layer, H_2		Potential Liquefaction-Induced Surface Manifestation (Y/N)?
		(ft)	(m)	(ft)	(m)	
GS26-CPT-1	0+58	28	8.5	5	1.5	N
GS26-CPT-2	3+66	8	2.4	33	10.1	Y
GS26-CPT-3	6+93	7	2.1	22	6.7	Y
C-108 (HDR 2014)	9+12	15	4.6	31	9.4	Y
GS26-CPT-5	10+35	14	4.3	3	0.9	N
GS26-CPT-6	15+13	33	10.1	10	3.0	N
B-06 (HDR 2024)	16+39	34	10.4	4	1.2	N
GS26-CPT-7	18+27	27	8.2	1	0.3	N
GS26-CPT-8	20+86	23	7.0	3	0.9	N
C-106 (HDR 2024)	24+64	24	7.3	4	1.2	N
GS26-CPT-10	27+60	43	13.1	0	0.0	N
GS26-CPT-11	29+38	25	7.6	6	1.8	N
GS26-CPT-12	32+08	25	7.6	14	4.3	Y
GS26-CPT-13	33+23	23	7.0	11	3.4	Y
C-105 (HDR 2024)	34+23	22	6.7	11	3.4	Y
A-SCPT4 (ARUP 2021)	37+61	36	11.0	1	0.3	N
A-B3 (ARUP 2021)	37+43	36	11.0	4	1.2	N
GS26-CPT-14	38+19	36	11.0	4	1.2	N
GS26-CPT-15	42+41	46	14.0	1	0.3	N
A-B2 (ARUP 2021)	43+62	8	2.4	3	0.9	Y
C-104 (HDR 2024)	43+89	7	2.1	5	1.5	Y
A-CPT3 (ARUP 2021)	45+05	11	3.4	3	0.9	N
GS26-CPT-16	48+94	44	13.4	5	1.5	N
GS26-CPT-17	48+94	44	13.4	5	1.5	N
GS26-CPT-18	51+60	40	12.2	8	2.4	N



Notes:
1) Investigation points plotted on Ishihara (1985) Figure (b).

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Appendix A.4:
Levee/Floodwall Engineering Analysis
Sheet Pile Wall

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MAY 2026
SHERWOOD DESIGN ENGINEERS, LTD.

SAFER BAY EPA SHEET PILE WALL

60% DESIGN REPORT

PROJECT NO.	DOCUMENT NO.
A299737	001

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
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APPENDICES

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A.2 Geotechnical information
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Appendix C Sheet pile wall settlement analysis

Appendix D FLAC2D outputs

1 Design assumptions

1.1 Floodwall configuration

The SAFER Bay East Palo Alto Project subject to this report extends approximately 5,250 feet along an existing embankment from the O'Connor Pump Station at San Francisquito Creek to Bay Road in East Palo Alto.

The Project utilizes a sheet pile floodwall with embankment.

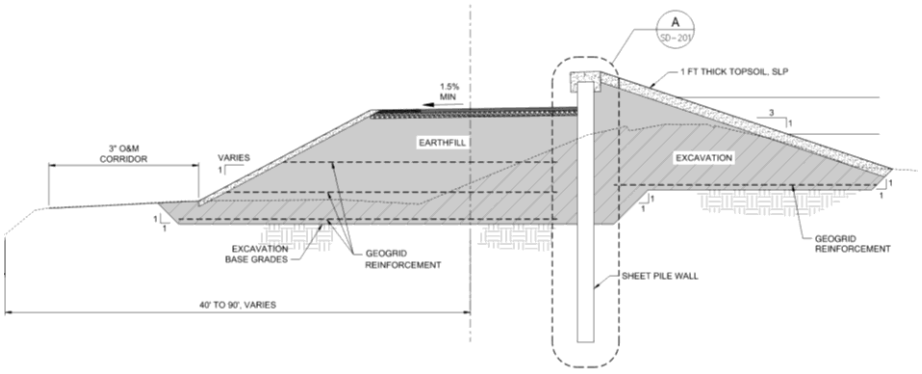


Figure 1-1 Typical floodwall with levee embankment design cross-section

Geometry of embankment has been provided by the Client and attached in Appendix A.1.

1.2 Sheet pile

Sheet pile is assumed to be AZ 20-800 with properties shown in Figure 1-2. Sheet pile will be ASTM A572 grade 50 with yield strength, F_y = 50 ksi. Plastic section modulus, Z = 43.3 in^3 per foot of wall.

Section	Dimensions				Properties						
	b	h	t_f	t_w	A	G_{sp}	G_w	I_y	w_{eLy}	S_y	W_{ply}
	in	in	in	in	in^2/ft	lb/ft	lb/ft^2	in^4/ft	in^3/ft	in^3/ft	in^3/ft
AZ 20-800	31.50	17.72	0.375	0.375	6.66	59.50	22.67	329.9	37.2	21.7	43.3

Figure 1-2 AZ 20-800 sheet pile properties

1.3 Corrosion mitigation

The required design life of the seawall is 50 years. Sheet piles will not be coated.

Corrosion rates considered follow the latest Caltrans (2022) guidance:

- 0.001 in per year for soil embedment zone,
- 0.0015 in per year for fill or disturbed natural soils,
- 0.002 in per year for atmospheric zone.

1.4 Water levels

The proposed embankment/floodwall is intended to provide 100-year flood protection with 3.5 feet of Sea Level Rise (SLR). The hydraulic loading scenarios and categories considered (according to USACE EM1110-2-2502) are summarized in table below:

Table 1-1: Hydraulic loading and loading scenarios

Type	**Current Day Loading** (w/o SLR)	**Anticipated Future Loading** (+3.5 ft SLR)	**Loading category**
HTOL	-	16.5	Extreme
DWSE	11.0	14.5	Unusual
MHHW	7.3	10.8	Usual

Notes: HTOL: Hydraulic Top of Levee, water at the top of floodwall elevation =100 yr plus 2 ft of freeboard plus 3.5 ft of SLR
DWSE: Design Water Surface Elevation of 11.0 ft NAVD88, based on the 100-yr flood elevation under current-day conditions.
MHHW: Mean Higher High Water, daily high tide.

Vertical datum for the project shall be in feet North American Vertical Datum of 1988 (NAVD88).

1.5 Soil profiles and properties

The soils stratigraphy and soil parameters used for the analyses were provided by Geosyntec and included in Appendix A.2.

1.6 Seismic parameters

Seismic design of the embankment / floodwall system has been based on ER 1110-2-1806 (2024), with reference to EM 1110-2-2502, and EM 1110-2-1913 for deformation assessment of the flood protection system. The project is classified as a critical structure because breach of the system could lead to significant economic consequences, property damage, and potential loss of life.

Two seismic hazard levels have been adopted in the design:

- the Operating Basis Earthquake (OBE) and
- the Maximum Design Earthquake (MDE).

The OBE represents a service-level event expected to occur within the service life of the structure, with the design objective that the levee / floodwall system remains serviceable following the event. For a critical structure, the OBE is taken with a minimum return period of 475 years.

The MDE represents the maximum design-level event, with the objective that the system does not breach and cause catastrophic flooding. Based on ER 1110-2-1806 (2024) for critical projects the MDE is taken as the larger of the 2,475-year return period ground motion or the Maximum Credible Earthquake (MCE). The MDE earthquake is taken as the 2,475 year earthquake based on the information provided by Geosyntec.

The updated simplified design parameters adopted for the present design stage are summarized in the following table:

Table 1-2: Simplified seismic design parameters

Design ground motion scenario	**Earthquake Return Period** (yr)	**Probability of Exceedance**	**PGA** (g)	**Rrup** (km)	**PGV*** (cm/s)	**Mw** (-)
OBE	475	10% in 50 years	0.51	15	78	7.5
MDE	2,475	2% in 50 years	0.77	13	137	7.9

*Evaluated by COWI using USGS Earthquake Hazard Tool for Lat 37.4686º, Long -122.1262, NSHM US 2023 and site class with Vs30 185 m/s.

Mw and Rrup from (modal values) deaggregation 475 and 2,475 at PGA are provided by Geosyntec. For the MDE, the 2,475-year Uniform Hazard Spectrum (UHS) provided by Geosyntec is adopted, as it is larger compared to the 84th percentile Maximum Credible Earthquake (MCE) UH spectrum. The UHSs PSA values provided by Geosyntec and shown in Table 1-3 are used as simplified input for the current design stage.

Table 1-3: 475 years and 2,475 years PSA (g) UHS

Period T (s)	Horizontal PSA (g)	
	MDE	**OBE**
0.01	0.771	0.507
0.02	0.773	0.505
0.03	0.758	0.497
0.05	0.837	0.541
0.075	1.062	0.678
0.1	1.261	0.809
0.15	1.501	1.003
0.2	1.637	1.121
0.25	1.776	1.219
0.3	1.932	1.306
0.4	2.082	1.355
0.5	2.095	1.33
0.75	1.83	1.117
1	1.656	0.953
1.5	1.321	0.729
2	1.111	0.575
3	0.763	0.38
4	0.551	0.265
5	0.417	0.195
7.5	0.226	0.102
10	0.136	0.061

Post-earthquake deformation acceptance criteria adopted in the design (260511 SAFER Bay EPA Final Design Criteria Report, May 2026) are a maximum allowable vertical displacement of 1.0 ft following the OBE and 3.4 ft following the MDE. If stability criteria are not satisfied simplified seismic analysis methods to estimate vertical displacement are applied at the present design stage. Given the critical project and high seismic hazard (ER 1110-2-1806, 2024) advanced analyses such as time-history, site response, and fully nonlinear deformation analyses are required for subsequent design phases.

1.7 Liquefaction

Geosyntec has performed the liquefaction assessment for the SAFER Bay EPA project site and has concluded that the liquefaction susceptibility is expected to vary throughout the soil profile. Liquefaction susceptibility is not expected to vary significantly between the OBE and MDE event. CPT based liquefaction hazard assessment for OBE and MDE (following Idriss and Boulanger 2008 method) and liquefaction induced settlements for coarse grained soil layers to a depth below ground level of 60-70 ft was carried out by Geosyntec. Within such depth both coarse grained units 4 and unit 5 are mostly liquefying as shown in Appendix A.3. A summary of the estimate of liquefaction induced settlements is shown in Table 1-4. For ACS6 results from closest CPT are not available and the highest of settlements at other location is assumed as representative. The post-liquefaction vertical settlement under either scenario varies along the alignment but is expected to range from approximately 0-6 inches for either seismic loading scenario. Geosyntec has provided the residual strength of liquefied soil and are reflected in Appendix A, Table 1.

Table 1-4: Summary of estimated liquefaction induced vertical settlements

Geo Section	CPT	Liquefaction Settlement OBE (in)	Liquefaction Settlement MDE (in)
ACS1 (10+35)	CPT-05	2.8	3.1
ACS2 (19+50)	CPT-06	3.4	3.5
ACS3 (27+50)	CPT-10	0.3	0.3
ACS4 (32+00)	CPT-12	5.5	5.7
ACS5 (38+00)	CPT-14	1.2	1.7
ACS6 (44+00)	NA	5.5	5.7

2 Loads and Load combinations

2.1 Earth Loads (EH/EV)

In the GeoStudio/FLAC2D models, lateral and vertical earth loads (EH and EV) are taken into account through the computed soil stress field generated by embankment self-weight, staged construction, groundwater conditions, and applied surface surcharge; therefore, earth pressures acting on the embedded sheet pile are derived directly from the analysis rather than applied as separate external loads.

2.2 Vehicle Live load (V)

Uniform surcharge loading (H-20 truck) at the trail area is assumed to be 480 psf acting at the crest on the entire width of the trail (behind the new floodwall). See §2.1.

2.3 Hydrostatic load (Hs)

Hydrostatic load is based on the project hydraulic loading scenarios defined in §1.4 Water levels. Accordingly, still water levels of 10.8 ft NAVD88, 14.5 ft NAVD88, and 16.5 ft NAVD88 are adopted for the Usual, Unusual, and Extreme load cases, respectively, with 16.5 ft NAVD88 corresponding to the HTOL at the top of floodwall elevation.

2.4 Seismic load (EQ)

Seismic load is applied using the OBE and MDE design ground motions defined in §1.6. Accordingly, the current design adopts PGA = 0.51 g, Mw = 7.5 for the OBE load case and PGA = 0.77 g, Mw = 7.9 for the MDE load case. The OBE is used as the serviceability-level earthquake, while the MDE is used as the extreme design event for which severe damage may occur, provided that breach and catastrophic flooding are avoided and the flood protection system can be repaired if required. At the current stage, these seismic loads are represented using simplified inputs consistent with the design basis described in §1.6.

2.5 Load combinations

Sheet pile design load combinations presented in Table 2-1 is based on floodwall loads combinations from USACE EM 1110-2-2502.

Table 2-1: Sheet pile design load combinations

LC	Load description	Load category	Load combination
LC-1A	Infrequent surge	**Unusual**	**D + EH + EV + Hs**
LC-1B	Maximum hydrostatic	**Extreme**	**D + EH + EV + Hs**
LC-2	Normal operating	**Usual**	**D + EH + EV + Hs**
LC-3A	Earthquake – OBE	**Unusual**	**D + EH + EV + EQ + Hs**
LC-3B	Earthquake – MDE	**Extreme**	**D + EH + EV + EQ + Hs**

Notes: Groundwater level used in LC-3A and LC-3B is at 3.3 ft NAVD88 for both landside and water side.

3 Geotechnical Analyses

The embankment/floodwall configuration given by the Client has been evaluated considering most up to date geotechnical information developed by Geosyntec and included in Appendix A. Based on the updated site conditions, the following key considerations remain valid for the project at 60 percent design stage:

- Settlement: Load from the embankment fill will cause settlement over time, primarily from consolidation of near-surface YBM deposits (with additional contribution from underlying fine-grained deposits).

- Seepage: During periods of high water when the embankment/floodwall are loaded, seepage can occur both through more pervious foundation layers (underseepage), and through more pervious reaches of the embankment (through-seepage). Both underseepage and through-seepage can initiate embankment failure modes including erosion, piping, slope instability which could lead to breach and interior flooding.

- Stability: The stability evaluation of the combined embankment/floodwall are updated to confirm that levee performance standards (i.e., factors of safety) can be achieved under key loading conditions described in Section 3.5.

Discussion of settlement, seepage, and stability analyses performed for the recommended embankment/floodwall to develop the 60 percent design is presented in the subsections below.

3.1 Cross sections for analyses

Geotechnical analyses were performed to address the key considerations described above based on the project cross sections and latest geotechnical information.

Five analysis cross sections (ACS) were developed to evaluate the settlement, seepage, and stability performance of the embankment/floodwall. Each cross section and the reasons for selection are described below:

- ACS1 (Station 10+35) shown in Appendix B.1, Figure 1 is taken along the southern part of the alignment adjacent to Faber Tract (east) and Bayshore Pond and the drainage channel (to the west). This cross section was selected to evaluate the effect of the pond on seepage and stability and to evaluate underseepage through the coarse-grained soil layers. A sheet pile wall extends to elevation - 40 ft. The new embankment lies within the footprint of the existing levee and is founded on YBM.

- ACS2 (Station 19+50) shown in Appendix B.1, Figure 2 is taken along the southern part of the alignment adjacent to Faber Tract (east) and the drainage channel (to the west). This cross section was selected to evaluate the effect of the drainage channel on seepage and stability. A sheet pile wall extends to

elevation - 25 ft. The new embankment lies within the footprint of the existing levee and is founded on YBM.

- ACS3 (Stations 27+50 / 32+00) shown in Appendix B.1, Figures 3a and 3b are taken along the middle part of the alignment adjacent to Faber Tract (to the east) and the drainage channel (to the west). These sections where selected to evaluate seepage and stability with a sheet pile wall toe within a coarse-grained soil layer. The sheet pile wall extends to elevation – 25 ft.

- ACS4 (Station 38+00) shown in Appendix B.1, Figure 4 is taken along the northern part of the alignment along the western margin of Laumeister Marsh and adjacent to the 1990 Bay Road property (to the west). This cross section was used to evaluate the effect of coarse-grained soil layer underlying YBM on underseepage performance of the embankment/floodwall system. A sheet pile wall extends to elevation – 25 ft. The new embankment lies outside the footprint of the existing levee, however, is founded on YBM.

- ACS5 (Station 44+00) shown in Appendix B.1,Figure 5 is taken along the northern portion of the alignment along the western margin of Laumeister Marsh and adjacent to open space within the 1990 Bay Road property. The embankment and sheet pile wall configuration is similar to ACS4. Similarly to ACS4, this cross section was selected to evaluate the effect of coarse-grained soil layer underlying YBM on underseepage performance of the embankment/floodwall system

The soil stratigraphy and soil parameters used for the analyses are based on the subsurface conditions by Geosyntec presented in Appendix A.2.

It should be noted that the soil parameters of the new embankment fill used in the analysis were assumed based on the parameters of the existing fill (Unit 1) or/and parameters developed for the 30 percent design phase, refer to Appendix A.2, Table 2.

3.2 Evaluation criteria

Geotechnical performance of the combined embankment / sheet pile floodwall system is evaluated with respect to settlement, seepage, static stability, and seismic response, using the project assumptions given in §1.4 Water levels, §1.5 Soil profiles and properties, and §1.6 Seismic parameters. The applicable hydraulic loading categories are Usual, Unusual, and Extreme, corresponding to the water levels defined in in §1.4.

Based on the updated project Design Criteria report, the geotechnical acceptance criteria that belong there are:

- Vertical exit gradient minimum FOS
  - 2.8 for Usual WSE = 10.8 ft
  - 2.0 for Unusual WSE = 14.5 ft
  - 1.6 for Extreme WSE = 16.5 ft

- Landside steady-state stability minimum FOS for deep-seated failure beneath the sheet wall
  - 1.6 for Usual
  - 1.5 for Unusual
  - 1.4 for Extreme

- Slope stability minimum FOS for surfaces that pass through the embankment fill only and do not pass through or beneath the floodwall
  - 1.5 for Usual
  - 1.4 for Unusual
  - 1.3 for Extreme

- End-of-construction stability minimum FOS
  - 1.3

- Waterside rapid drawdown minimum FOS
  - 1.3 for Unusual
  - 1.1 for Extreme

- Seismic acceptance criteria
  - OBE allowable vertical displacement = 1.0 ft
  - MDE allowable vertical displacement = 3.4 ft

The soil stratigraphy and engineering parameters used in the analyses are those provided by Geosyntec, as noted in §1.5 and included in Appendix A.

Seismic evaluation is based on the OBE and MDE design hazard levels described in §1.6, with seismic design referenced to ER 1110-2-1806 and deformation assessment referenced to EM 1110-2-2502 and EM 1110-2-1913.

The specific checks and acceptance criteria adopted for settlement, seepage, static stability, and seismic response are presented in §3.3 to §3.6, respectively.

The evaluation criteria are applied to assess the performance of the combined embankment and embedded sheet pile wall system, including verification of global stability, seepage control (cut-off performance), deformation compatibility, and structural demand on the sheet pile, based on the analysis results described in §4.

3.3 Settlements analysis

Settlement analyses were performed by Geosyntec using the computer program Settle3 from Rocscience which allows for estimating time-dependent soil settlement. The analysis was performed at five different cross sections (stations) using the stratigraphy and soil parameters provided in Appendix A.2.

The results and general assumptions to the calculations are provided in Appendix A.4

3.4 Seepage analysis

Through-seepage, underseepage and steady-state seepage were considered in the design. The through-seepage was assessed qualitatively. The underseepage and steady state seepage for each analysis cross section and hydraulic loading conditions were analysed in the finite element computer program SEEP/W.

3.4.1 Seepage analysis modelling

The permeability parameters for each soil layer used in the analysis are provide in Appendix A.2.

The boundary conditions of the models were modelled as follows:

- Nodes along the waterside ground surface were set to a constant head equal to a relevant hydraulic loading condition
- Nodes along the waterside vertical edge were set to a no-flow. The no-flow boundary was set based on the assumption that the waterside blanket is continuous. The waterside boundary was placed 2,000 ft from the centreline of the trail.
- Nodes along the landside ground surface were set to a potential seepage face.
- Nodes along the landside vertical edge were set to a constant head equal to the landside edge of the model or landside levee toe elevation whichever is lower. The landside boundary was placed 2,000 ft from the centreline of the trail.
- Nodes along the bottom of the model were set to a no-flow boundary condition.

The sheet pile wall was modelled as a water impervious barrier.

3.4.2 Through-seepage

Similarly to the assessment provided in the 30 percent design, the potential for the occurrence of through-seepage is considered very low. The proposed construction of the levee comprises medium to high plasticity cohesive soil with an embedded sheet pile wall, which creates an effective cut-off barrier.

3.4.3 Underseepage

Underseepage was analysed by comparing the ratio between the average vertical exit gradient at the landside toe of the embankment and the critical exit gradient to the minimum required factor of safety (chapter 3.2). The average vertical exit gradient, i_e, was calculated as the vertical head drop across the landside blanket layer divided by the thickness of the blanket layer. The critical exit gradient, i_c, was calculated as the ratio of the blanket's buoyant unit weight (maximum total unit weight of 112.5 pcf can be assumed) to the unit weight of water.

The underseepage analysis results are summarized in table given in Appendix B.2.

All underseepage analysis meet required minimum vertical gradient FOS for the levee toe. The vertical gradient criterion was not met at the pond, located further landside from the embankment toe, for the usual hydraulic loading condition reaching slightly lower than required FOS of 2.7. However, following the recommendations provided in EM 1110-2-2502, for areas located further from the levee toe the minimum factor of safety of 1.6 can be applied based on EM 1110-2-1913.

3.5 Stability analysis

Stability analyses were performed for the representative analysis cross sections defined in §3.1 using the GeoStudio modelling approach adopted for the project. The analyses evaluate the behaviour of the combined embankment / embedded sheet pile wall system under the hydraulic and construction loading conditions defined in §1.4 and Section 2, and are assessed against the minimum acceptance criteria given in §3.2. The assessment includes long-term steady-state stability, end-of-construction stability, and rapid drawdown stability, as applicable. The analyses are used to confirm that the proposed embankment geometry and sheet pile wall configuration provide adequate global stability and are consistent with the structural design assumptions presented in Section 4. Further results are presented in Appendix B.

3.5.1 Modelling details

Soil parameters utilized in the GeoStudio models were provided by Geosyntec and are shown in Appendix A.2.

Similarly to the 30 percent design, the new embankment fill was reinforced with geogrids. Fortrac 55 geogrid by Huesker was selected for analysis with an uniaxial long-term design strength of 2,207 pounds per foot of length. The geogrid layers were modelled as “reinforcement lines”. For the landside steady state stability analysis the pullout resistance was calculated based on provided interface shear angle and surface area factor. The interface shear angle of 20˚ and 21˚ was chosen for the bottom layer atop of YBM soil and for overlying layers embedded completely in the new fill, respectively. The surface area factor was set to 2. For the rapid drawdown and end of construction analysis the pullout resistance was specified to 350 psf and 750 psf for the bottommost and overlying layers, respectively.

Tension cracks were modelled along the crest in rapid drawdown and steady state landside slope stability analysis. It was assumed that the tension crack zone could penetrate to a depth of 4 ft or the upper geogrid layer, whichever is shallower. The tension cracks were modelled as filled with water. Similarly to the 30 percent design, the tension cracks were not modelled for the end of construction stability analysis assuming that the construction period is not long enough for the tension crack to develop.

3.5.2 Landside steady-state stability

Landside steady-state stability analyses were performed assuming steady-state seepage conditions and long-term effective strength parameters. The analyses used the steady-state pore-water pressures obtained from the seepage analyses described in §3.4, and assessed potential slip surfaces passing through and beneath the embankment crest area to evaluate the long-term stability of the combined embankment / embedded sheet pile wall system. Results of the landside stability analyses are presented in Appendix B.3.

The analyses indicate that the representative analysis cross sections satisfy the minimum required factors of safety defined in §3.2.

3.5.3 Waterside rapid drawdown stability

Waterside rapid drawdown conditions were evaluated by assuming an instantaneous reduction in waterside level from the flood level to the waterside toe elevation. This represents a full drawdown condition and is considered a conservative modelling assumption. Rapid drawdown analyses were performed using both drained and undrained strength parameters in accordance with the staged rapid drawdown procedure of Duncan et al. (1990).

The analyses used the seepage conditions corresponding to the flood levels defined in §1.4, and the results are presented in Appendix B.4. The representative project cross sections satisfy the minimum required rapid drawdown stability criteria defined in §3.2.

3.5.4 End of construction stability

End of construction (EOC) stability was analyzed for both the waterside and landside slopes, and the more critical case (i.e., the side with lower stability FOS) between the landside and waterside stability is reported herein.

Results of the EOC stability analyses are presented in Appendix B.5. The proposed configurations at each representative cross section exceed the minimum required EOC stability FOS of 1.3.

Based on the results of these EOC stability analyses, proposed embankment/floodwall can be constructed in one stage (i.e., staged embankment construction would not be needed).

3.6 Seismic stability analysis

Seismic analysis was carried out for the representative project sections selected for seismic evaluation using the design earthquake levels defined in §1.6 Seismic parameters and assessed against the acceptance criteria given in §3.2. The seismic assessment of the embankment / embedded sheet pile wall system was undertaken using a combination of pseudo-static stability analysis, deformation assessment, and structural demand analysis.

Pseudo-static seismic stability analyses were performed in GeoStudio using the SLOPE/W module, in which earthquake loading is represented by a static inertial horizontal force (Kh) acting on the potential sliding mass equal to 0.5 times the PGA at outcrop (EM 1110-2-2502) provided in section 1.6. These analyses were used to assess the seismic stability of the embankment / floodwall system and to determine the corresponding factors of safety under the applicable seismic loading conditions.

Seismic deformations of the embankment were evaluated separately using the Bray and Macedo (2019) method, based on the project seismic inputs defined in §1.6 and the yield condition derived from the stability analyses. These deformation estimates were used to assess compliance with the project seismic allowable vertical displacement performance criteria described in §3.2.

According to EM 1110-2-2502 pseudostatic methods with the limit equilibrium approach can be used to estimate displacement if soil are not susceptible to liquefaction and suitable minimum safety factor using pseudostatic methods (Kh) range between 1.1 and 1.2. An approximate estimate of slope displacement was carried out decoupling the settlement due to earthquake and that due to liquefaction (“psedostatic + liquefaction” in Appendix B.7). First the FoS with the pseudostatic method in GeoStudio is evaluated neglecting the effect of liquefaction on soil strength (assuming FoS equal 1.2 is the acceptable threshold), in case the FoS criterium is not complied, in case the FoS criterium is not satisfied the Bray and Macedo (2019) method to evaluate total displacement is applied and then the vertical settlement from liquefaction (see 1.7) is added on top. In an alternative approach (“post-liquefaction pseudostatic” in Appendix B.7) the pseudostatic method in GeoStudio includes the effect of liquefaction on soil strength (assuming FoS equal 1.1 is the acceptable threshold) then, if the FoS criterium is not satisfied the Bray and Macedo (2019) method is applied to evaluate the total displacement. If the total displacement is below threshold vertical displacement the simplified check is considered satisfied and viceversa.

It is noted that in both cases pseudostatic analyses, as end of construction analyses, can consider composite failure surface constrained within the critical Young Bay Mud. For the case of “post-liquefaction pseudostatic” in some cases the effect of removing this constraint and introducing reduced soil strength do not lead to a lower FoS. In other cases the liquefying layer is too deep and has no effect on the FoS. In these two cases the “post-liquefaction pseudostatic” method is considered not applicable (“NA”).

The structural demand on the embedded sheet pile wall under seismic loading was evaluated separately using pseudostatic analyses in FLAC2D where together with the vertical gravity vector of amplitude g an horizontal vector of amplitude 0.5 PGA was applied acting in the critical (landward) direction. These analyses (see Appendix D) were used to obtain the seismic forces in the sheet pile for the structural verification presented in §4.3 and §4.5. A Mohr Coulomb soil model is used where, based on the results from liquefaction assessment, the strength properties of the soil are reduced to residual where applicable. A linear stiffness model is adopted considering a reduced stiffness obtained from small strain stiffness corresponding to the applicable Vs_{100} and reducing it by a factor 2.

The sheet pile wall was modelled as a structural beam using the parameters stated in Figure 1-2.

Where applicable, the seismic assessment considers the influence of cyclic strength degradation and post-liquefaction strength reduction, consistent with the project design basis presented in §1.6 and §1.7. Results of the seismic analyses are presented in Appendices B.6 to B.8 and Appendix D.

Based on the results shown in Appendix B.6 to B.8 it is concluded that sections ACS1 (10+35), ACS3 (32+00) and ACS5 (44+00) are, in order of decreasing likelihood, the most likely cross sections not to comply with the vertical displacement threshold. Based on the liquefaction potential (Appendix A.3) ACS1 is deemed representative of the sections between 2+00 and 12+50, ACS3 of the sections between 30+00 and 38+50 and ACS5 of the sections between 43+00 and 46+00. As shown in Figure 3-1, these reaches are ranked in order of decreasing seismic risk, with the thickness of the red bars providing a visual indication of the relative severity and priority for non-linear effective stress time-history analyses.

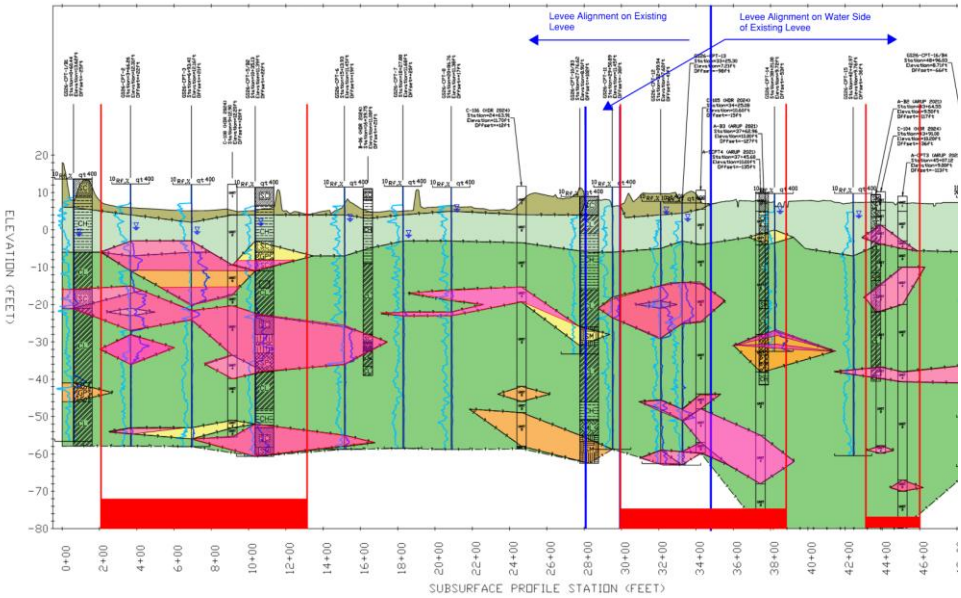


Figure 3-1 Graphical representation of areas with varying seismic risk along the alignment

4 Sheet pile wall design

This section presents the structural verification of the cantilever sheet pile floodwall for the 60% design, including the selection of a sheet pile section (type/size and steel grade) and associated embedment assumptions sufficient to support design development and to enable quantification of the BoQ.

The outcomes of this section are the governing demand/capacity checks, a recommended preliminary sheet pile section, estimated long term settlement and tip levels required along the alignment.

4.1 GeoStudio analysis models

The sheet pile wall was modelled using a finite element software SIGMA/W by GeoStudio, which performs stress–deformation and soil–structure interaction analyses.

Both, drained and undrained, soil conditions were analysed. The drained and undrained soil conditions were modelled using Mohr-Coulomb and Tresca (for cohesive soils) model, respectively.

The sheet pile wall was modelled as a structural beam using the parameters stated in Figure 1-2. The elastic modulus of steel was assumed as 200 GPa.

The boundary conditions were set to fully fixed at the bottom of the model and a horizontal fixation was applied along the left and right-side edge of the model.

The traffic load was modelled as constant stress of 480 psf in vertical direction along the entire width of the trail.

The landside water level was assumed at 3.3 ft NAVD88. The waterside water level was assumed accordingly to the hydraulic loading condition.

4.2 GeoStudio analyses results

The maximum bending moment along the sheet pile wall for different hydraulic loading conditions is presented in Table 4-1.

Table 4-1 Maximum bending moment along the sheet pile wall obtained for selected analysis cross sections

LC \ STA	10+35	19+50	27+50 / 32+00	38+00	44+00
1A - Unusual - M_{max} (kip-ft/ft)	21.6	21.0	30.0	17.5	10.0
1B - Extreme - M_{max} (kip-ft/ft)	26.6	21.2	43.0	18.5	15.2
2 - Usual - M_{max} (kip-ft/ft)	15.9	20.7	20.8	20.8	8.9

Notes: M_{max} = maximum bending moment per foot of wall
STA = station of selected analysis cross section

Further GeoStudio outputs are presented in Appendix B.9.

4.3 FLAC2D models and analyses results

A 1-ft width of the cantilevered sheet pile was modelled in FLAC2D. FLAC2D results after OBE and MDE are presented in Table 4-2.

Table 4-2 FLAC2D analysis results along floodwall alignment

LC \ STA	10+35	27+50	44+00
3A - OBE - M_{max} (kip-ft/ft)	22.3	pending	pending
3B - MDE - M_{max} (kip-ft/ft)	65.5	pending	pending

Notes: M_{max} = maximum bending moment per foot of wall
STA = station of selected analysis cross section

Further FLAC2D outputs are presented in Appendix D.

4.4 Corroded sheet pile properties

Following §1.2, some corrosion of sheet piles is expected after the 50-year design life. Estimated sheet pile thickness losses and reduced elastic section moduli of the AZ 20-800 sheet pile wall are as follows:

- Above trail elevation:
  - Landside in atmospheric zone and waterside in fill/disturbed natural soils: 0.175-in loss in 50 years
  - Reduced elastic section modulus, S = 22.9 in^3 per ft of wall

- Below trail:
  - Both landside and waterside in fill/disturbed natural soils: 0.15-in loss in 50 years
  - Reduced elastic section modulus, S = 25.0 in^3 per ft of wall

4.5 Sheet pile structural design

GeoStudio/FLAC2D analysis results are summarized in Table 4-1 and Table 4-2 respectively. Structural verification of AZ 20-800 sheet pile is presented in Table 4-3. The check takes the elastic instead of the plastic section modulus as input, so the calculation is slightly conservative. Allowable stresses depend on categories given by EM 1110-2-2504:

- Usual: $F_b=0.5F_y=25.0$ ksi
- Unusual: may be increased by 33%. $F_b=1.33(0.5F_y)= 33.2$ ksi
- Extreme: may be increased by 75%. $F_b=1.75(0.5F_y)= 43.7$ ksi

Table 4-3 AZ 20-800 sheet pile wall structural verification

LC	M_{max} (kip-ft/ft)	S_{red} (in^3/ft)	$f_b=M_a/S$ (ksi)	F_b (ksi)	D/C (-)
1A - Unusual	30.0	22.9	15.72	33.25	0.48
1B - Extreme	43.0	22.9	22.53	43.75	0.51
2 - Usual	20.8	22.9	10.90	25.00	0.44
3A - Unusual	22.3	22.9	11.68	33.25	0.35
3B - Extreme	65.5	22.9	34.32	43.75	0.78

Notes: M_{max} = maximum bending moment per foot of wall
S_{red} = elastic section modulus per foot of wall reduced by corrosion
f_b= actual bending stress
F_b= allowable bending stress
D/C = demand-capacity ratio

4.6 Sheet pile wall embedment

The floodwall will consist of a steel sheet pile wall embedded within the embankment along the waterside crown hinge point. The sheet pile wall will include a concrete cap. The sheet piles will extend to a tip elevation as per Table 4-4.

Table 4-4 Sheet pile wall tip elevations along floodwall alignment as per ACS

ACS / STA	Start STA	End STA	Tip elevation (ft NAVD88)
1 / 10+35	0+00	18+00	-40.0
2 / 19+50	18+00	24+64	-25.0
3 / 27+50 && 32+00	24+64	35+50	-25.0
4 / 38+50	35+50	43+00	-25.0
5 / 44+00	43+00	52+00	-25.0

Notes: ACS = analysis cross section
STA = station

These tip elevations were selected to achieve adequate embedment and to provide for groundwater cutoff for underseepage, based on geotechnical analyses presented in §3.

4.7 Sheet pile wall settlement analysis

As mentioned previously, placement of the new embankment fill will induce settlement of the underlying soils. Relative downward movement of the surrounding soil in the upper part of the sheet pile generates negative skin friction, which is resisted by positive shaft resistance mobilised at greater depth where the relative movement between pile and soil reduces. The depth at which negative skin friction changes to positive shaft resistance is referred to herein as the equilibrium plane. (Fellenius, “Basic of foundation design”, 2026).

Sheet pile wall settlement was evaluated using the unified method described above. Negative skin friction and positive shaft resistance were calculated from the effective stress profile using the adopted β-method parameters for the relevant soil units. The ß coefficients for different soil layers are provided in Appendix A.2. The soil settlement profile for station 42+50, presented in Appendix A.4, was used as a reference due to largest calculated soil settlement. The sheet pile wall shortening due to drag force was assumed negligible and the contribution of toe resistance conservatively neglected.

For the assessed section, the calculated equilibrium plane occurs at approximately elevation -11 ft NAVD83. The resulting estimated sheet pile head settlement is 1.8 in.

On this basis, the predicted sheet pile settlement is considered limited for the current design stage. Extension of the sheet pile length solely to reduce settlement is therefore not recommended as a practical settlement-mitigation measure.

The calculation details are presented in Appendix C.

5 Limitations

This report has been prepared for the use of Sherwood Engineers, in accordance with generally accepted engineering practice. No warranty, express or implied, is made. The analyses and recommendations submitted are preliminary and intended for use in 60% design phase. The analyses and recommendations are based on the data available to COWI at the time of preparing this report.

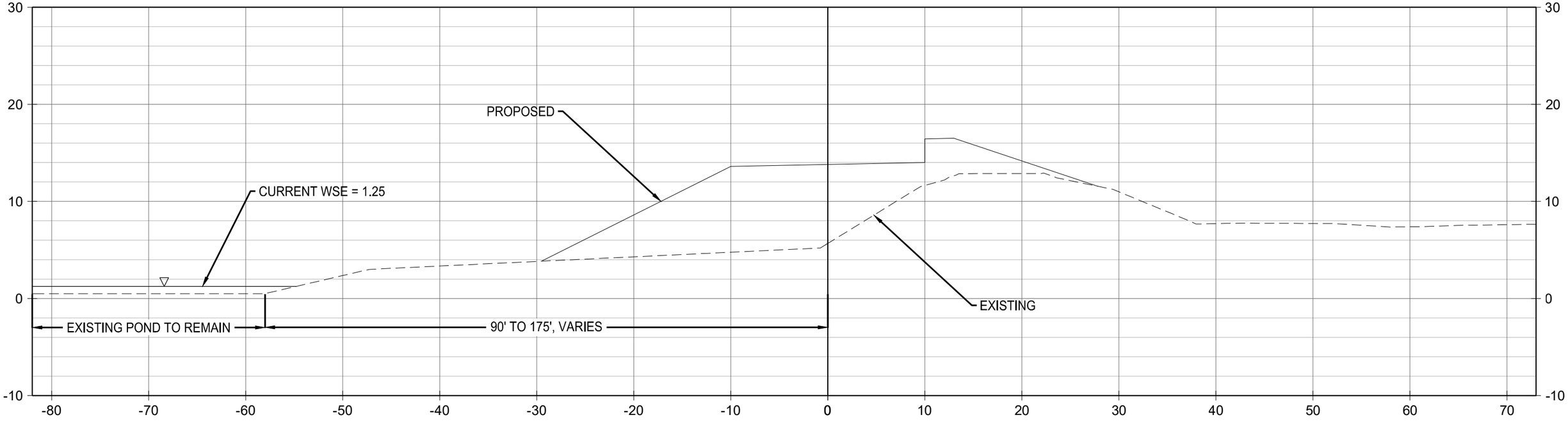
Main limitation related to the seismic assessment of the levee embankments are related to the simplified representation of the seismic load using a pseudostatic approach rather than a time history approach. In order to better capture the local effect of local soil conditions, a time history approach ideally would require the propagation of the seismic shaking starting from an engineering bedrock having Vs30 (m/s) not below 360 m/s. This would require the Vs measurements to be sufficiently deep so that the harmonic average in the deepest 30m (Vs30) reaches 360 m/s.

Due to the criticality of the project and the high seismicity of the area after shocks analyses (ER-1110-2-1806) using post mainshock conditions should be carried out for both OBE and MCE.

The time history analyses should consider non-linear soils models, able to account for cyclic response and effect of deviatoric and volumetric shear strains including stiffness and strength degradation due to excess pore water pressure build up during shaking.

Appendix A Inputs and parameters

A.1 Geometry of embankments



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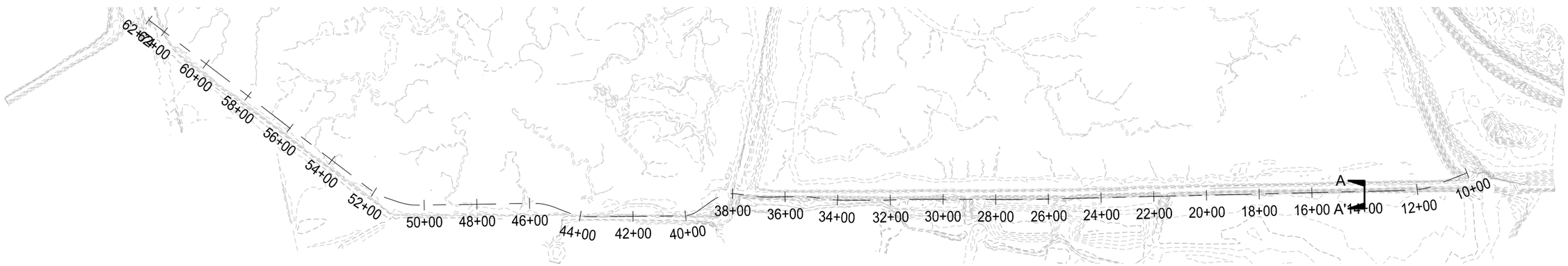

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DRAINAGE CHANNEL SECTION - STA 11+25 TO STA 15+50

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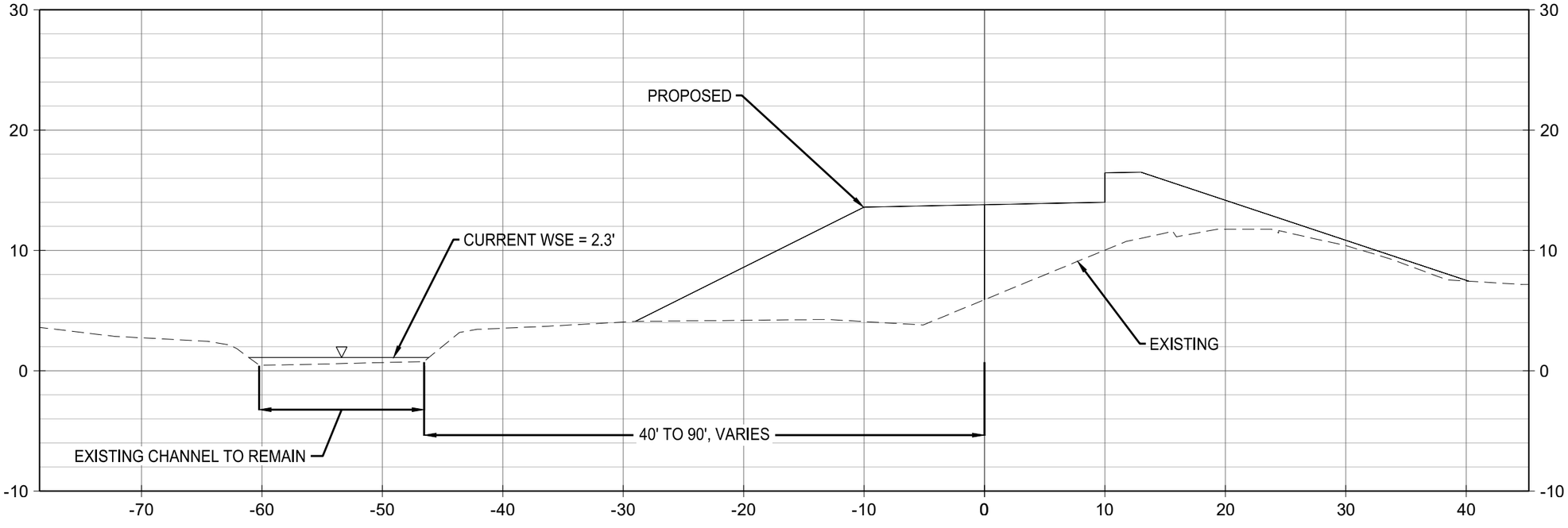
EAST PALO ALTO, CA

April 21, 2026



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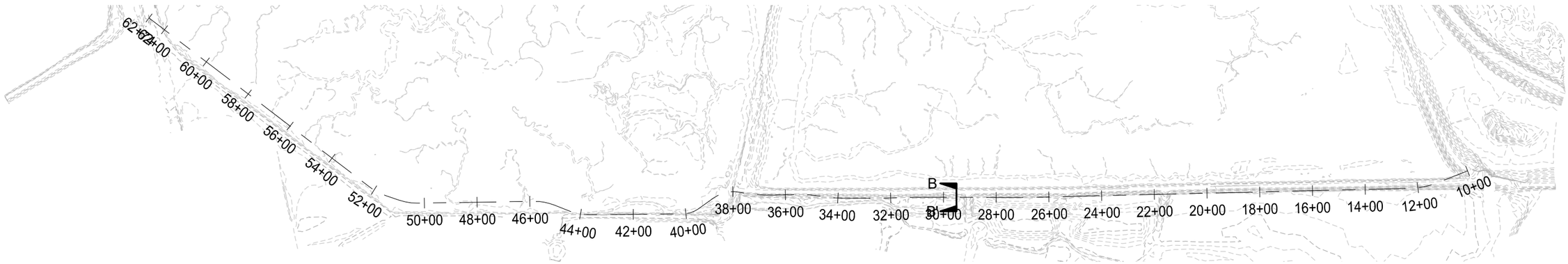

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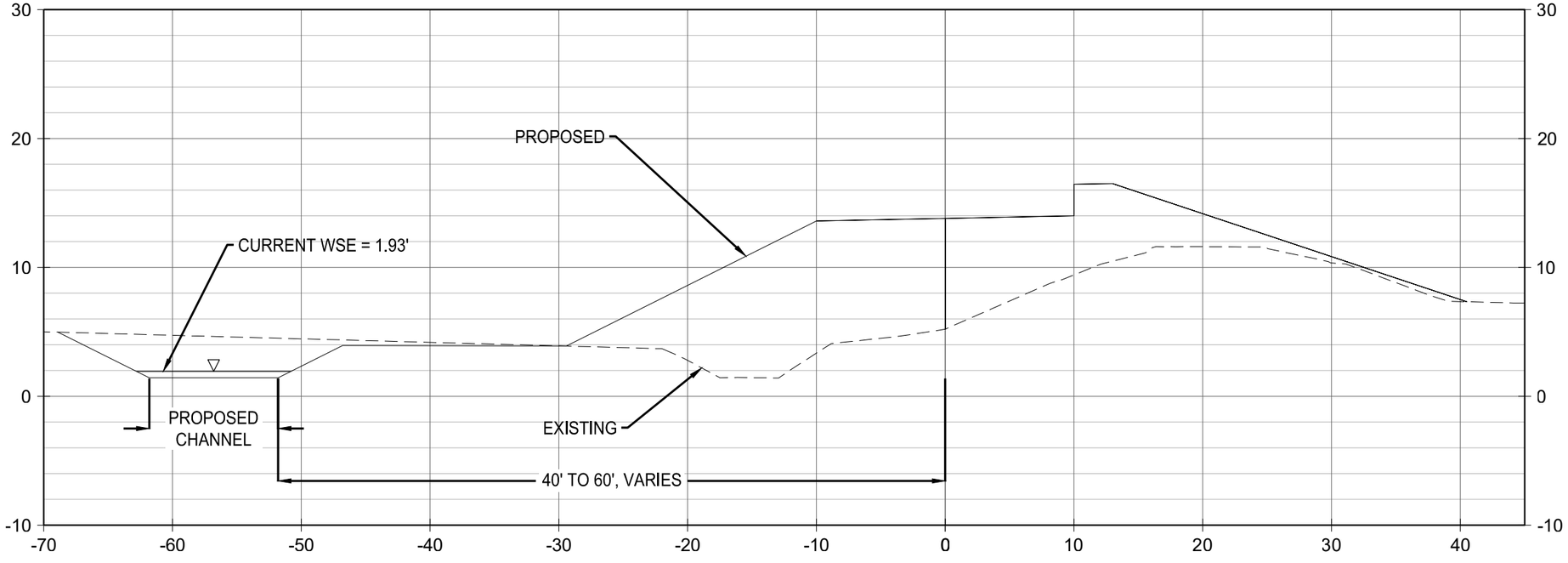
April 21, 2026



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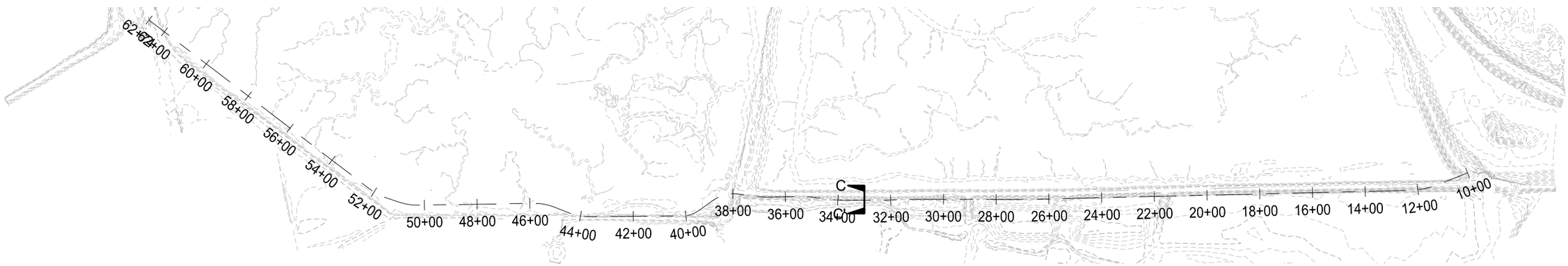

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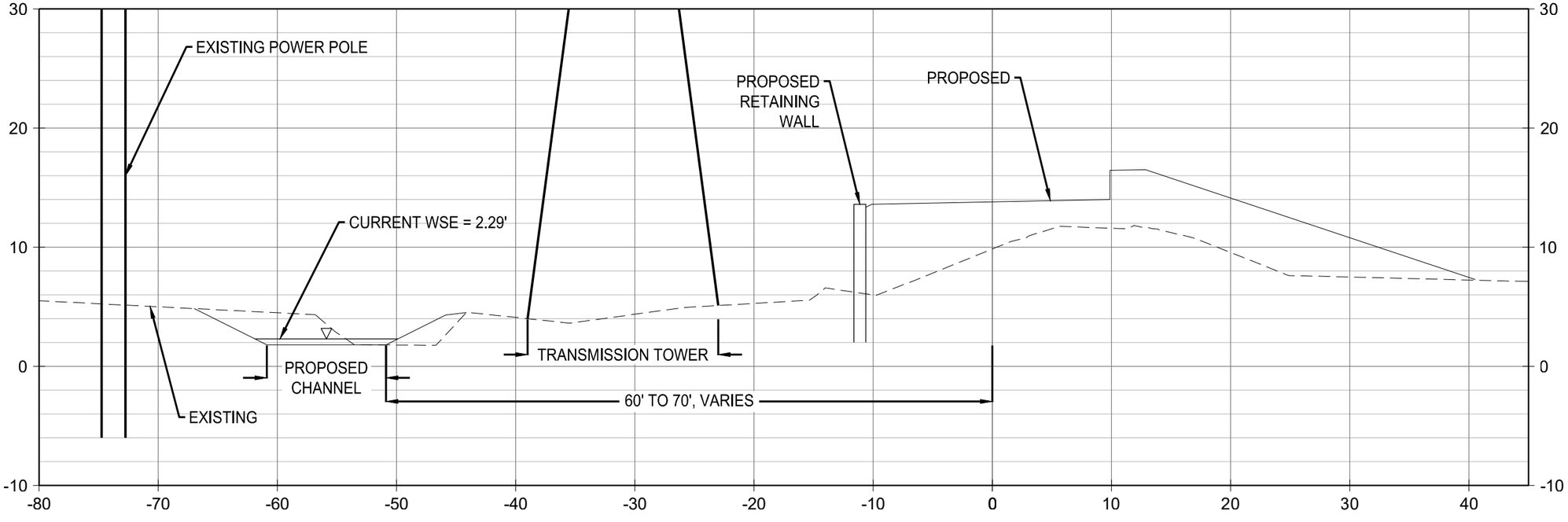

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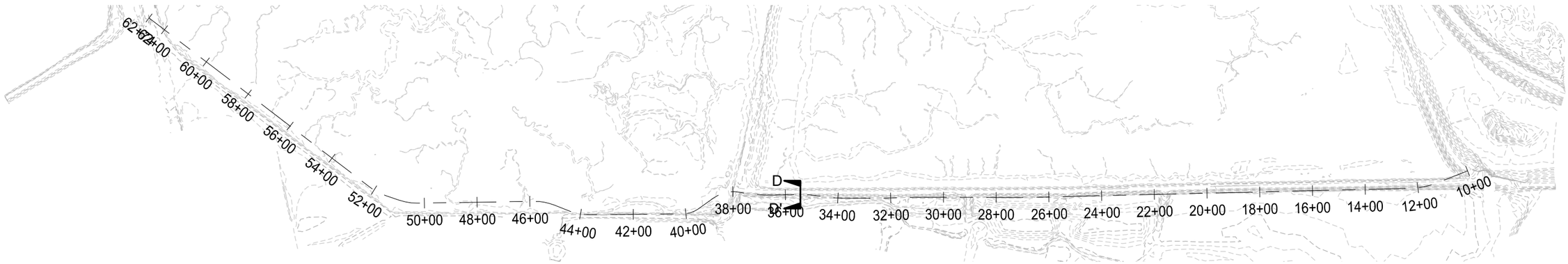

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DRAINAGE CHANNEL SECTION - STA 35+25 TO STA 35+55

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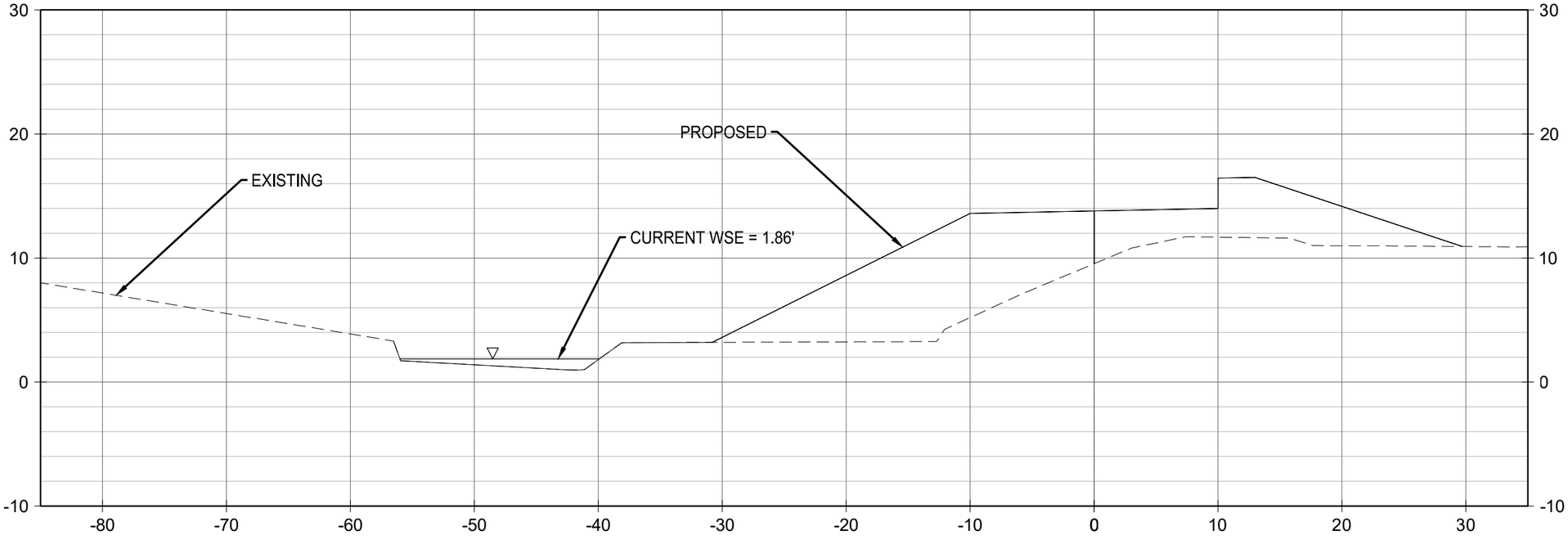
April 21, 2026



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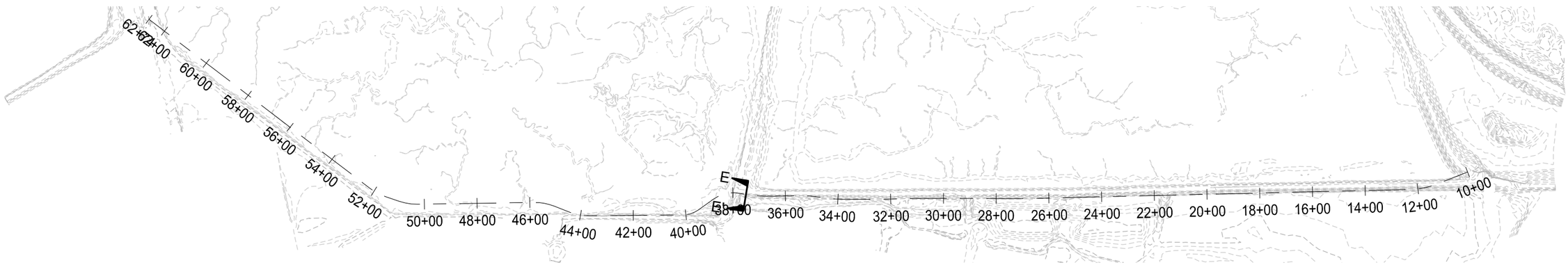

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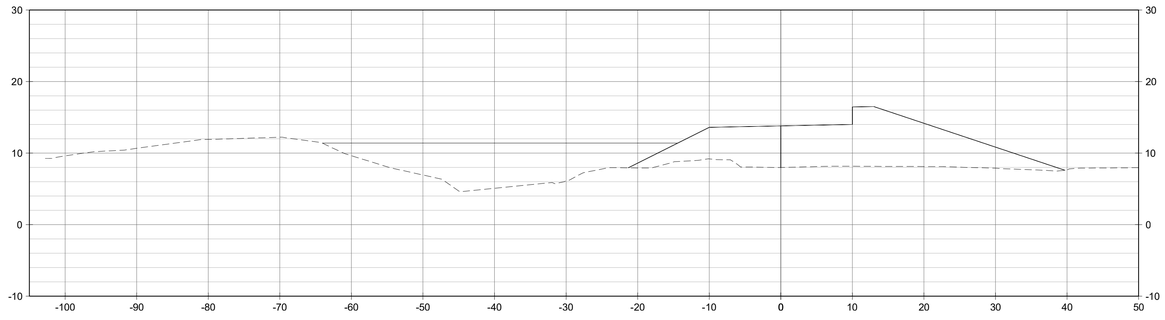

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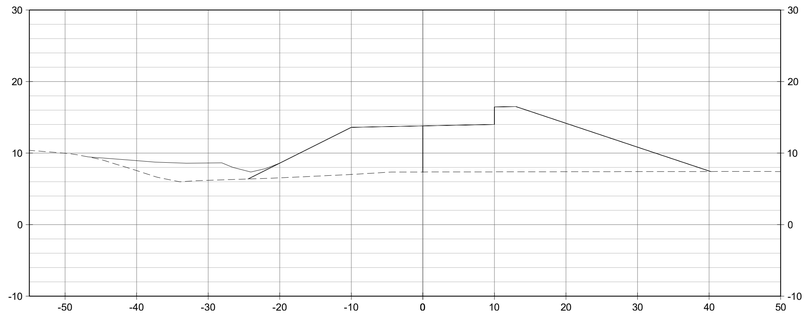

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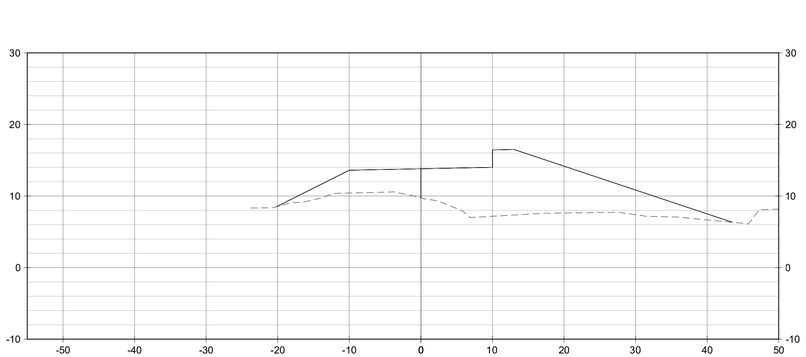

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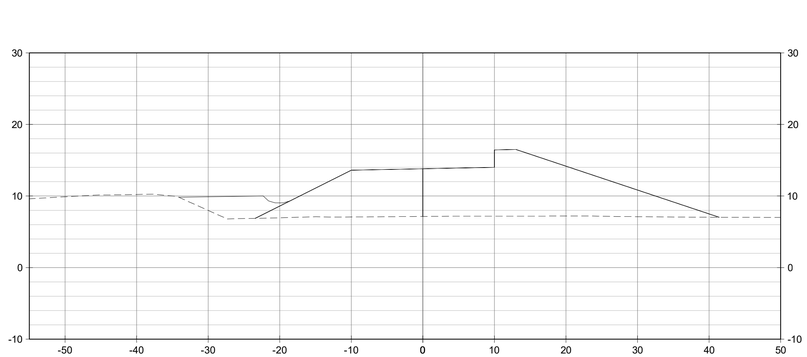

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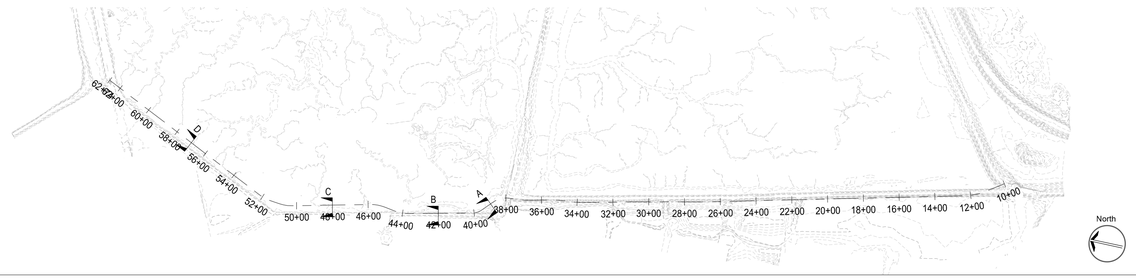

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NO.	DESCRIPTION OF REVISION	DATE

CHECKED BY: D. SAND

DRAFTED BY: T. STIMPSON

SCALE:

LAST REVISION: 2/27/2026

PREPARED BY

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for

CITY OF EAST PALO ALTO
DEPARTMENT OF PUBLIC WORKS
SAN MATEO COUNTY, CALIFORNIA

SHEET TITLE: TYPICAL SECTIONS

PROJECT/JOB TITLE: SAFER Bay Project
SOUTH OF BAY ROAD

SHEET NUMBER: CX-103

PROJECT/JOB NUMBER: 25-027

CITY OF EAST PALO ALTO
INCORPORATED 1983

60% SUBMITTAL - NOT FOR CONSTRUCTION

P:\2025\25-027_SAFER_EPA\03_Design\01_CAD\01_CD\CX-101_Typical-Sections.dwg,CX-103_Typical Sections,Ellen Obenshain,4/23/2026 6:45:25 PM,ARCH expand D (36.00 x 24.00 Inches),1:1

A.2 Geotechnical information

LEGEND

- INVESTIGATION LOCATION
- LEVEE DESIGN 5' CONTOUR
- EXISTING GROUND ELEVATION 5' CONTOUR
- EXISTING GROUND ELEVATION 1' CONTOUR
- SAFER LEVEE CONTROL LINE

NOTES

CPT = CONE PENETRATION TEST
B = BORING
GS26- = GEOSYNTEC EXPLORATORY BORINGS (GEOSYNTEC 2026)
A- = ARUP EXPLORATORY BORINGS (ARUP 2021)
C-/B- = HDR EXPLORATORY BORINGS (HDR 2024)

1. TOPOGRAPHIC MAP BY MARK THOMAS, DATED MAY 2026 USING NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND NATIONAL GEODETIC VERTICAL DATUM OF 1988 (NAVD88).
2. EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS.
3. HISTORICAL EXPLORATION LOCATIONS ARE BASED ON THE LOCATIONS SHOWN ON THE PRELIMINARY GEOTECHNICAL REPORT FOR SOUTH OF BAY ROAD (HDR 2024) AND THE GEOTECHNICAL DATA REPORT SFPUC REVANSWOOD FACILITY TO BEDWELL BAYFRONT PARK (HDR 2025).

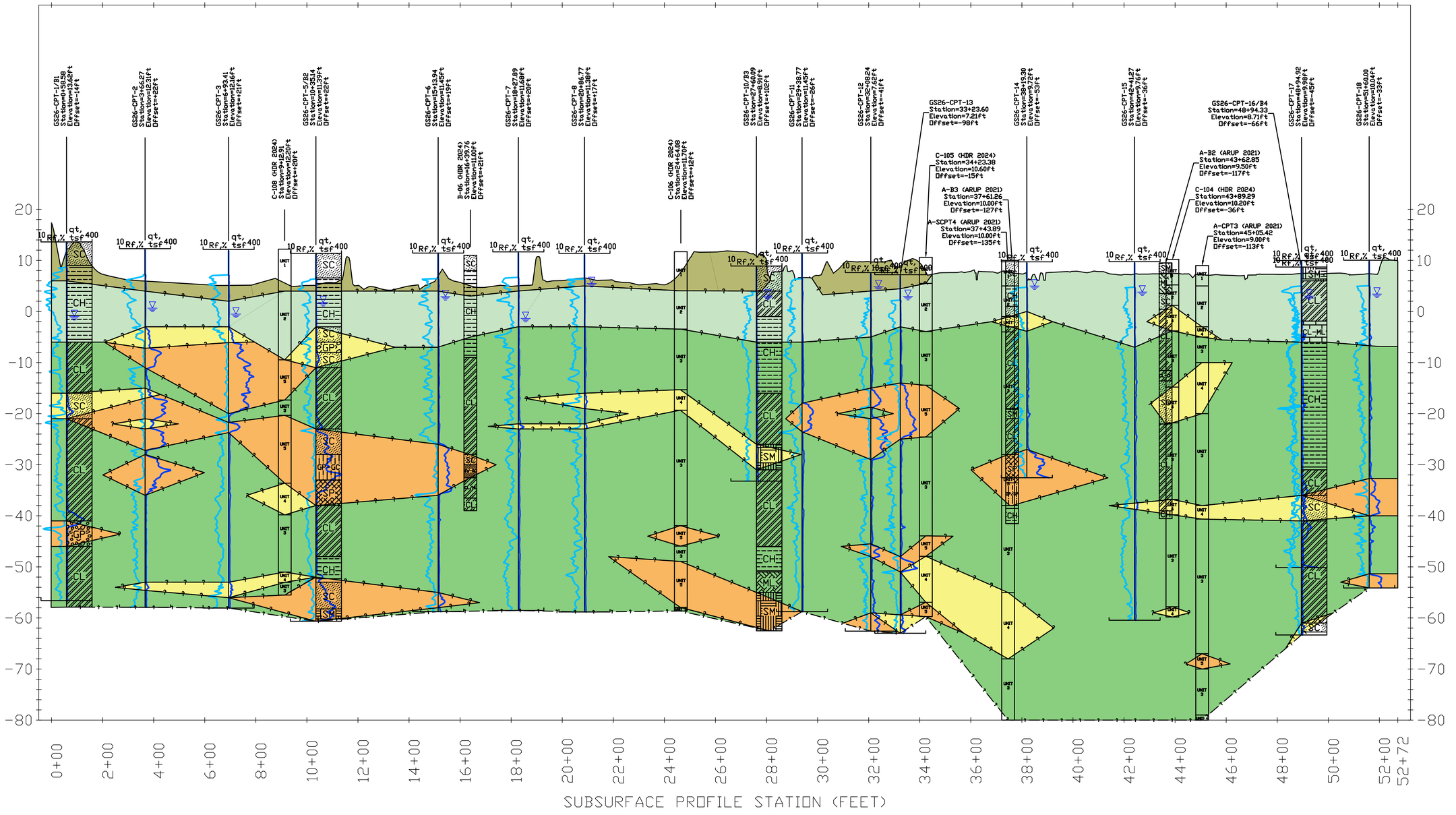
INVESTIGATION LOCATIONS
SAFER EAST PALO ALTO
STA 0+00 TO 52+27
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | MAY 2026

FIGURE 1

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WG3607 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS



GEOLOGIC CROSS SECTION

VERTICAL EXAGGERATION: 20

LEGEND

- UNIT 1 (FILL) - SANDY CLAY (CL/CH) TO SILTY SAND (SC/SM), SOFT TO MEDIUM STIFF
- UNIT 2 (YOUNG BAY MUD [YBM]) - VERY SOFT TO MEDIUM STIFF
- UNIT 3 (OLD BAY CLAY [OBC]) - MEDIUM STIFF TO VERY STIFF
- UNIT 4 (COARSE-GRAINED SOIL 1) - LOOSE TO MEDIUM DENSE
- UNIT 5 (COARSE-GRAINED SOIL 2) - MEDIUM DENSE TO VERY DENSE
- GROUNDWATER LEVEL ENCOUNTERED DURING INVESTIGATION (NOTE 5)

NOTES

CPT = CONE PENETRATION TEST
B = BORING
OFFSET = DISTANCE FROM SAFER BAY CONTROL LINE TO INVESTIGATION LOCATION (BAY SIDE IS POSITIVE AND LAND SIDE IS NEGATIVE).

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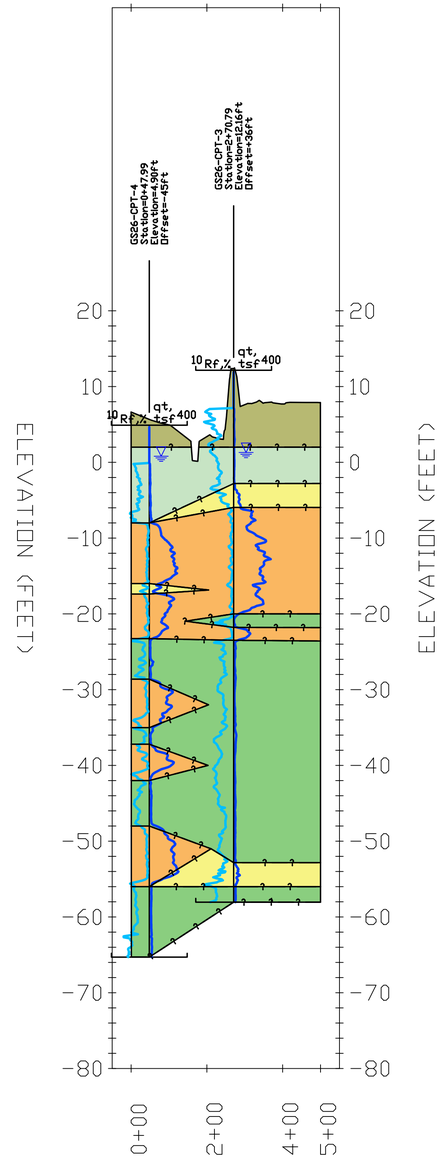
2. EXPLORATION LOCATIONS ARE BASED ON GPS SURVEY OF AS-BUILT LOCATIONS.
3. TOPOGRAPHIC SURVEY BY MARK THOMAS, DATED MAY 2026 WAS USED FOR THIS CROSS SECTION. HORIZONTAL DATUM USED WAS NORTH AMERICAN DATUM OF 1983 CALIFORNIA STATE PLANE ZONE 3, US FOOT (NAD83, CA ZONE 3) AND THE VERTICAL DATUM USED WAS THE NATIONAL GEODETIC VERTICAL DATUM OF 1988 (NAVD88).
4. GROUNDWATER ELEVATIONS ARE BASED ON THE ESTIMATED HYDROSTATIC PORE PRESSURE AT THE END OF CPT PORE PRESSURE DISSIPATION TESTS TAKEN IN GRANULAR MATERIAL.
5. CPT DATA FROM HISTORICAL INVESTIGATIONS WERE INTERPRETED CONSISTENT WITH THE GEOSYNTEC CPTS AND USED TO EVALUATE THE DEPTHS AND THICKNESSES OF UNITS 1 THROUGH 5.

GEOLOGIC CROSS SECTION
SAFER EAST PALO ALTO
STA 0+00 TO 52+27
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

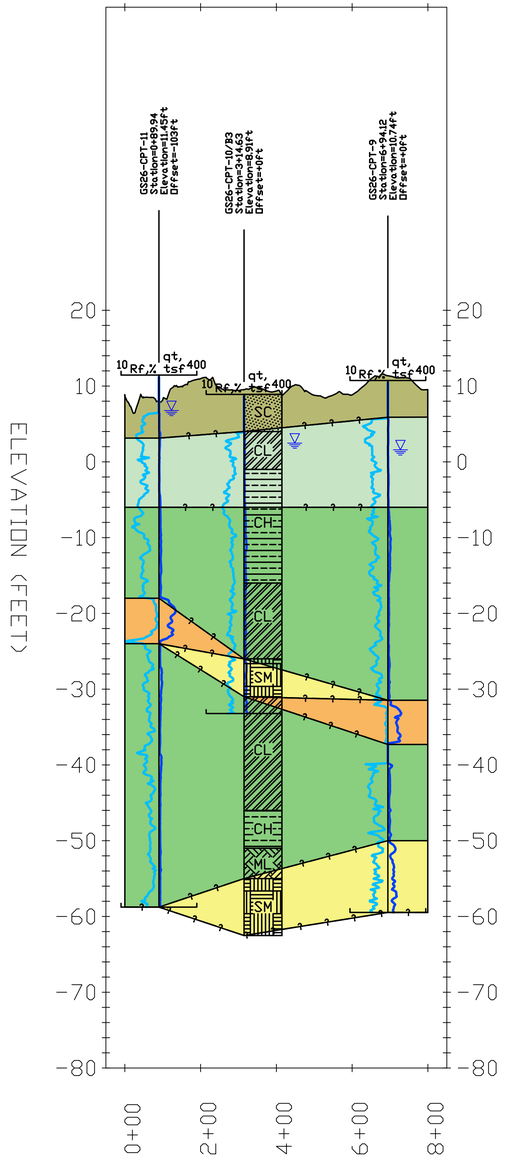
PROJECT NO: 26007427 | MAY 2026 | FIGURE 2

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\W3\3007 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS



1/1 STA 7+10 CROSS SECTION

VERTICAL EXAGGERATION: 20



2/1 STA 27+80 CROSS SECTION

VERTICAL EXAGGERATION: 20

LEGEND

- UNIT 1 (FILL) - SANDY CLAY (CL/CH) TO SILTY SAND (SC/SM), SOFT TO MEDIUM STIFF
- UNIT 2 (YOUNG BAY MUD (YBM)) - VERY SOFT TO MEDIUM STIFF
- UNIT 3 (OLD BAY CLAY (OBC)) - MEDIUM STIFF TO VERY STIFF
- UNIT 4 (COARSE-GRAINED SOIL 1) - LOOSE TO MEDIUM DENSE
- UNIT 5 (COARSE-GRAINED SOIL 2) - MEDIUM DENSE TO VERY DENSE
- GROUNDWATER LEVEL ENCOUNTERED DURING INVESTIGATION (NOTE 5)

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PERPENDICULAR CROSS SECTION
SAFER EAST PALO ALTO
STA 7+10 AND STA 27+80
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | MAY 2026

FIGURE 3

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\SWG3807 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

Table 1: Summary of Material Properties
SAFER Bay Project (South of Bay Road)
East Palo Alto, California

Soil Unit	Material	USCS	Total Unit Weight[1]	Strength					Settlement										Permeability		Pile Capacity		
				Undrained/Total Stress Parameters			Drained/Effective Stress Parameters					Modified Compression Indices			Coefficient of Consolidation	Oedometric Modulus	Elastic Modulus	Coefficient of Volume Compressibility	Hydraulic Conductivity	Anisotropy Ratio	Shaft Friction Factor[5][6][8]	Shaft Adhesion Factor [7]	Limiting Shaft Friction[8]
				Undrained Shear Strength[2]	Total Cohesion	Total Friction Angle	Effective Cohesion	Effective Friction Angle	Void Ratio	Porosity	Initial Overconsolidation Ratio	Modified Compression Index	Modified Recompression Index	Modified Secondary Compression Index									
			γ_t (pcf)	S_u (psf)	c_R (psf)	φ_R (degrees)	c' (psf)	φ' (degrees)	e -	n -	OCR -	C_{cc} -	C_{rc} -	C_{ac} -	c_v (ft^2/yr)	E_{oed} (ksf)	E' (ksf)	m_v (1/ksf)	K_h (cm/s)	K_h/K_v -	β -	α -	q_s (psf)
Unit 1	Existing Fill	CL,CH,SC,SM	120	--	--	--	0	30	--	--	--	--	--	--	--	--	200	--	2.0×10^{-6}	4	0.29	--	1400
Unit 2	Young Bay Mud [YBM] (Under Existing Levee Footprint)	CL,CH	115	400	400	0	200	30	1.1	0.52	2.0	0.20	0.03	0.01	15	50	--	2.0×10^{-2}	1.5×10^{-6}	4	0.20	1.0	400
	Young Bay Mud [YBM] (Outside Existing Levee Footprint)			200	200	0			1.3	0.57	1.0					50		2.0×10^{-2}	1.5×10^{-6}		0.20	1.0	200
Unit 3	Marsh	CL,CH,ML	120	500 + 21z	500 + 21z	0	0	30	0.9	0.47	2.2	0.20	0.02	0.01	20	150	--	6.5×10^{-3}	4.0×10^{-6}	4	0.23	0.9	450 + 19z
Unit 4	Coarse-Grained Soil 1	SC,SW,SM,GC,GP	125	-- $(S_r/\sigma'_{vc} = 0.17)^{[3][4]}$	--	--	0 --	30 --	--	--	-- --	-- --	-- --	-- --	-- --	-- --	500 --	-- --	3.0 X 10^{-4} --	4	0.29 --	-- --	1400 --
Unit 5	Coarse-Grained Soil 2	SC,SP,SM,GP,GP-GC	125	-- $(S_r/\sigma'_{vc} = 0.40)^{[3][4]}$	--	--	0 --	38 --	--	--	-- --	-- --	-- --	-- --	-- --	-- --	1000 --	-- --	1.0 X 10^{-2} --	4	0.37 --	-- --	1700 --

Abbreviations:
-- : Not applicable based on soil type
σ'_v: Vertical effective stress at the depth of interest
ksi: kips per square inch
pcf: pounds per cubic foot
pci: pounds per cubic inch
psf: Pounds per square foot
z: depth below top of OBC
S_r: Residual shear strength
σ'_{vc}: effective stress

Notes:
1. Total unit weight refers to the moist density. For soil beneath the groundwater table, the effective unit weight can be estimated by subtracting the unit weight of water (62.4 pcf).
2. Undrained shear strength can be used to represent total stress cohesion (c) of cohesive soil layers at 5% strain, for sheetpile analysis in LPILE.
3. Values in parentheses are provided to model conditions after soil liquefaction due to seismic loading, and assume use of a soft clay p-y model to represent liquefied soil.
4. Post liquefaction residual shear strength ratio (S_r/σ'_{vc}) for Unit 4 and 5 is based on the relationship provided in Figure 89 of: Idriss, I.M., and Boulanger, R.W., 2008. "Soil Liquefaction During Earthquakes". Earthquake Engineering Research Institute (EERI) Monograph MNO-12.
5. Shaft friction factor for Unit 1, 4, & 5 is based on API (2002): American Petroleum Institute. 2002. *API Recommended Practice 2A-WSD-Planning, Designing, and Constructing Fixed Offshore Platforms - Working Stress Design*. 21st ed.
6. Shaft friction factor for Unit 2 & 3 is based on FHWA (2016): *Design and Construction of Driven Pile Foundations - Volume I* (Publication No. FHWA-NHI-16-009) and USACE (1991): *Design of Pile Foundations* (EM-1110-2-2906).
7. Shaft Adhesion Factor is based on DM7.2 (Foundation and Earth Structures) UFC 3-220-20 (16 January 2025).
8. Limiting Shaft Friction values represent ultimate, not allowable loads. To obtain allowable loads, the structural engineer should apply appropriate factors of safety (FS) when using allowable stress design, or by applying load and resistance factors when using load and resistance factor design.
9. All parameters shown represent conditions for static loading below the groundwater table unless noted otherwise.

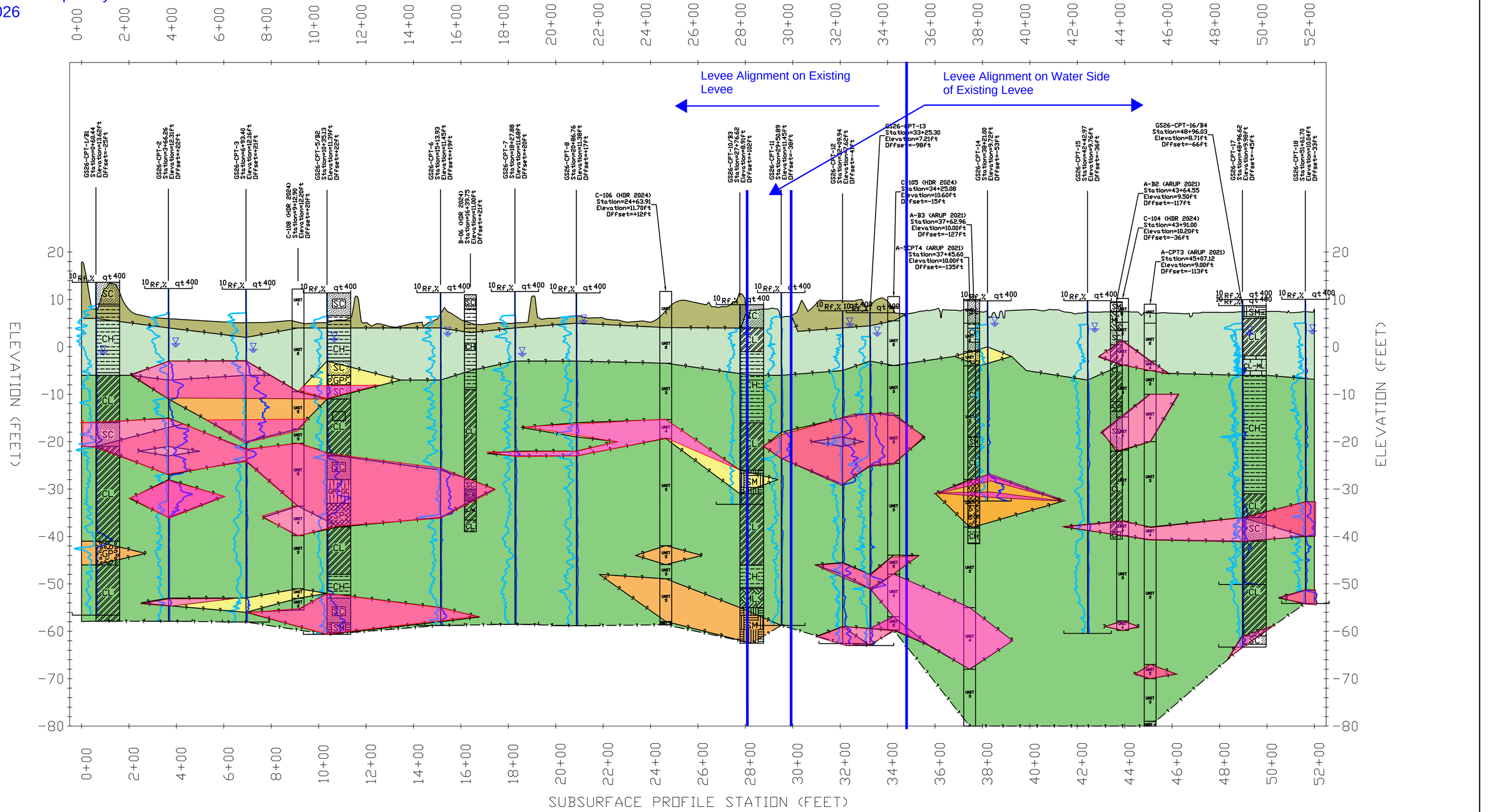
Table 2: Assumed Material Properties of New Embankment Fill
SAFER Bay Project (South of Bay Road)
East Palo Alto, California

Material	USCS	Total Unit Weight[1]	Strength					Settlement										Permeability		Pile Capacity		
			Undrained/Total Stress Parameters			Drained/Effective Stress Parameters					Modified Compression Indices											
			Undrained Shear Strength[2]	Total Cohesion	Total Friction Angle	Effective Cohesion	Effective Friction Angle	Void Ratio	Porosity	Initial Overconsolidation Ratio	Modified Compression Index	Modified Recompression Index	Modified Secondary Compression Index	Coefficient of Consolidation	Oedometric Modulus	Elastic Modulus	Coefficient of Volume Compressibility	Hydraulic Conductivity	Anisotropy Ratio	Shaft Friction Factor[3][4][5]	Shaft Adhesion Factor [7]	Limiting Shaft Friction[8]
		γ_t (pcf)	S_u (psf)	c_R (psf)	φ_R (degrees)	c' (psf)	φ' (degrees)	e -	n -	OCR -	C_{cc} -	C_{rc} -	C_{ac} -	c_v (ft^2/yr)	E_{oed} (ksf)	E' (ksf)	m_v (1/ksf)	K_h (cm/s)	K_h/K_v -	β -	α -	q_s (psf)
New Embankment Fill	CL/SC	125	750	750	0	75	30	1.5	0.6	--	--	--	--	--	--	200	--	2.0 X 10^{-6}	4	0.29	--	

A.3 Liquefaction potential for OBE and MDE

SAFER (South of Bay Road)
Liquefaction Susceptibility Evaluation
21 May 2026

SUBSURFACE PROFILE STATION (FEET)



SUBSURFACE PROFILE STATION (FEET)

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GEOLOGIC CROSS SECTION
SAFER EAST PALO ALTO
STA 0+00 TO 52+27
EAST PALO ALTO, CALIFORNIA

Geosyntec consultants

PROJECT NO: 26007427 | MAY 2026 | FIGURE 1

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\W3807 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

A.4 Settlement analyses

Table 1: Settlement Results
SAFER Bay Project (South of Bay Road)
East Palo Alto, California

Station	Overbuild Height	Post-Construction Levee Top Elevation	Estimated Settlement during Embankment construction	50 Year Post-Construction Settlement	Total 50 Year Settlement	Long-Term Levee Top Elevation (ft NAVD88)
	(ft)	(ft)	(in.)	(in.)	(in.)	(ft)
7+00	0.5	13.5	2.5	5.3	7.8	**13.1**
15+00	1.0	14.0	1.9	9.9	11.8	**13.2**
28+50	1.5	14.5	4.8	15.9	20.8	**13.2**
42+50	1.5	14.5	4.9	16.6	21.5	**13.1**
49+00	1.5	14.5	5.1	16.3	21.4	**13.1**

Notes:

1. Water table elevation set to the ground surface elevation.
2. Levee construction duration assumed to be three months.
3. For sections of the levee alignment outside of the existing levee footprint, total settlement was calculated after the top two feet of Young Bay Mud was excavated and replaced with new fill during the first month of construction.
4. 50 year post-construction, relative to completion of earthwork.
5. Soil properties obtained from Summary of Material Properties table.
9. Soil layers obtained from Figure: Geologic Cross Section, SAFER East Palo Alto, STA 0+00 to 52+27, East Palo Alto, California.
10. Fill unit weight taken from ULE Guidance Document for Geotechnical Analyses (DWR 2015): Section 5.8.6 Table 5-5
11. Overbuild slopes assumed to be steepened in accordance with Figure 1-2 of EM 1110-2-1913 (USACE 2022) for design long-term slopes of 2H:1V landside and 3H:1V waterside.
12. Height of pile cap fill set to 3.5ft.

SAFER (South of Bay Road)
Levee Cross Section Locations for Settlement Analysis

SUBSURFACE PROFILE STATION (FEET)

0+00 2+00 4+00 6+00 8+00 10+00 12+00 14+00 16+00 18+00 20+00 22+00 24+00 26+00 28+00 30+00 32+00 34+00 36+00 38+00 40+00 42+00 44+00 46+00 48+00 50+00 52+00

Levee Alignment on Existing Levee

Levee Alignment on Water Side of Existing Levee

GS26-CPT-1/B1 Station=0+00.44 Elevation=13.62ft Offset=-25ft

GS26-CPT-2 Station=3+56.26 Elevation=11.21ft Offset=-22ft

GS26-CPT-3 Station=6+93.40 Elevation=11.66ft Offset=-21ft

C-108 (HDR 2024) Station=12+29.30 Elevation=12.20ft Offset=-20ft

GS26-CPT-5/B2 Station=10+35.13 Elevation=11.39ft Offset=-22ft

GS26-CPT-6 Station=15+13.93 Elevation=11.45ft Offset=-19ft

B-06 (HDR 2024) Station=16+39.75 Elevation=11.00ft Offset=-21ft

GS26-CPT-7 Station=18+27.88 Elevation=11.68ft Offset=-20ft

GS26-CPT-9 Station=20+86.76 Elevation=11.38ft Offset=-17ft

C-106 (HDR 2024) Station=24+63.91 Elevation=11.70ft Offset=-12ft

GS26-CPT-10/B3 Station=27+76.62 Elevation=8.91ft Offset=-102ft

GS26-CPT-11 Station=29+30.89 Elevation=9.46ft Offset=-38ft

GS26-CPT-12 Station=32+09.94 Elevation=9.34ft Offset=-41ft

GS26-CPT-13 Station=33+25.30 Elevation=7.21ft Offset=-98ft

C-105 (HDR 2024) Station=34+25.08 Elevation=10.60ft Offset=-15ft

A-B3 (ARUP 2021) Station=37+62.96 Elevation=10.00ft Offset=-127ft

A-CPT4 (ARUP 2021) Station=37+45.60 Elevation=10.00ft Offset=-135ft

GS26-CPT-14 Station=38+21.00 Elevation=9.72ft Offset=-53ft

GS26-CPT-15 Station=42+42.97 Elevation=9.76ft Offset=-36ft

A-B2 (ARUP 2021) Station=43+64.55 Elevation=9.50ft Offset=-117ft

C-104 (HDR 2024) Station=43+91.00 Elevation=10.20ft Offset=-36ft

A-CPT3 (ARUP 2021) Station=45+07.12 Elevation=9.00ft Offset=-113ft

GS26-CPT-16/B4 Station=48+96.03 Elevation=8.71ft Offset=-66ft

GS26-CPT-17 Station=49+96.62 Elevation=9.98ft Offset=-45ft

GS26-CPT-18 Station=51+04.70 Elevation=10.04ft Offset=-33ft

ELEVATION (FEET)

20 10 0 -10 -20 -30 -40 -50 -60 -70 -80

10 Rf,% qt 400

Section 1: Thickest Sand Section below levee footprint

Section 2: Thickest Clay Section below levee footprint

Section 3: Alignment on marsh

Section 4: Thickest Clay Section in marsh area

Section 5: Thickest Sand section in marsh area near gate structure

STA 0+00 to 12+00
6-inch overbuild

STA 12+50 to 27+50
12 inch overbuild

STA 28+00 to Bay Rd
18-inch overbuild

Transition zones

SUBSURFACE PROFILE STATION (FEET)

LEGEND

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GEOLOGIC CROSS SECTION
SAFER EAST PALO ALTO
STA 0+00 TO 52+27
EAST PALO ALTO, CALIFORNIA

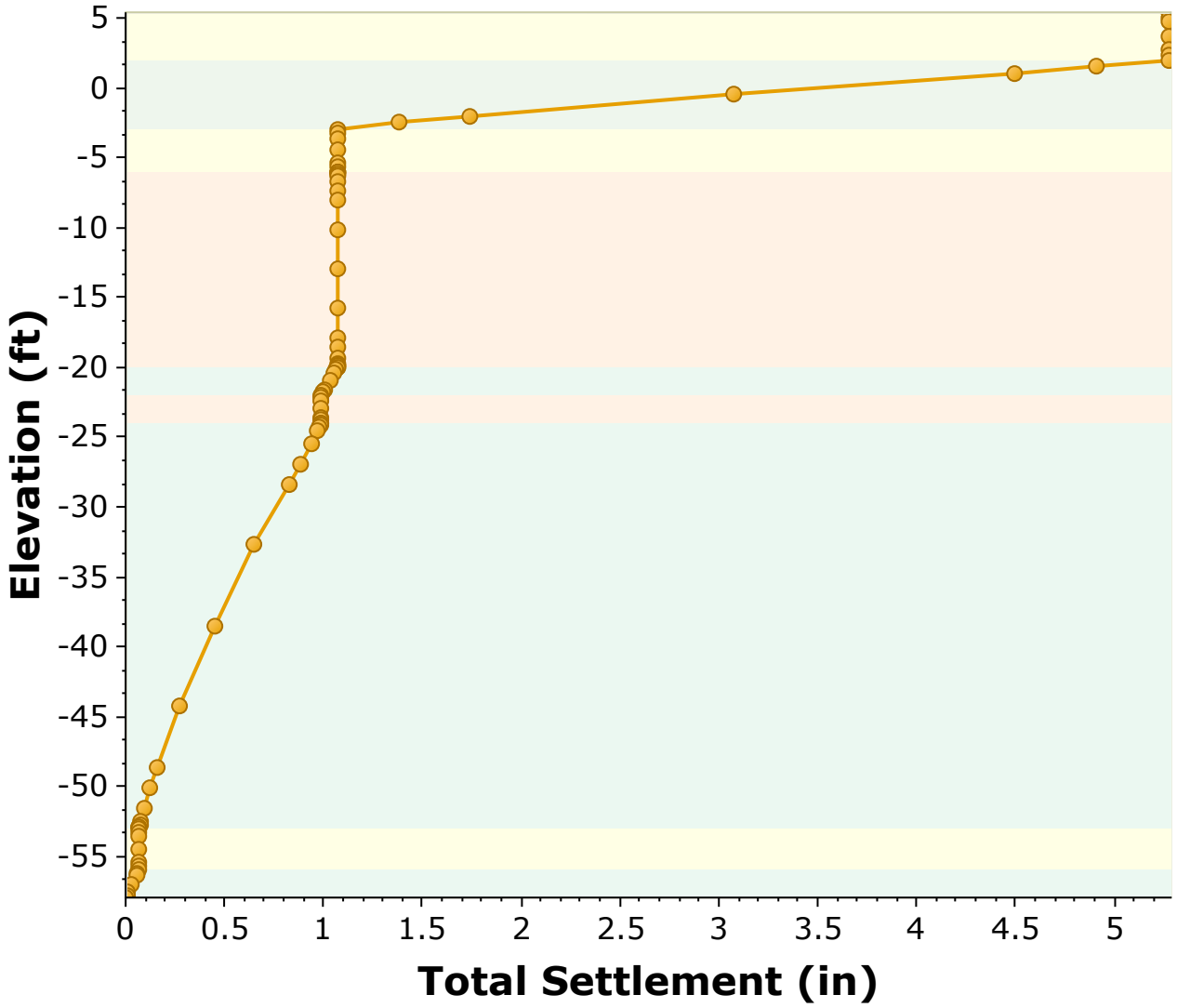
Geosyntec consultants

PROJECT NO: 26007427 | MAY 2026 | FIGURE 1

P:\CADD\CIVIL 3D\SHERWOOD DESIGN ENGINEERS\WO3807 - SAFER EPA\SOUTH OF BAY ROAD - CROSS SECTIONS

DRAFT

Total Settlement vs. Elevation



Center of Levee (50 Years = 50.25 y)

Predicted settlement vs. depth beneath levee control line.

Settlement profile is estimated at 50-years, relative to the end of earthwork construction. Assumes sheet pile will be installed at the completion of earthwork.

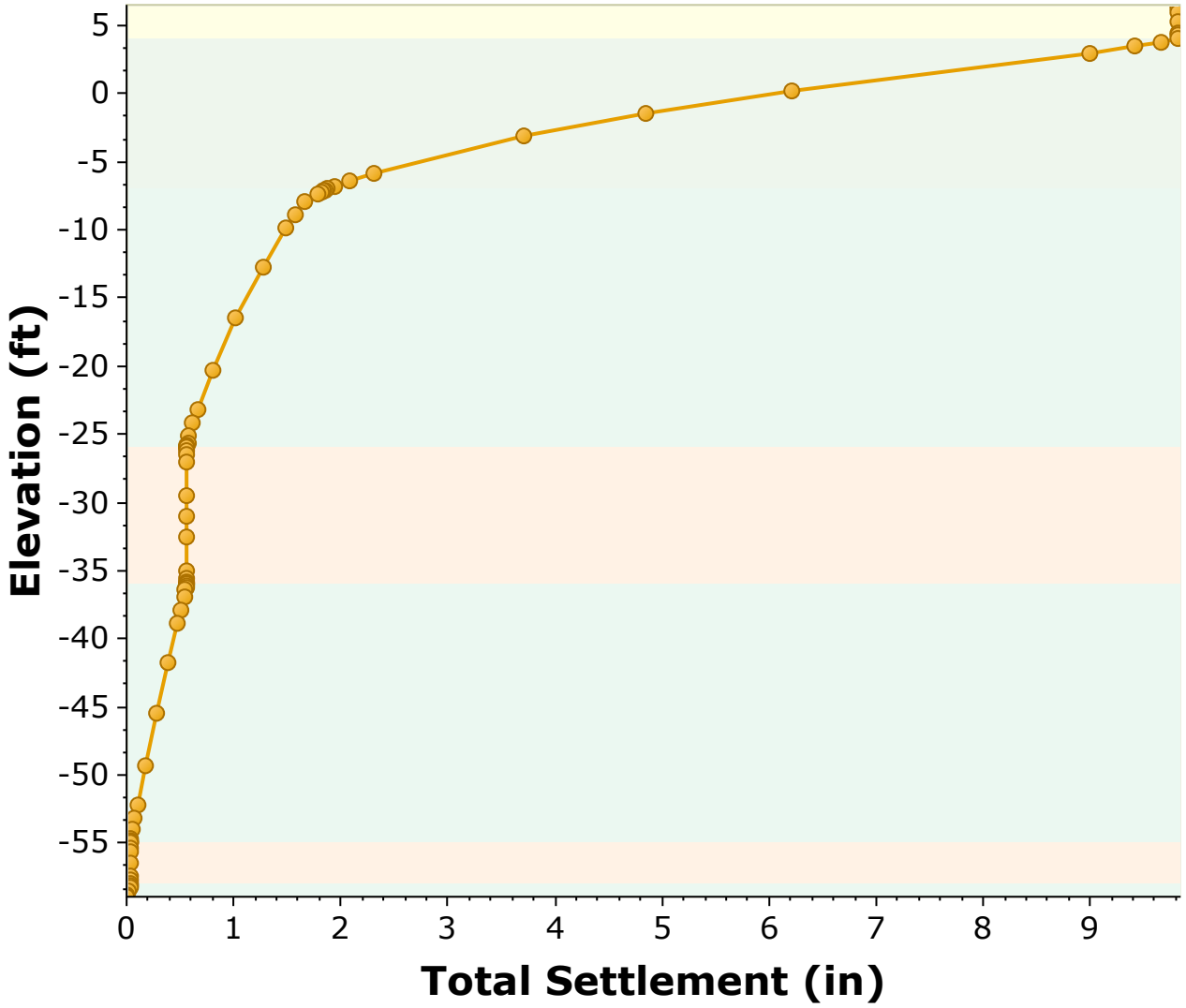
Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y

ROCSCIENCE	*Project*	SAFER (South of Bay Road)	
	Analysis Description	Settlement Analysis of STA 7+00	
	Drawn By	Armaan Khamenian	*Company* Geosyntec Consultants, Inc.
	Date	5/6/2026, 2:09:25 PM	*File Name* SAFER (South of Bay Road) - STA 7+00 (Overbuild).s3z

SETTLE3 5.027

DRAFT

Total Settlement vs. Elevation



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Predicted settlement vs. depth beneath levee control line.

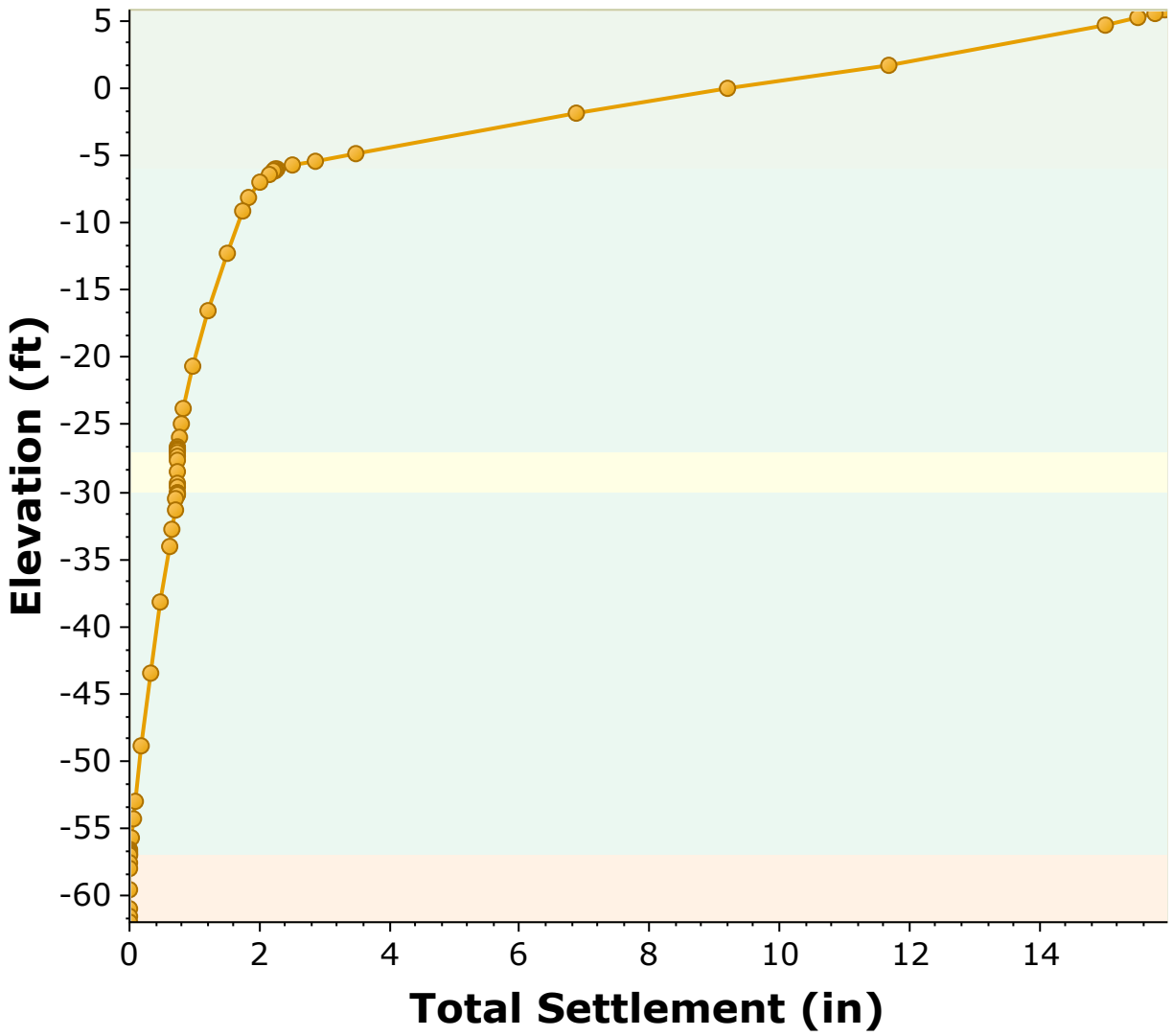
Settlement profile is estimated at 50-years, relative to the end of earthwork construction. Assumes sheet pile will be installed at the completion of earthwork.

Reference Stage: Stage 3 (3 Months - End of Constr.) = 0.25 y

ROCSCIENCE	*Project* SAFER (South of Bay Road)	
	Analysis Description Settlement Analysis of STA 15+00	
	Drawn By Armaan Khamenian	*Company* Geosyntec Consultants, Inc.
SETTLE3 5.027	*Date* 5/6/2026, 2:09:25 PM	*File Name* SAFER (South of Bay Road) - STA 15+00 (Overbuild).s3z

DRAFT

Total Settlement vs. Elevation



Center of Levee (50 Years = 50.3333 y)

Predicted settlement vs. depth beneath levee control line.

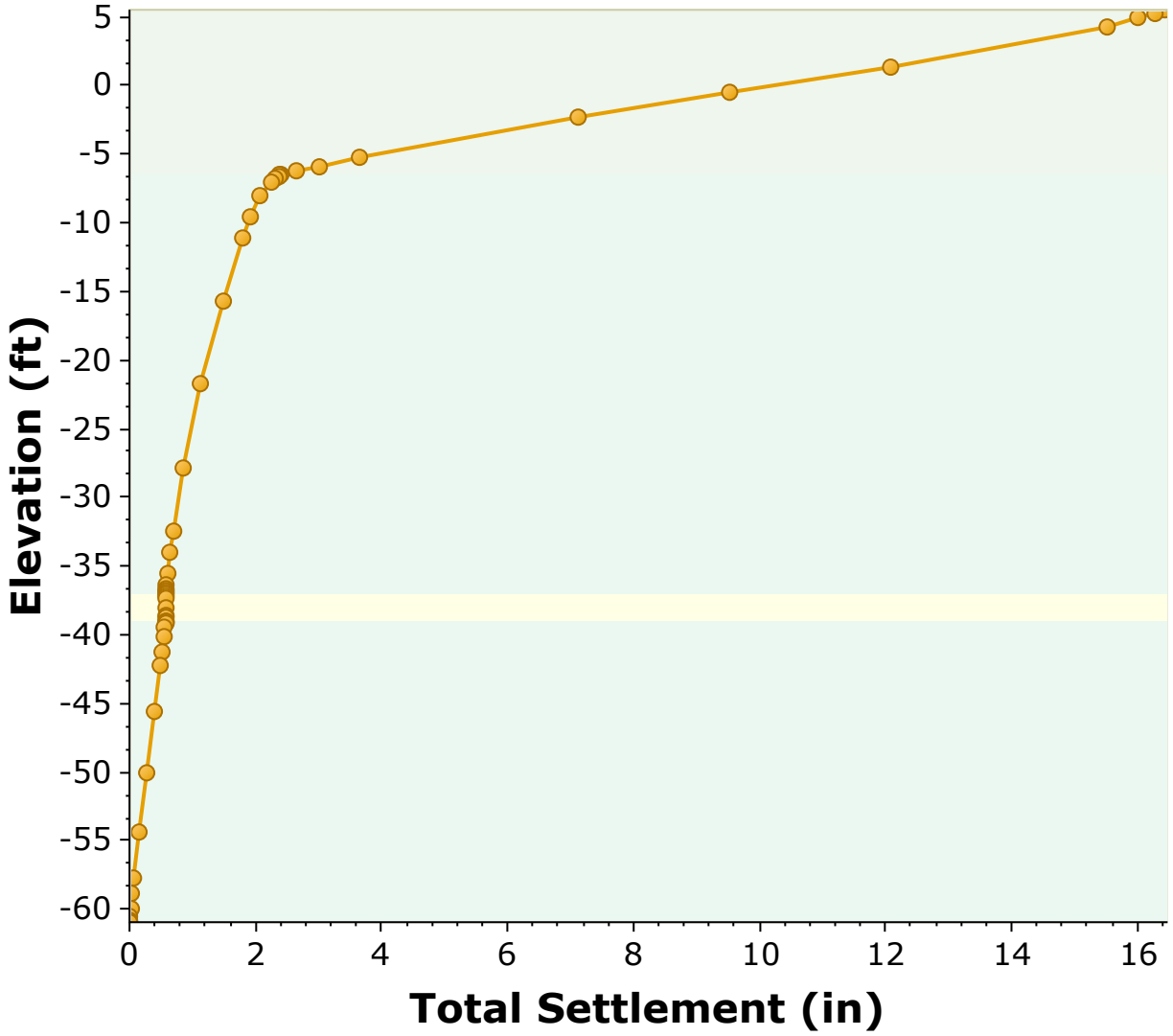
Settlement profile is estimated at 50-years, relative to the end of earthwork construction. Assumes sheet pile will be installed at the completion of earthwork.

Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y

ROCSCIENCE	*Project* SAFER (South of Bay Road)	
	Analysis Description Settlement Analysis at STA 28+50	
	Drawn By Armaan Khamenian	*Company* Geosyntec Consultants, Inc.
SETTLE3 5.027	*Date* 5/6/2026, 2:09:25 PM	*File Name* SAFER (South of Bay Road) - STA 28+50 (Overbuild).s3z

DRAFT

Total Settlement vs. Elevation



Center of Levee (50 Years = 50.3333 y)

Predicted settlement vs. depth beneath levee control line.

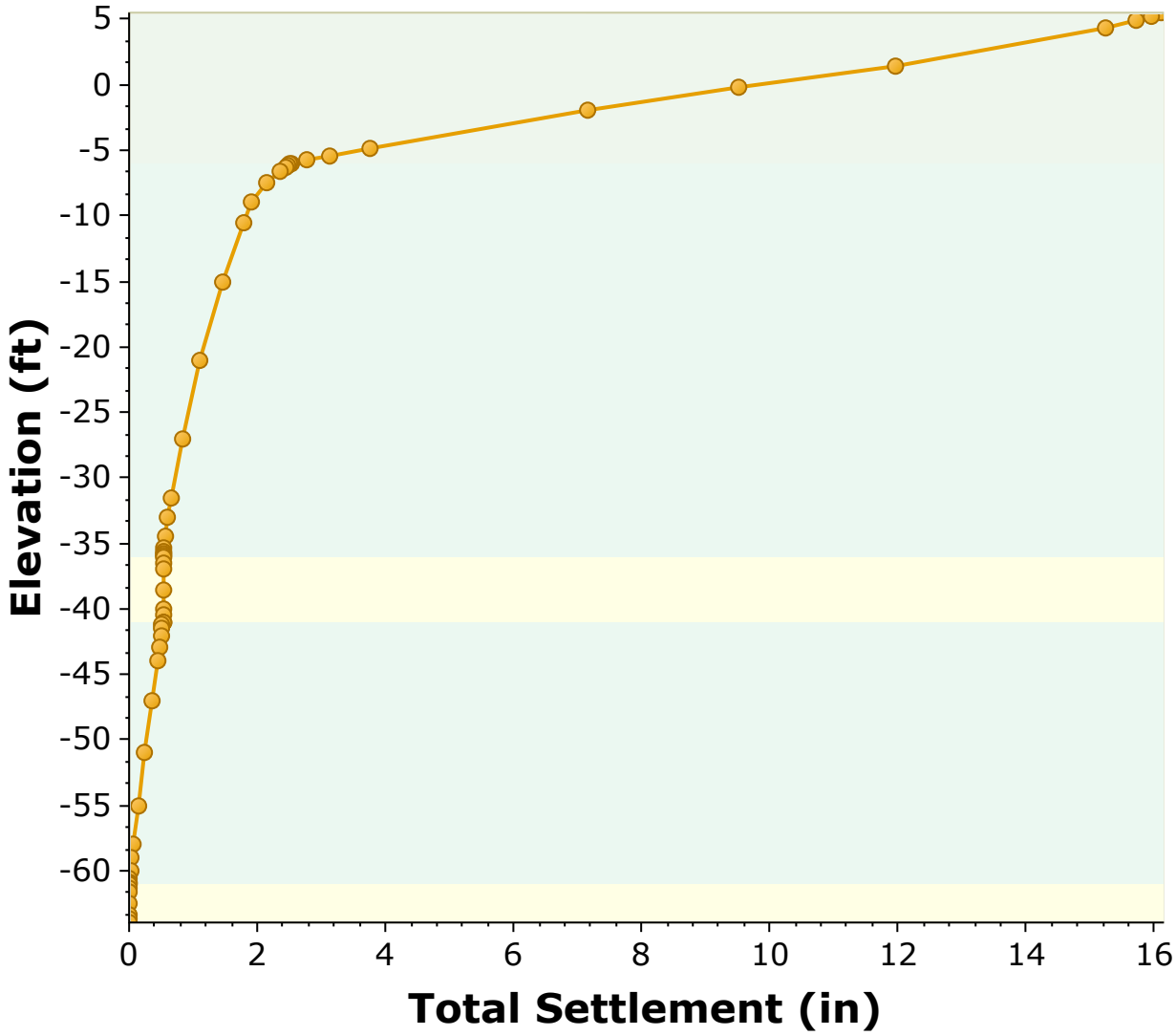
Settlement profile is estimated at 50-years, relative to the end of earthwork construction. Assumes sheet pile will be installed at the completion of earthwork.

Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y

ROCSCIENCE	*Project* SAFER (South of Bay Road)	
	Analysis Description Settlement Analysis at STA 42+50	
	Drawn By Armaan Khamenian	*Company* Geosyntec Consultants, Inc.
SETTLE3 5.027	*Date* 5/6/2026, 2:09:25 PM	*File Name* SAFER (South of Bay Road) - STA 42+50 (Overbuild).s3z

DRAFT

Total Settlement vs. Elevation



Center of Levee (50 Years = 50.3333 y)

Predicted settlement vs. depth beneath levee control line.

Settlement profile is estimated at 50-years, relative to the end of earthwork construction. Assumes sheet pile will be installed at the completion of earthwork.

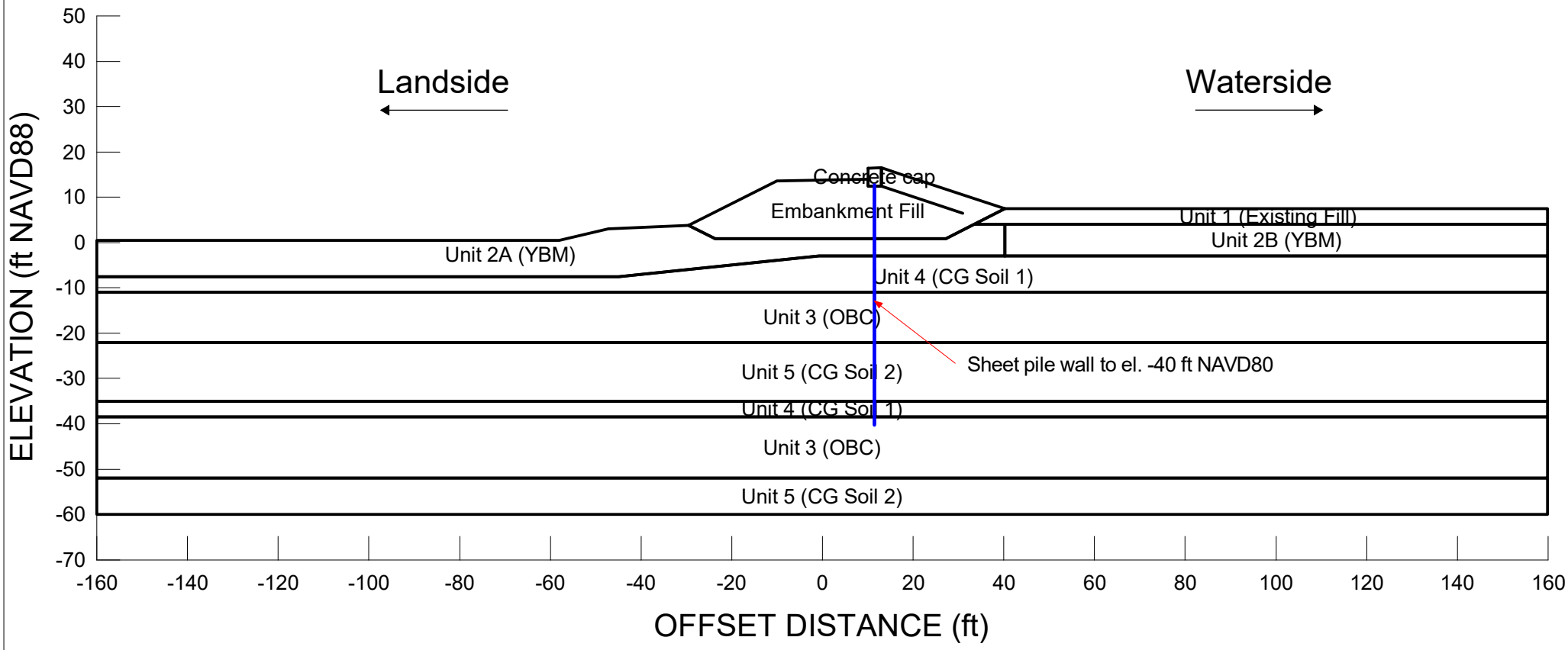
Reference Stage: Stage 4 (4 Months - End of Constr.) = 0.3333 y

ROCSCIENCE	*Project* SAFER (South of Bay Road)	
	Analysis Description Settlement Analysis at STA 49+00	
	Drawn By Armaan Khamenian	*Company* Geosyntec Consultants, Inc.
SETTLE3 5.027	*Date* 5/6/2026, 2:09:25 PM	*File Name* SAFER (South of Bay Road) - STA 49+00 (Overbuild).s3z

Appendix B GeoStudio outputs

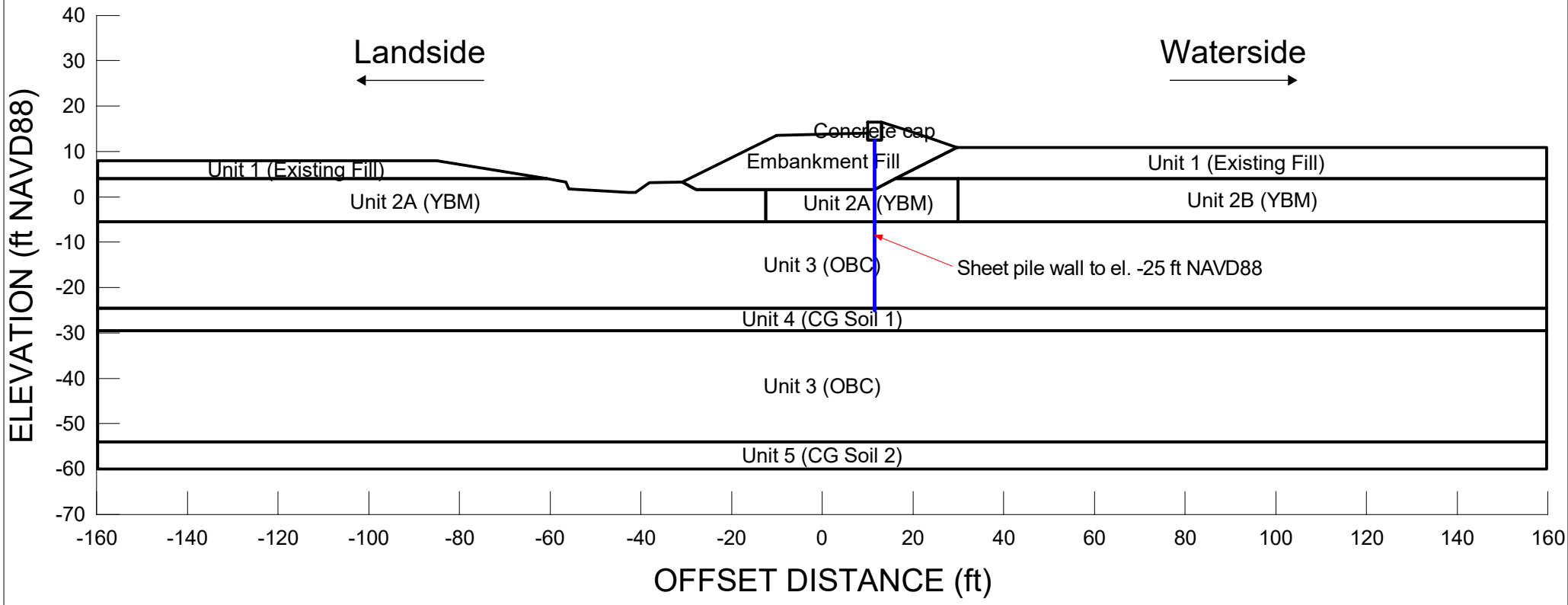
B.1 Analysis cross sections figures

Analysis cross section 1 (ACS1)
St 10+35



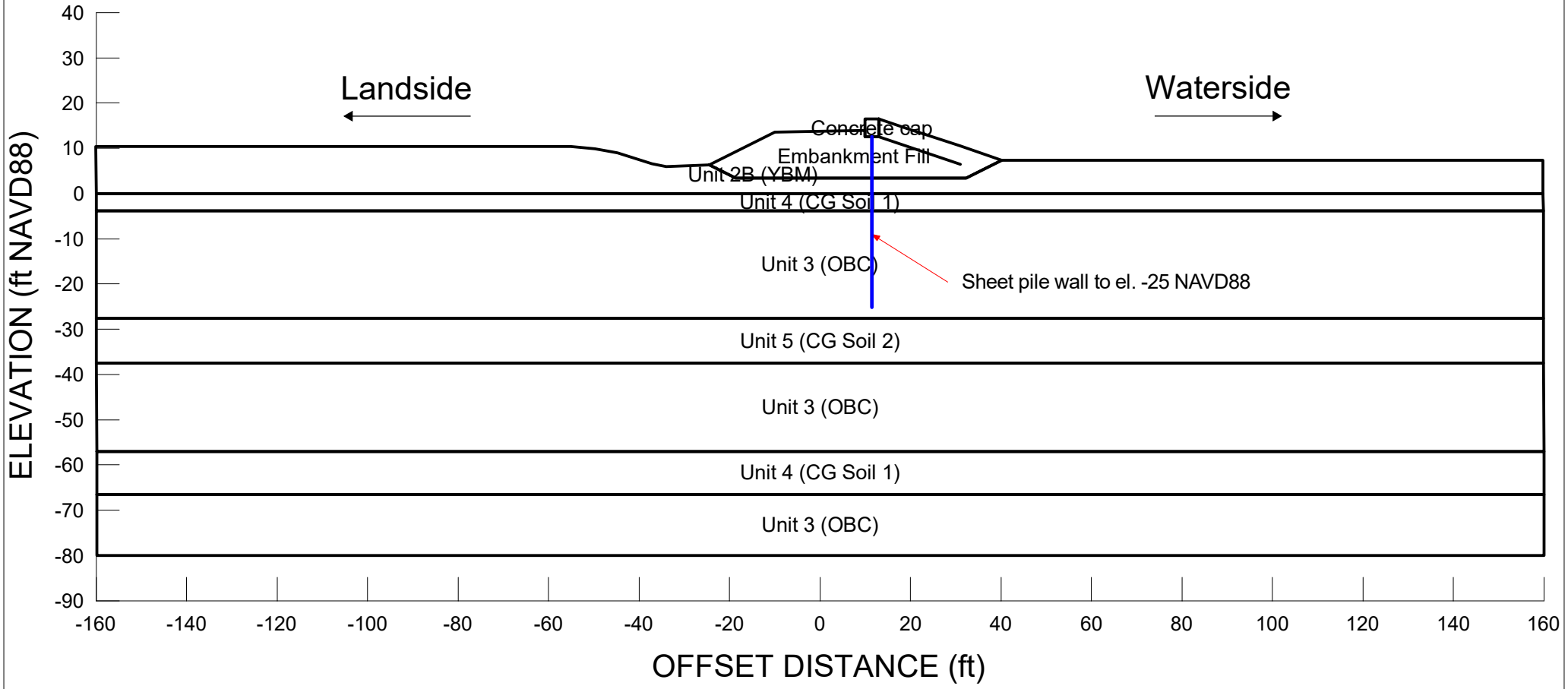
Appendix B.1
Figure 1

Analysis cross section 3 (ACS3)
St 27+50

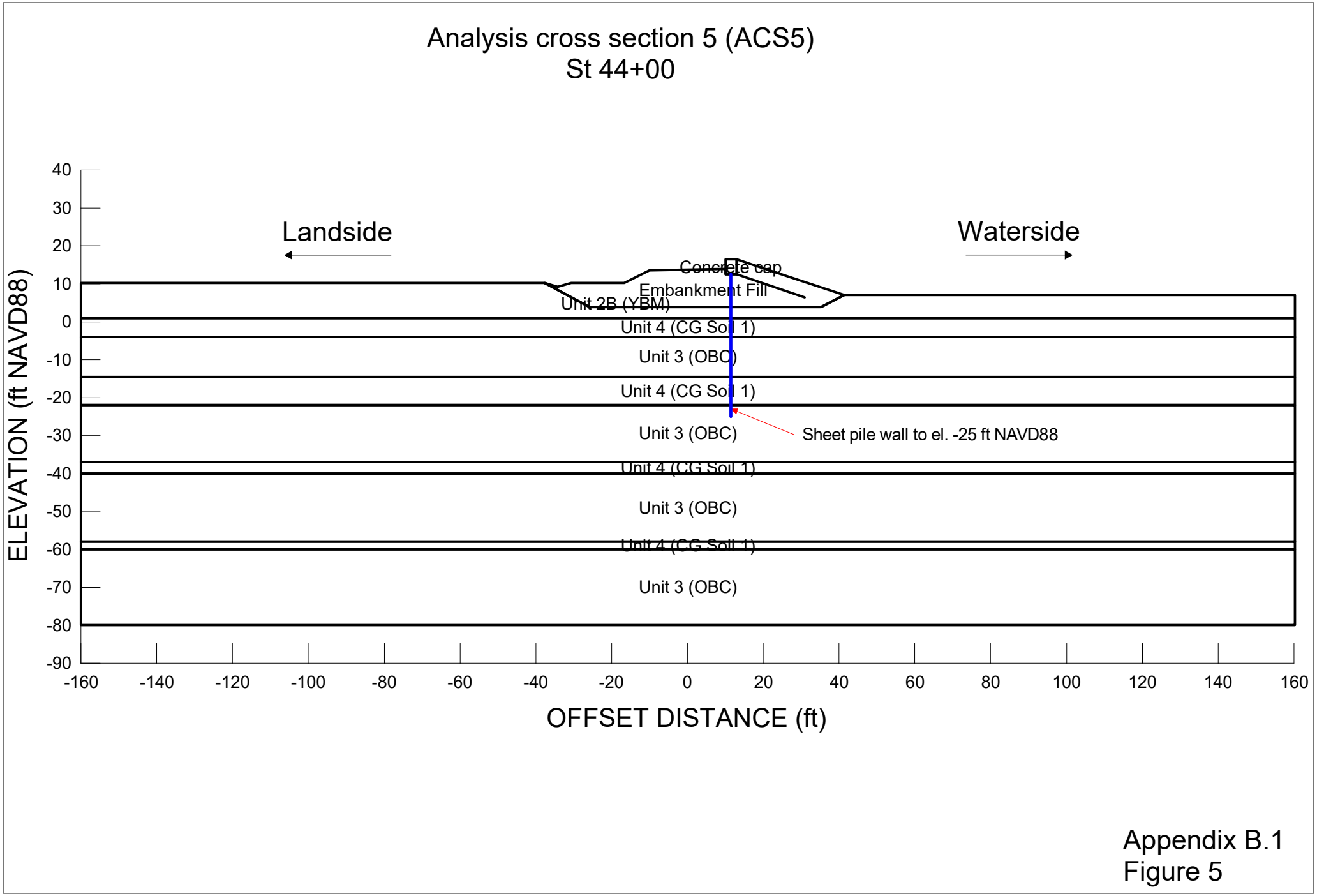


Appendix B.1
Figure 3a

Analysis cross section 4 (ACS4)
St 38+00

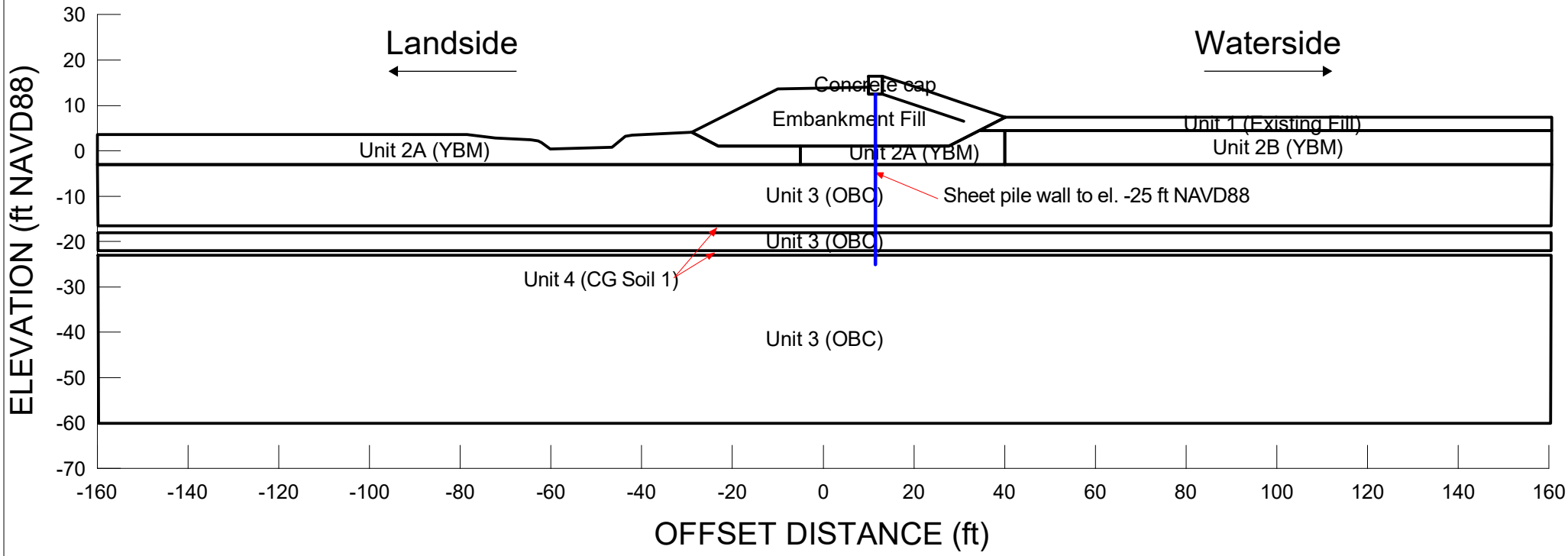


Appendix B.1
Figure 4



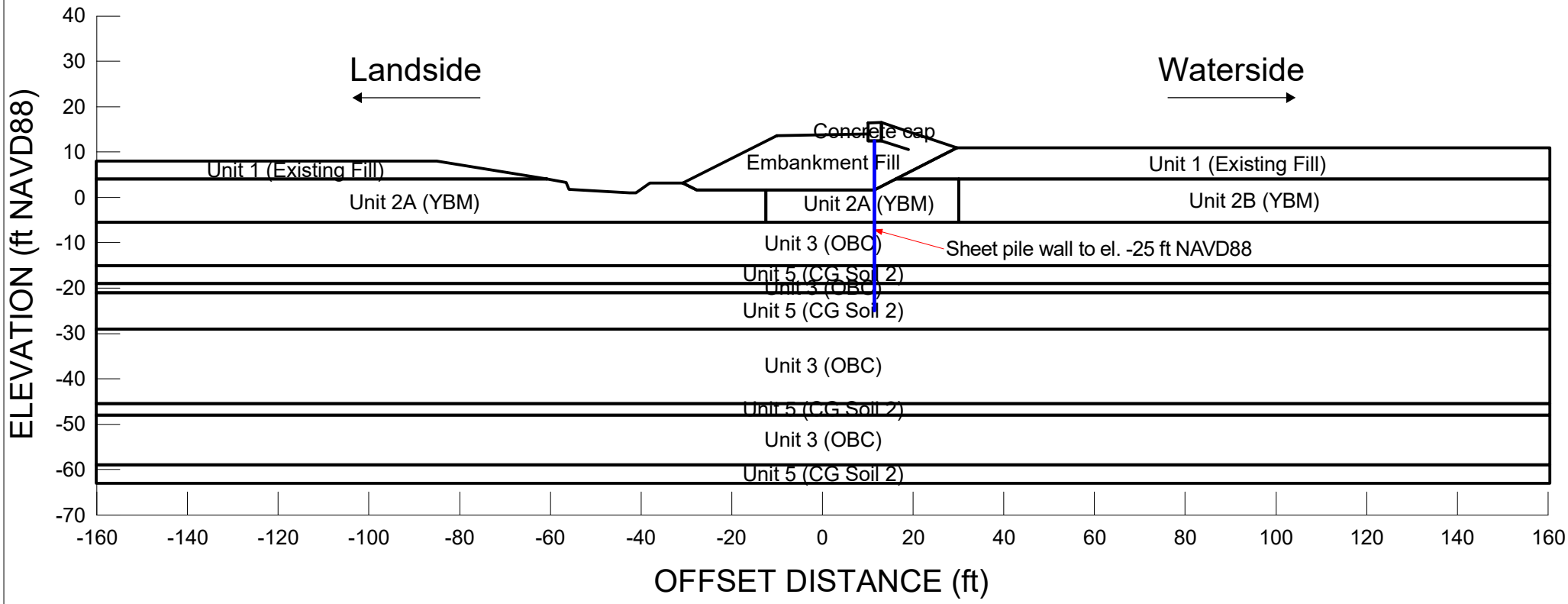
Appendix B.1
Figure 5

Analysis cross section 2 (ACS2)
St 19+50



Appendix B.1
Figure 2

Analysis cross section 3 (ACS3)
St 32+00



Appendix B.1
Figure 3b

B.2 Steady state underseepage analysis results

					Usual (required FOS of **2.8**)		Unusual (required FOS of **2.0**)		Extreme (required FOS of **1.6**)	
Analysis cross section	Blanket layers	Average unit wieght of blanket layer (pcf)	Critical exit gradient i_c	Exit gradient calculation location	Gradient, i_e	**FOS**	Gradient, i_e	**FOS**	Gradient, i_e	**FOS**
ACS1	Unit 2B	112,5	0,80	Embankment toe	0,016	**50,2**	0,019	**42,8**	0,020	**39,5**
		112,5	0,80	Pond	0,298	**2,69**	0,335	**2,40**	0,353	**2,27**
ACS2	Unit 2 and 3	112,5	0,80	Embankment toe	0,034	**23,6**	0,054	**14,8**	0,072	**11,1**
		112,5	0,80	Drainage channel	0,207	**3,88**	0,249	**3,22**	0,270	**2,98**
ACS3 (27+50)	Unit 2 and 3	112,5	0,80	Embankment toe	0,105	**7,68**	0,164	**4,88**	0,194	**4,14**
		112,5	0,80	Drainage channel	0,199	**4,03**	0,264	**3,05**	0,295	**2,72**
ACS3 (32+00)	Unit 2 and 3	112,5	0,80	Embankment toe	0,090	**8,92**	0,140	**5,71**	0,166	**4,85**
		112,5	0,80	Drainage channel	0,242	**3,31**	0,300	**2,68**	0,328	**2,44**
ACS4	Unit 2B	112,5	0,80	Embankment toe	0,198	**4,06**	0,389	**2,07**	0,482	**1,67**
ACS5	Unit 2B	112,5	0,80	Embankment toe	0,000	-	0,183	**4,38**	0,227	**3,53**

B.3 Landside steady-state stability analysis results

Analysis cross section	Landside steady state FOS for different loading conditions:			Drainage channel steady state FOS for different loading conditions:		
	Usual **FOS>1,6**	Unusual **FOS>1,5**	Extreme **FOS>1,4**	Usual **FOS>1,6**	Unusual **FOS>1,5**	Extreme **FOS>1,4**
ACS1	2,27	2,24	2,23	-	-	-
ACS2	1,98	1,92	1,89	3,18	2,94	2,84
ACS3 (27+50)	1,96	1,85	1,80	-	-	-
ACS3 (32+00)	2,02	1,94	1,91	6,03	5,81	5,71
ACS4	2,43	2,28	2,20	-	-	-
ACS5	3,19	3,09	3,04	-	-	-

B.4 Waterside rapid drawdown stability analysis results

Analysis cross section	Rapid drawndown FOS for different loading conditions:		
	Usual	Unusual **FOS>1,3**	Extreme **FOS>1,1**
ACS1	1,96	1,90	1,89
ACS2	2,84	2,79	2,79
ACS3 (27+50)	-	3,50	3,48
ACS3 (32+00)	-	3,38	3,39
ACS4	2,40	2,34	2,33
ACS5	1,62	1,51	1,50

B.5 End of construction stability analysis results

Analysis cross section	End of construction stability Required **FOS>1.3**	
	Landside	Waterside
ACS1	1,67	3,31
ACS2	2,06	3,41
ACS3 (27+50)	1,72	5,58
ACS3 (32+00)	1,73	5,58
ACS4	1,88	3,04
ACS5	2,71	2,95

B.6 Seismic slope stability analysis results

pseudostatic+liquefaction		OBE		MDE		
Section	Side	FoS	Figure (FoS)	FoS	Figure (FoS)	Figure (Ky)
		(target >1.2)		(target >1.2)		
10+35	Landside	1.12	B8-1	0.89	B8-2	B8-9
19+50	Landside	1.22	B8-11	1.01	B8-12	B8-17
27+50	Landside	1.07	B8-19	0.89	B8-20	B8-23
32+00	Landside	1.06	B8-25	0.88	B8-26	B8-31
38+00	Landside	1.13	B8-34	0.94	B8-35	B8-42
44+00	Landside	1.23	B8-45	0.94	B8-46	B8-53
10+35	Waterside	1.79	B8-3	1.35	B8-4	-
19+50	Waterside	1.57	B8-13	1.22	B8-14	-
27+50	Waterside	1.60	B8-21	1.16	B8-22	B8-24
32+00	Waterside	1.60	B8-27	1.16	B8-28	B8-32
38+00	Waterside	1.60	B8-36	1.28	B8-37	-
44+00	Waterside	1.53	B8-47	1.18	B8-48	B8-54

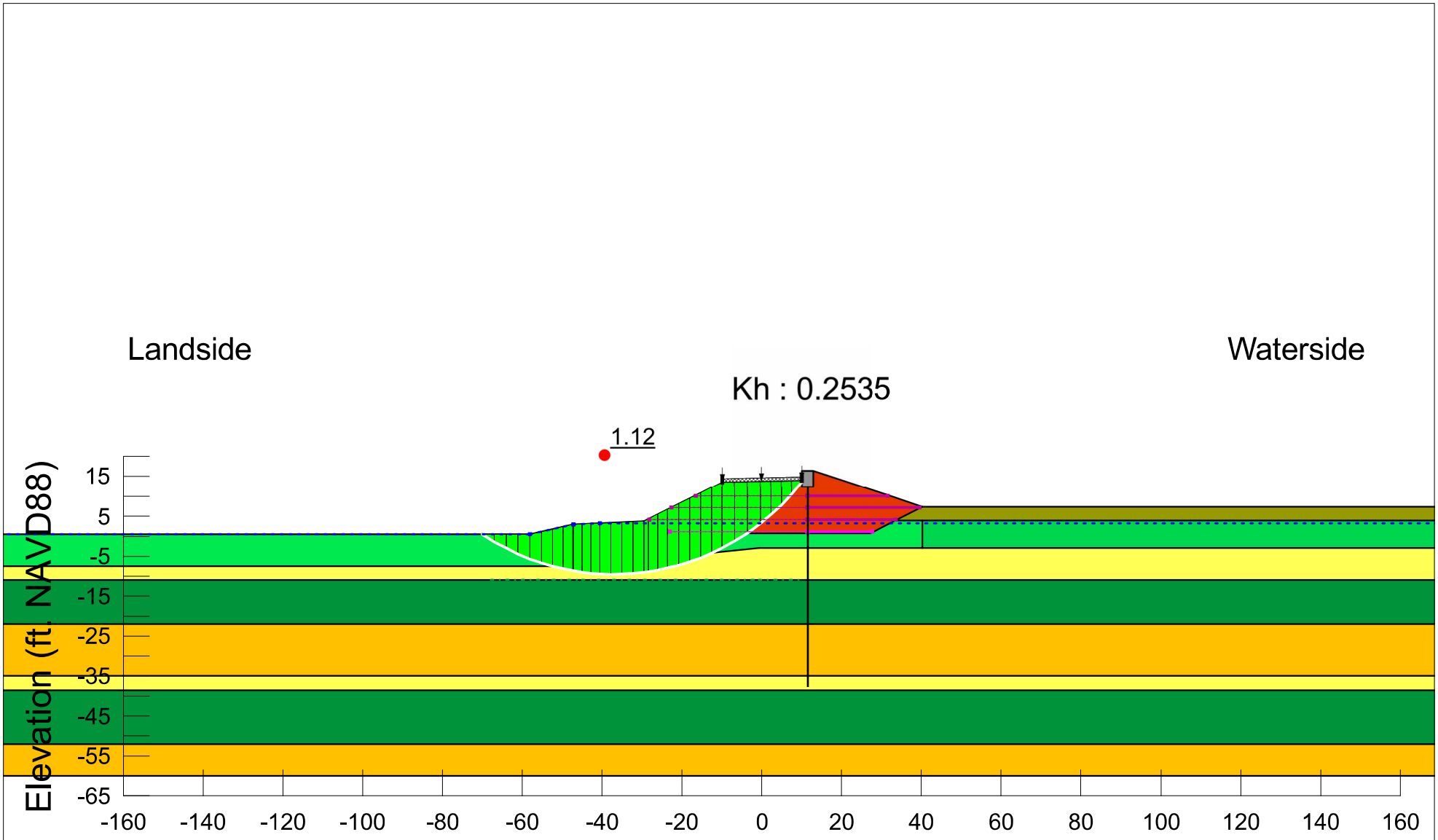
post-liquefaction pseudostatic		OBE		MDE		
Section	Side	FoS	Figure (FoS)	FoS	Figure (FoS)	Figure (Ky)
		(target >1.1)		(target >1.1)		
10+35	Landside	0.90	B8-5	0.79	B8-6	B8-10
19+50	Landside	1.17	B8-15	0.96	B8-16	B8-18
27+50	Landside	NA	-	NA	-	-
32+00	Landside	0.84	B8-29	0.76	B8-30	B8-33
38+00	Landside	1.04	B8-38	0.84	B8-39	B8-43
44+00	Landside	0.91	B8-49	0.69	B8-50	B8-55
10+35	Waterside	1.25	B8-7	1.13	B8-8	-
19+50	Waterside	NA	-	NA	-	-
27+50	Waterside	NA	-	NA	-	-
32+00	Waterside	NA	-	NA	-	-
38+00	Waterside	1.28	B8-40	1.03	B8-41	B8-44
44+00	Waterside	1.16	B8-51	1.00	B8-52	B8-56

B.7 Seismic displacement results

pseudostatic+liquefaction		OBE			MDE		
Section	Side	disp (ft) (target <1)			disp (ft) (target <3.4)		
		d84%	d50%	d16%	d84%	d50%	d16%
10+35	Landside	0.2	0.4	0.8	0.5	1.2	2.7
19+50	Landside	FoS compliant	FoS compliant	FoS compliant	0.3	0.8	1.7
27+50	Landside	0.1	0.3	0.7	0.5	1.2	2.6
32+00	Landside	0.3	0.5	0.8	0.5	1.1	2.4
38+00	Landside	0.1	0.3	0.6	0.4	0.9	2.0
44+00	Landside	FoS compliant	FoS compliant	FoS compliant	0.3	0.8	1.8
10+35	Waterside	FoS compliant	FoS compliant	FoS compliant	FoS compliant	FoS compliant	FoS compliant
19+50	Waterside	FoS compliant	FoS compliant	FoS compliant	FoS compliant	FoS compliant	FoS compliant
27+50	Waterside	FoS compliant	FoS compliant	FoS compliant	0.2	0.6	1.2
32+00	Waterside	FoS compliant	FoS compliant	FoS compliant	0.2	0.6	1.2
38+00	Waterside	FoS compliant	FoS compliant	FoS compliant	FoS compliant	FoS compliant	FoS compliant
44+00	Waterside	FoS compliant	FoS compliant	FoS compliant	5.9	13.7	28.9

post-liquefaction pseudostatic		OBE			MDE		
Section	Side	disp (ft) (target <1)			disp (ft) (target <3.4)		
		d84%	d50%	d16%	d84%	d50%	d16%
10+35	Landside	0.3	0.6	1.3	1.9	4.7	10.1
19+50	Landside	FoS compliant	FoS compliant	FoS compliant	0.4	0.9	2.0
27+50	Landside	NA	NA	NA	NA	NA	NA
32+00	Landside	0.4	0.9	2.0	1.3	3.2	6.8
38+00	Landside	0.2	0.4	0.8	0.6	1.4	2.9
44+00	Landside	0.2	0.6	1.2	0.8	2.1	4.4
10+35	Waterside	FoS compliant	FoS compliant	FoS compliant	FoS compliant	FoS compliant	FoS compliant
19+50	Waterside	NA	NA	NA	NA	NA	NA
27+50	Waterside	NA	NA	NA	NA	NA	NA
32+00	Waterside	NA	NA	NA	NA	NA	NA
38+00	Waterside	FoS compliant	FoS compliant	FoS compliant	0.3	0.6	1.4
44+00	Waterside	FoS compliant	FoS compliant	FoS compliant	0.3	0.7	1.5

B.8 Seismic slope stability output figures

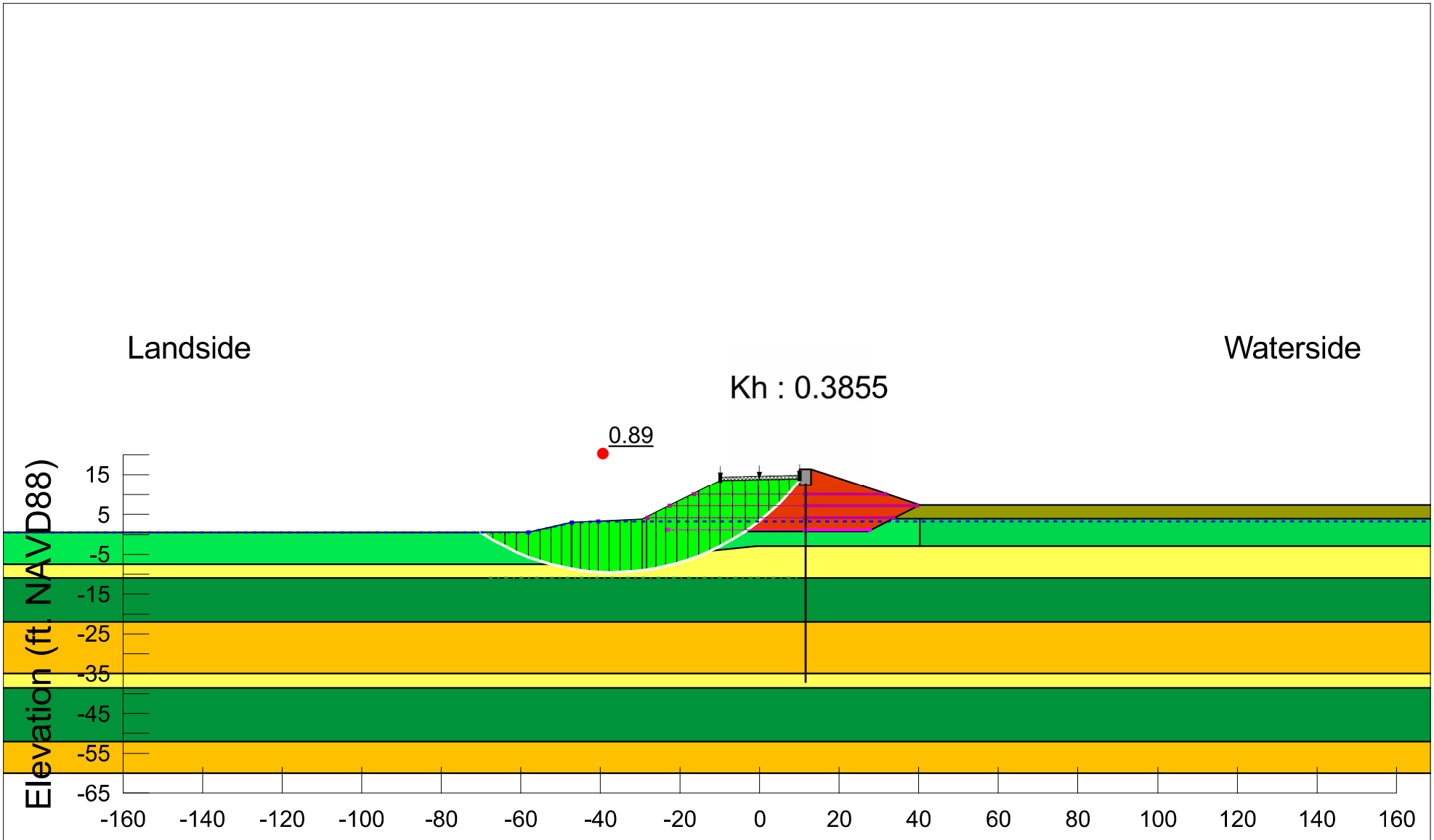


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-1

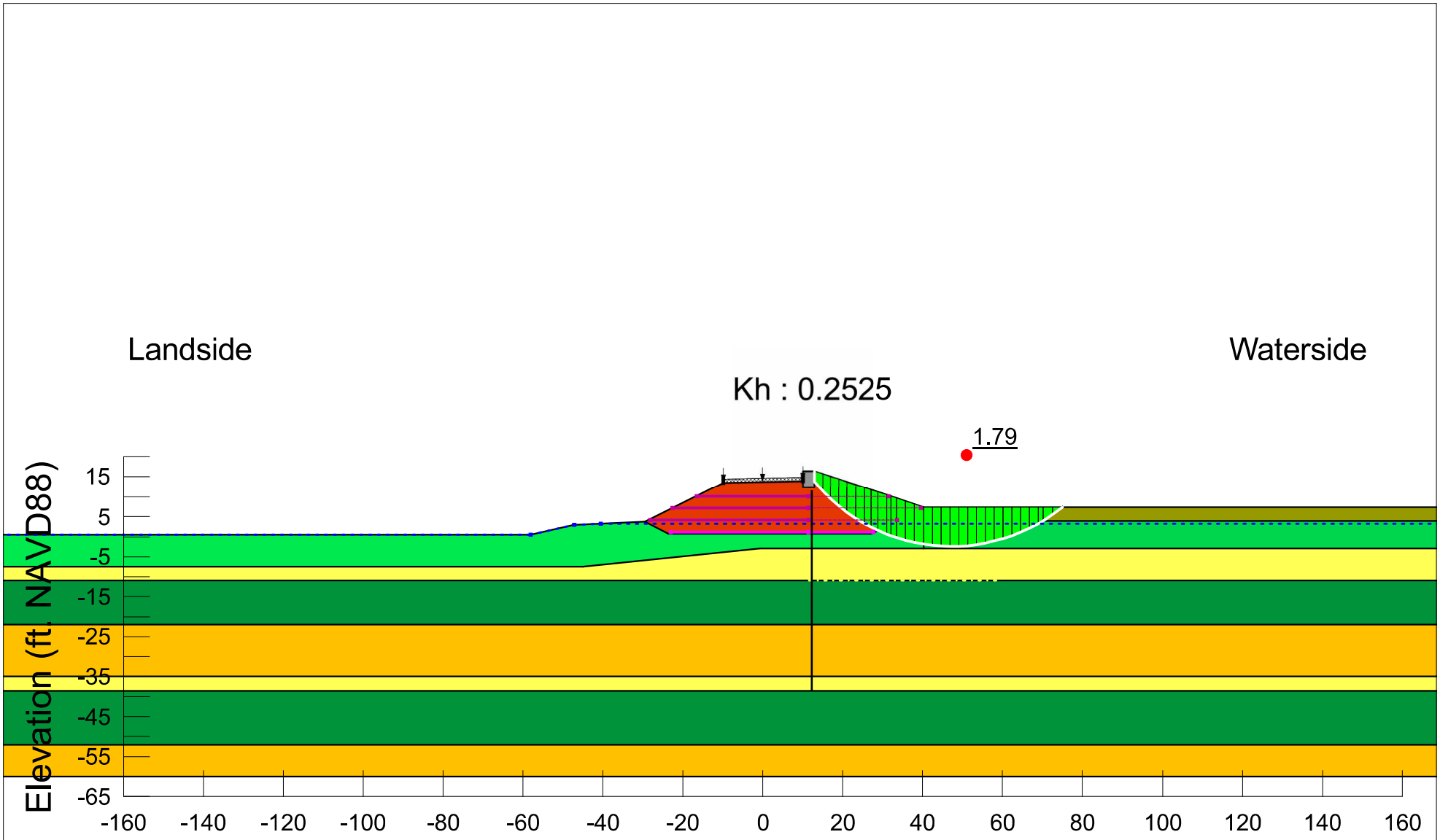


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-2

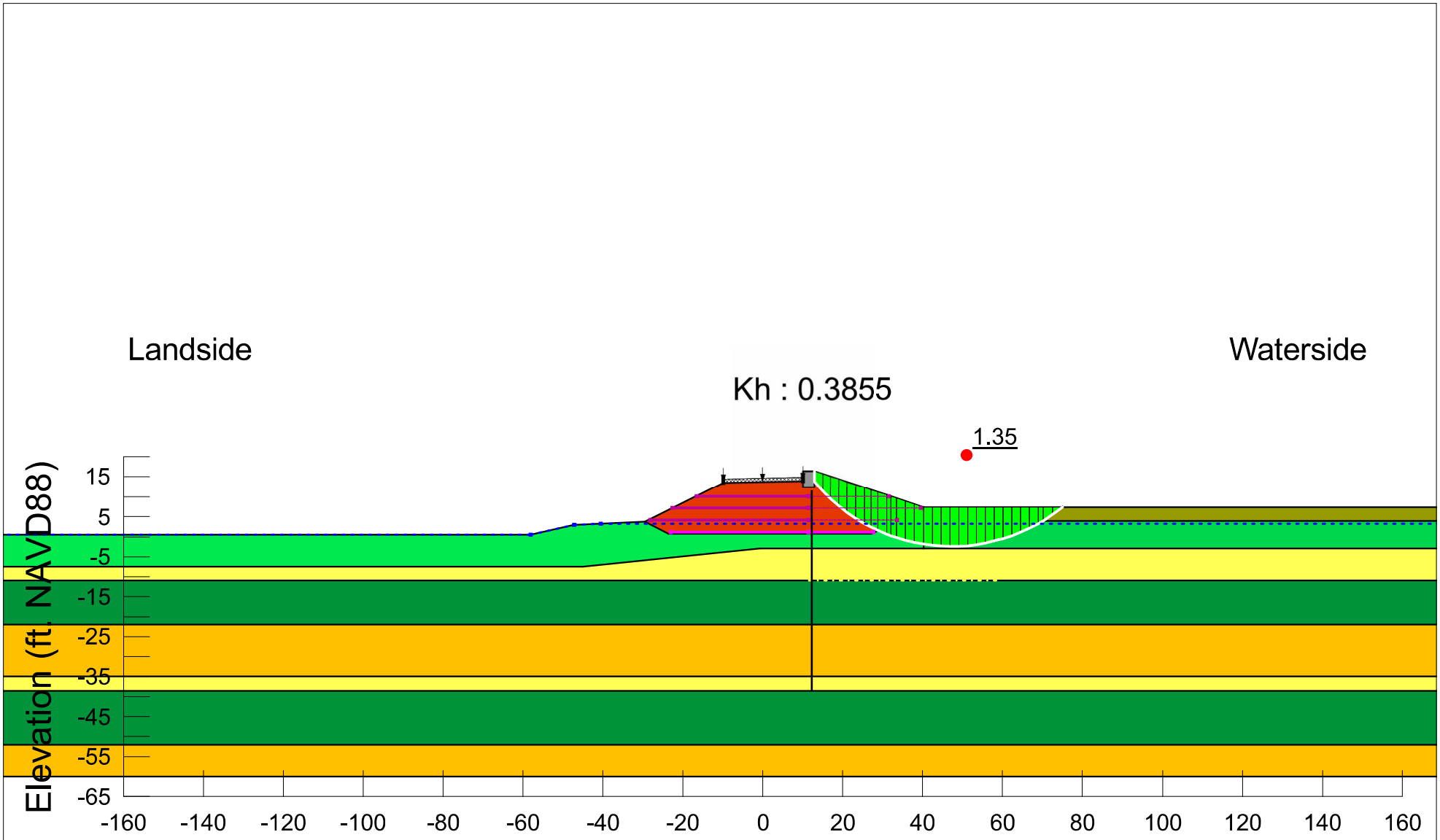


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-3

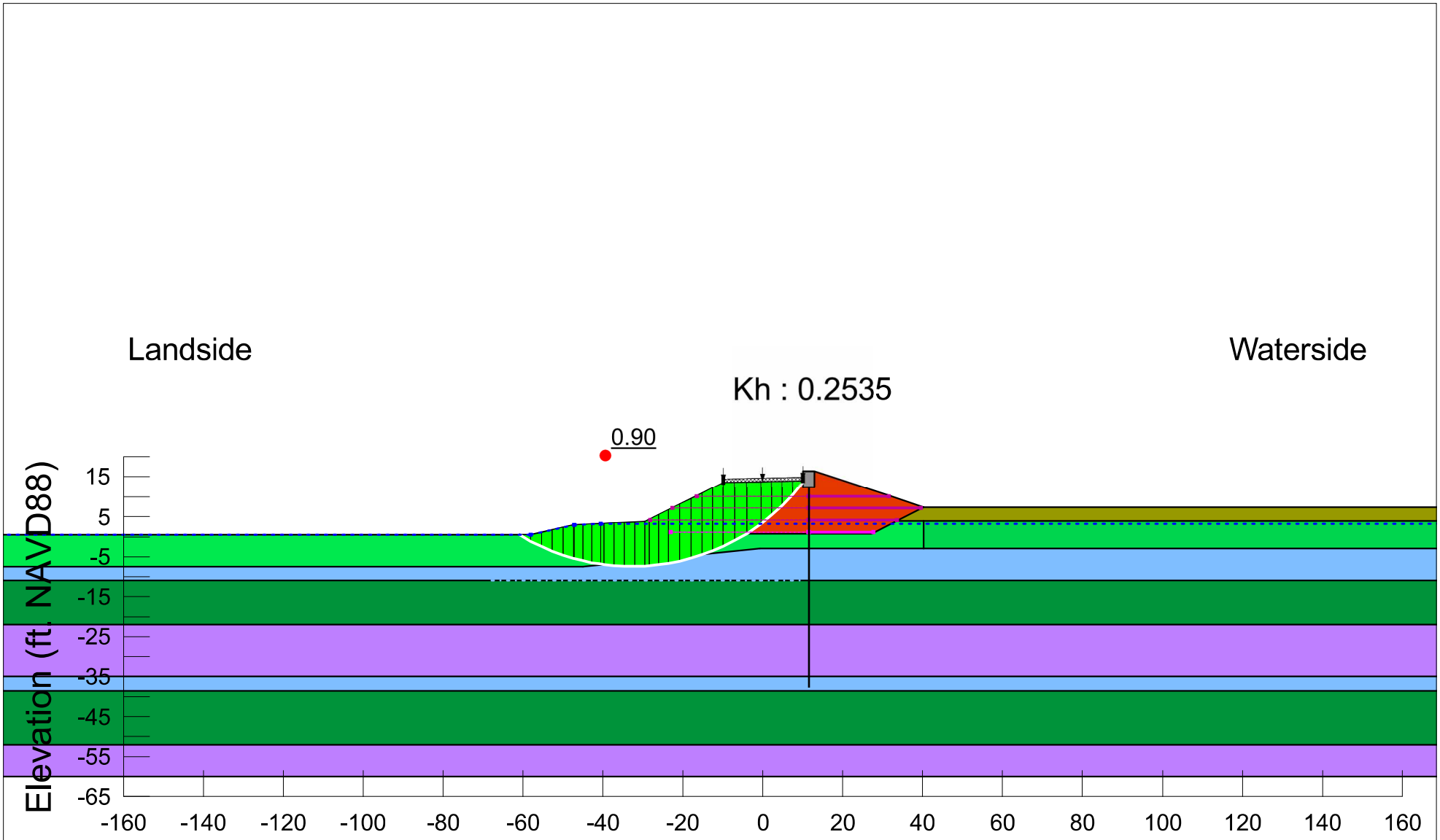


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-4

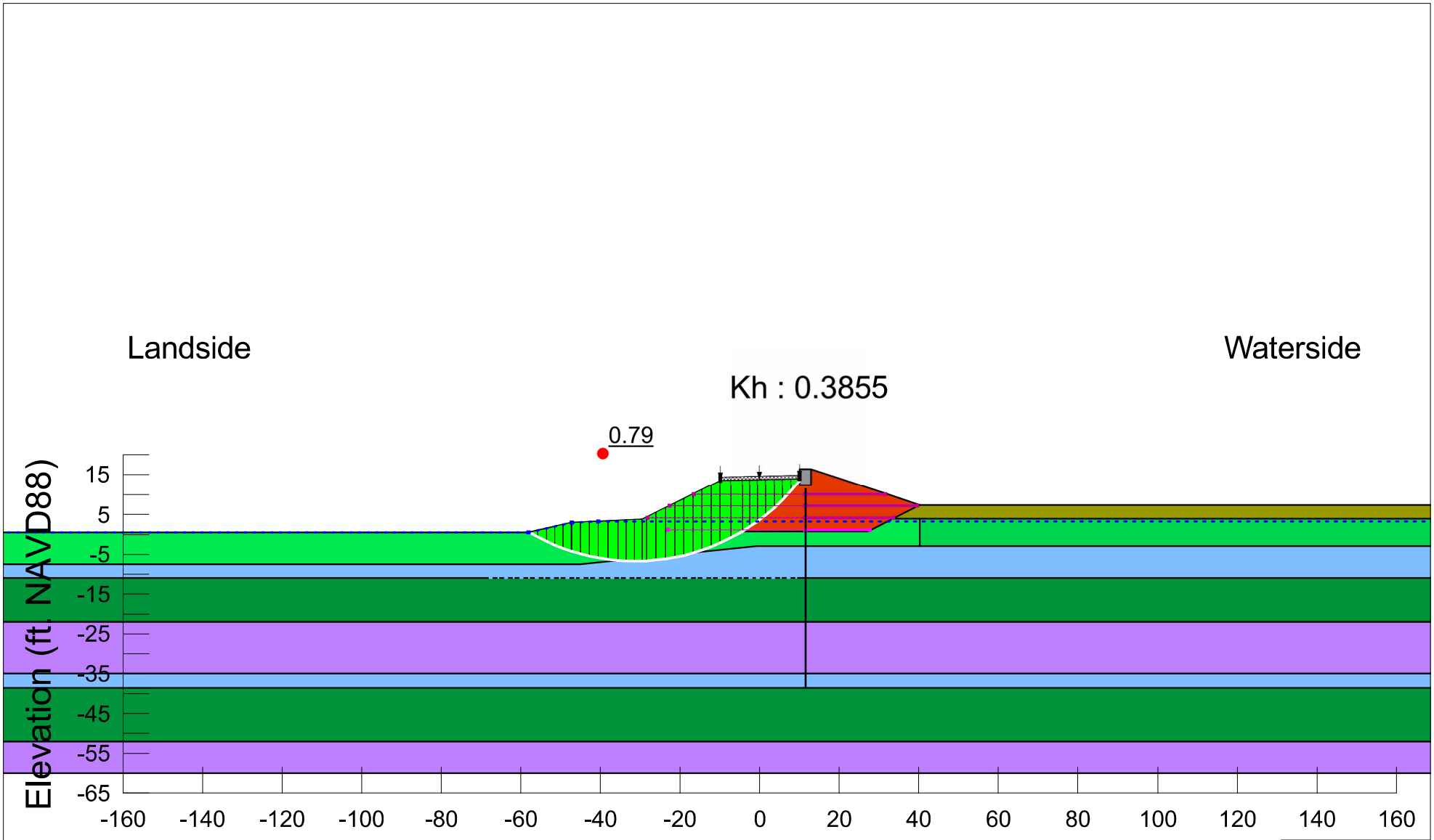


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-5

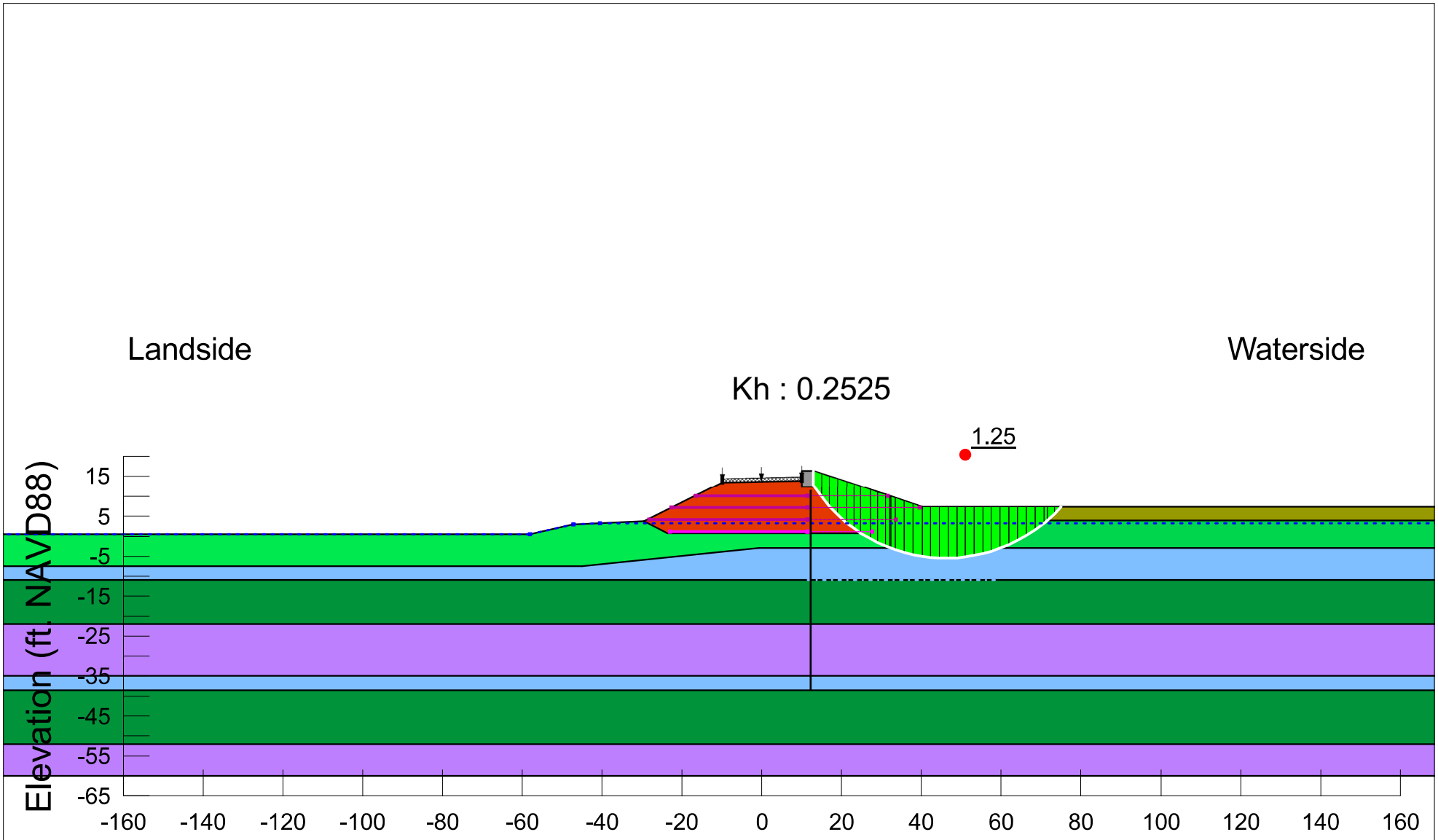


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-6

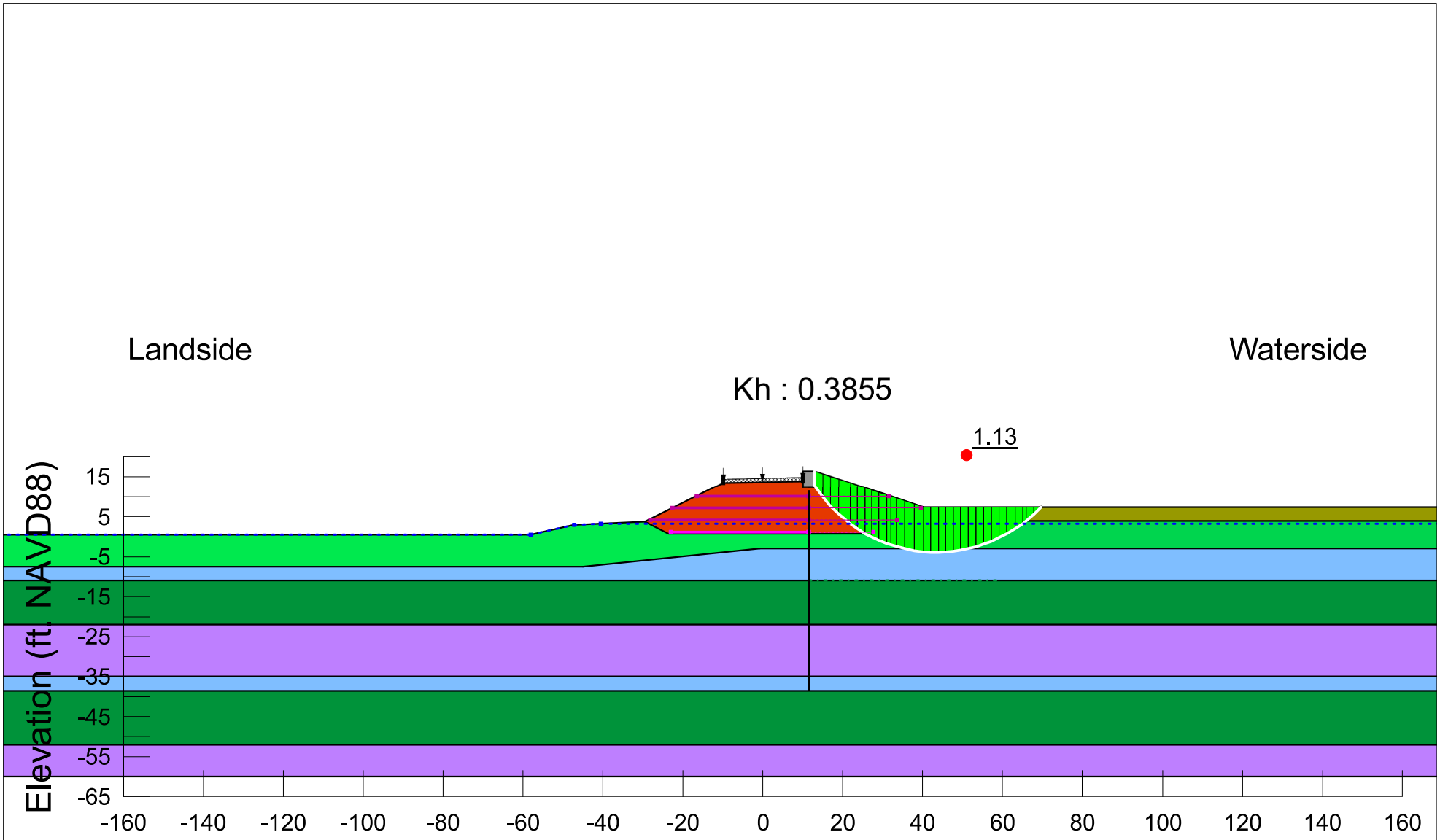


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-7

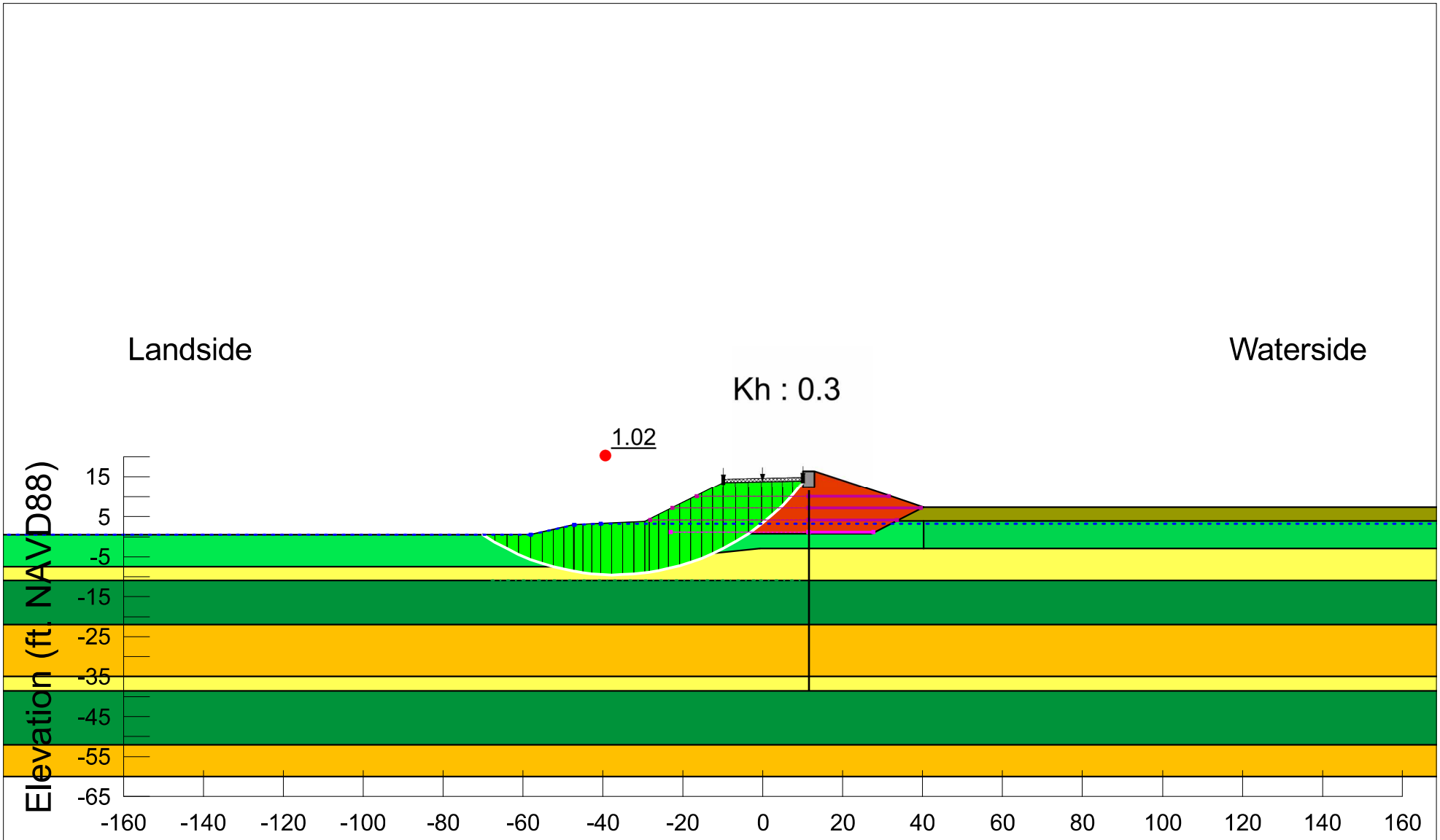


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-8

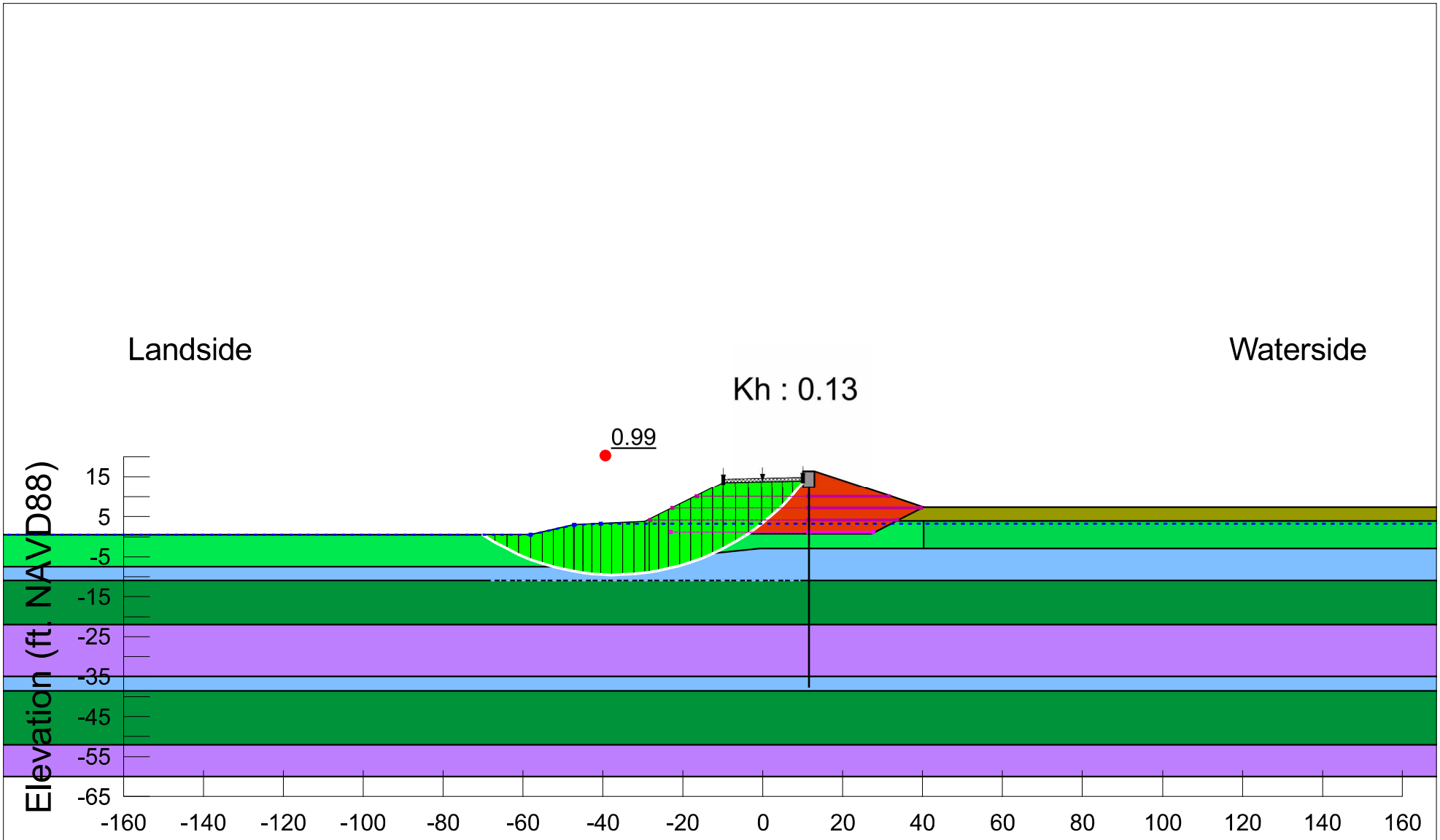


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-9

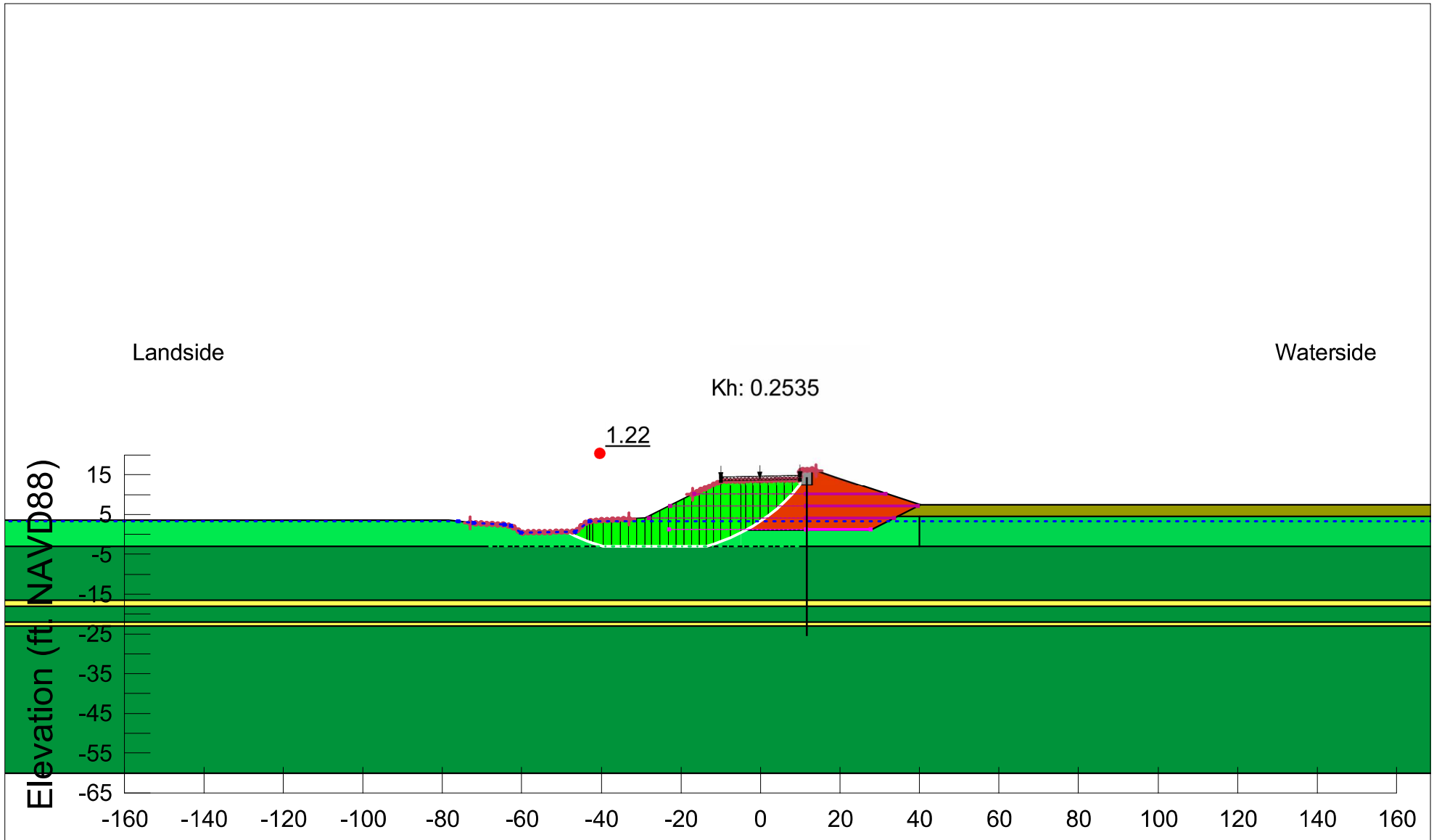


CROSS SECTION ACS1 (STATION 10+35)
CUTOFF WALL TO -40 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-10

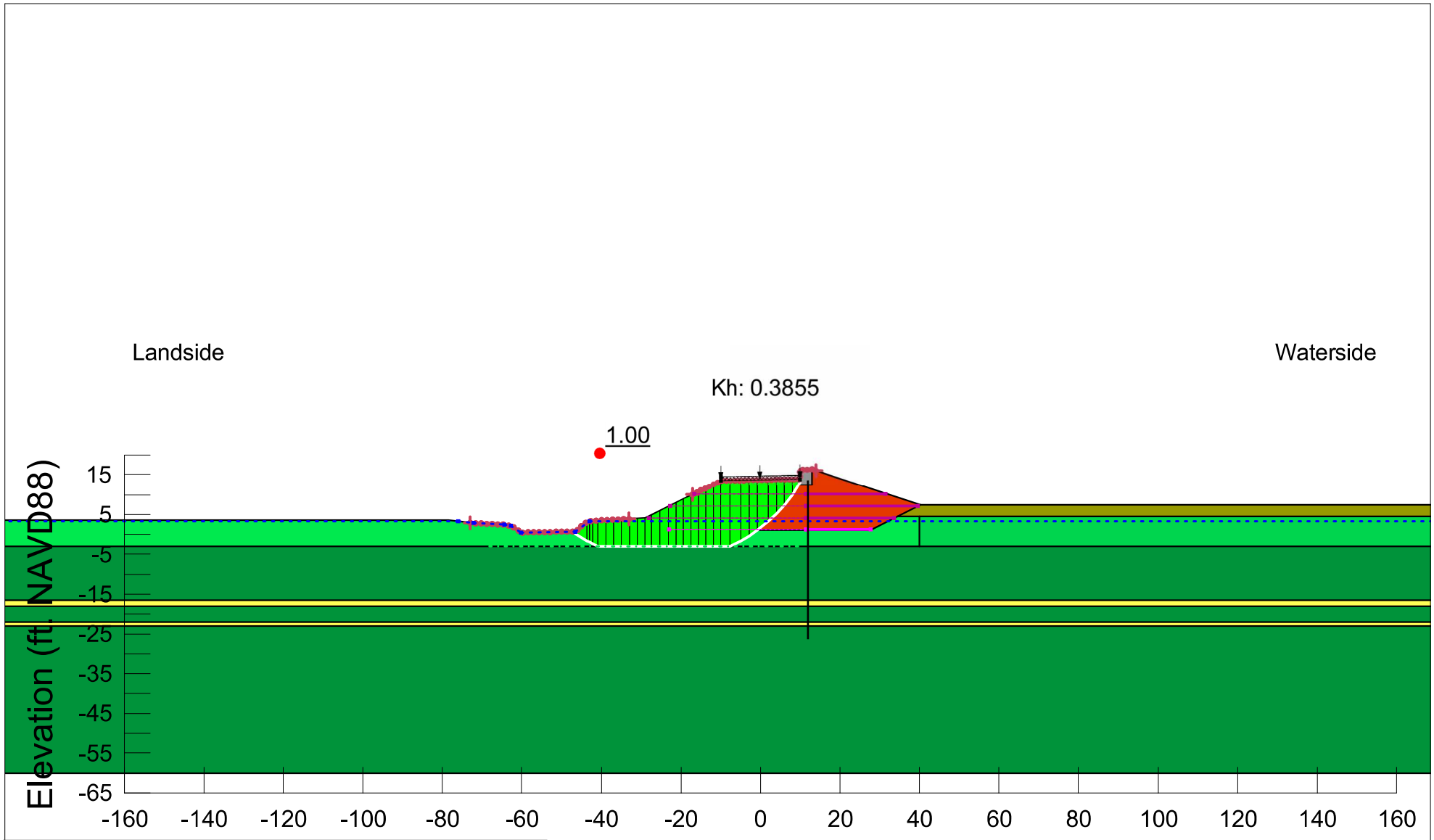


CROSS SECTION ACS2 (STATION 19+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-11

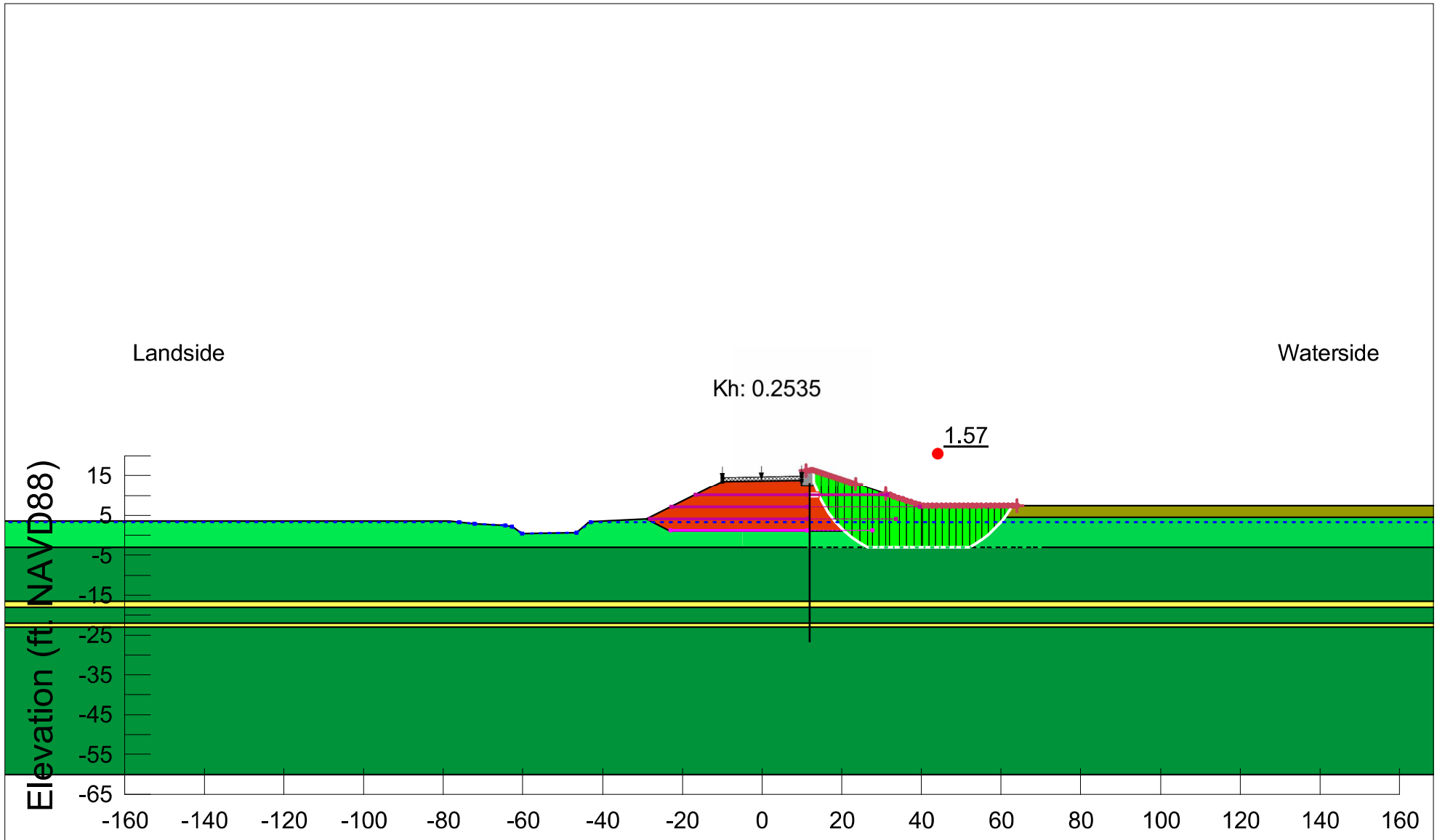


CROSS SECTION ACS2 (STATION 19+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-12

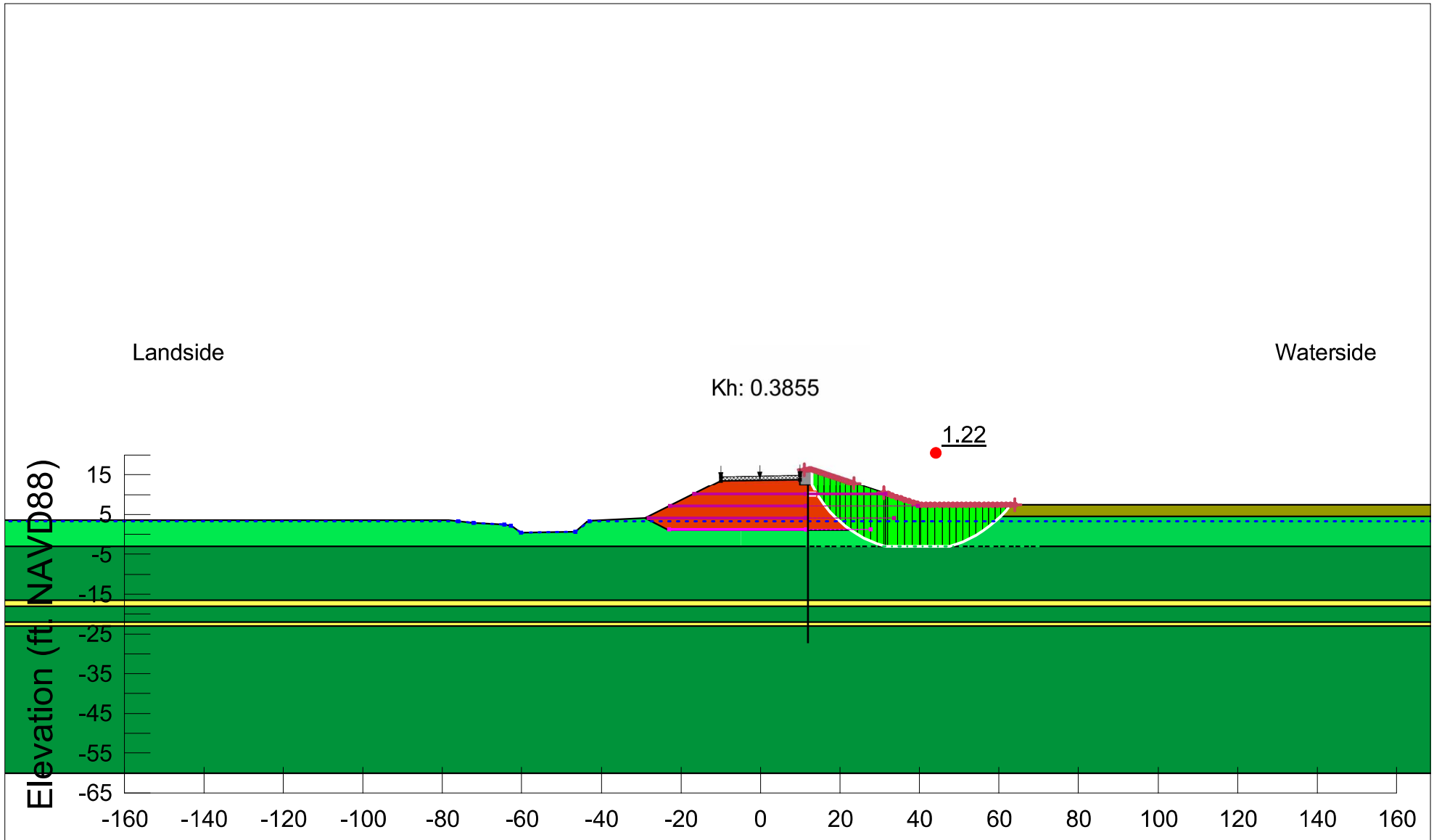


CROSS SECTION ACS2 (STATION 19+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-13

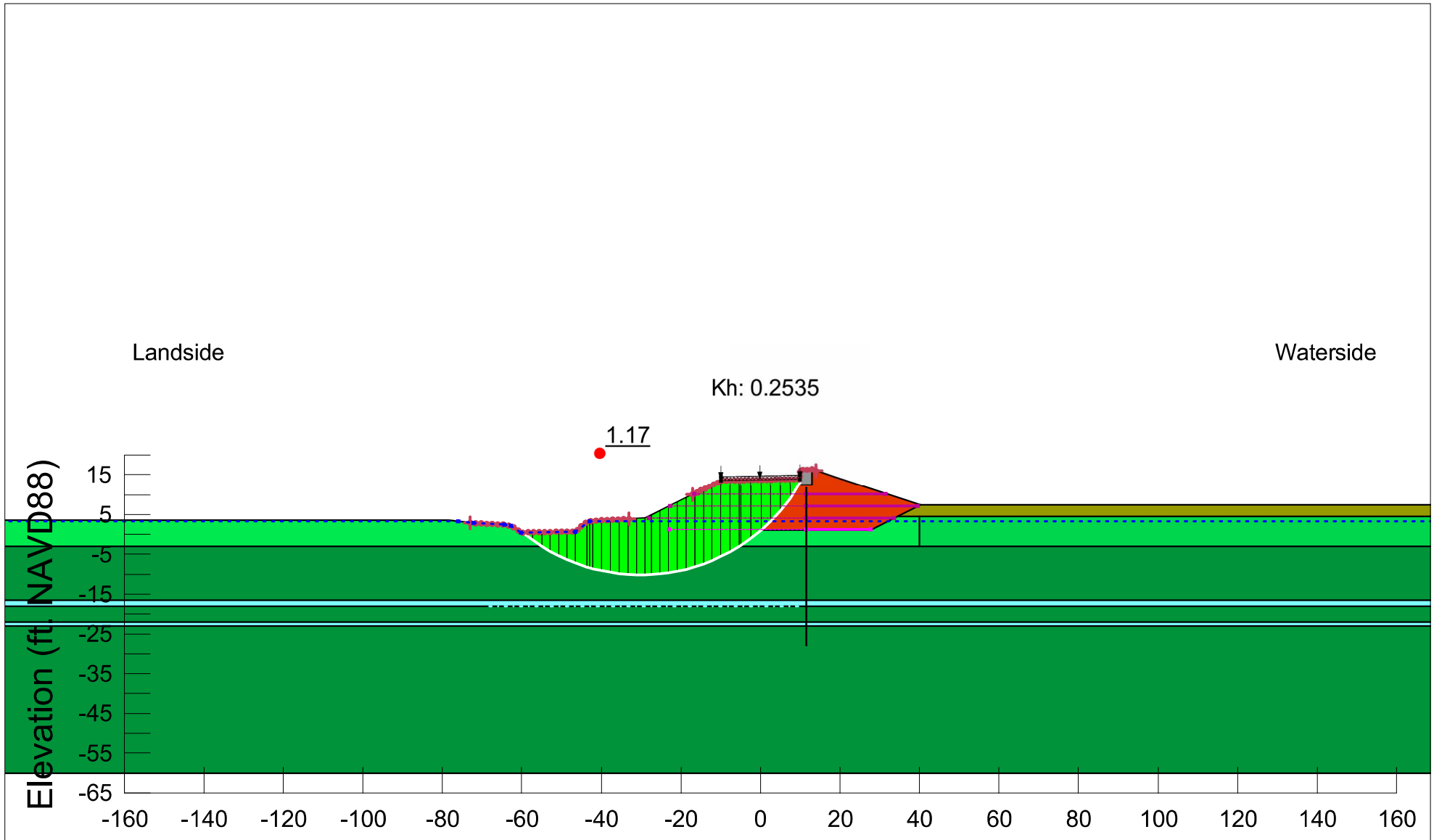


CROSS SECTION ACS2 (STATION 19+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-14

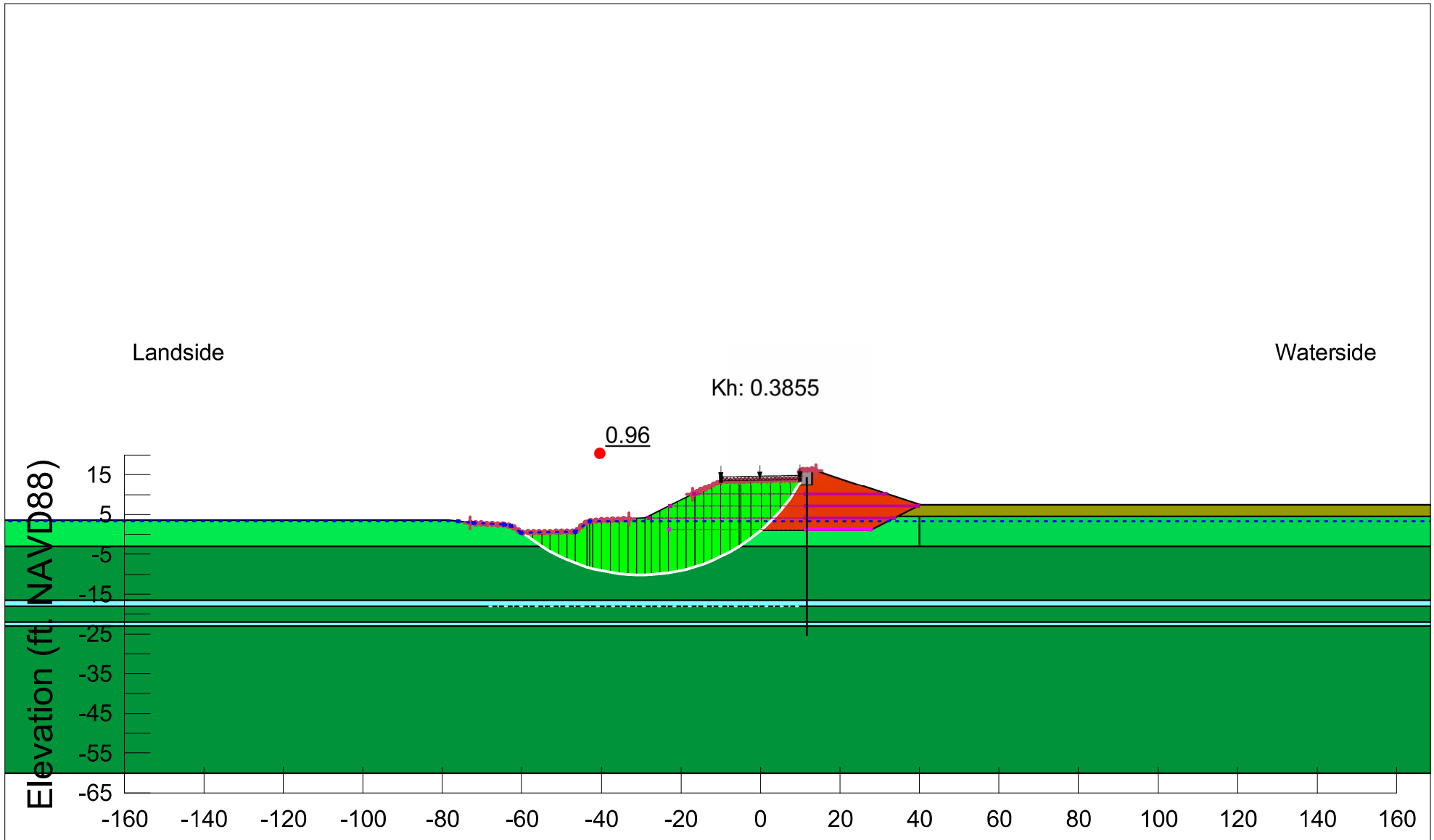


CROSS SECTION ACS2 (STATION 19+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-15

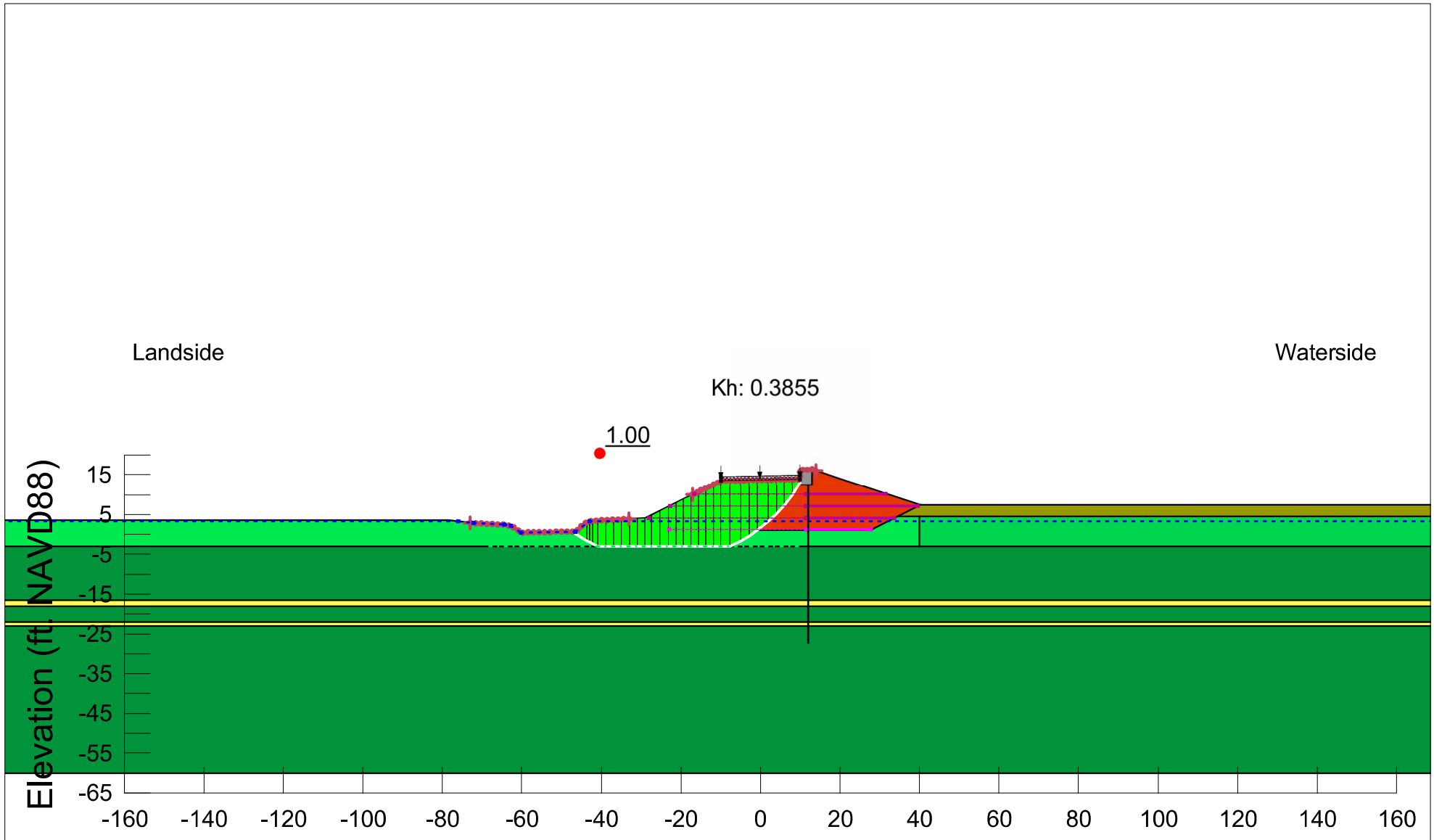


CROSS SECTION ACS2 (STATION 19+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-16

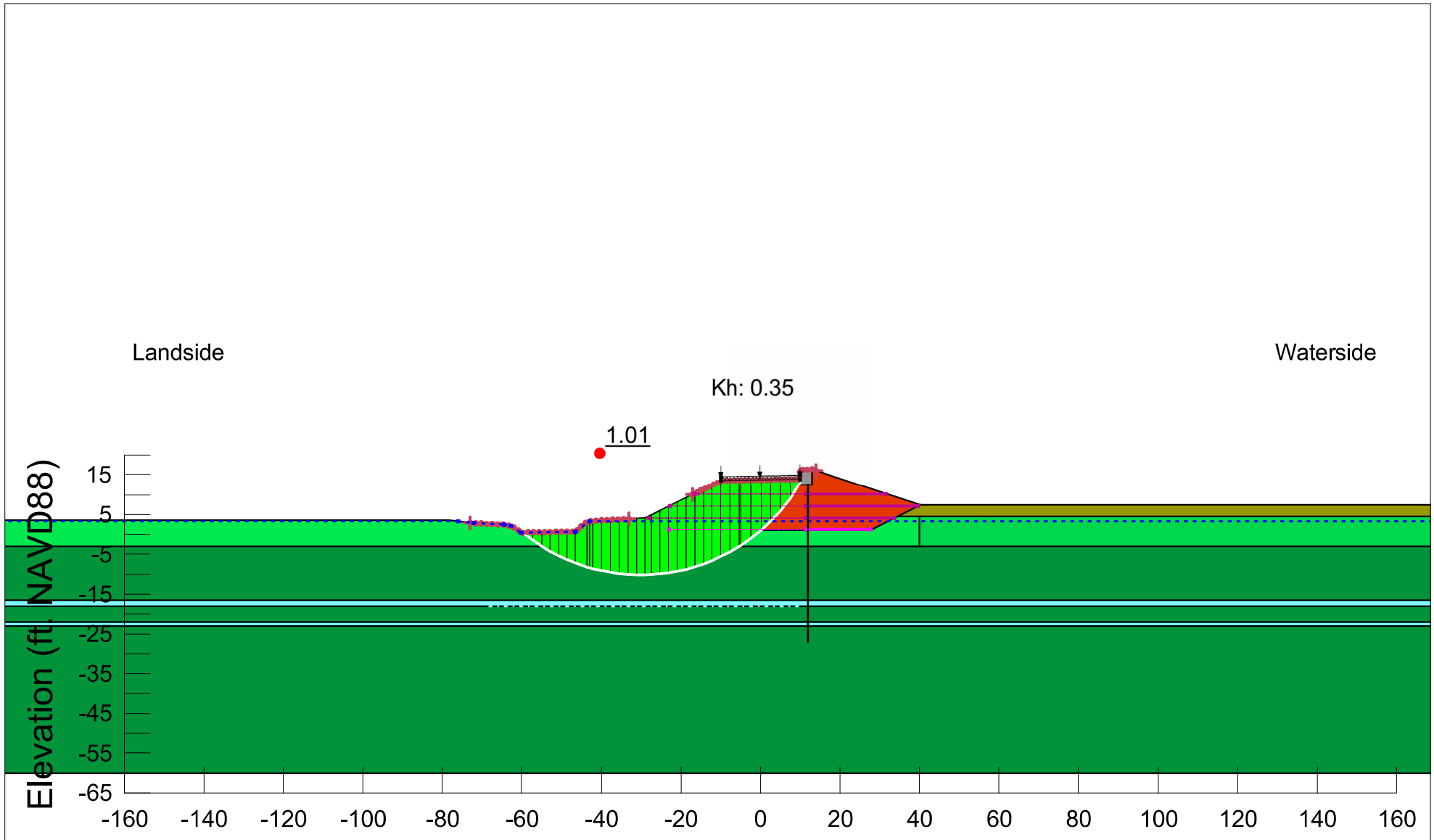


CROSS SECTION ACS2 (STATION 19+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-17



CROSS SECTION ACS2 (STATION 19+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - YIELD COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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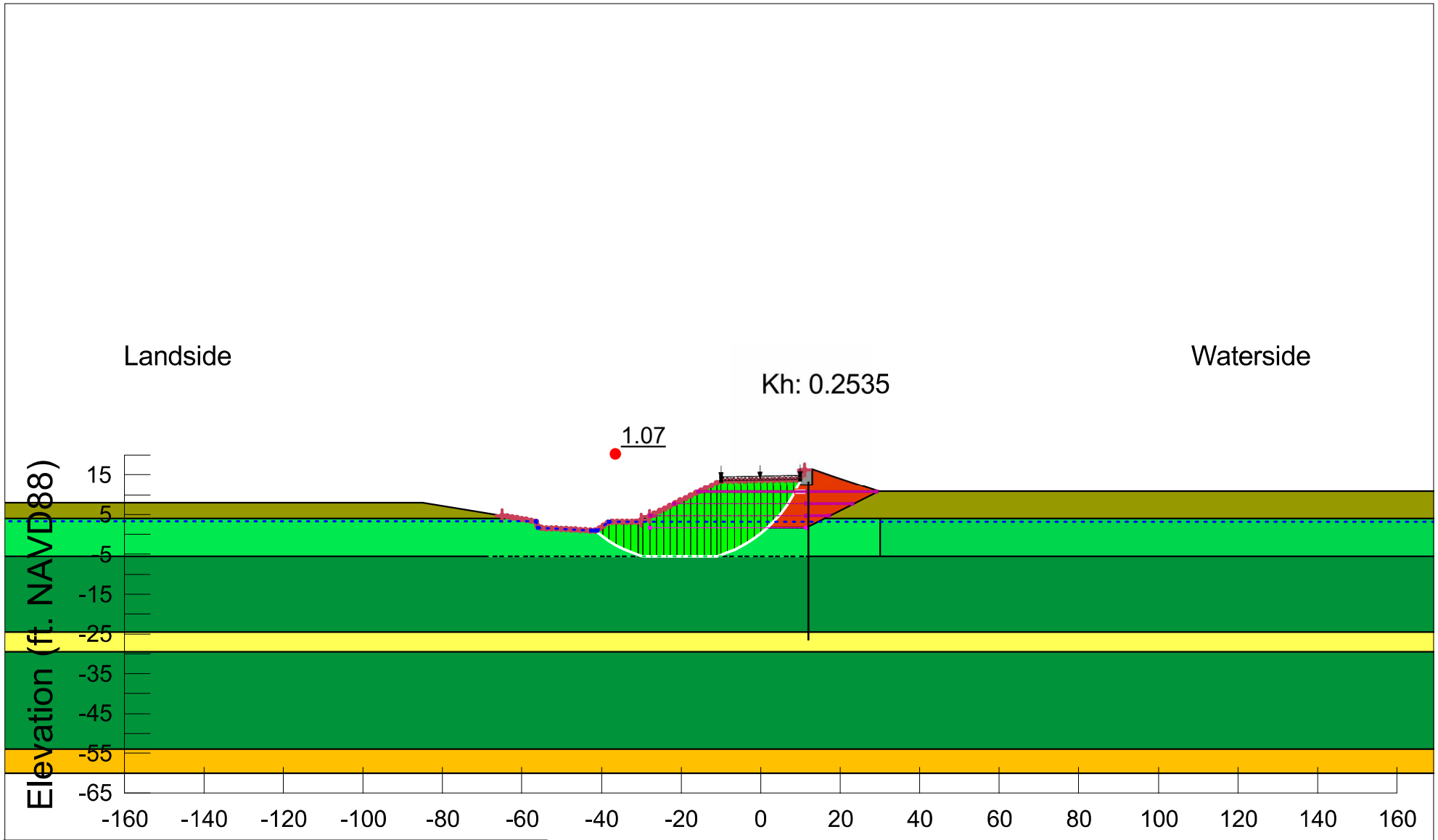
FIGURE B8-18



CROSS SECTION ACS2 (STATION 27+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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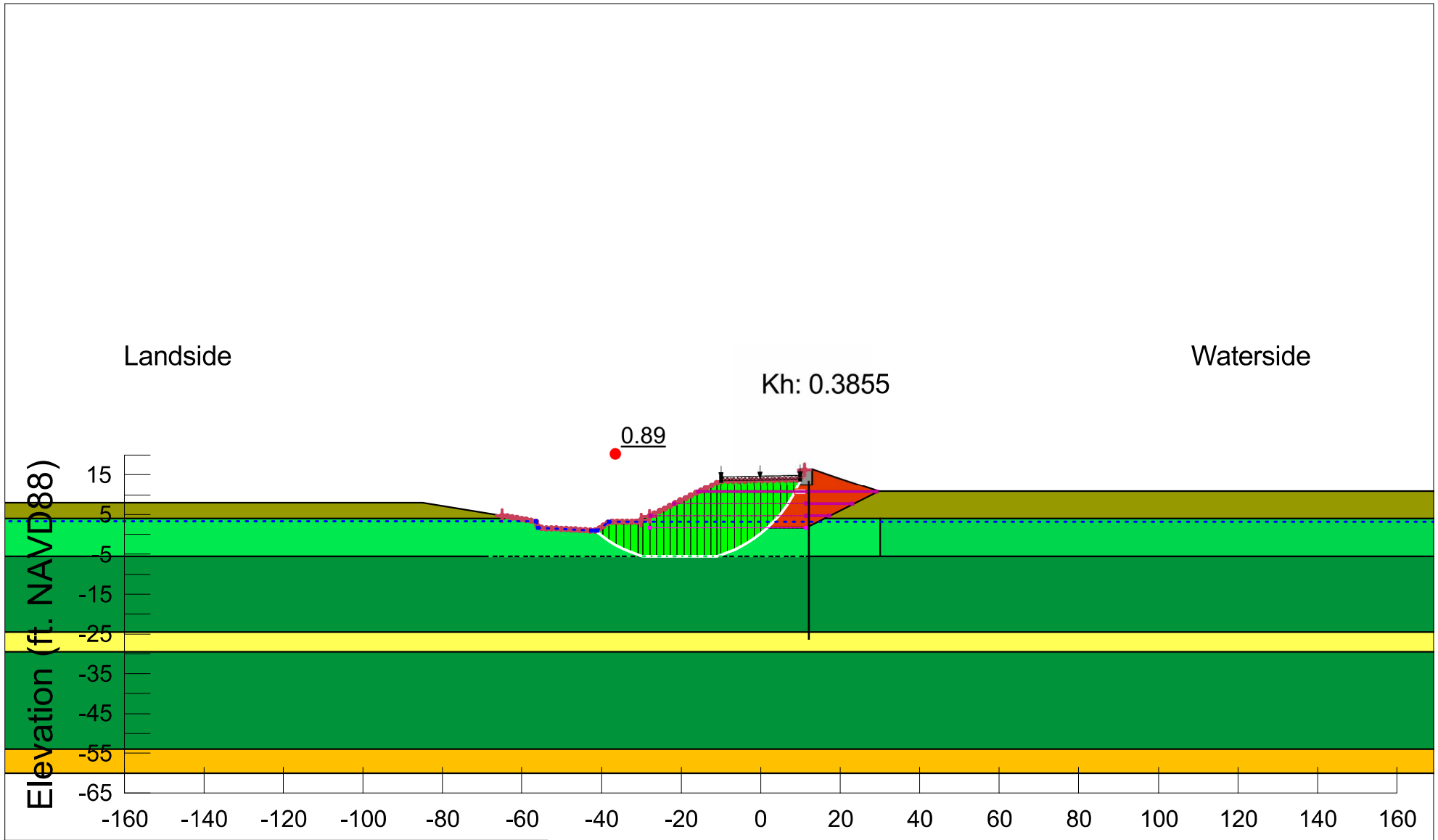
FIGURE B8-19



CROSS SECTION ACS2 (STATION 27+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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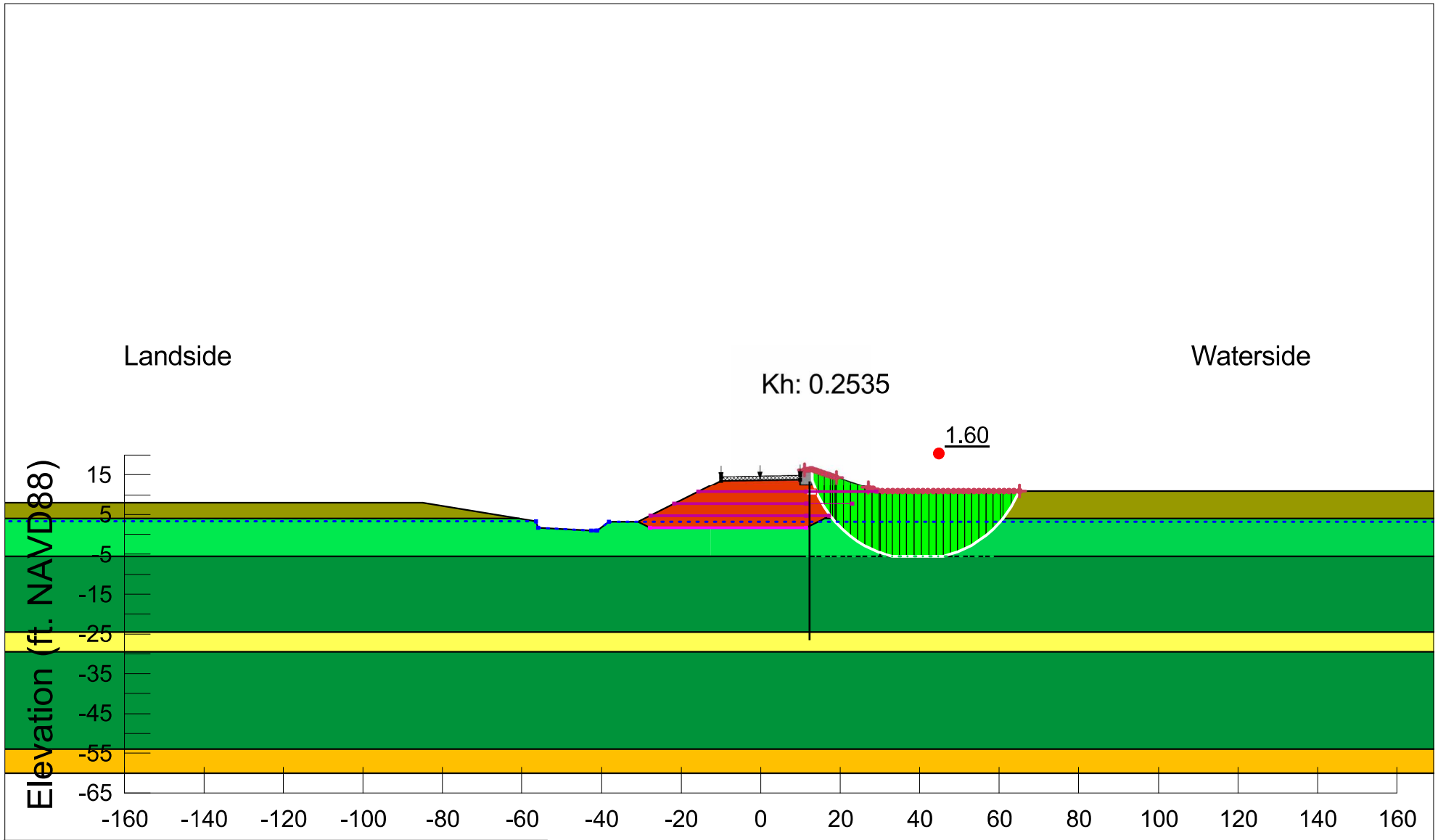
FIGURE B8-20



CROSS SECTION ACS2 (STATION 27+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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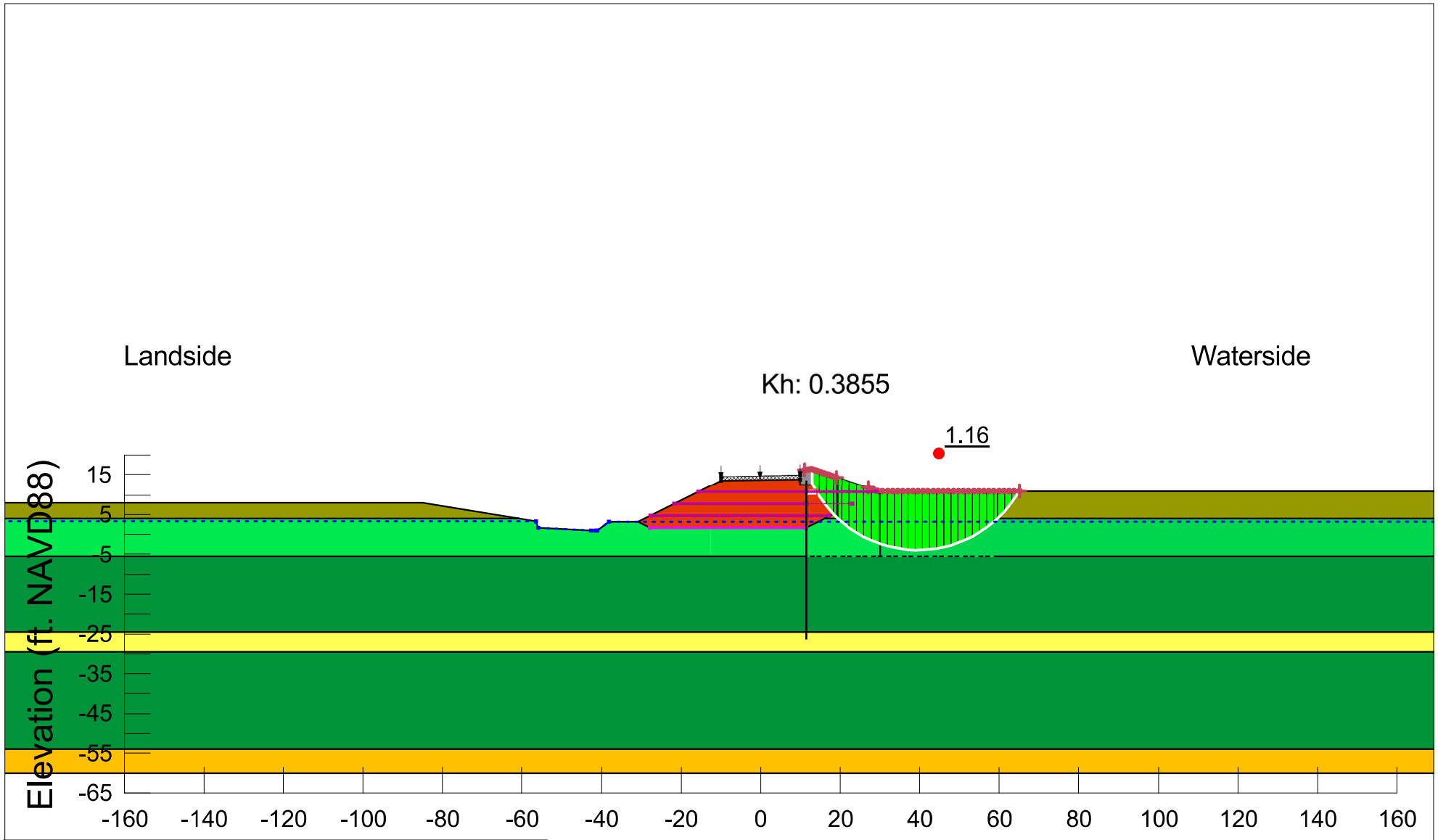
FIGURE B8-21

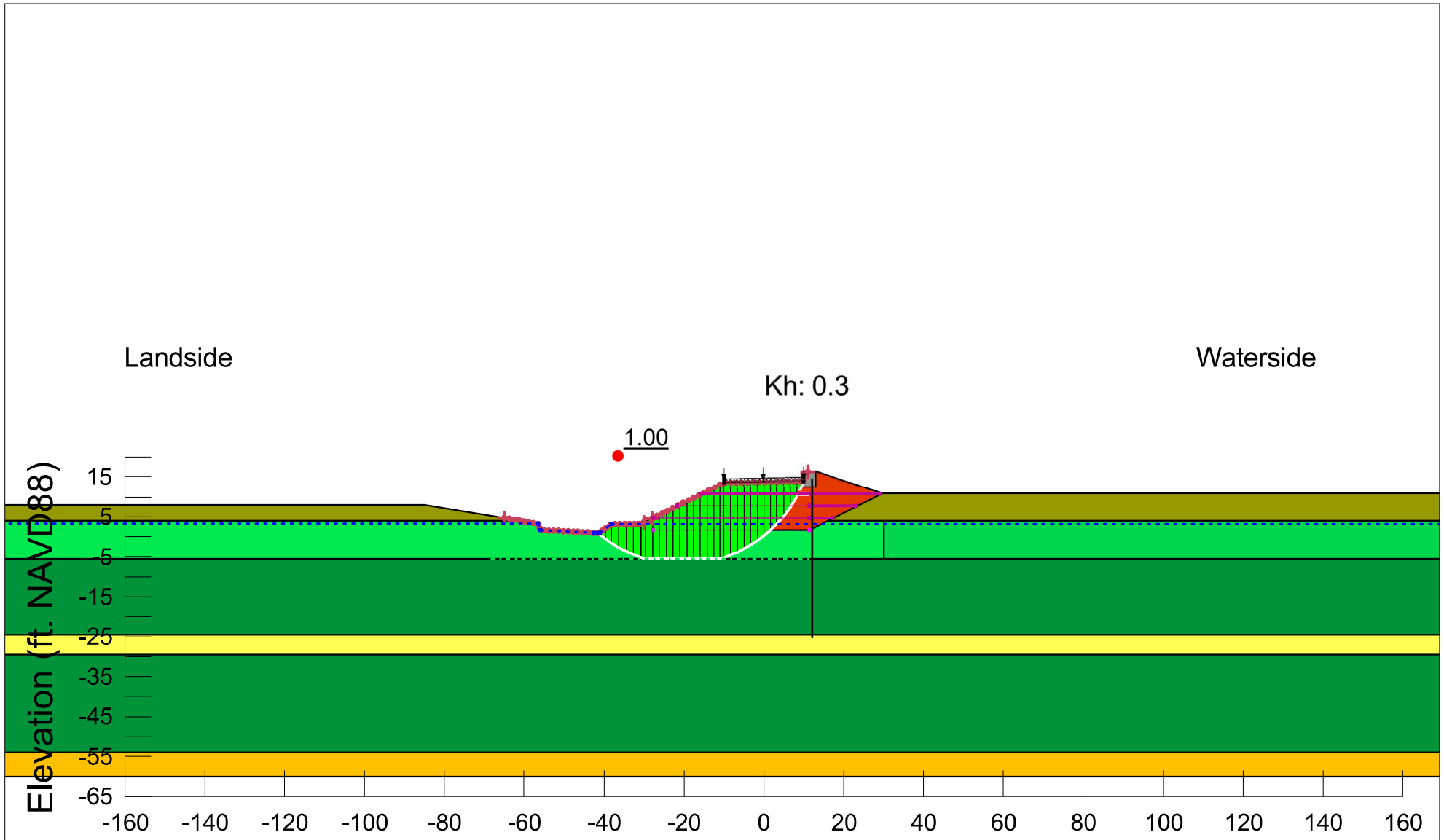


CROSS SECTION ACS2 (STATION 27+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-22



CROSS SECTION ACS2 (STATION 27+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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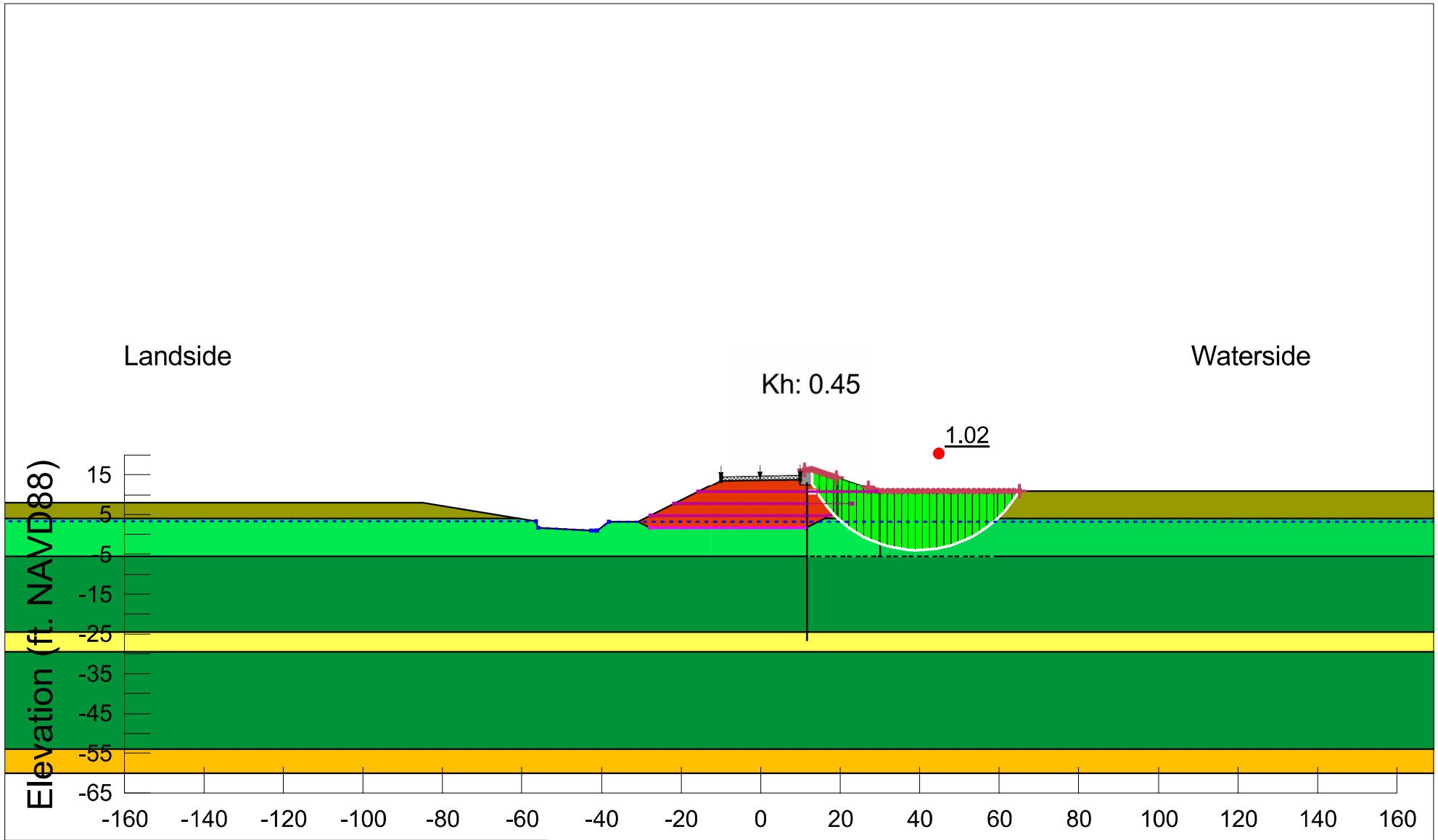
FIGURE B8-23



CROSS SECTION ACS2 (STATION 27+50)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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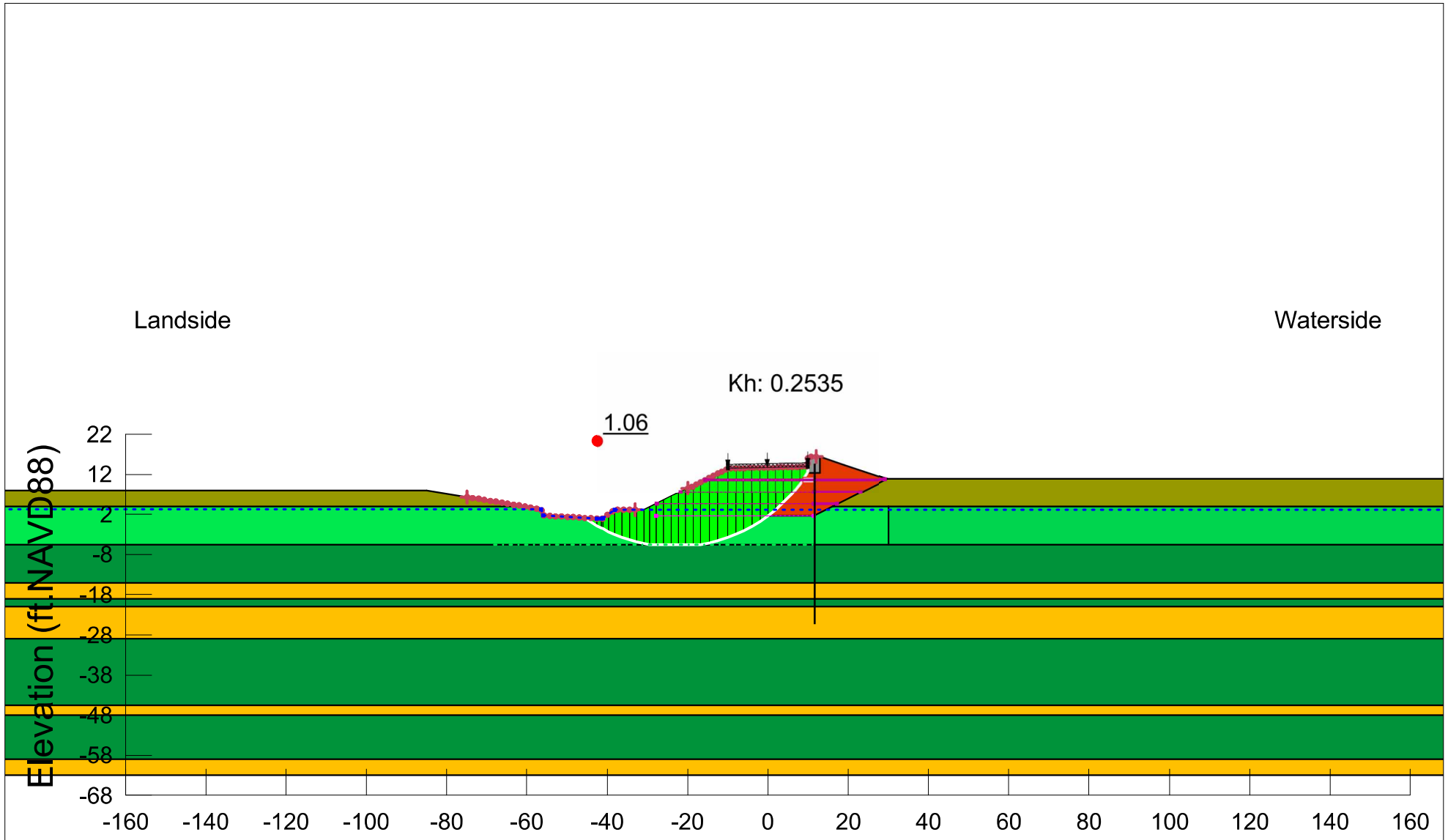
FIGURE B8-24



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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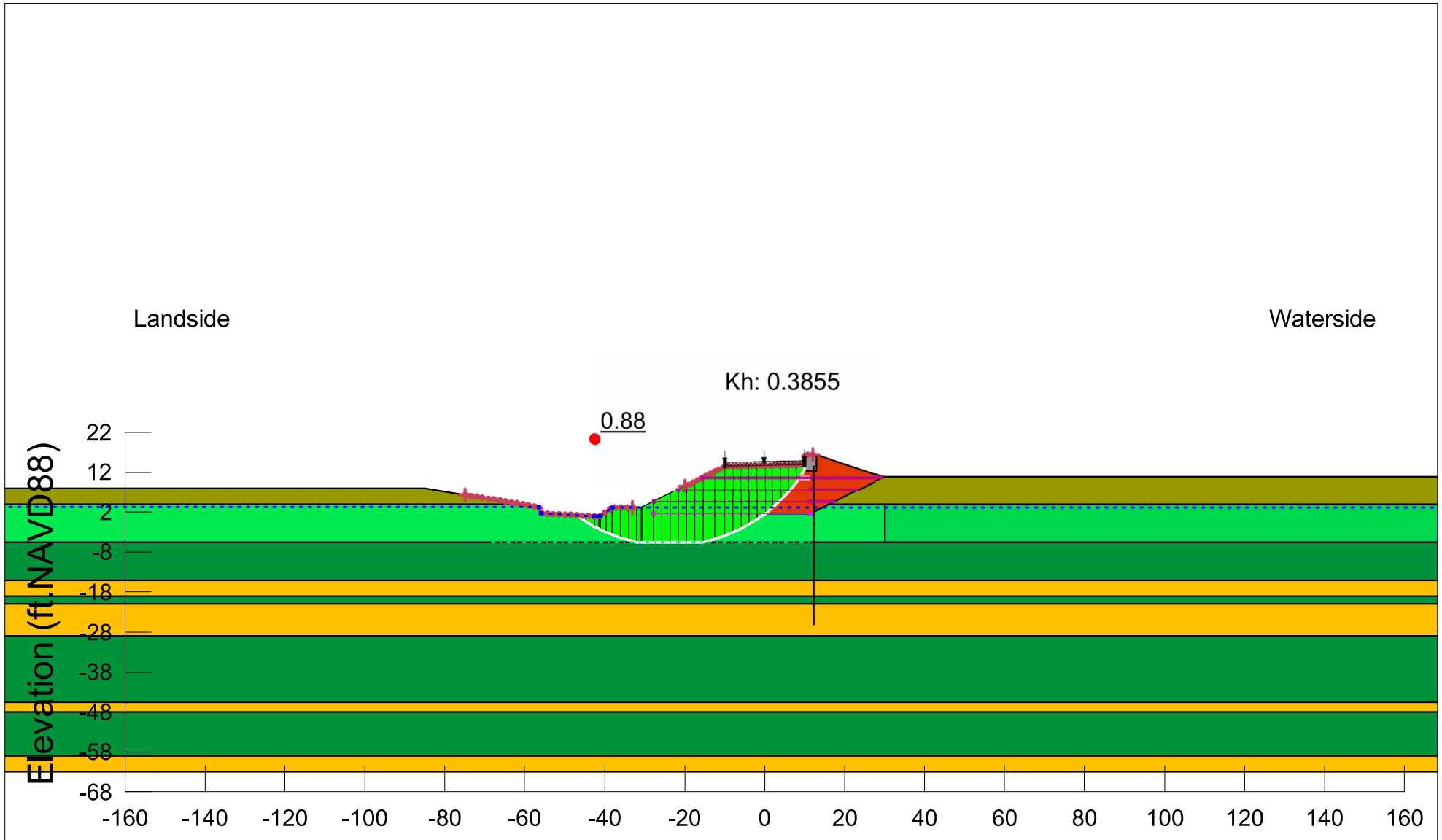
FIGURE B8-25



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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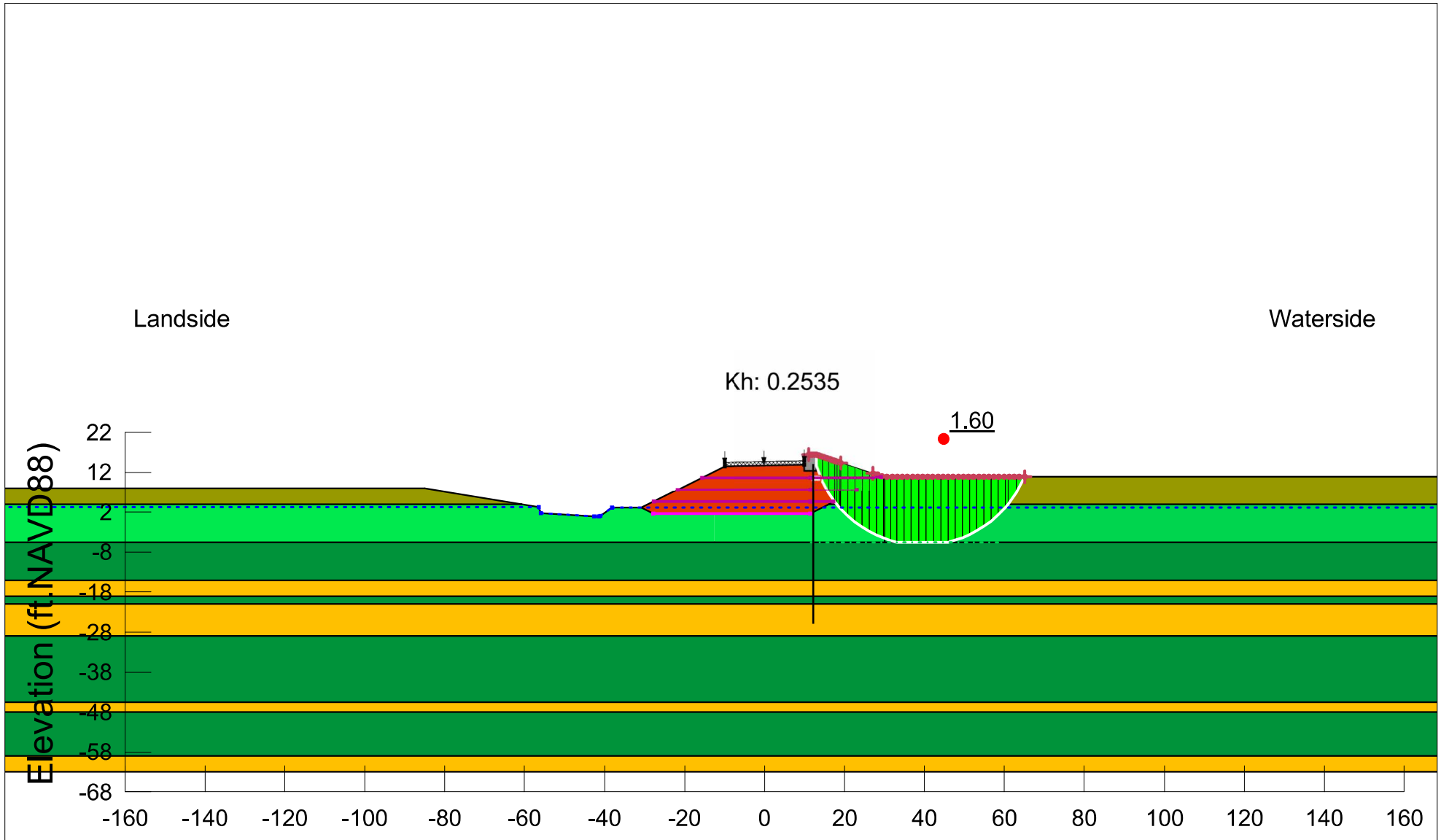
FIGURE B8-26



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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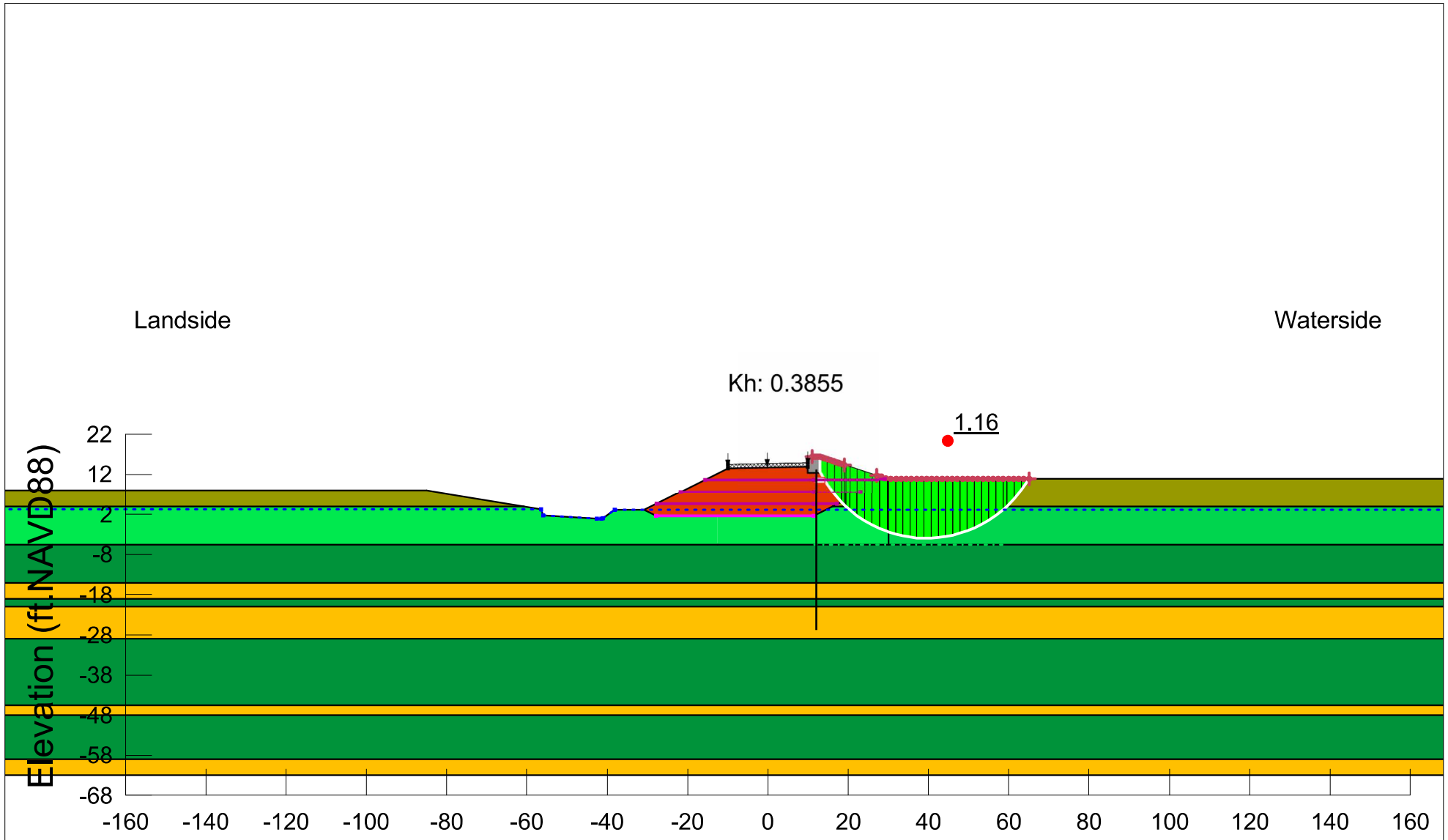
FIGURE B8-27



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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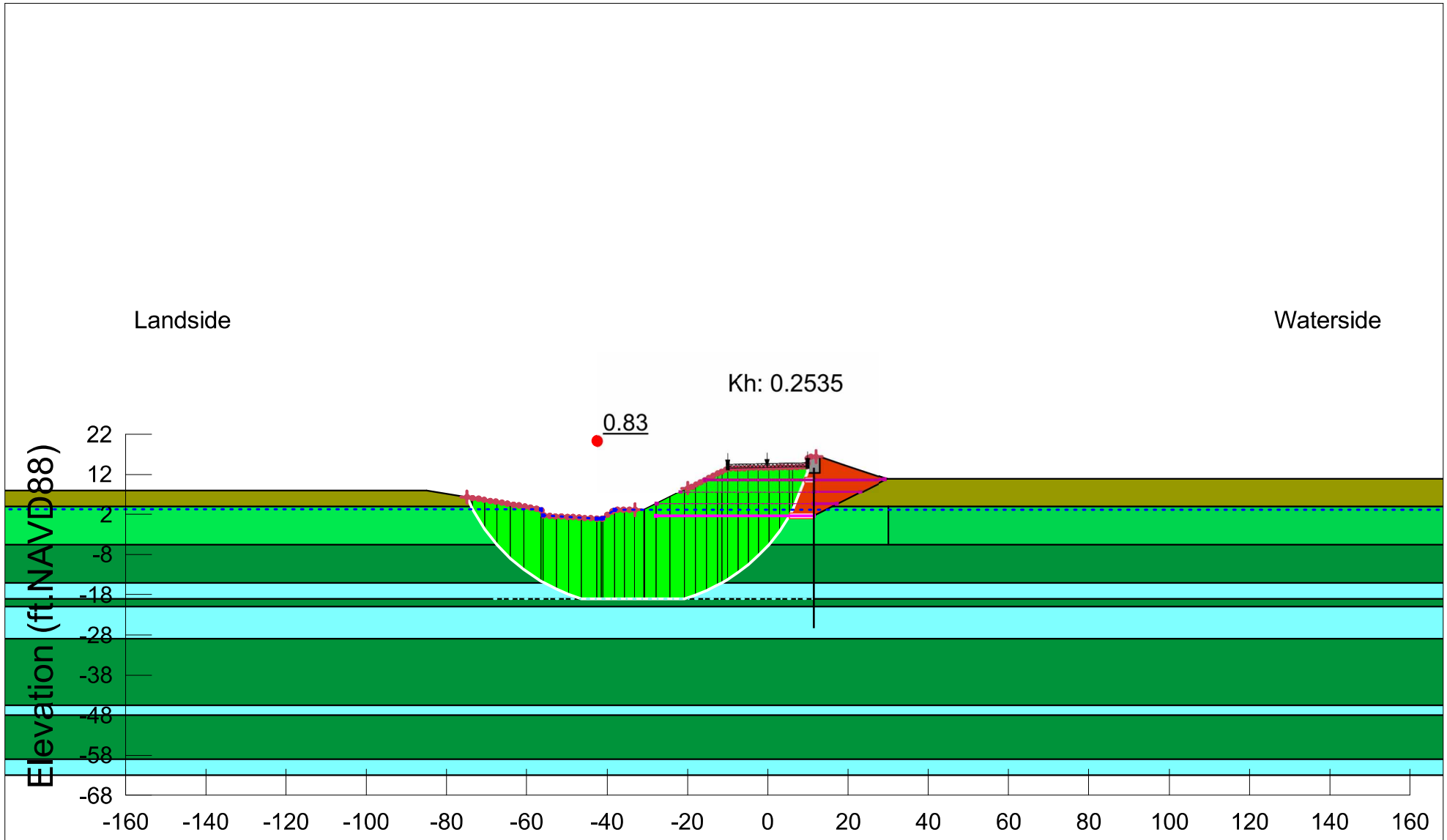
FIGURE B8-28



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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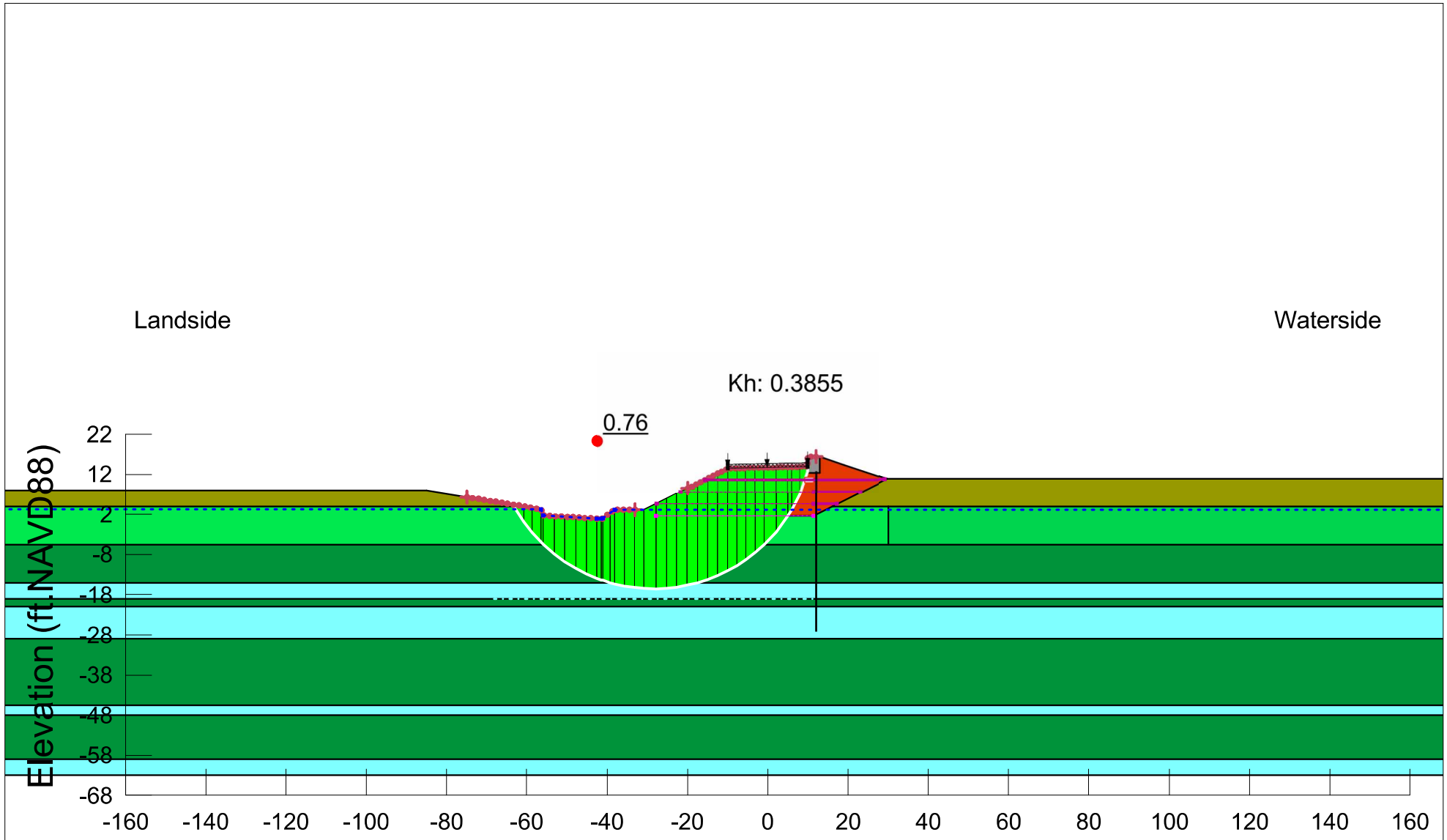
FIGURE B8-29



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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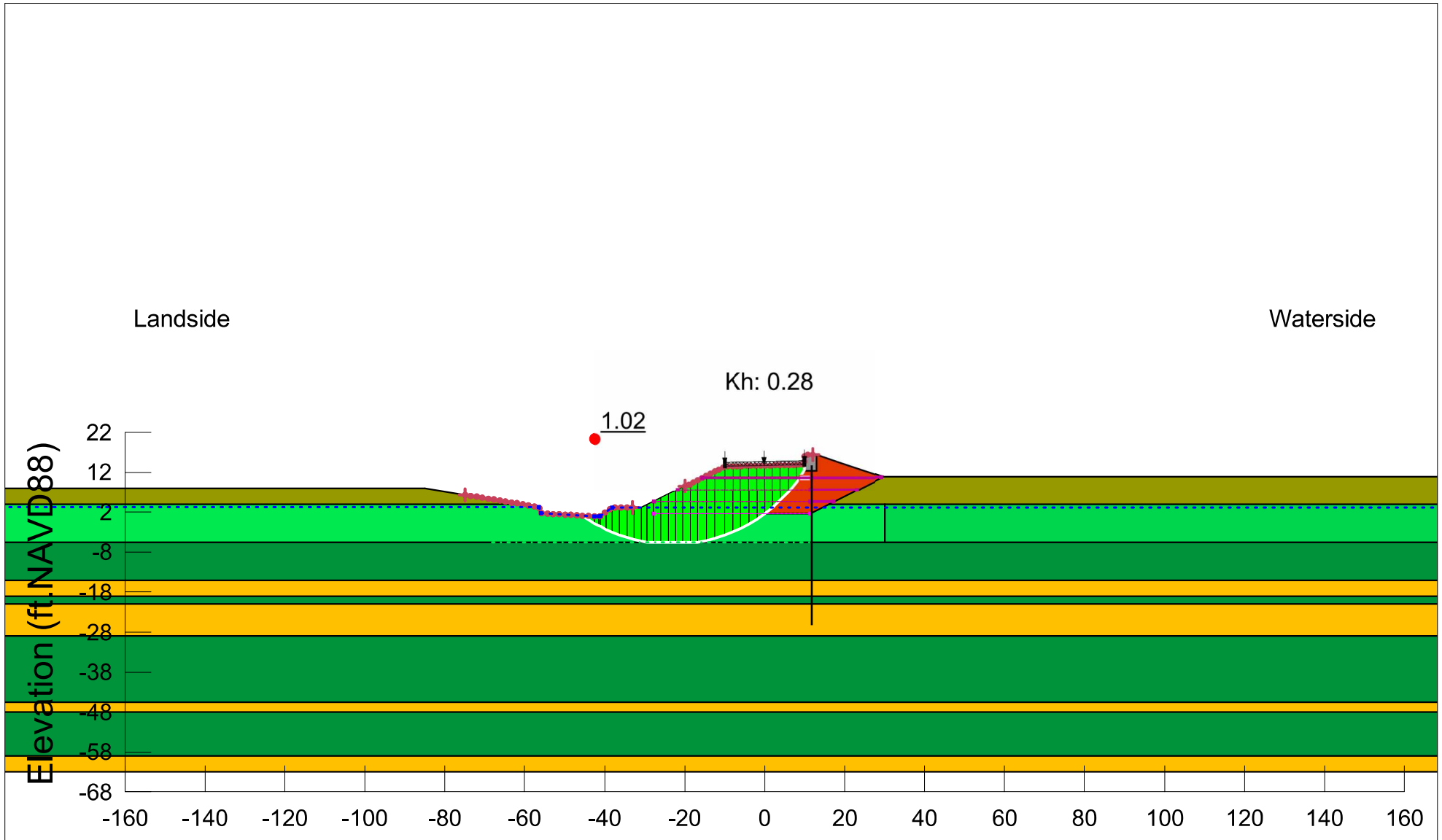
FIGURE B8-30



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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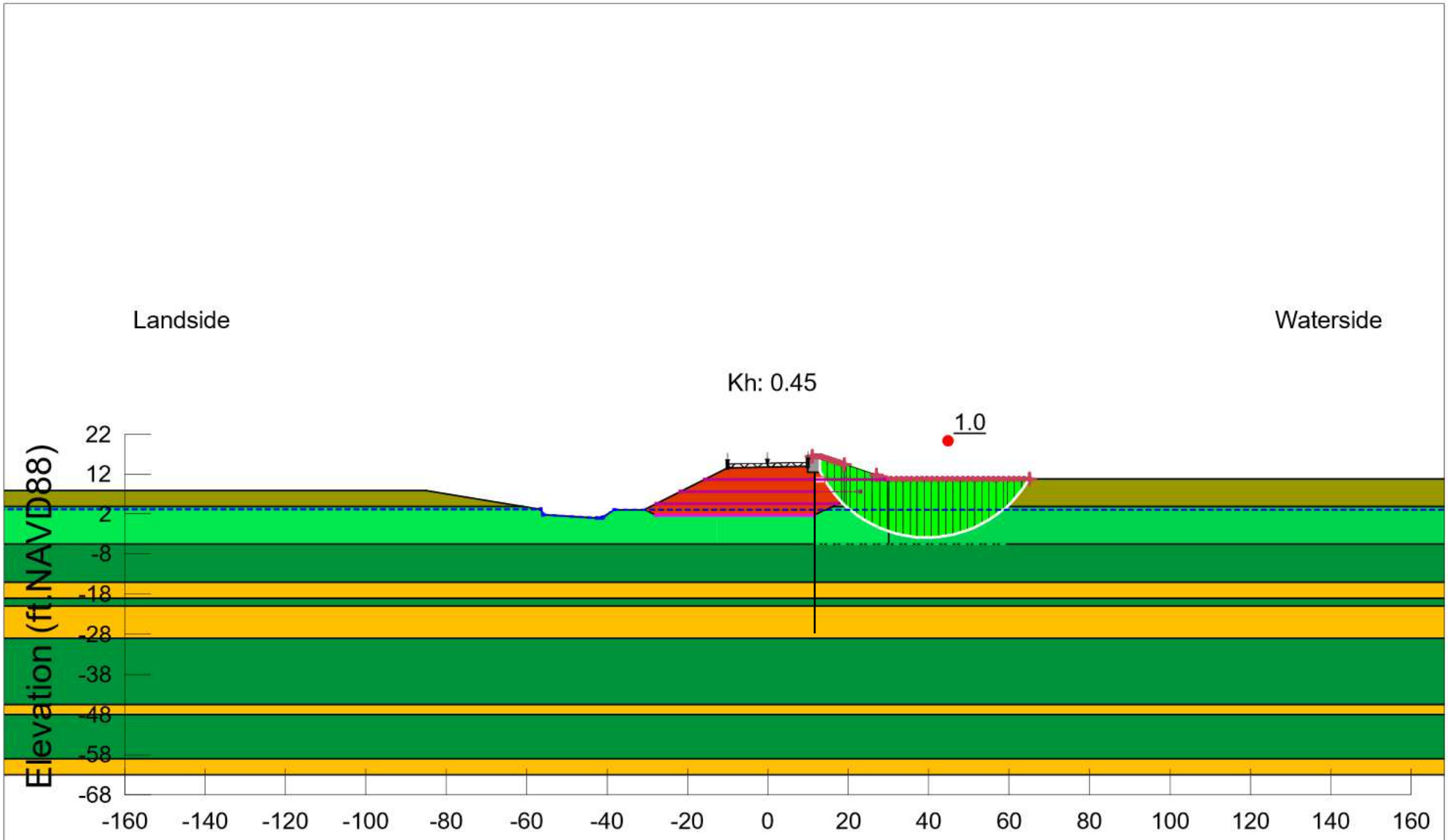
FIGURE B8-31



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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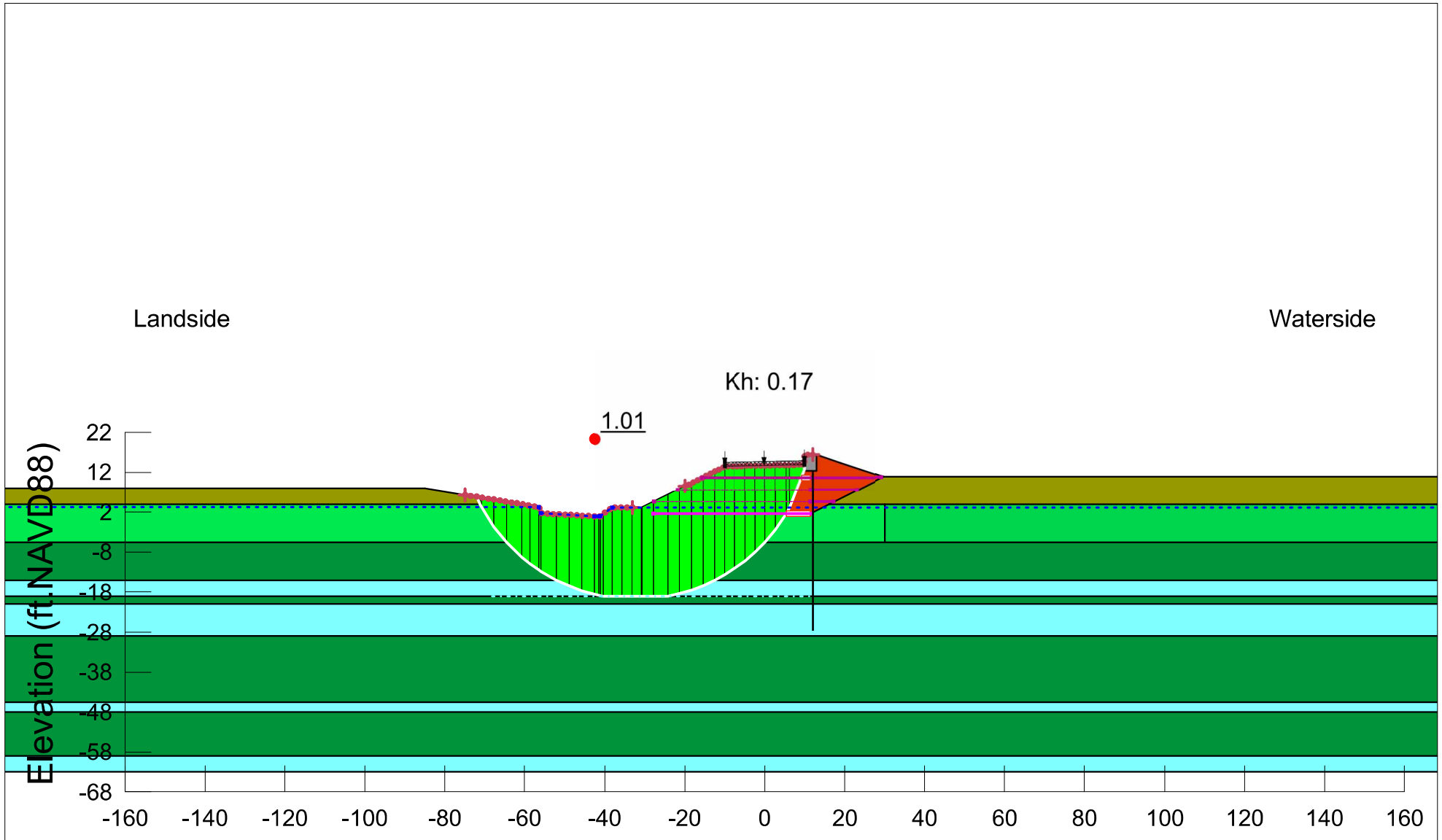
FIGURE B8-32



CROSS SECTION ACS3 (STATION 32+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - YIELD COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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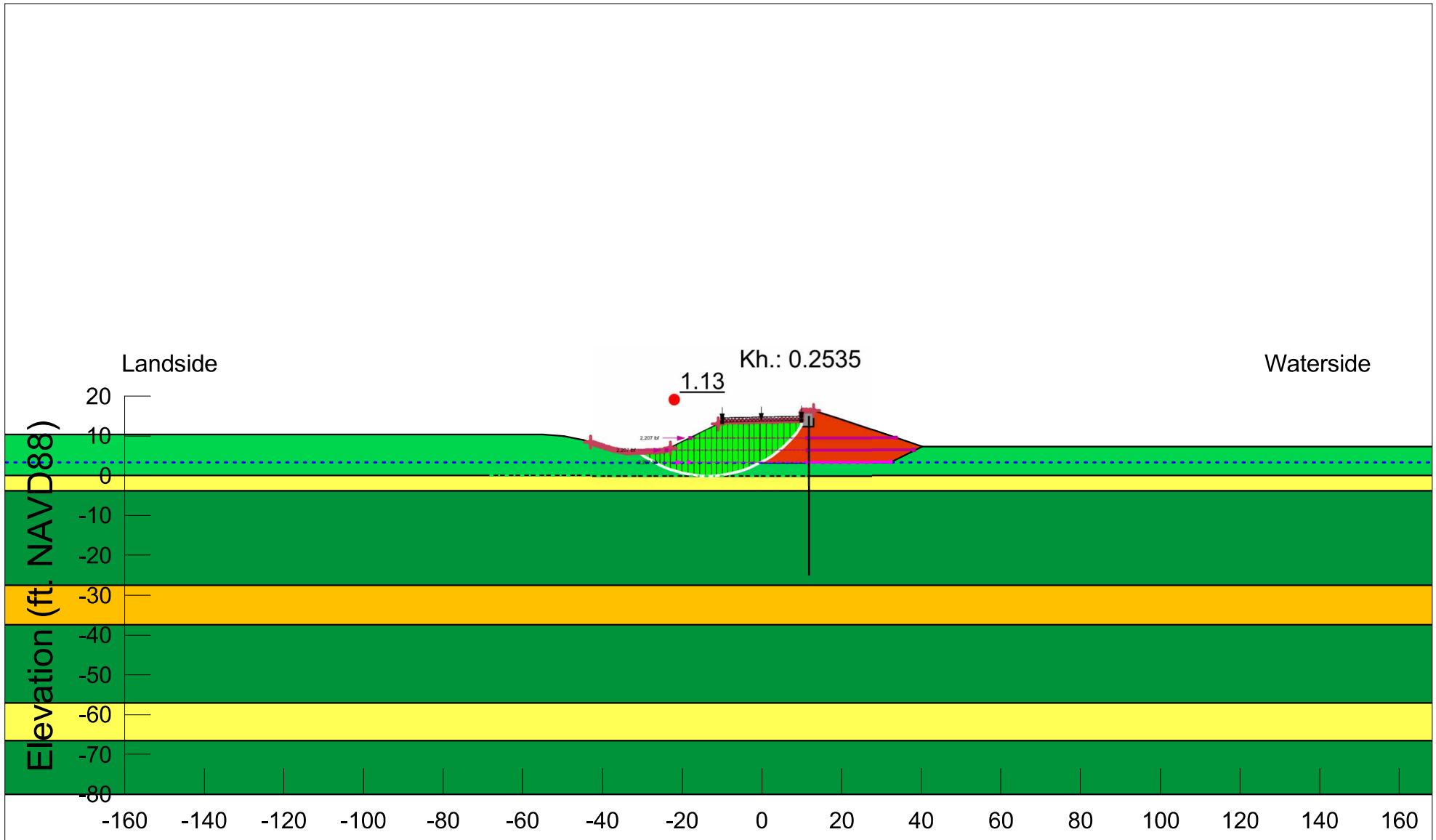
FIGURE B8-33



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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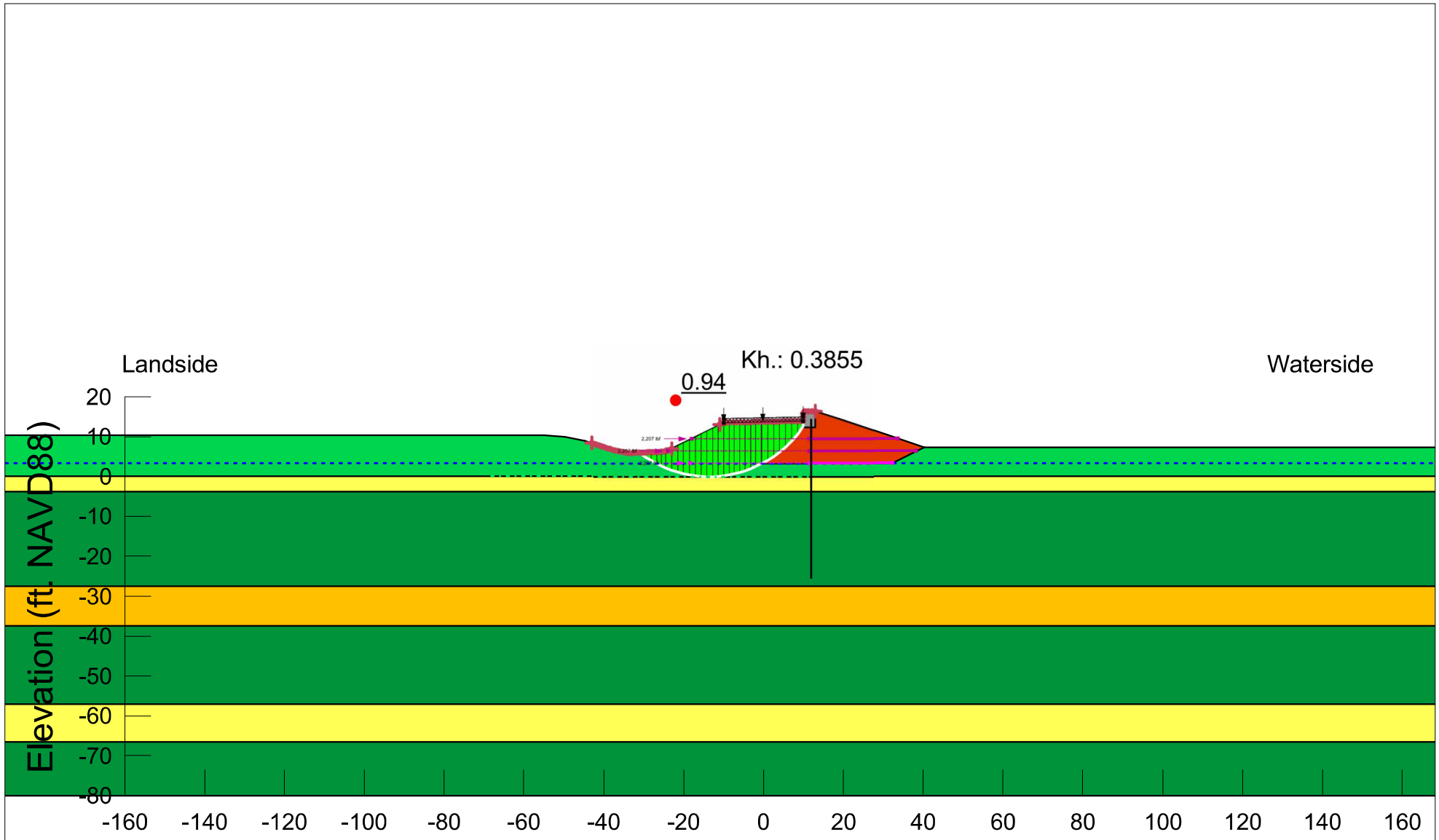
FIGURE B8-34



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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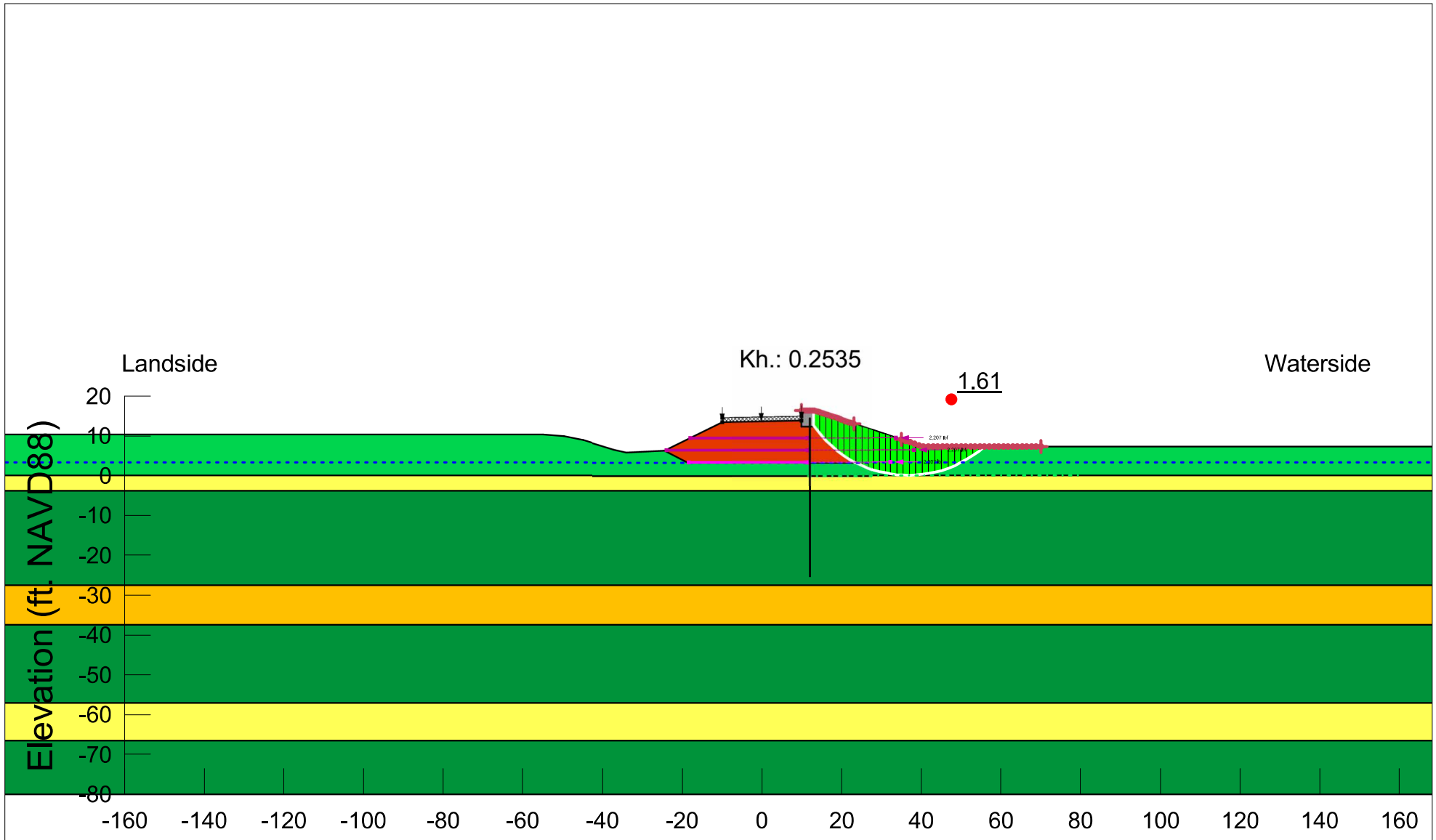
FIGURE B8-35



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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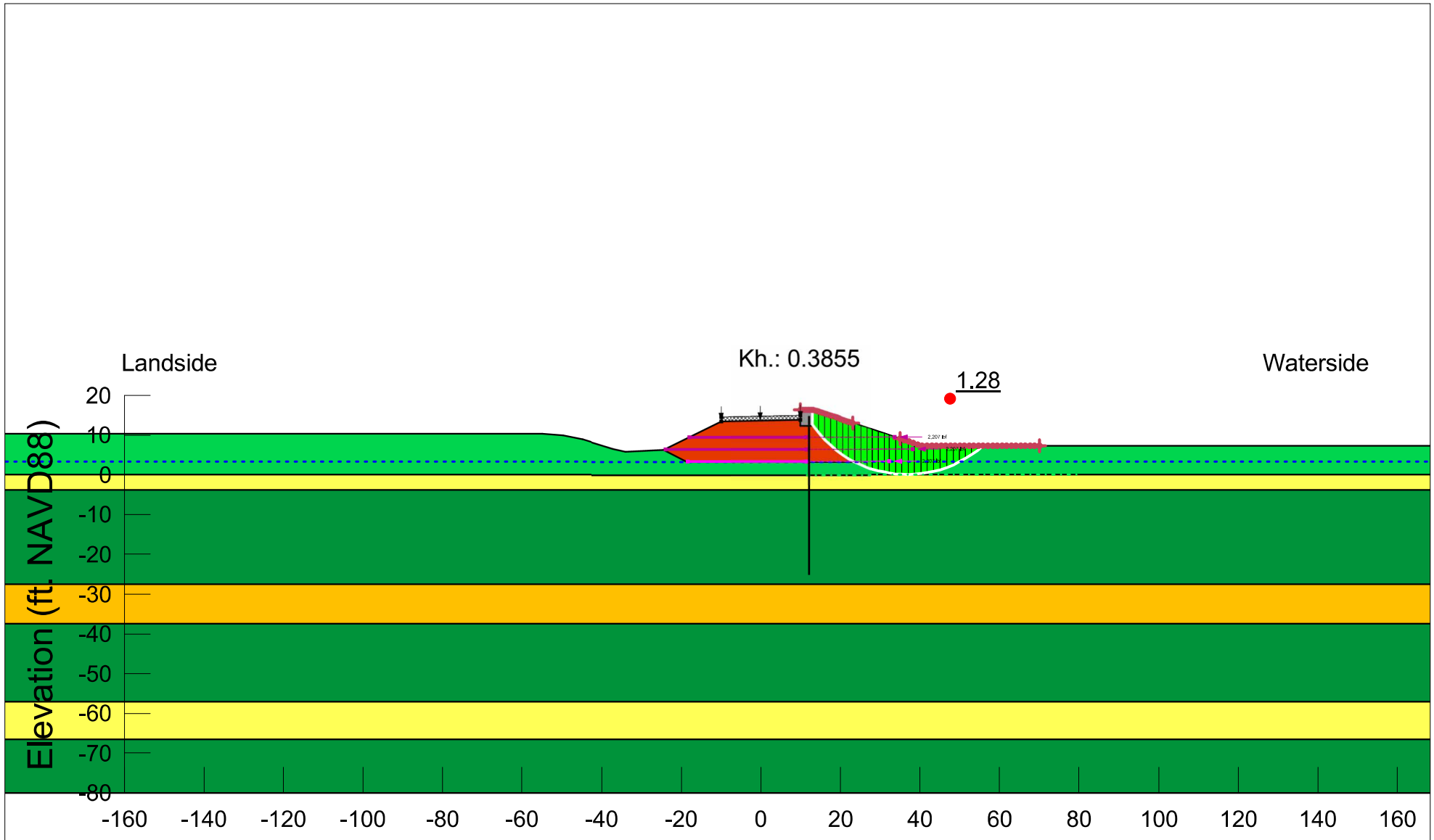
FIGURE B8-36



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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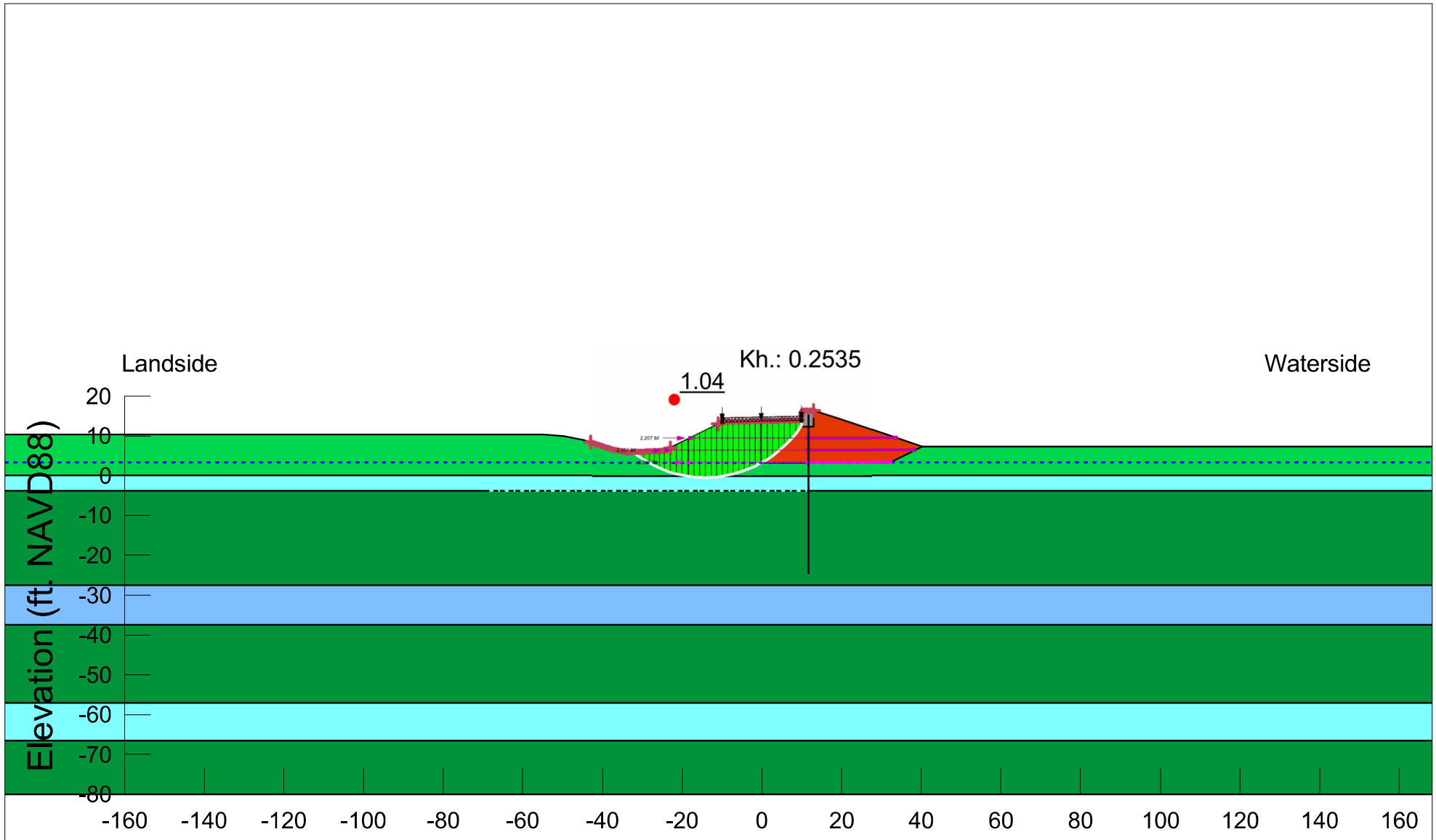
FIGURE B8-37



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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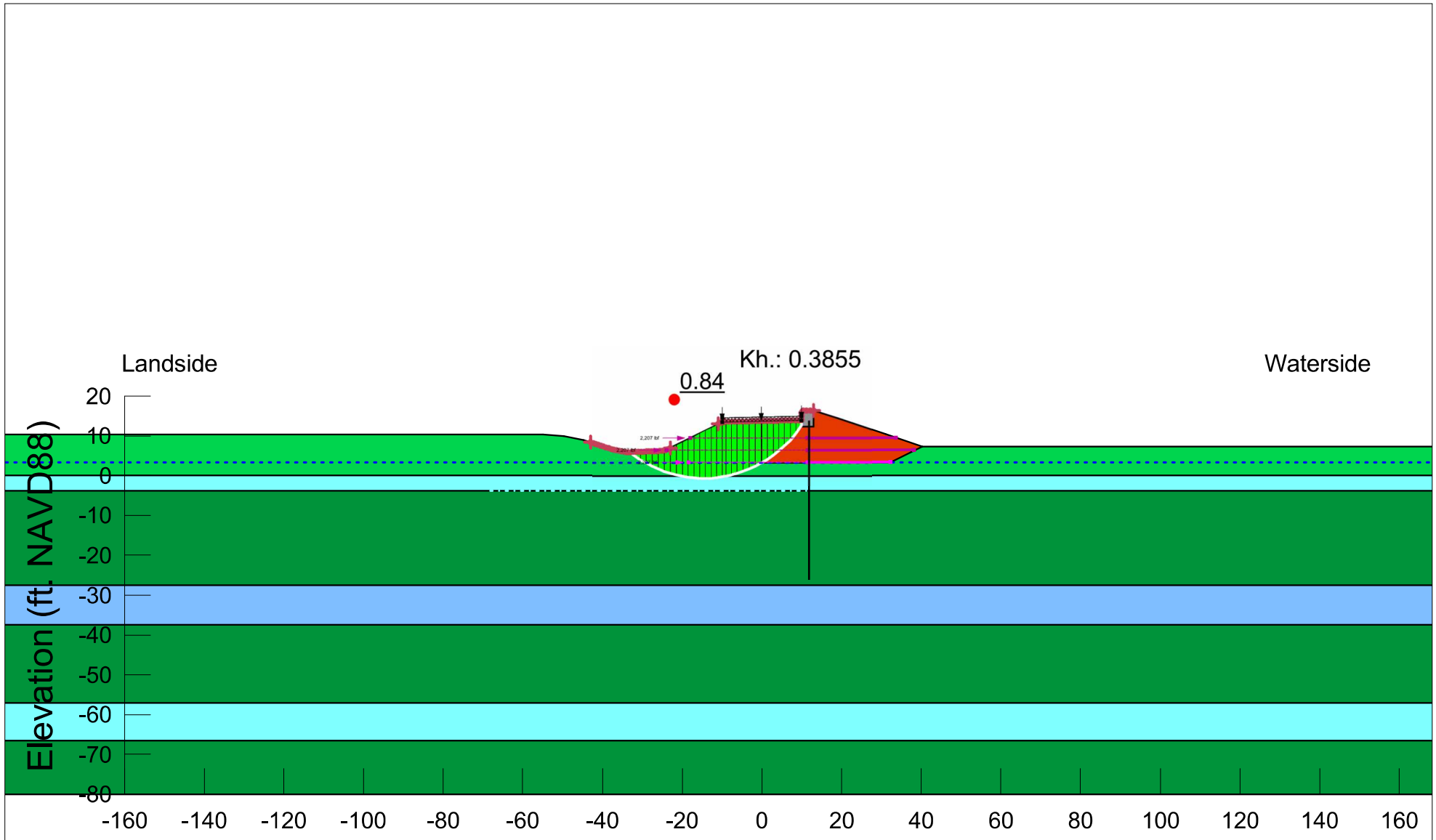
FIGURE B8-38



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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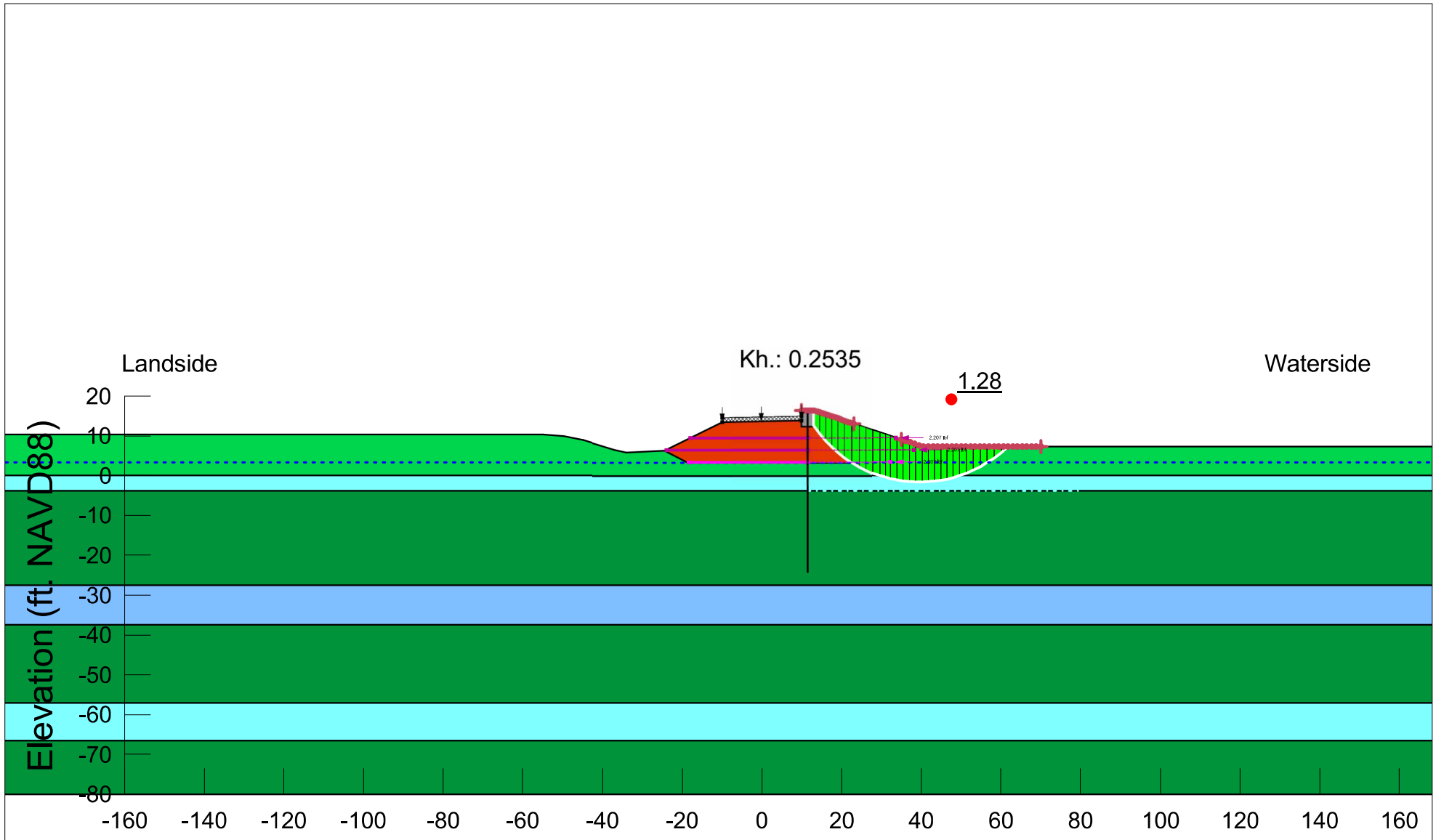
FIGURE B8-39



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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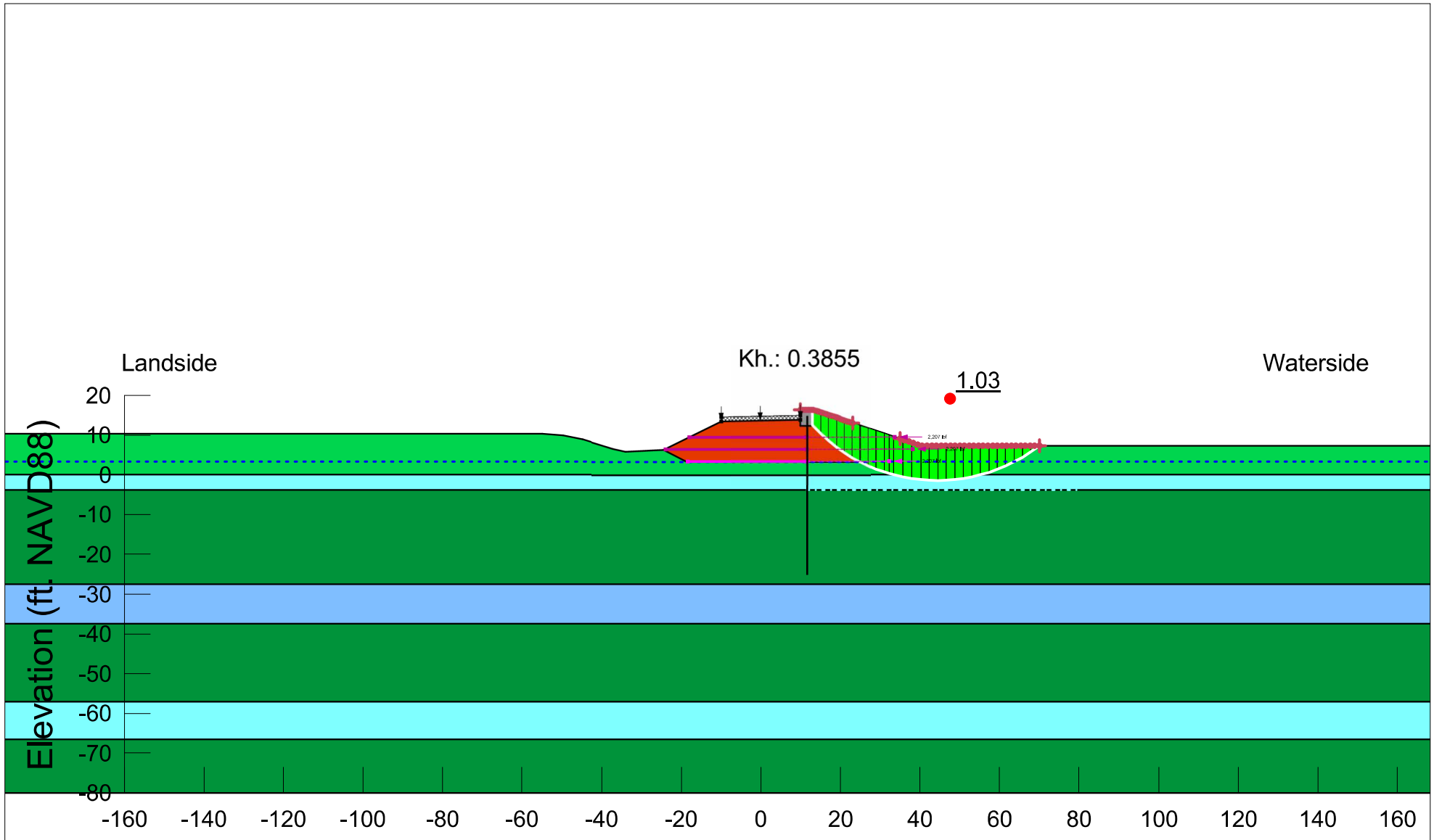
FIGURE B8-40



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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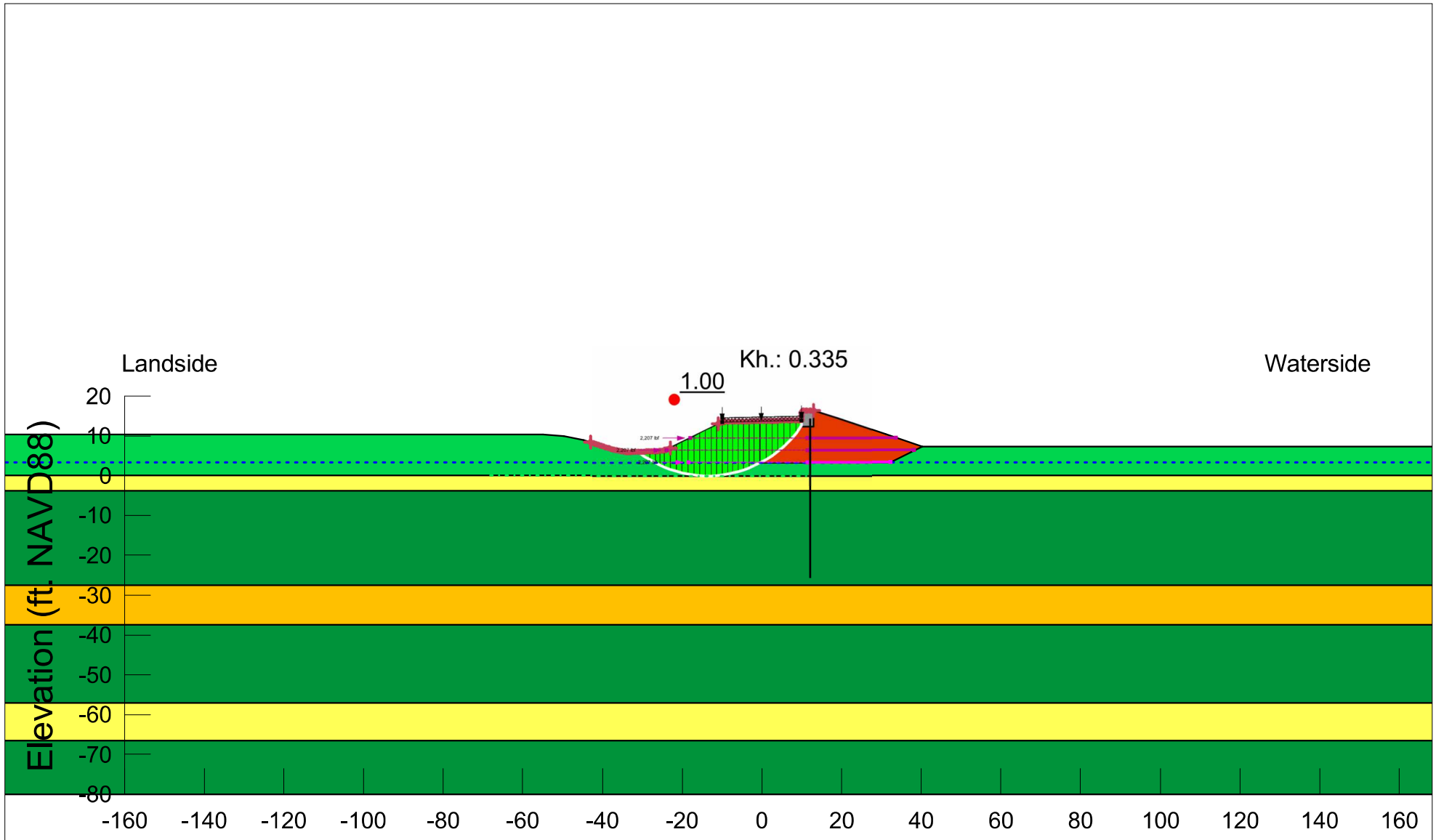
FIGURE B8-41



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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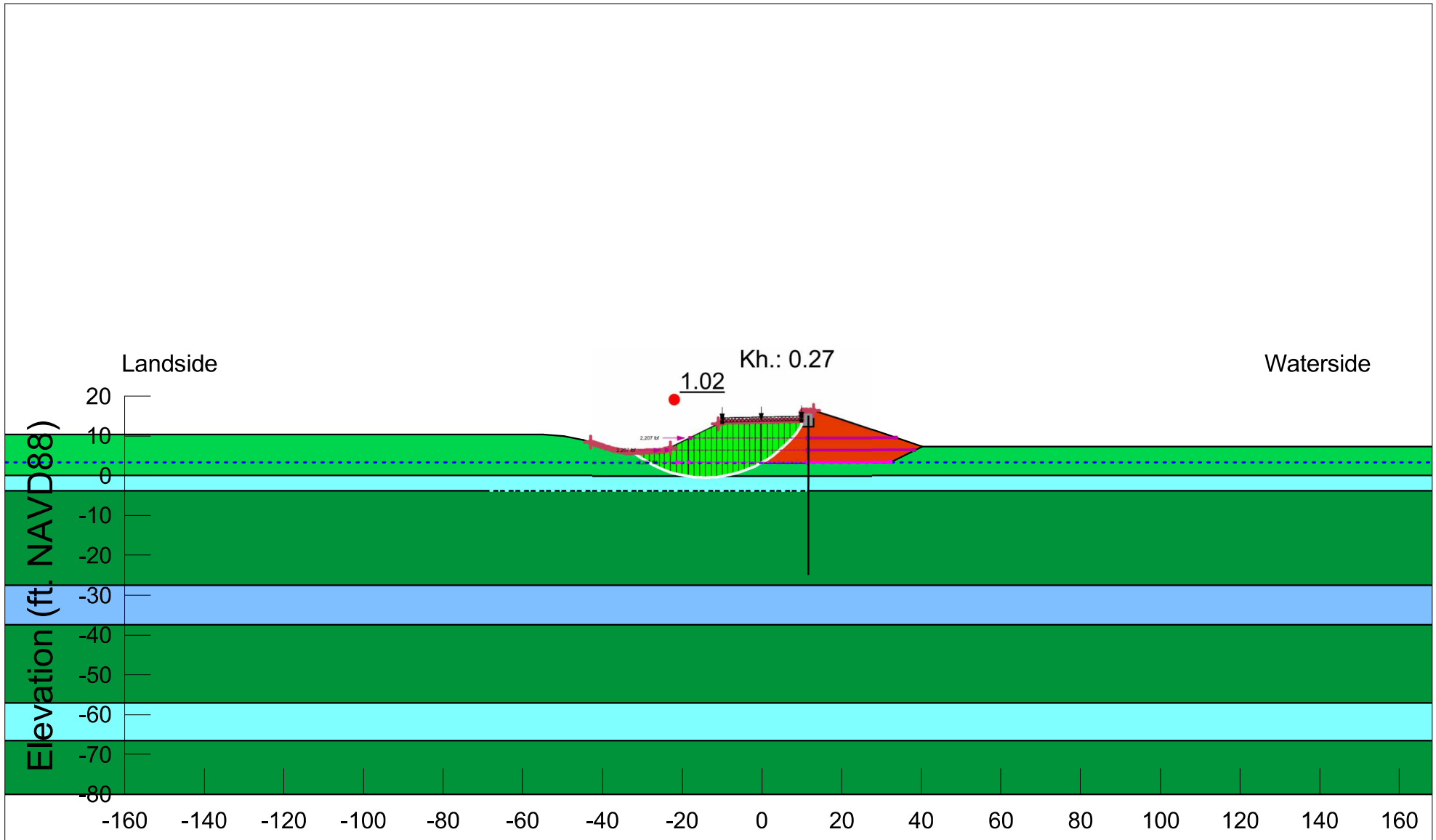
FIGURE B8-42



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - YIELD COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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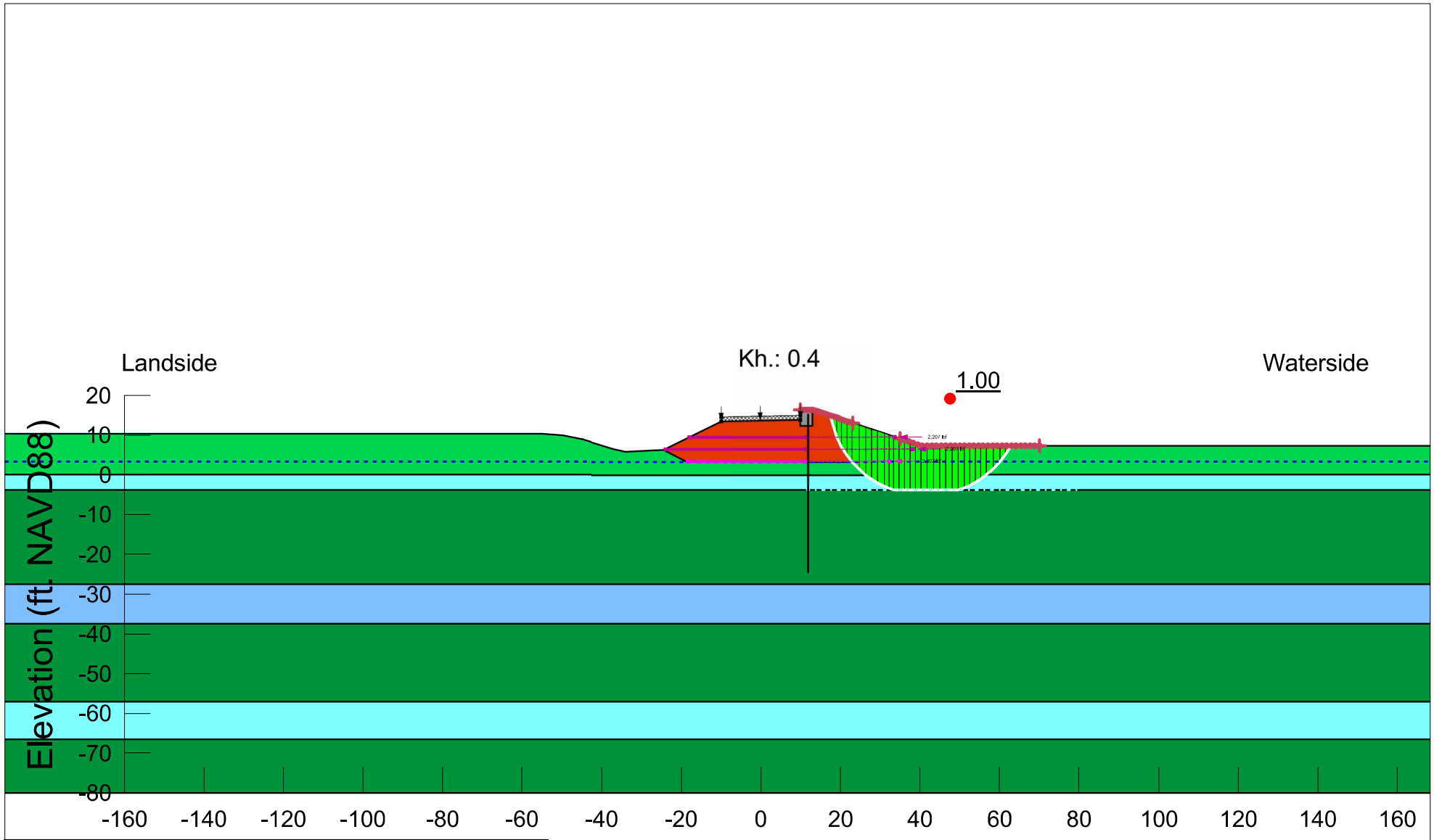
FIGURE B8-43



CROSS SECTION ACS4 (STATION 38+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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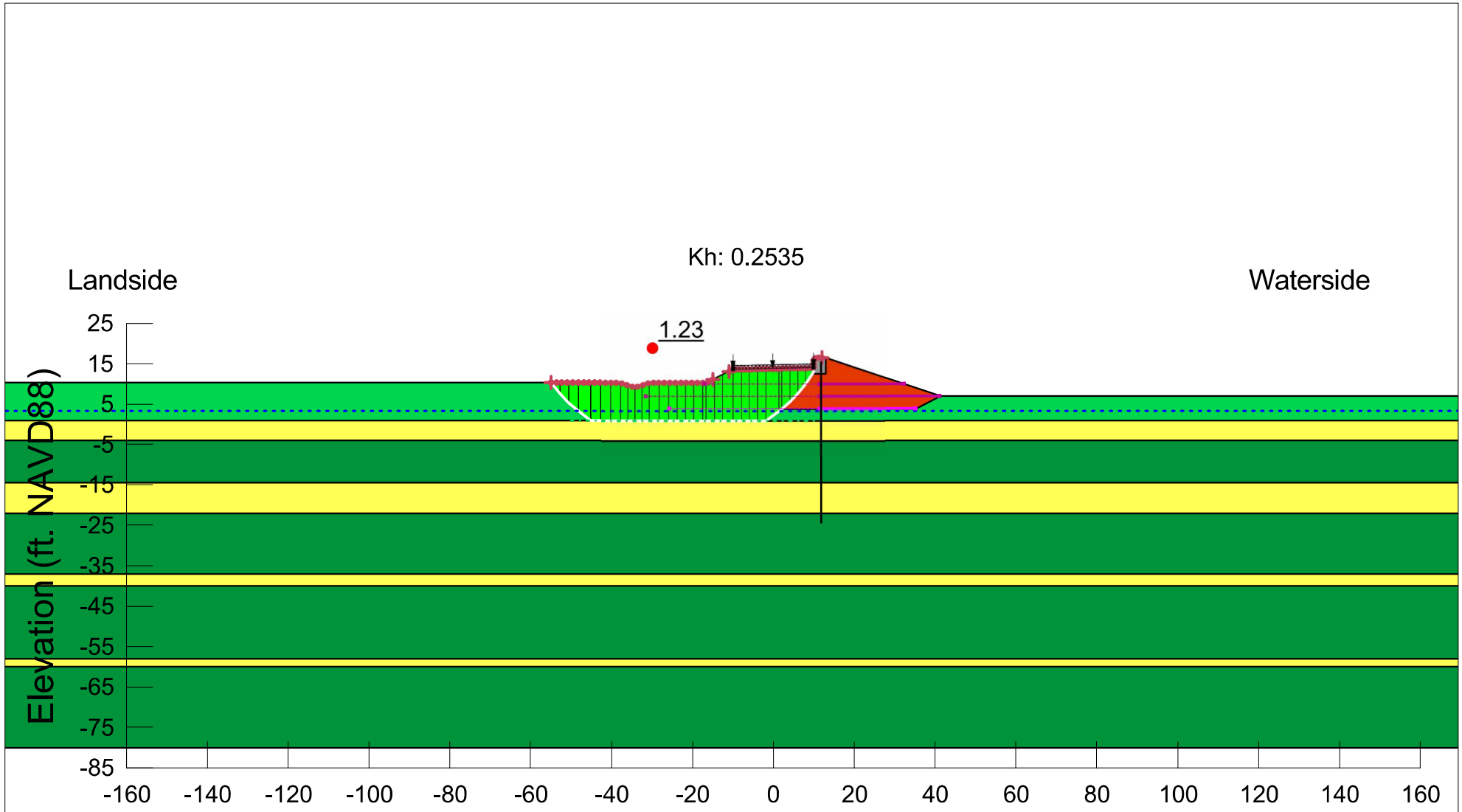
FIGURE B8-44

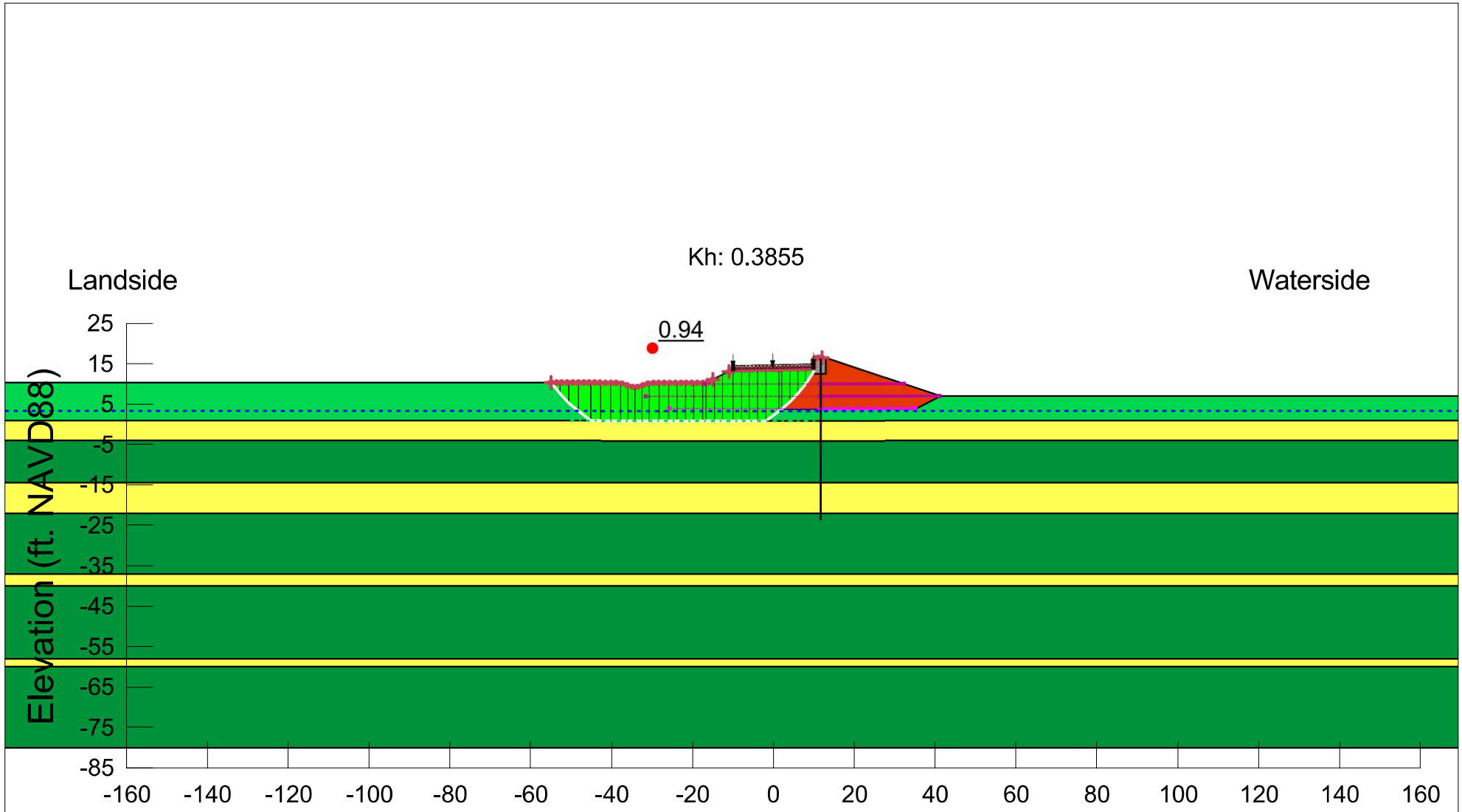


CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-45



CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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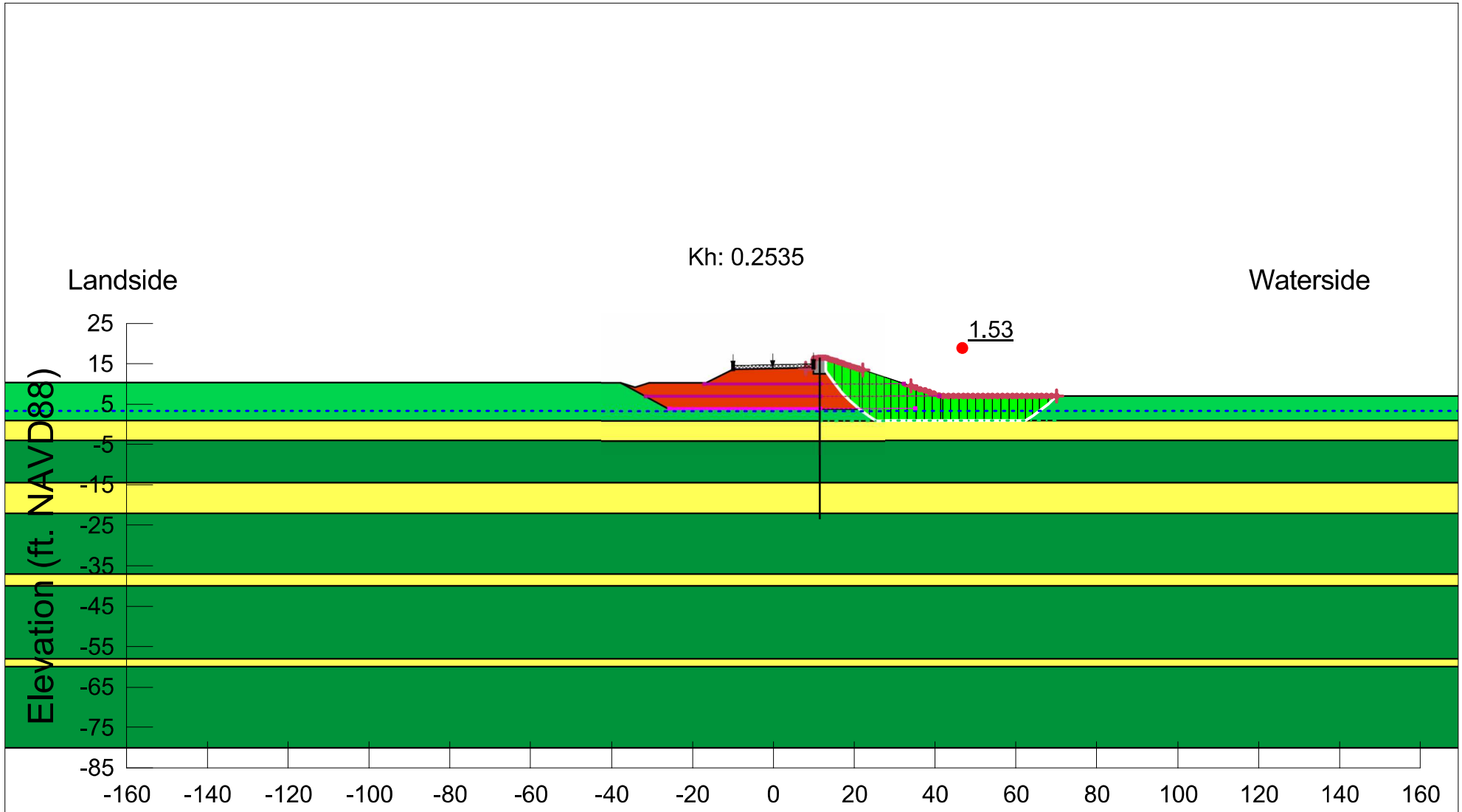
FIGURE B8-46



CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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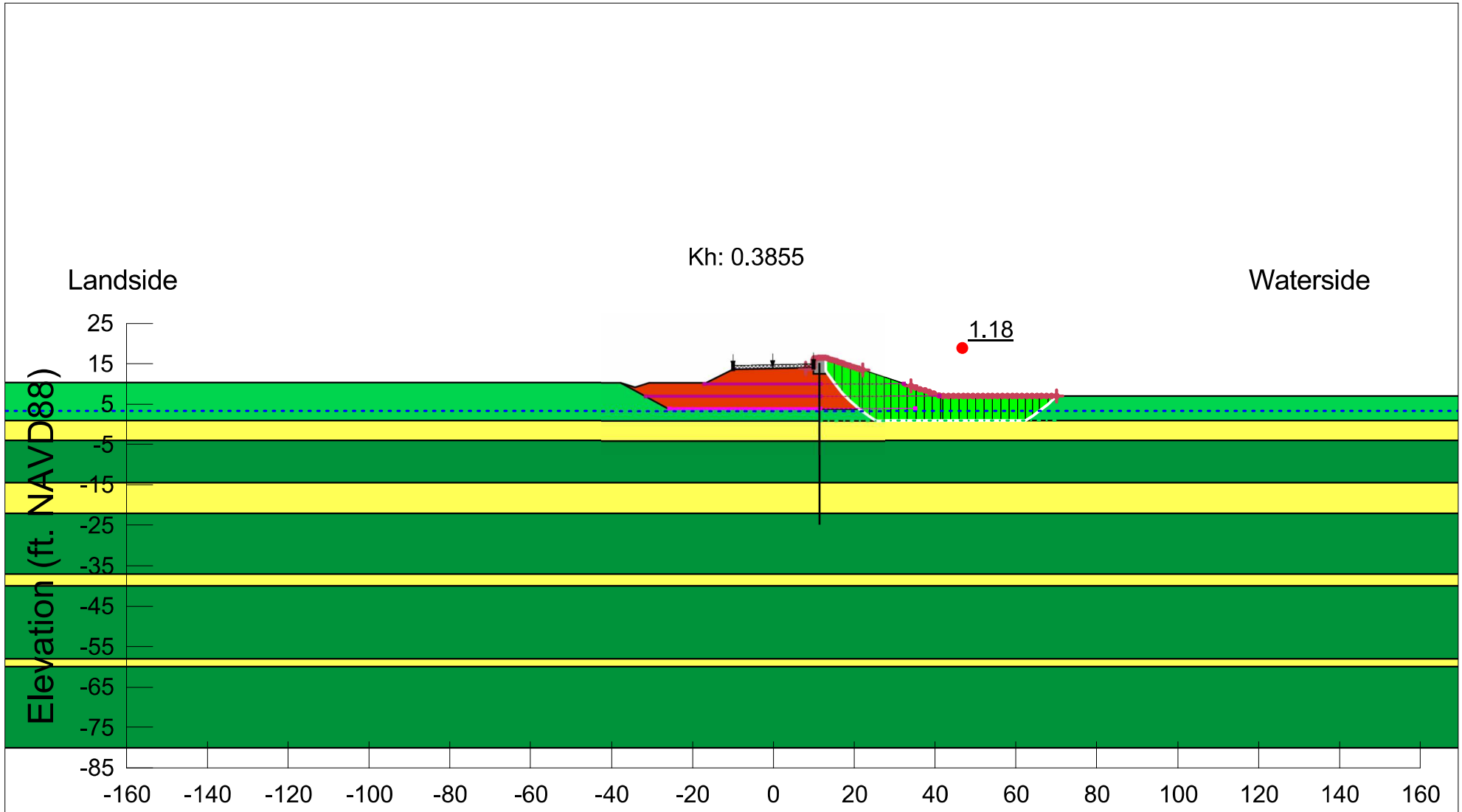
FIGURE B8-47



CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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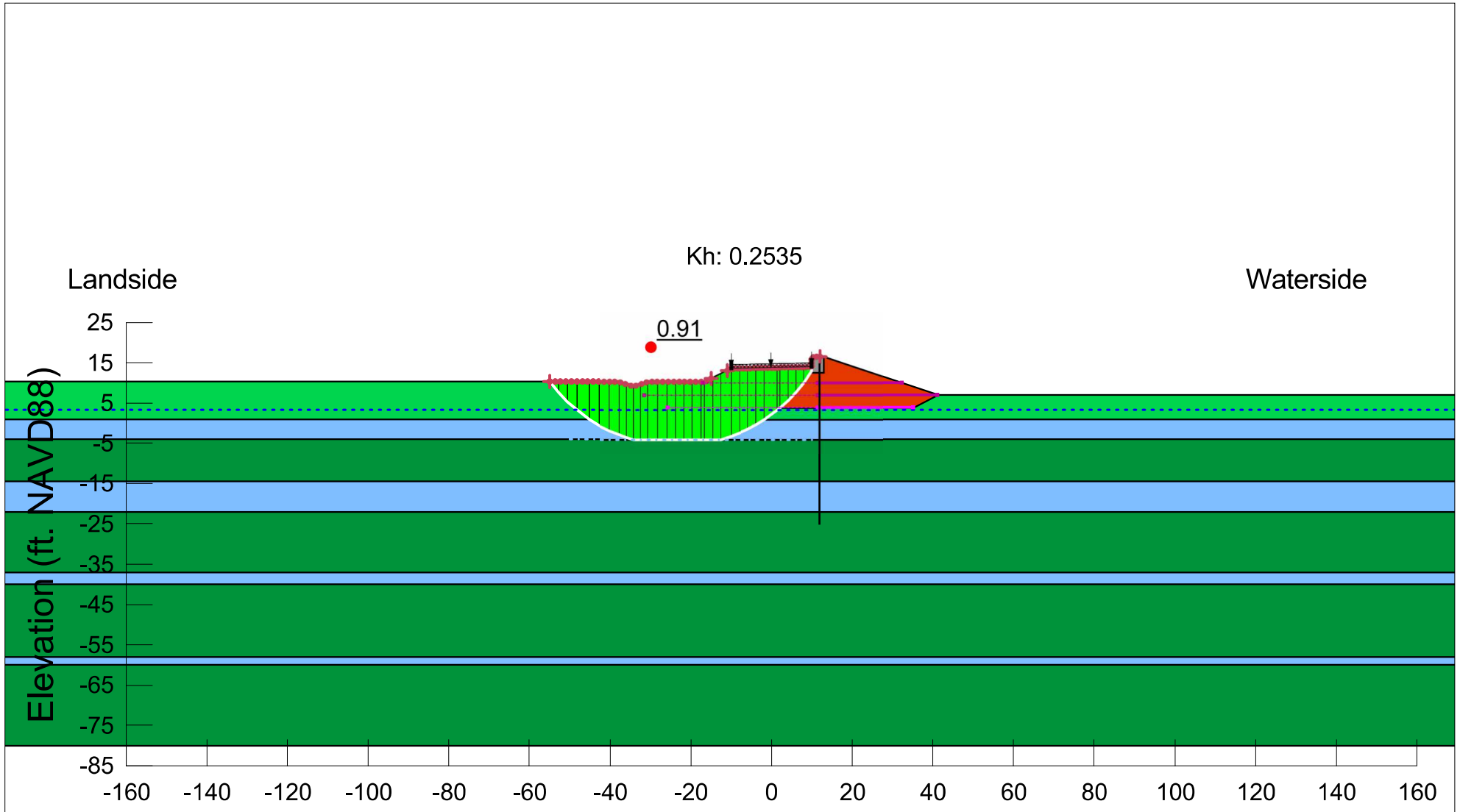
FIGURE B8-48



CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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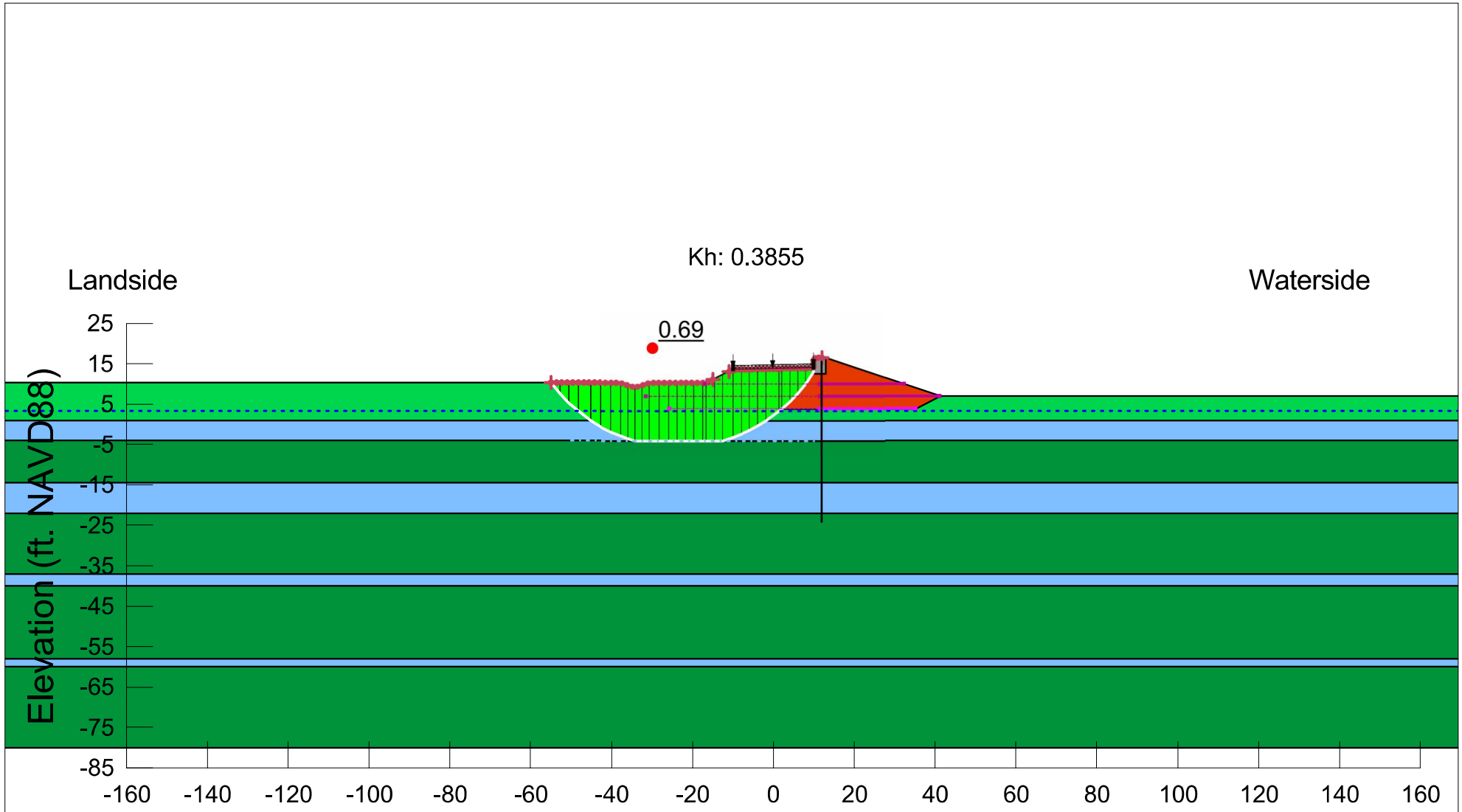
FIGURE B8-49



CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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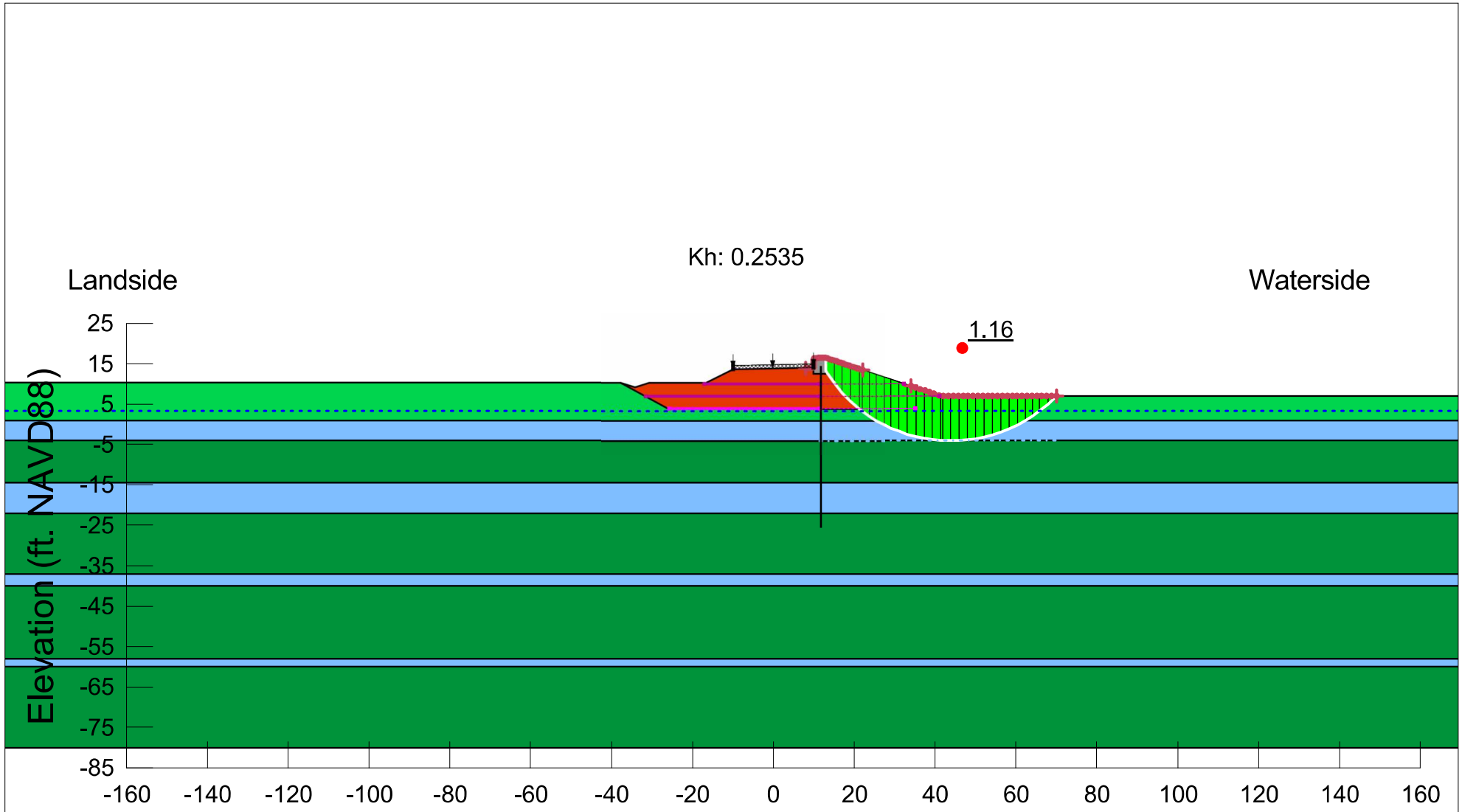
FIGURE B8-50



CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - OBE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

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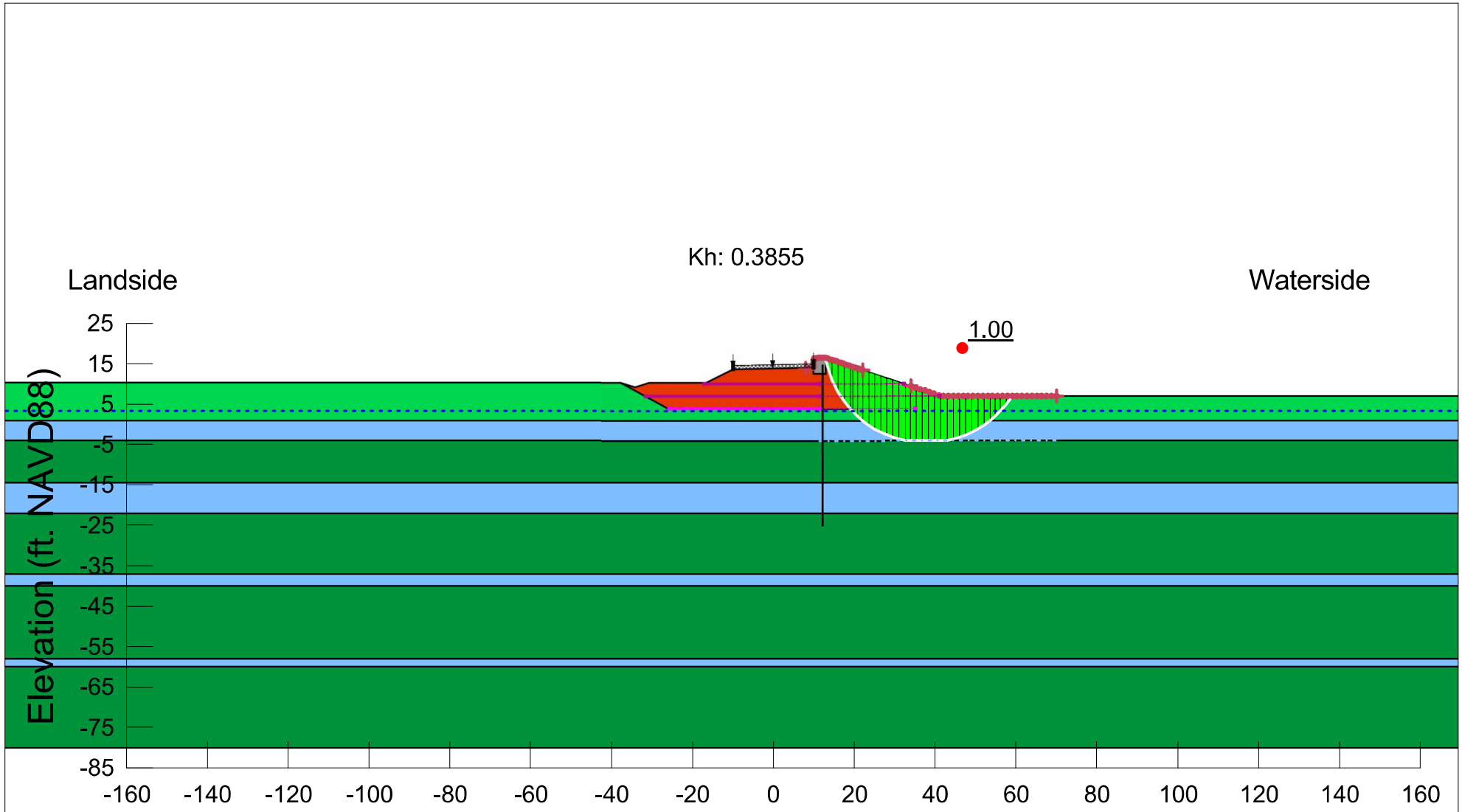
FIGURE B8-51

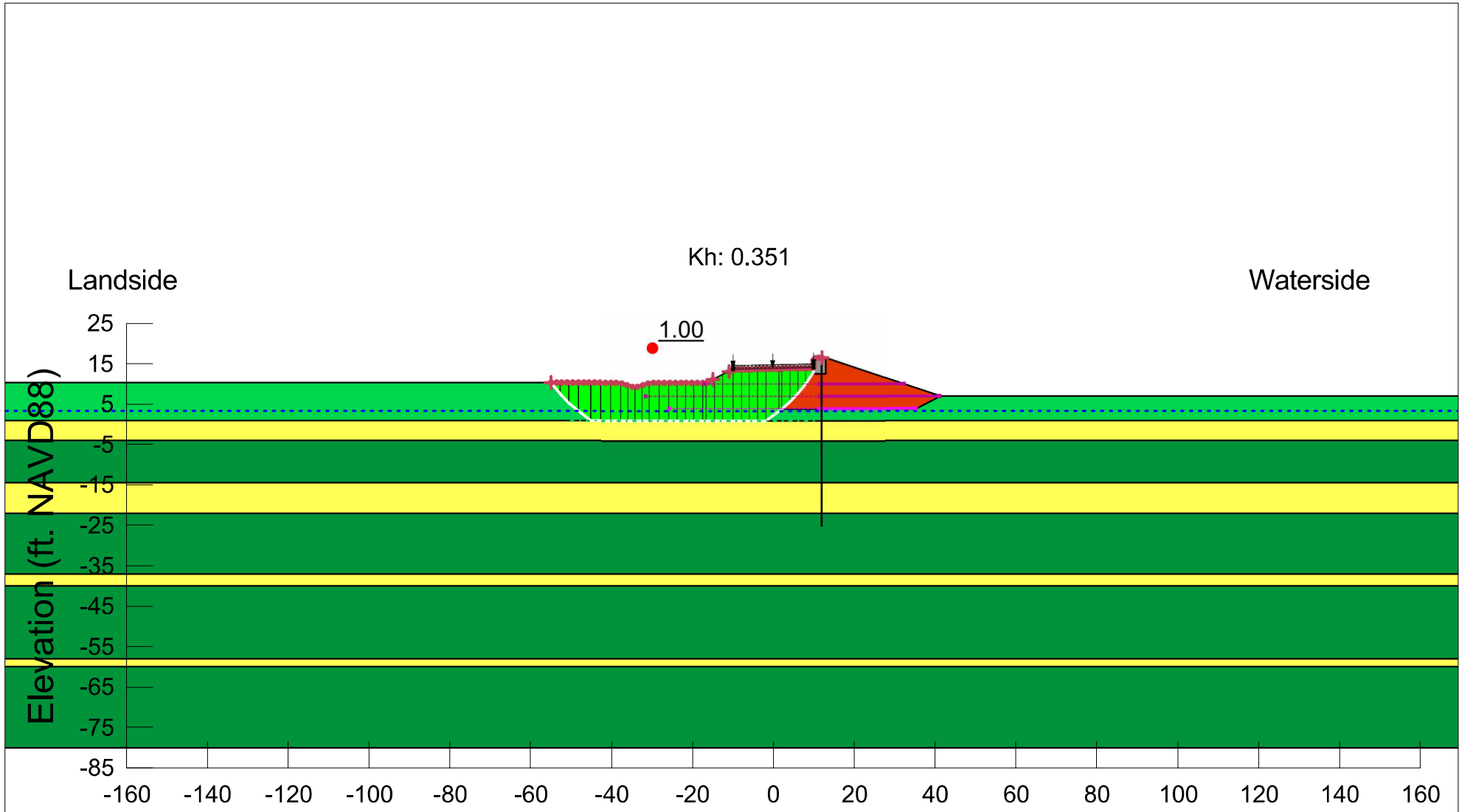


CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - MDE

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-52

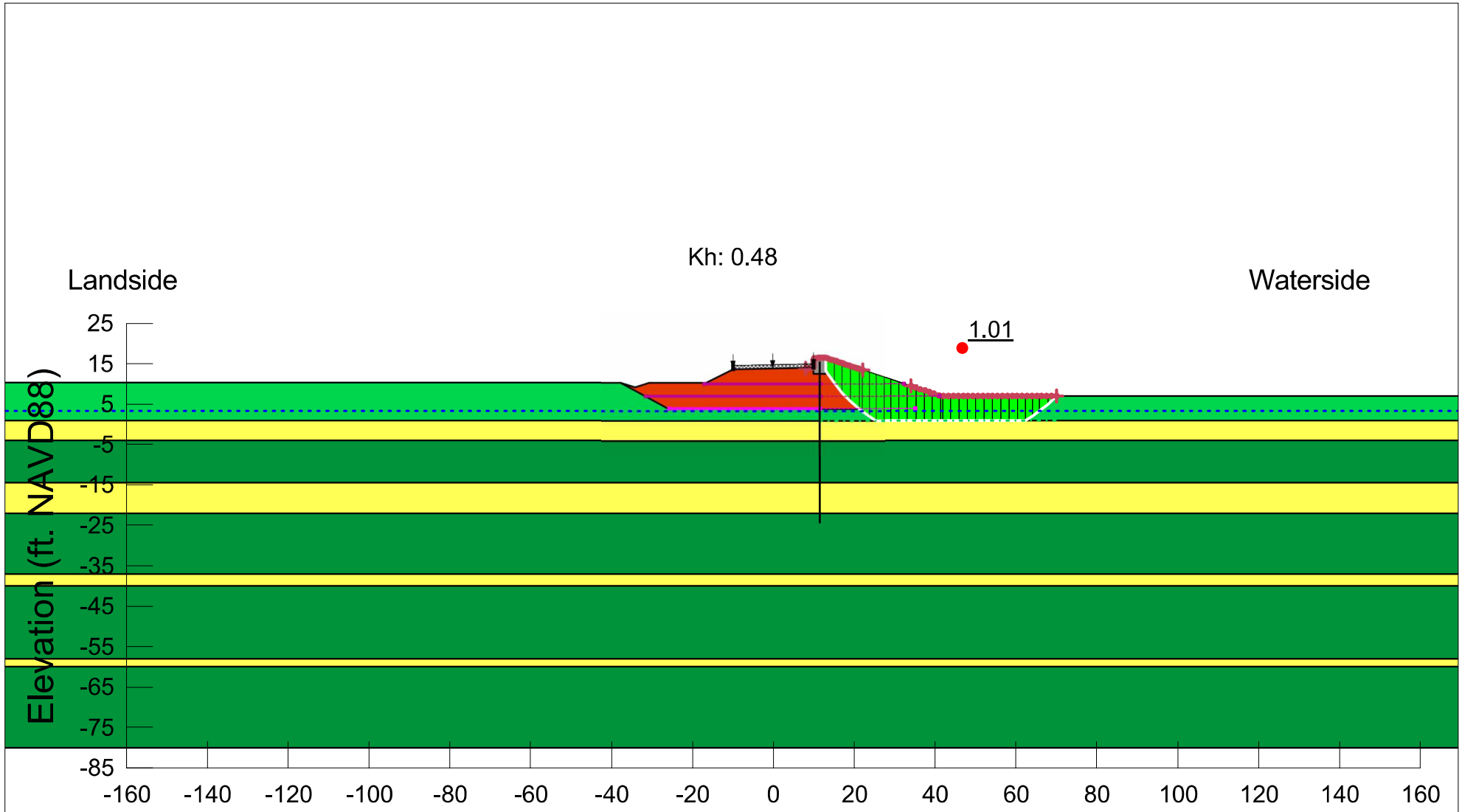


CROSS SECTION ACS5 (STATION 44+00)
CUTOFF WALL TO -25 FT, WITH GEOGRID,
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-53



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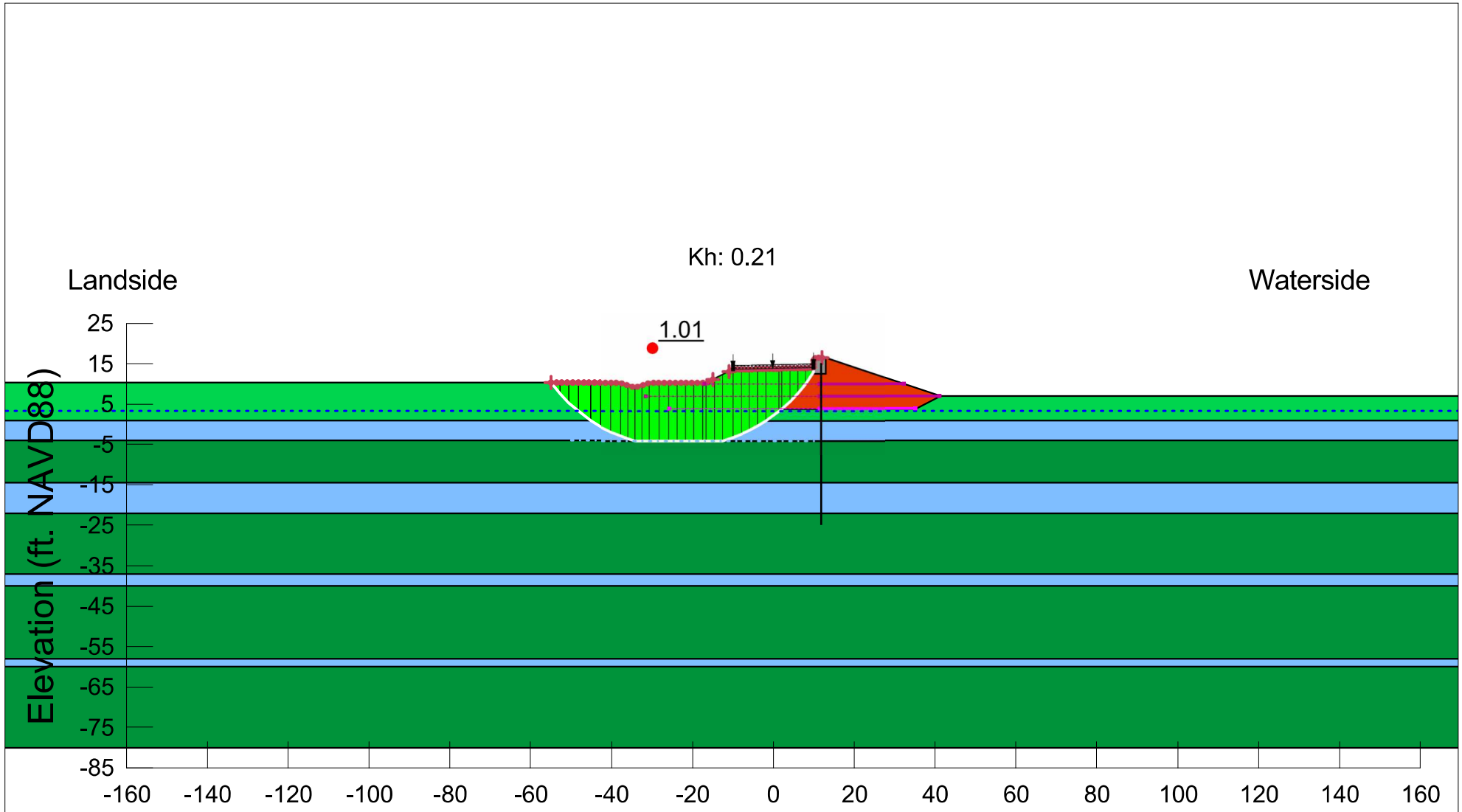
PSEUDOSTATIC SEISMIC STABILITY - YIELD
COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-54



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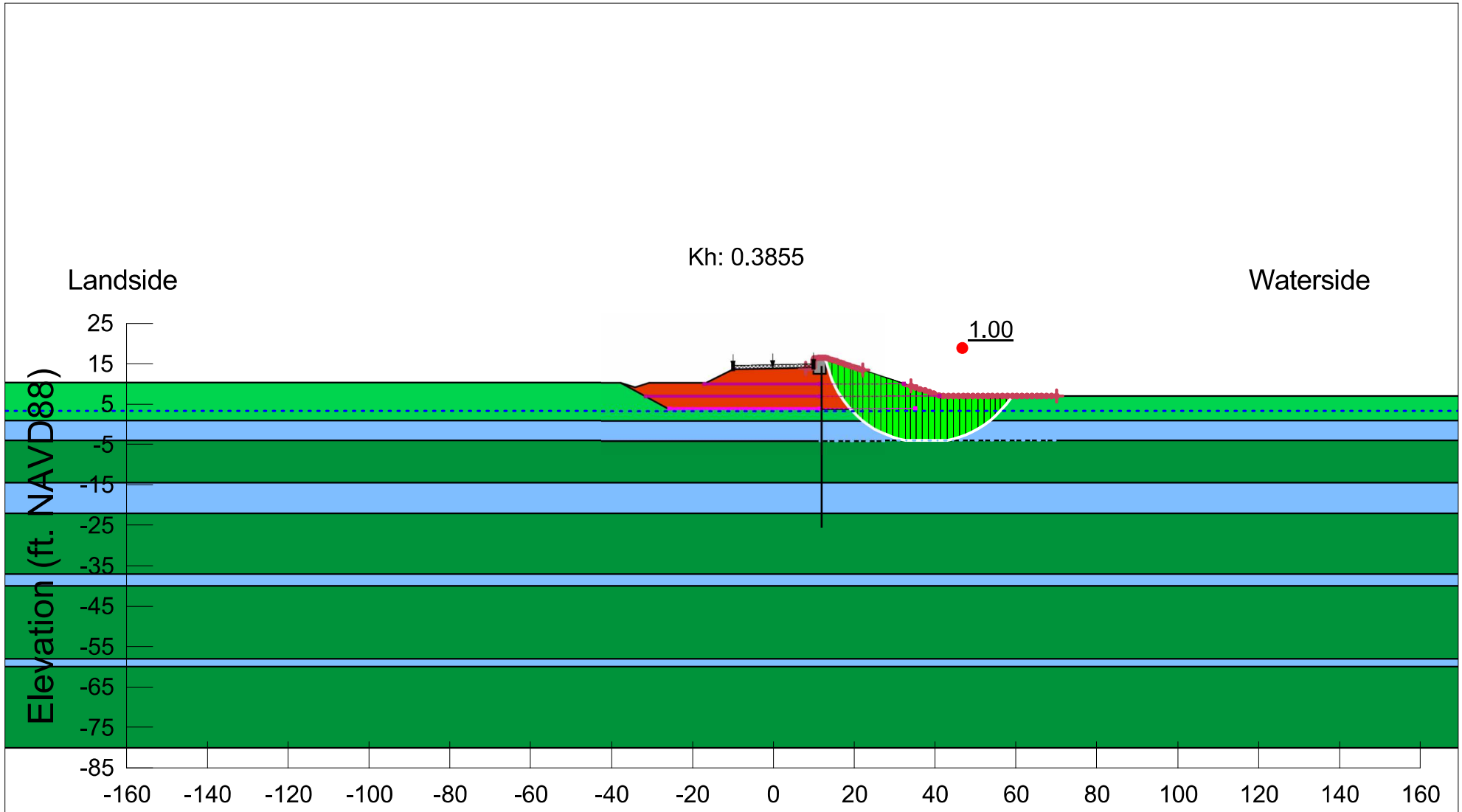
CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - YIELD COEFFICIENT

SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

JUNE 2026

FIGURE B8-55



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CUTOFF WALL TO -25 FT, WITH GEOGRID, POST LIQUEFACTION PSEUDOSTATIC SEISMIC STABILITY - YIELD COEFFICIENT

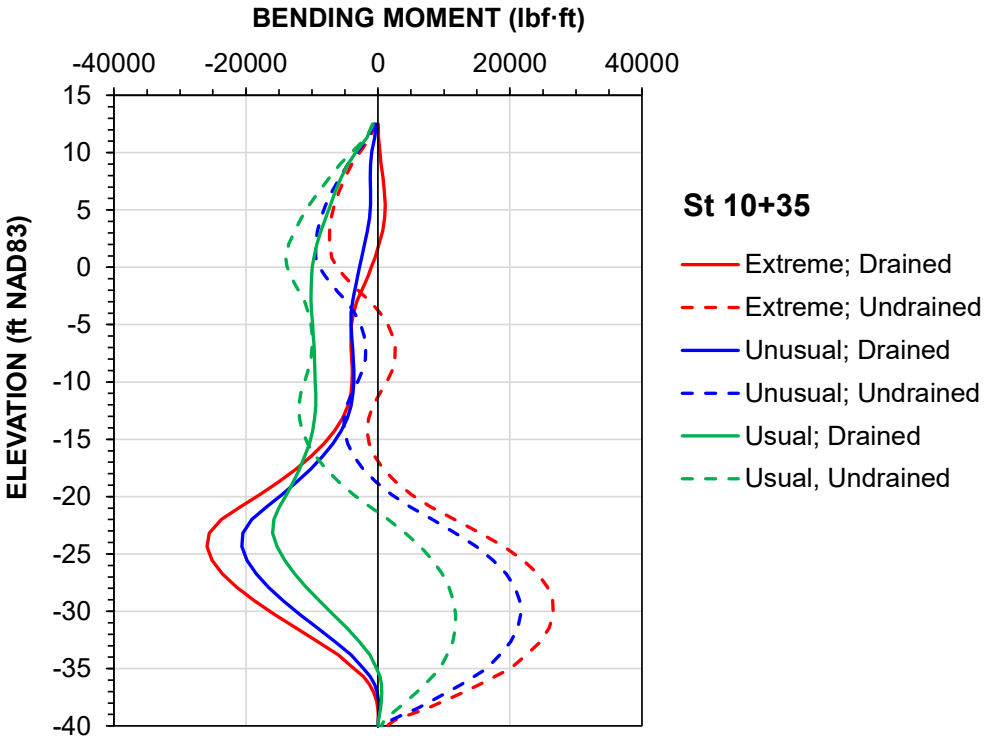
SAFER BAY- SOUTH OF BAY ROAD (60% DESIGN) - SAN MATEO COUNTY - CALIFORNIA

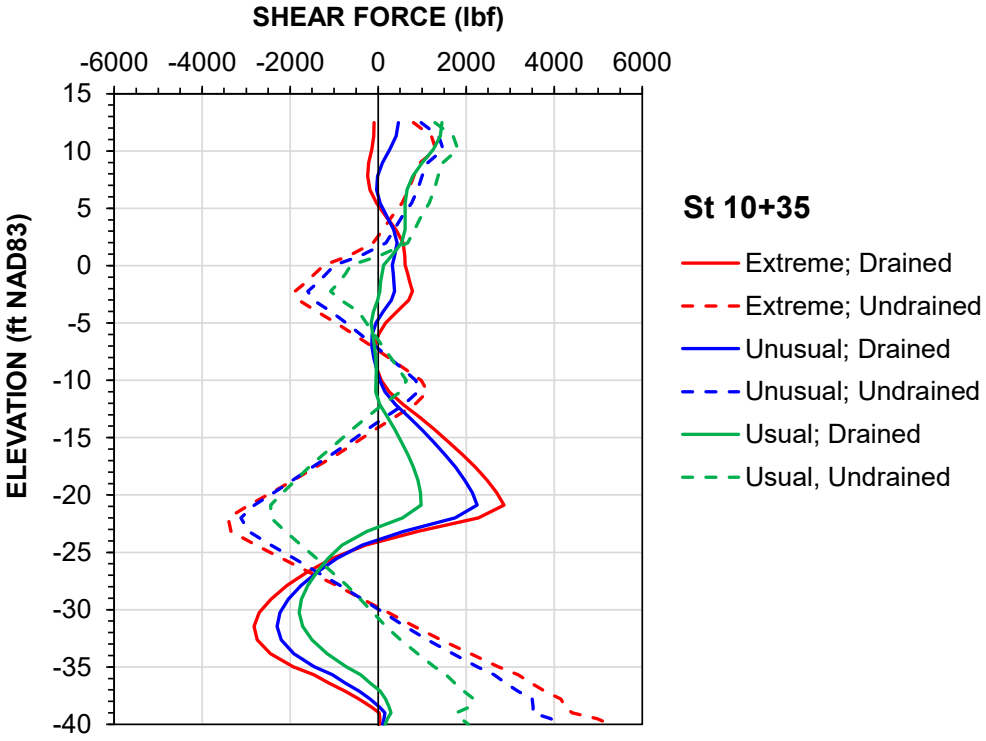
JUNE 2026

FIGURE B8-56

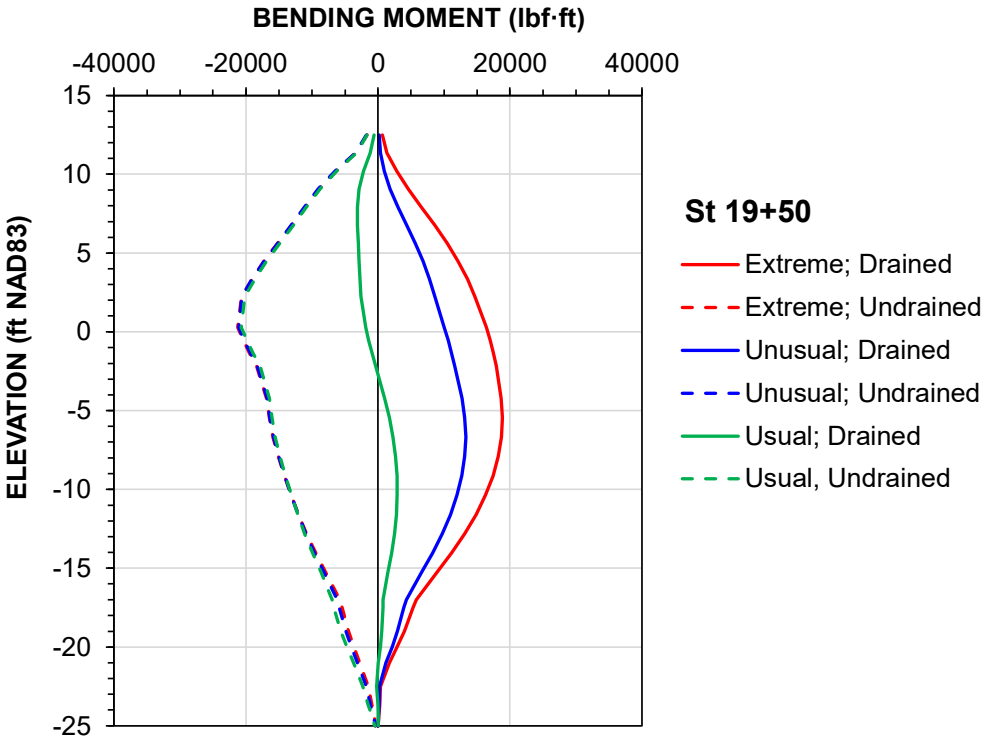
B.9 Sheet pile structural forces diagrams (pending)

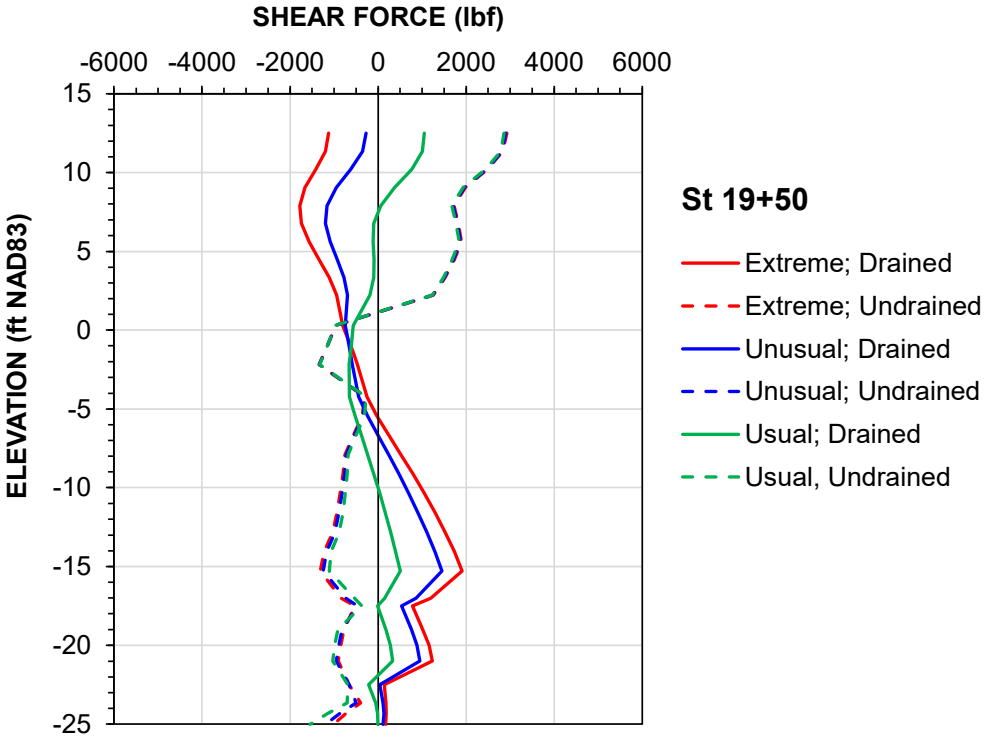
Station 10+35



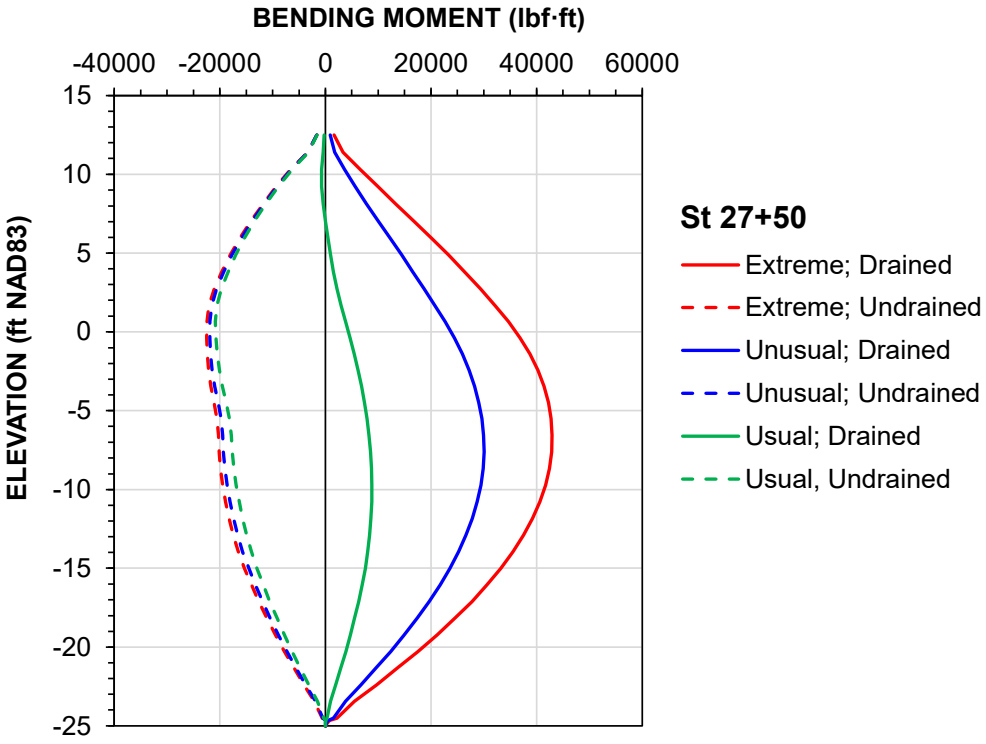


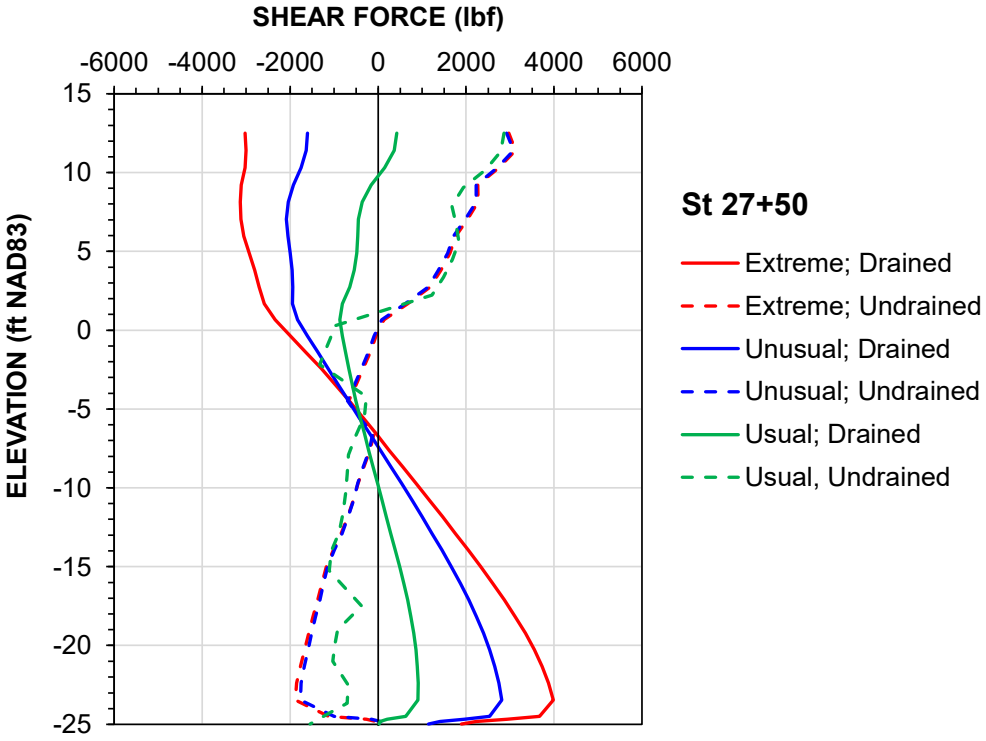
Station 19+50



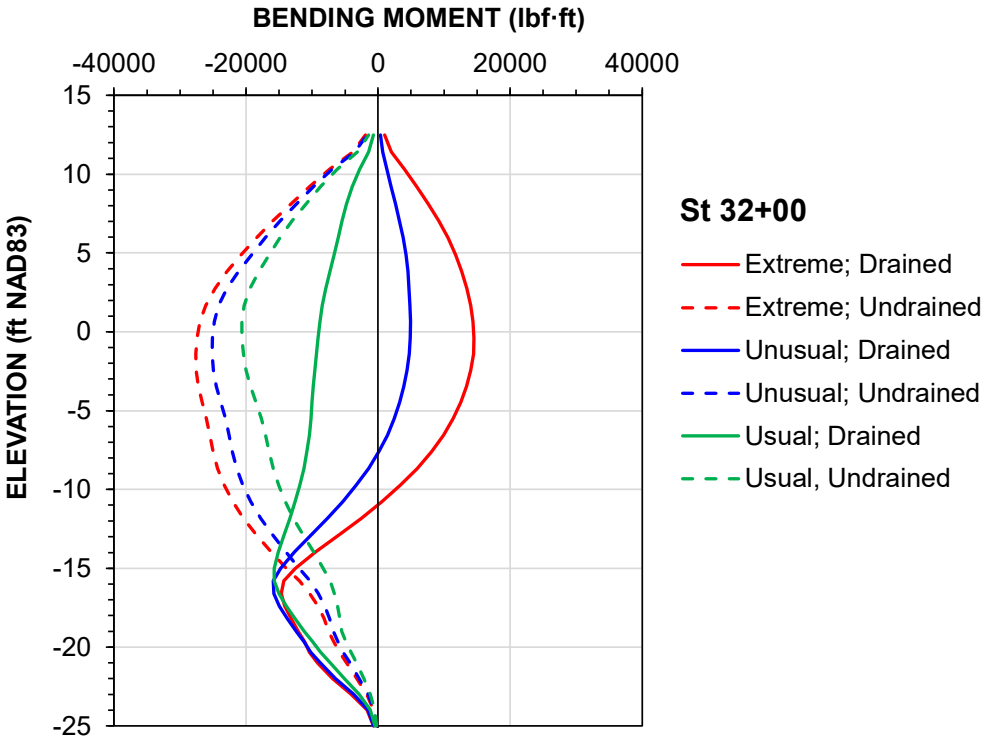


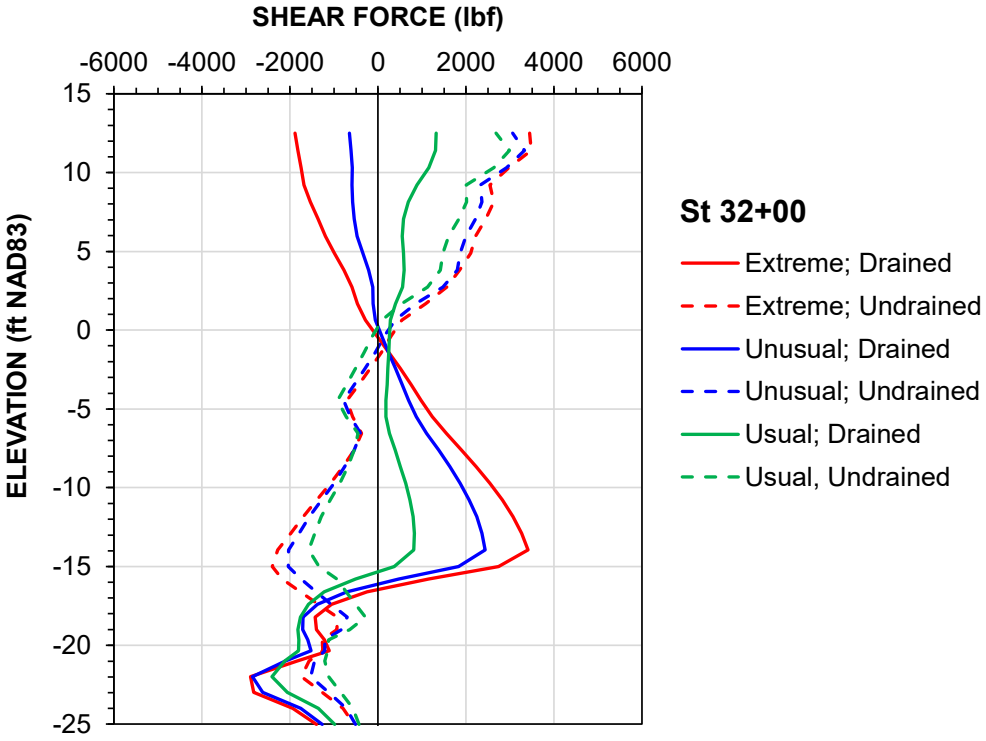
Station 27+50



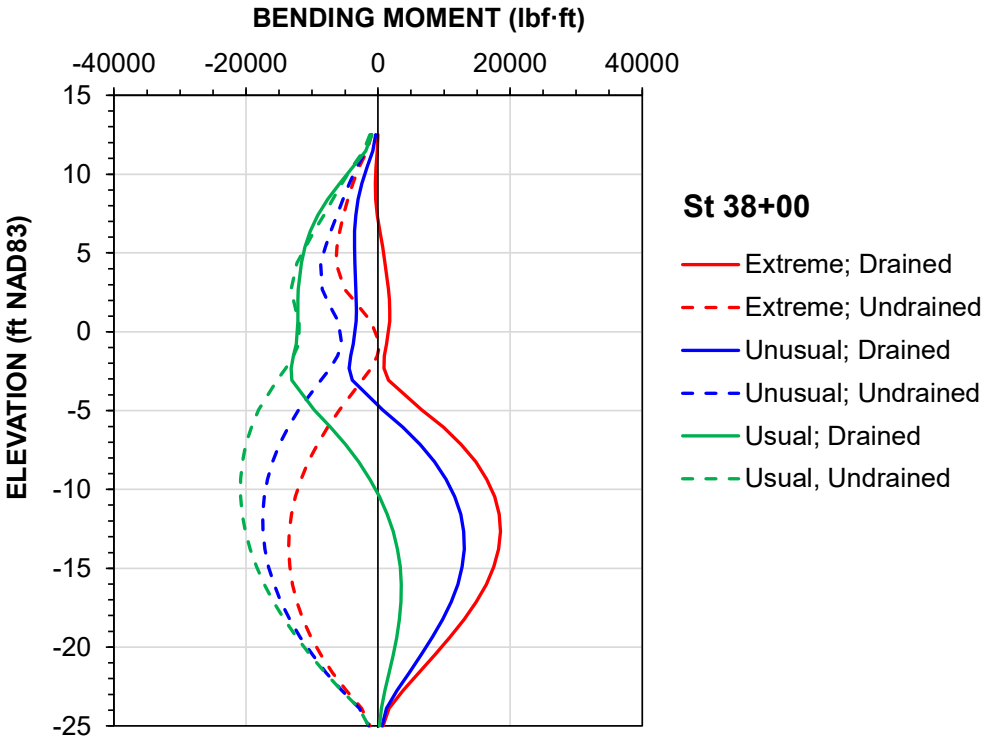


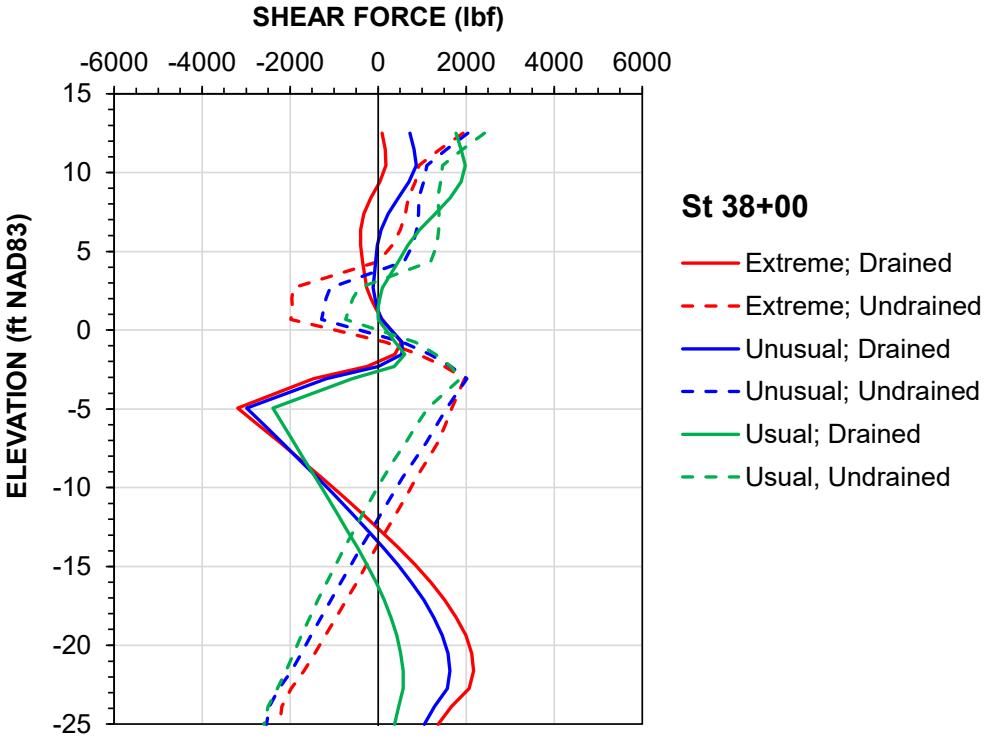
Station 32+00





Station 38+00





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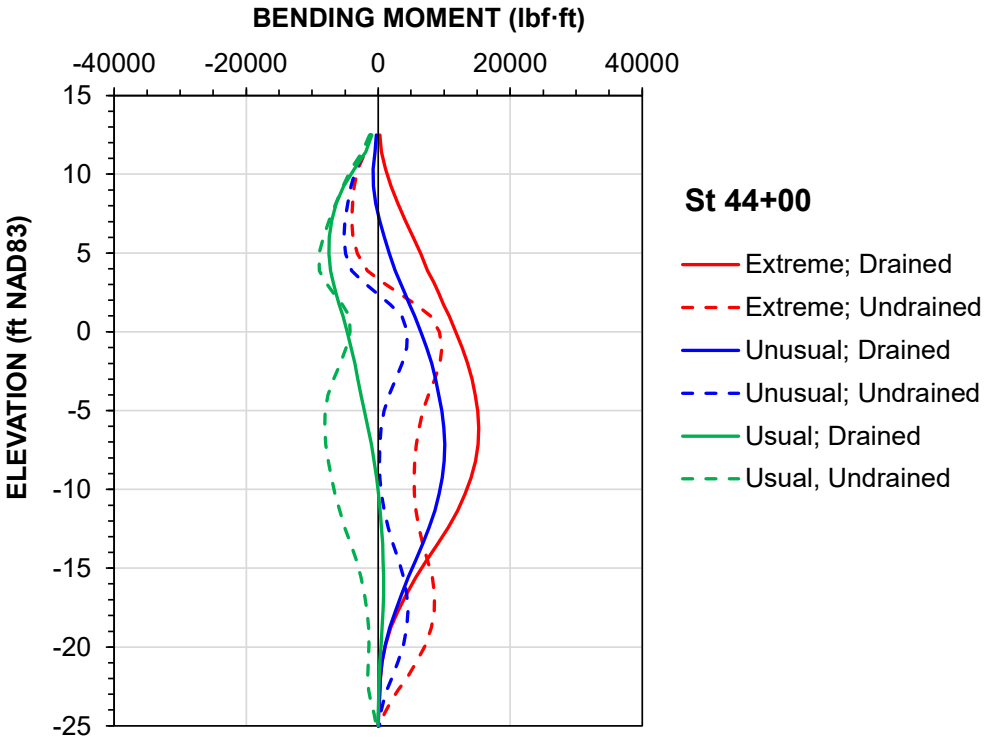
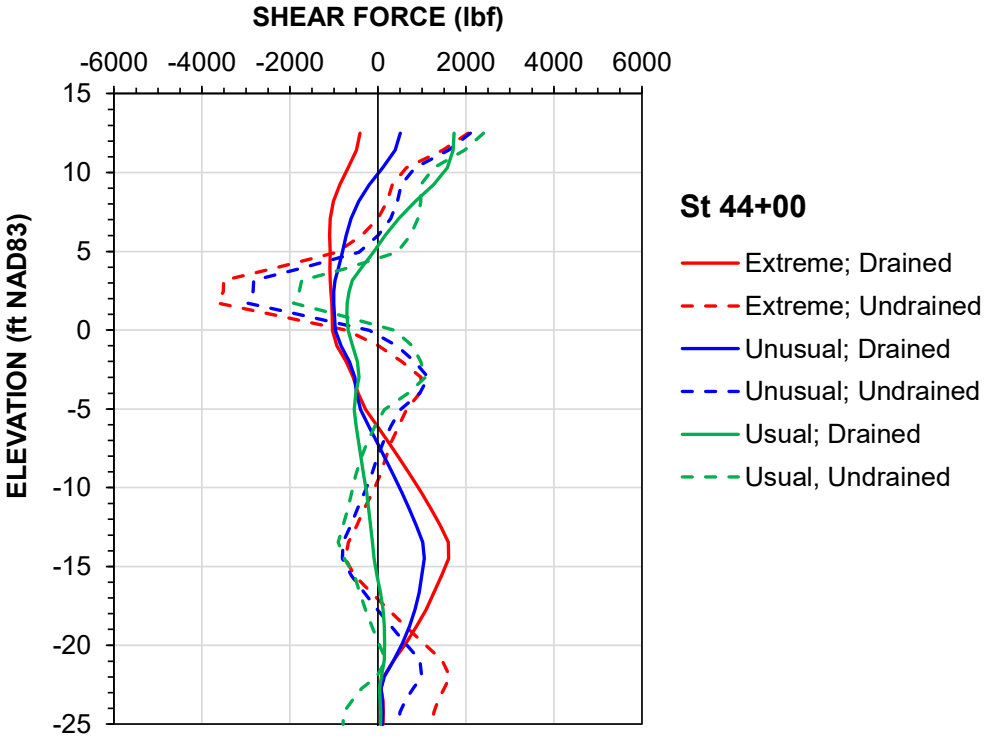




Appendix C Sheet pile wall settlement analysis

Appendix C

Sheet pile wall type		AZ 20-800
As per ft of wall	ft2	2,64
Unit weight of water	pcf	62,4
Assumed GWL	ft NAD83	3,3

Reference station: 42+50

Soil layer	Elevation Top	Elevation Bottom	Depth Top	Depth Bottom	Total unit weight	Beta-coefficient
	(ft NAD83)		(ft)		(pcf)	-
Fill (embankment)	14	5	0	9	125	0,29
Unit 2B	5	-6,5	9	20,5	115	0,2
Unit 3	-6,5	-37	20,5	51	120	0,23
Unit 4	-37	-39	51	53	125	0,29
Unit 3	-39	-61	53	75	120	0,23

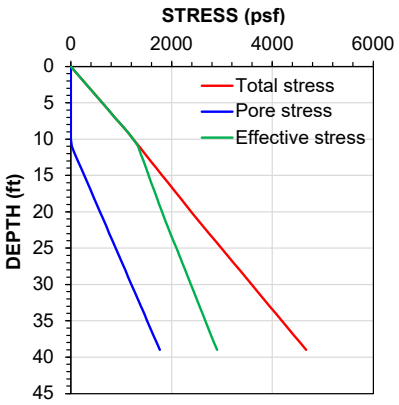
Elevation (ft NAD83)	Depth (ft)	Total stress (psf)	Pore stress (psf)	Effective stress (psf)	Beta-coefficient -	Unit shaft resistance (psf)	Shaft resistance (lbf)	Elevation of the equilibrium plane (EP)	Negative skin friction[1] (lbf)	Positive shaft resistance[2] (lbf)
14	0	0	0	0	0,29	0	0	-11	19278	21483
13	1	125	0	125	0,29	36	96			
12	2	250	0	250	0,29	73	191			
11	3	375	0	375	0,29	109	287			
10	4	500	0	500	0,29	145	383			
9	5	625	0	625	0,29	181	479			
8	6	750	0	750	0,29	218	574			
7	7	875	0	875	0,29	254	670			
6	8	1000	0	1000	0,29	290	766			
5	9	1125	0	1125	0,29	326	861			
4	10	1240	0	1240	0,2	248	655			
3	11	1355	19	1336	0,2	267	706			
2	12	1470	81	1389	0,2	278	733			
1	13	1585	144	1441	0,2	288	761			
0	14	1700	206	1494	0,2	299	789			
-1	15	1815	268	1547	0,2	309	817			
-2	16	1930	331	1599	0,2	320	844			
-3	17	2045	393	1652	0,2	330	872			
-4	18	2160	456	1704	0,2	341	900			
-5	19	2275	518	1757	0,2	351	928			
-6	20	2390	580	1810	0,2	362	956			
-7	21	2508	643	1865	0,23	429	1132			
-8	22	2628	705	1922	0,23	442	1167			
-9	23	2748	768	1980	0,23	455	1202			
-10	24	2868	830	2038	0,23	469	1237			
-11	25	2988	892	2095	0,23	482	1272			
-12	26	3108	955	2153	0,23	495	1307			
-13	27	3228	1017	2210	0,23	508	1342			
-14	28	3348	1080	2268	0,23	522	1377			
-15	29	3468	1142	2326	0,23	535	1412			
-16	30	3588	1204	2383	0,23	548	1447			
-17	31	3708	1267	2441	0,23	561	1482			
-18	32	3828	1329	2498	0,23	575	1517			
-19	33	3948	1392	2556	0,23	588	1552			
-20	34	4068	1454	2614	0,23	601	1587			
-21	35	4188	1516	2671	0,23	614	1622			
-22	36	4308	1579	2729	0,23	628	1657			
-23	37	4428	1641	2786	0,23	641	1692			
-24	38	4548	1704	2844	0,23	654	1727			
-25	39	4668	1766	2902	0,23	667	1762			

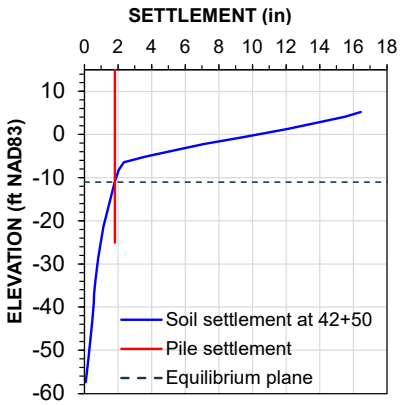
Notes:

[1] Negative skin friction calculated from the pile head to the elevation of equilibrium plane (EP)

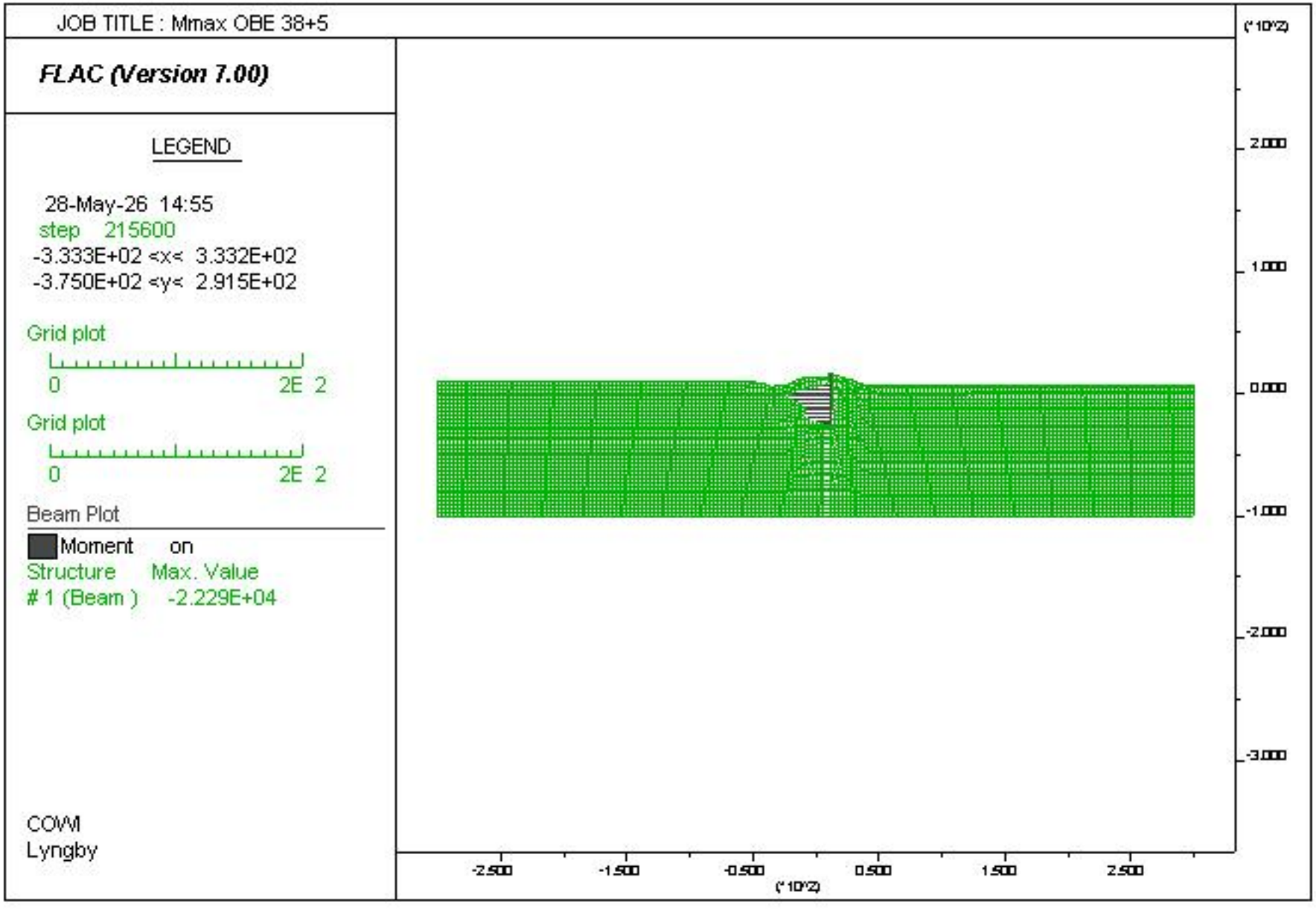
[2] Positive shaft resisance calculated from the elevation of equilibrium plane (EP) to the sheet pile toe

Estimated pile head settlement (in): 1,8





Appendix D FLAC2D outputs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FLAC (Version 7.00)
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28-May-26 14:55
step 215600
-3.333E+02 <x< 3.332E+02
-3.750E+02 <y< 2.915E+02
Grid plot
0
2E 2
Grid plot
0
2E 2
Beam Plot
Moment on
Structure Max. Value
1 (Beam) -2.229E+04
COWI
Lyngby
(*10^2)
2.000
1.000
0.000
-1.000
-2.000
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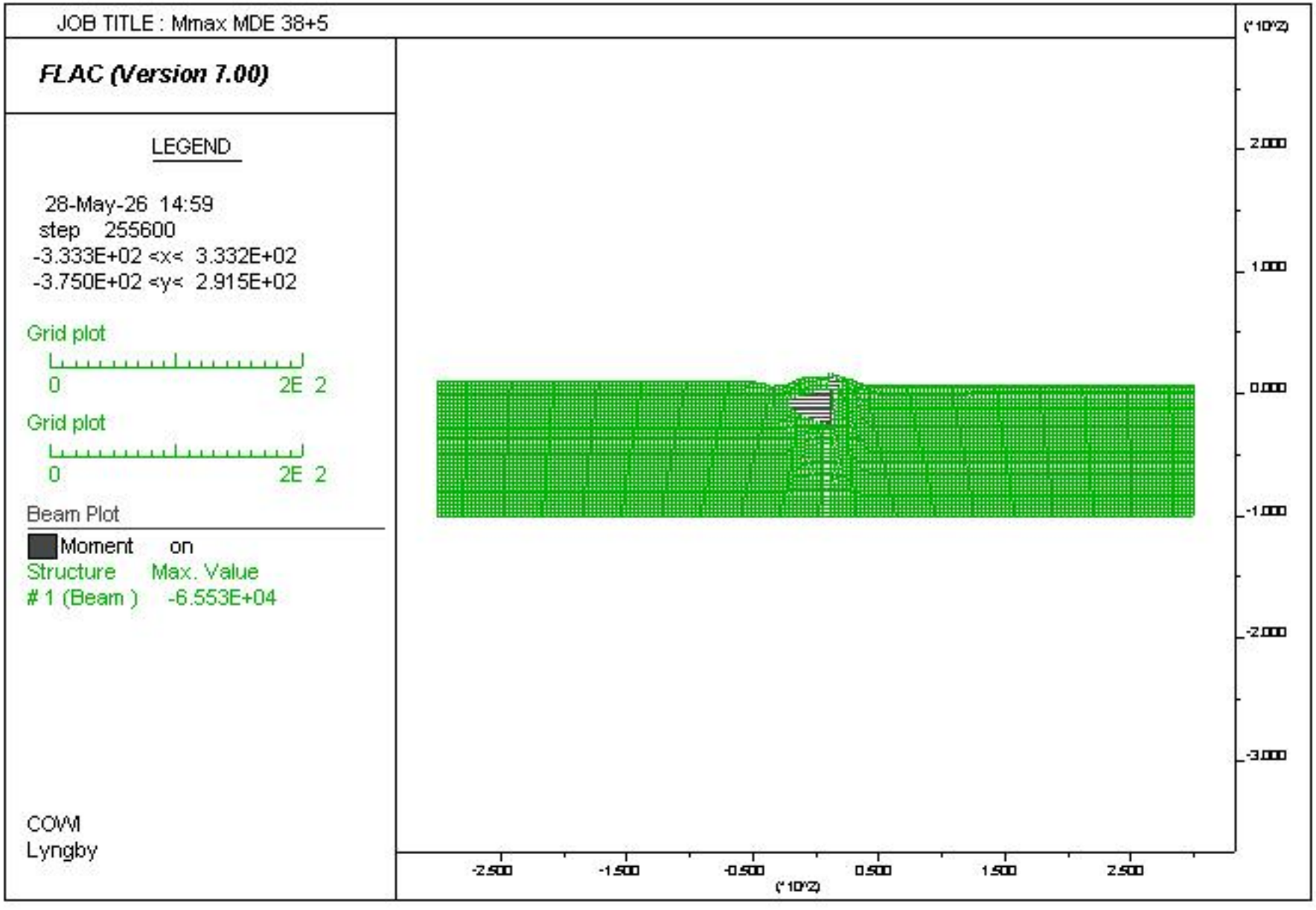
0.500
1.500
2.500
(*10^2)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Grid plot
0
2E 2
Grid plot
0
2E 2
Beam Plot
Moment on
Structure Max. Value
1 (Beam) -6.553E+04
COWI
Lyngby
(*10^2)
2.000
1.000
0.000
-1.000
-2.000
-3.000
-2.500
-1.500
-0.500
0.500
1.500
2.500
(*10^2)

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Appendix A.5:
Levee/Floodwall Engineering Analysis Retaining Wall

ADDRESS COWI North America, Inc.
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Oakland, CA 94607
USA

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FAX +1 510 839 9715
WWW cowi.com

MAY 2026
SHERWOOD DESIGN ENGINEERS, LTD.

SAFER BAY EPA RETAINING WALL

60% DESIGN REPORT

PROJECT NO.	DOCUMENT NO.
A299737	001

VERSION	DATE OF ISSUE	DESCRIPTION	PREPARED	CHECKED	APPROVED
1.0	27-05-2026	First issue	JKBA/GLNI	DLHR	JMC

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APPENDICES

Appendix A Retaining Wall Calculations
A.1 Retaining Wall Concrete Checks
A.2 Pipe Pile Design (PENDING)

1 Design Assumptions

1.1 Retaining Wall Configuration

The SAFER Bay East Palo Alto Project subject to this report extends approximately 5,250 feet along an existing embankment from the O'Connor Pump Station at San Francisquito Creek to Bay Road in East Palo Alto.

The Project utilizes a retaining wall near the transmission tower located on site.

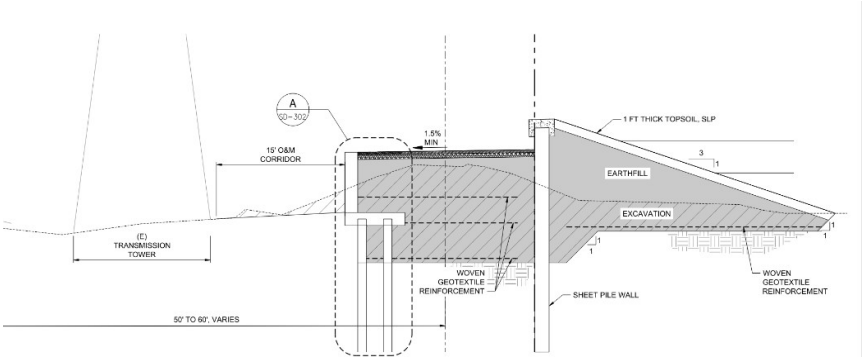


Figure 1-1 Typical retaining wall at transmission tower

1.2 Retaining Wall Composition

The retaining wall contains of two elements, a concrete L wall, and a pile supported foundation.

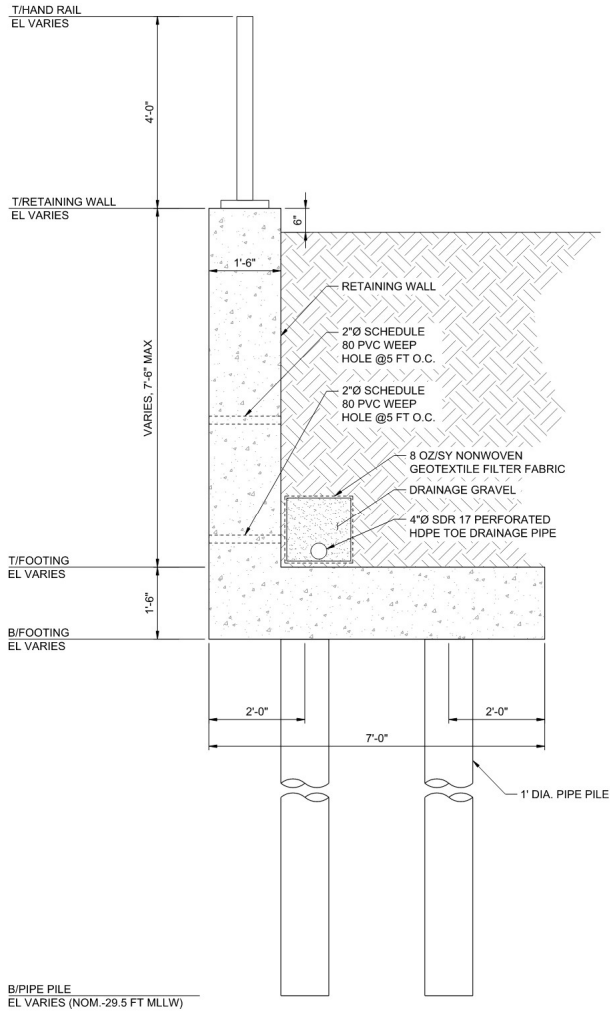


Figure 1-2 Typical Retaining Wall Section

1.2.1 Concrete

The reinforced concrete portion is specified as 5000psi compressive 28-day strength concrete with 60ksi rebar.

1.2.2 Steel Pipe Pile

A set of two of steel piles are set into the footing at 10ft spacing. These pipe piles are specified as 1ft diameter piles confirming to ASTM A252 Gr.50.

1.3 Codes and References

The following codes and references are used for design.

› USACE, Retaining Walls and Floodwalls, EM 1110-2-2502, August 1, 2022

- ASCE 7-22, American Society of Civil Engineers (ASCE), Minimum Design Loads for Buildings and Other Structures, 2022.
- ACI 318, Building Code Requirements for Structural Concrete (ACI 318-25) and Commentary (ACI 318R-25), 2025.

1.4 Soil Profiles and Properties

The soils stratigraphy and soil parameters used for the analyses were provided by Geosyntec and are not reported here.

1.5 Seismic Parameters

Seismic design of the retaining wall system has been based on EM 1110-2-2502.

Two seismic hazard levels have been adopted in the design:

- the Operating Basis Earthquake (OBE) and
- the Maximum Design Earthquake (MDE).

The updated simplified design parameters adopted for the present design stage are summarized in the following table:

Table 1-1: Simplified seismic design parameters

Design ground motion scenario	**Earthquake Return Period** (yr)	**Probability of Exceedance**	**PGA** (g)
OBE	475	10% in 50 years	0.51
MDE	2,475	2% in 50 years	0.77

2 Loads and Load Combinations

2.1 Earth Loads (EH/EV)

Lateral earth pressures were calculated and considered through the computed soil stress field generated by embankment self-weight, groundwater conditions, and applied surface surcharge.

2.2 Vehicle Live Load (V)

Uniform surcharge loading (H-20 truck) at the trail area is assumed to be 480 psf acting at the crest on the entire width of the trail (behind the new floodwall). See §2.1.

2.3 Hydrostatic Load (Hs)

Hydrostatic load is not considered for the retaining wall structure as it lies behind the flood protection system.

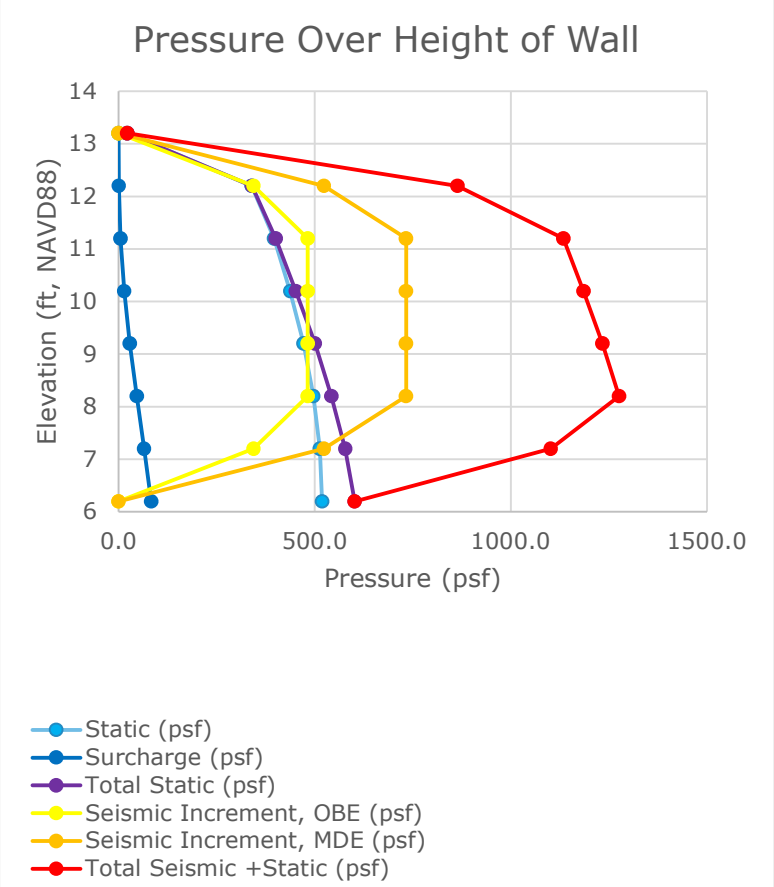
2.4 Seismic load (EQ)

Seismic load is applied using the OBE and MDE design ground motions defined in §1.5. Accordingly, the current design adopts PGA = 0.51 g for the OBE load case and PGA = 0.77 g for the MDE load case. The OBE is used as the serviceability-level earthquake, while the MDE is used as the extreme design event for which severe damage may occur. These loads are adopted as equivalent pressures along the heigh of the retaining wall.

2.5 Pressure Diagram

Table 2-1 Load pressures along height of wall

EL (ft, NAVD88)	Z (ft)	Static (psf)	Surcharge (psf)	Total Static (psf)	Seismic Increment, OBE (psf)	Seismic Increment, MDE (psf)	Total Seismic +Static (psf)
13.2	**0**	22.3	0.0	22.3	0.0	0.0	22.3
12.2	**1**	339.4	0.7	340.1	344.3	523.6	863.7
11.2	**2**	396.1	5.0	401.1	482.1	733.1	1134.2
10.2	**3**	437.9	14.6	452.4	482.1	733.1	1185.5
9.2	**4**	471.5	28.9	500.3	482.1	733.1	1233.4
8.2	**5**	496.6	46.2	542.8	482.1	733.1	1275.9
7.2	**6**	513.1	64.8	577.9	344.3	523.6	1101.5
6.2	**7**	518.9	83.2	602.0	0.0	0.0	602.0



2.6 Load combinations

Retaining wall load combinations presented in Table 2-2 is based on floodwall loads combinations from USACE EM 1110-2-2502, Table C.2.

Table 2-2: Sheet pile design load combinations

LC	Load description	Load category	Load combination
LC-1	Normal Operating	**Unusual**	**D + EH + EV + Hs**
LC-2	Normal operating +Surcharge	**Usual**	**D + EH + EV + Hs + ES**
LC-3A	Earthquake – OBE	**Unusual**	**D + EH + EV + EQ + Hs**
LC-3B	Earthquake – MDE	**Extreme**	**D + EH + EV + EQ + Hs**

3 Concrete Design

The following is a summary of calculation included in Appendix A.1.

3.1 Location of Evaluation

The concrete wall strength is evaluated at the top of footing where loading is the highest.

3.2 Demand Capacity Ratios

The following DCR (Demand Capacity Ratios) are taken from Appendix A.1. See attached calculations for detailed design.

Table 3-1 Concrete DCR Table

Failure Mode	DCR
Shear	0.7
Bending	0.9

4 Pipe Pile Design

This section presents the structural verification of the pipe pile deep foundation to be included in Appendix A.2.

This section and associated appendix are pending and will be included in future design submittals.

Appendix A Retaining Wall Calculations

A.1 Retaining Wall Concrete Checks

PROJECT	**SAFER Bay South of Bay Road**	JOB NO.	A299737
SUBJECT	Retaining Wall Calculations	ISSUE NO.	1
PREPARED	EVVI	DATE OF ISSUE	06/3/2026
CHECKED	JMC		

COWI

Background/Purpose

The following calculations checks the retaining wall for the SAFER Bat South of Bay Road project. The retaining wall is located at the transmission tower located near STA 25+00.

Design Methodolodgy

This calculation follows the project basis of design including material strengths, codes, and standards listed below.

Codes and Standards

The following codes and standards are used in this calculation.

- USACE, Retaining Walls and Floodwalls, EM 1110-2-2502, August 1, 2022
- ASCE 7-22, American Society of Civil Engineers (ASCE), Minimum Design Loads for Buildings and Other Structures, 2022.
- ACI 318, Building Code Requirements for Structural Concrete (ACI 318-25) and Commentary (ACI 318R-25), 2025.

Materials

The following lists the materials used in this calculation.

Concrete

$f'c := 5\ ksi$ — Concrete compressive strength

$\gamma_c := 150\ pcf$ — Unit weight of concrete

$\lambda_c := 1.0$

Steel

$f_y := 60\ ksi$ — Steel yield stress, Rebar

Soil

$\phi_s := 30\ deg$ — Coefficient of soil friction

$\gamma_s := 120\ pcf$ — Unit weight of soil

$\mu := 0.5 \cdot \tan(\phi_s) = 0.29$ — Sliding friction coefficient

Water

$\gamma_w := 62.4\ pcf$ — Unit weight of water

Geometry

This section calcuates the anchor capacity

$EL_{top_soil} := 13.2\ ft$ — Elevation of the top of soil

$EL_{top_wall} := EL_{top_soil} + 0.5\ ft = 13.70\ ft$ — Elevation of the top of wall

$EL_{top_foot} := 6.2\ ft$ — Elevation of the top of footing

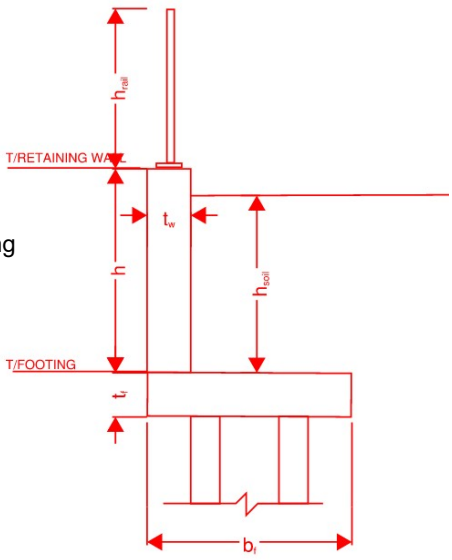
$h := EL_{top_wall} - EL_{top_foot} = 7.50\ ft$ — Height of wall

$h_{soil} := EL_{top_soil} - EL_{top_foot} = 7.00\ ft$ — Height of retained soil

$t_w := 1.5\ ft$ — Wall thickness

$b_f := 7\ ft$ — Footing width

$t_f := 1.5\ ft$ — Footing thickness

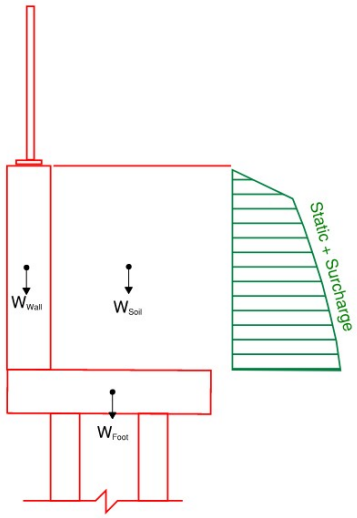


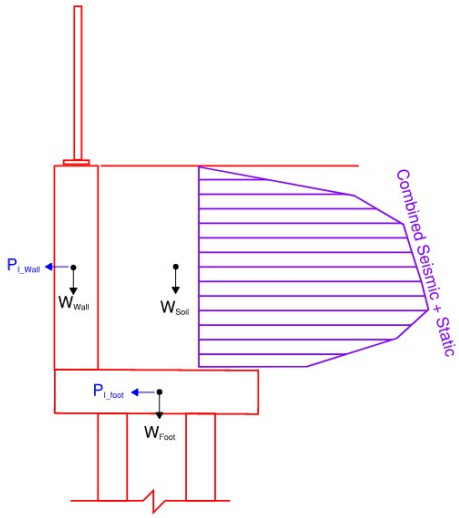
$h_{rail} := 4\ ft$ Height of railing

$EL_{bot_foot} := EL_{top_foot} - t_f = 4.70\ ft$ Elevation of bottom of footing

Loads

Load Diagram





Soil Loading

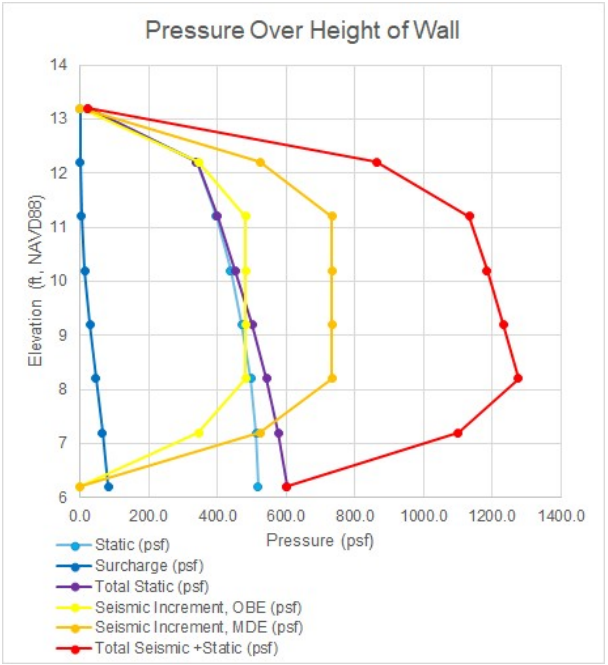
EL (ft, NAVD88)	Z (ft)	Static (psf)	Surcharge (psf)	Total Static (psf)	Seismic Increment, OBE (psf)	Seismic Increment, MDE (psf)	Total Seismic +Static (psf)
13.2	0	22.3	0.0	22.3	0.0	0.0	22.3
12.2	1	339.4	0.7	340.1	344.3	523.6	863.7
11.2	2	396.1	5.0	401.1	482.1	733.1	1134.2
10.2	3	437.9	14.6	452.4	482.1	733.1	1185.5
9.2	4	471.5	28.9	500.3	482.1	733.1	1233.4
8.2	5	496.6	46.2	542.8	482.1	733.1	1275.9
7.2	6	513.1	64.8	577.9	344.3	523.6	1101.5
6.2	7	518.9	83.2	602.0	0.0	0.0	602.0

$P_S := 3.13\ \frac{kip}{ft}$ Resultant Pressure (Total Static)

$EL_S := 9.16\ ft$ Resultant location (Elevation)

$P_{EQ} := 7.11\ \frac{kip}{ft}$ Resultant Pressure (Total Seismic +Static)

$EL_{EQ} := 9.46\ ft$ Resultant location (Elevation)



Hydrostatic Loading

Hydrostatic loading is not considered on the retatining wall as it is behind the flood protection wall at the other side of the levee.

Dead Load

$W_{wall} := \gamma_c \cdot (h \cdot t_w) = 1.69 \ \frac{kip}{ft}$ Weight of the wall

$W_{foot} := \gamma_c \cdot (t_f \cdot b_f) = 1.58 \ \frac{kip}{ft}$ Weight of the footing

$W_{soil} := \gamma_s \cdot (h \cdot (b_f - t_w)) = 4.95 \ \frac{kip}{ft}$ Weight of the soil above the footing

$W_{total} := W_{wall} + W_{foot} + W_{soil} = 8.21 \ \frac{kip}{ft}$ Total Weight

Inertial Forces of wall structure

$PGA_{MDE} := 0.77$ PGA (MDE)

$k_h := \frac{2 \cdot PGA_{MDE}}{3} = 0.51$ EM 1110-2-2502 Section 6.9.6.1.2

$P_{I_wall} := k_h \cdot W_{wall} = 0.87 \ \frac{kip}{ft}$ Inertial force for the Wall

$P_{I_foot} := k_h \cdot W_{foot} = 0.81 \ \frac{kip}{ft}$ Inertial force for the Footing

$P_I := P_{I_wall} + P_{I_foot} = 1.67 \ \frac{kip}{ft}$ Total inertial force

$x_I := \frac{(h \cdot t_w) \cdot \left(\frac{h}{2} + t_f\right) + (t_f \cdot b_f) \cdot \left(\frac{t_f}{2}\right)}{(h \cdot t_w) + (t_f \cdot b_f)} = 3.08 \ ft$ Height of intertial force

Load Combination

Load combinations are taken from EM 1110-2-2502, Table C.2.

Table C.2
Example Load Combinations for an Earth Retaining Wall

Load Case	Load Description	Load Category	Load Combination
1	Normal Operating	Usual	$D + EH + EV + HS_U$
2	Normal Operating + Surcharge	Unusual	$D + EH + EV + HS_U + ES_N$
3A	Earthquake – OBE	Unusual	$D + EH + EV + EQ + HS_e$
3B	Earthquake – MDE	Extreme	$D + EH + EV + EQ + HS_e$
4	Maximum Hydrostatic	Maximum	$D + EH + EV + HS_{pr}$
5	Construction	Unusual	$D + EH + EV + ES_N$

Load combinations 2, and a maximum of 3A and 3B are considered in this design.

Check Overturning

$RM := W_{wall} \cdot \frac{t_w}{2} + W_{foot} \cdot \frac{b_f}{2} + W_{soil} \cdot \left(t_w + \frac{b_f - t_w}{2}\right) = 27.82 \ \frac{kip \cdot ft}{ft}$ Resitive Moment

$OM_s := P_S \cdot (EL_S - EL_{bot_foot}) = 13.96 \ \frac{kip \cdot ft}{ft}$ Overturning Moment (Static)

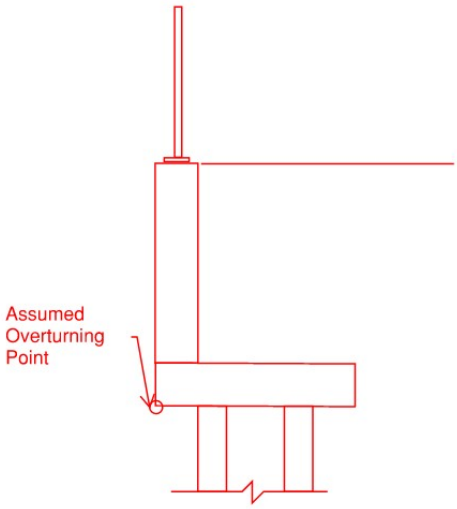
$OM_{eq} := P_{EQ} \cdot (EL_{EQ} - EL_{bot_foot}) + P_I \cdot x_I = 39.00 \ \frac{kip \cdot ft}{ft}$ Overturning Moment (Seismic)

$\frac{RM}{OM_s} = 1.99$ Safety Factor against Overturning (Static)

$$OVERTURNING_s := \begin{cases} \text{if } \frac{RM}{OM_s} > 1.5 \\ \quad \text{“Okay”} \\ \text{else} \\ \quad \text{“Add Piles/Redesign”} \end{cases} = \text{“Okay”}$$

$$\frac{RM}{OM_{eq}} = 0.71 \qquad \text{Safety Factor against Overturning (Seismic)}$$

$$OVERTURNING_{eq} := \begin{cases} \text{if } \frac{RM}{OM_{eq}} > 1.5 \\ \quad \text{“Okay”} \\ \text{else} \\ \quad \text{“Add Piles/Redesign”} \end{cases} = \text{“Add Piles/Redesign”}$$



Check Sliding

This section calcuates the anchor capacity

$RF := \mu \cdot W_{total} = 2.37 \ \frac{kip}{ft}$ Resisting force for Sliding

$SF_s := P_S = 3.13 \ \frac{kip}{ft}$ Sliding Force (Static)

$SF_{eq} := P_{EQ} + P_l = 8.78 \ \frac{kip}{ft}$ Sliding Force (Seismic)

$\frac{RF}{SF_s} = 0.76$ Safety Factor against sliding (Static)

$$SLIDING_s := \begin{cases} \text{if } \frac{RF}{SF_s} > 1.5 \\ \quad \text{“Okay”} \\ \text{else} \\ \quad \text{“Add Piles/Redesign”} \end{cases} = \text{“Add Piles/Redesign”}$$

$\frac{RF}{SF_{eq}} = 0.27$ Safety Factor against sliding (Seismic)

$$SLIDING_{eq} := \begin{cases} \text{if } \frac{RF}{SF_{eq}} > 1.1 \\ \quad \text{“Okay”} \\ \text{else} \\ \quad \text{“Add Piles/Redesign”} \end{cases} = \text{“Add Piles/Redesign”}$$

Sliding deemed inadequate. Add piles and check again.

Demands @ Base of Wall

This section calcuates the anchor capacity

Axial Demand @ Base of Wall

$A_u := W_{wall} = 1.69 \ \frac{kip}{ft}$

Shear Demand @ Base of Wall

$V_u := P_{EQ} + P_{l_wall} = 7.98 \ \frac{kip}{ft}$

Moment Demand @ Base of Wall

$M_u := P_{EQ} \cdot \left(EL_{EQ} - EL_{bot_foot}\right) + P_{l_wall} \cdot \left(\frac{h}{2}\right) = 37.09 \ \frac{kip \cdot ft}{ft}$

Reinforcing Steel

$b_{wall} := 1\ \frac{ft}{ft}$ 1ft width of wall

$d_{bar} := \frac{7}{8}\ in$ Rebar Diameter

$S_{bar} := 12\ in$ Rebar Spacing

$cover := 2\ in$ Rebar Cover

Free Body Diagram of wall cut

$A_s := \left(\frac{\pi \cdot d_{bar}^{\ 2}}{4}\right) \cdot \frac{12\ in}{S_{bar} \cdot ft} = 0.6\ \frac{in^2}{ft}$ Area of longitudinal steel in section

Check Wall For Shear @ Base

This section calcuates the anchor capacity

$\phi_{shear} := 0.75$ Shear reduction factor

Shear Capacity

$\lambda_s := \text{if}\left(\sqrt{\frac{2}{1+\frac{t_w}{10\ in}}} \geq 1,\ 1,\ \sqrt{\frac{2}{1+\frac{t_w}{10\ in}}}\right) = 0.85$ Size effect modification Factor ACI 318-25 Eqn 22.5.5.1.3

$\rho_{w_22} := \frac{A_s}{b_{wall} \cdot (t_w)} = 0.0028$ Steel Ratio per Area of Concrete

$V_C := \left(8 \cdot \lambda_s \cdot \lambda_c \cdot (\rho_{w_22})^{\frac{1}{3}} \cdot \sqrt{\frac{f'c}{psi}} \cdot psi + \frac{A_u}{6 \cdot b_{wall} \cdot t_w}\right) \cdot b_{wall} \cdot t_w = 15\ \frac{kip}{ft}$ Concrete Shear Capacity under Static Loading (ACI 318-25 Table 22.5.5.1)

$V_{C_min} := \lambda_c \cdot \sqrt{\frac{f'c}{psi}} \cdot psi \cdot b_{wall} \cdot t_w = 15\ \frac{kip}{ft}$ Min Concrete Shear Strength ACI 22.5.5.1.1

$V_{C_max} := 5 \cdot \lambda_c \cdot \sqrt{\frac{f'c}{psi}} \cdot psi \cdot b_{wall} \cdot t_w = 76\ \frac{kip}{ft}$ Max Concrete Shear Strength ACI 22.5.5.1.1

$V_c := \text{if}\left(V_C < V_{C_min},\ V_{C_min},\ \text{if}\left(V_C > V_{C_max},\ V_{C_max},\ V_C\right)\right) = 15\ \frac{kip}{ft}$

$\phi V_n := \phi_{shear} \cdot V_c = 11\ \frac{kip}{ft}$

DC Ratio

$$DC_V := \frac{V_u}{\phi V_n} = 0.7$$

$$CHECK := \begin{cases} \text{if } V_u < \phi V_n \\ \quad \text{"No Transverse Reinforcement Required"} \\ \text{else} \\ \quad \text{"Add Transverse Reinforcing"} \end{cases} = \text{"No Transverse Reinforcement Required"}$$

Check Wall For Bending @ Base

$\phi_{bending} := 0.9$ Bending reduction factor

Tension Capacity from Steel

$T_s := A_s \cdot f_y = 36.08 \ \frac{kip}{ft}$ Tension capacity for steel

Compression force @ Base of wall

$\beta_1 := 0.85$ Value for f'c =3500ksi, ACI 318-25 Table 22.2.2.4.3

$c := \frac{A_s \cdot f_y}{0.85 \cdot f'c \cdot b_{wall} \cdot \beta_1} = 0.83 \ in$ Depth of neutral axis

$a := \beta_1 \cdot c = 0.71 \ in$ ACI 318-25 22.2.2.4.1

$C_c := 0.85 \cdot f'c \cdot b_{wall} \cdot \beta_1 \cdot c = 36.08 \ \frac{kip}{ft}$ Compression force

Capacity @ base of wall

$d := t_w - \left(cover + \frac{d_{bar}}{2} \right) = 15.56 \ in$ distance between tension and compression forces

$M_n := C_c \cdot \left(d - \frac{a}{2} \right) = 45.73 \ \frac{kip \cdot ft}{ft}$ Nominal moment

$\phi M_n := \phi_{bending} \cdot M_n = 41.15 \ \frac{kip \cdot ft}{ft}$ Moment capacity

DC Ratio

$$DC_M := \frac{M_u}{\phi M_n} = 0.9$$

A.2 Pipe Pile Design (PENDING)