

**GEOTECHNICAL DATA REPORT
LAS GALLINAS LEVEE SYSTEM
SAN RAFAEL, CALIFORNIA**

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July 3, 2013



July 3, 2013
File: 96670

Mr. Neal Conatser
Marin County Department of Public Works
3501 Civic Center Drive, Room 304
San Rafael, CA 94903

**SUBJECT: Geotechnical Data Report
Las Gallinas Levee System
San Rafael, California**

Dear Mr. Conatser:

Kleinfelder is pleased to present the attached geotechnical data report for the Las Gallinas Levee System (LGLS) for your review. This report supersedes our draft geotechnical data report of July 6, 2010 and includes revised analyses and discussion based on input from the United States Army Corps of Engineers (USACE) and personnel from the Marin County Department of Public Works.

The scope of this investigation included evaluating approximately two miles of levees along the south (right) bank of the South Fork of Las Gallinas Creek and surrounding developments between Stations 0+00 and 108+00 in San Rafael, California. The purpose of our investigation was to explore subsurface conditions along the levee alignment and perform a geotechnical evaluation of existing levee conditions. The enclosed report contains a summary of our field explorations, laboratory testing, and engineering analyses, including a risk-based fragility assessment.

We appreciate the opportunity to provide our services for this project. If you have questions regarding this report or if we may be of further assistance, please contact us.

Respectfully submitted,

KLEINFELDER WEST, INC.


Craig A. Hall, PE, GE
Geotechnical Engineer



CAH/EMJ/es


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TABLE OF CONTENTS

<u>CHAPTER</u>	<u>PAGE</u>
EXECUTIVE SUMMARY	1
1. INTRODUCTION.....	7
1.1 GENERAL	7
1.2 PURPOSE OF PROJECT.....	7
1.3 DATUM.....	7
1.4 WATER SURFACE ELEVATION (WSE).....	8
1.4.1 General	8
1.5 FREEBOARD AND WAVE RUN-UP	8
2. PROJECT DESCRIPTION	10
2.1 PROJECT LOCATION AND DESCRIPTION.....	10
3. BACKGROUND INFORMATION	11
3.1 HISTORY OF LEVEE CONSTRUCTION.....	11
3.1.1 Reach 1.....	11
3.1.2 Reach 2.....	12
3.2 TYPICAL LEVEE GEOMETRY.....	12
3.3 PAST LEVEE PERFORMANCE.....	13
4. REVIEW OF EXISTING INFORMATION.....	16
5. SITE RECONNAISSANCE	17
6. SUBSURFACE INVESTIGATION.....	21
6.1 EXPLORATORY BORINGS	21
6.2 EXPLORATORY CPTS	22
6.3 PIEZOMETER INSTALLATION AND MONITORING	23
7. GEOTECHNICAL LABORATORY TESTING	25
8. SEISMICITY	26
9. SITE AND SUBSURFACE CONDITIONS.....	27
9.1 SITE CONDITIONS	27
9.1.1 Reach 1.....	27
9.1.2 Reach 2.....	27
9.2 SUBSURFACE CONDITIONS.....	28
9.2.1 Reach 1.....	28
9.2.2 Reach 2.....	29
10. GEOTECHNICAL ENGINEERING ASSESSMENT.....	30
10.1 OVERTOPPING	30
10.2 FUTURE SETTLEMENT.....	30
10.3 SEISMIC HAZARD.....	31
10.3.1 Liquefaction.....	31
10.3.2 Liquefaction-Induced Settlement	32
10.3.3 Lateral Spreading.....	33
10.3.4 Ground Lurching.....	33
10.3.5 Cyclic Densification	33
10.3.6 Ground Rupture.....	34
10.3.7 Strong Ground Shaking	34
11. RISK BASED ANALYSIS	35
11.1 METHODOLOGY AND APPROACH	36

11.1.1	Theoretical Underpinnings of Fragility Analysis	37
11.2	SEEPAGE	39
11.2.1	Seepage Reliability	39
11.2.2	Seepage Results	41
11.3	SLOPE STABILITY	44
11.3.1	Slope Stability Reliability	44
11.3.2	Cases Analyzed and Methods Used	45
11.3.3	Soil Strength.....	45
11.3.4	Independence of Material Properties	46
11.3.5	Result of Stability Analyses	46
11.4	JUDGMENT CURVES.....	50
11.4.1	Redwood Boxes	51
11.4.2	Encroachments	52
11.4.3	Animal Burrows	52
11.4.4	Erosion.....	52
11.4.5	Vegetation.....	53
11.4.6	Utilities	53
11.5	RESULTS OF RISK-BASED ANALYSES (SEEPAGE, STABILITY, AND JUDGMENT)	54
12.	LIMITATIONS	55
13.	REFERENCES.....	57

PLATES

1	Site and Boring Location Plan
2	Levee Alignment with Stationing
3	Settlement Monitoring Data (1962 to 2012)
4A – 4B	Piezometer Details
5	Seepage Fragility Analysis (4-foot Blanket)
6	Seepage Fragility Analysis (No Blanket)
7	Seepage Fragility Analysis (1-foot Blanket)
8	Results of Seepage Analysis (Existing Conditions: Case 1A “Expected” Case)
9a	Underseepage Probability of Failure (Full Scale)
9b	Seepage Probability of Failure (0 to 6 Percent Scale)
10	Slope Stability Fragility Analysis (Existing Conditions: Case 1A – “Expected” Case)
11	Results of Slope Stability Analysis (Existing Conditions: Case 1A – “Expected” Case)
12a	Stability Probability of Failure (Full Scale)
12b	Stability Probability of Failure (0 to 35 Percent Scale)
13	Judgment Probability of Failure
14	Combined Probability of Failure – Existing Conditions

APPENDICES

A	Logs of Borings (Plates A1 – A14)
B	Logs of Cone Penetration Tests
C	Laboratory Test Results (Plates C1 – C15)
D	Methodologies of Seepage and Stability Analysis
E	Risk Based Analysis

EXECUTIVE SUMMARY

This Executive Summary briefly outlines the major findings of our geotechnical investigation and risk-based analysis, and implications for future performance of the Las Gallinas Levee System (LGLS). Additional discussion of each item outlined below is presented in detail in the attached report, which should not be separated from this summary. This report includes conclusions regarding surface and subsurface conditions of the levees, potential failure modes for the levees, and the results of risk-based analysis (fragility analysis). All sections of this report should be reviewed to understand the results of the risk-based analysis.

Purpose of Study

The purpose of our geotechnical engineering investigation and analyses is to provide to the Marin County Flood Control and Water Conservation District (District) a preliminary summary of the project background information, subsurface conditions, geotechnical assessment of existing conditions of the LGLS, Reaches 1 and 2 (as defined below), and risk-based analyses of existing conditions. The results of our investigation and analyses are important to the community as they will be used to assess the integrity of the existing levee system surrounding Santa Venetia and will help provide a basis for assessing some of the various available alternatives to mitigate the levee system for flood protection. This information will also be made available to USACE for their use in assessing potential flood damages which could result in the event of a failure of the existing levee system.

Las Gallinas Levees

The LGLS, as defined for the purposes of this study, includes levees along the southern and eastern bank of the south fork of Las Gallinas Creek. LGLS partially surrounds the community of Santa Venetia located north and east of the City of San Rafael in eastern Marin County, California.

Stationing limits in this report reference a baseline along the inner (project) levee and does not necessarily match US Army Corps of Engineers (USACE) stationing for the exterior Santa Venetia Marsh Preserve Levee. Stationing begins with Station 0+00 at the southeastern end of the levee system near E. Vendola Drive near Pump Station #4 and increases northwestward to

Station 32+00 near Pump Station Number 5 at the northeast end of Vendola Drive (referred to in the report as Reach 1). The levee then extends to the southwest along Las Gallinas Creek to Station 108+00 at the southwest end of Vendola Drive (referred to in the report as Reach 2).

The LGLS was initially constructed by placing fill on tidal marshland. From historical topographic map research, it appears that initial fill was placed sometime between 1914 and 1942 (US Geological Survey, 1914 and US Army Corps of Engineers, 1942.) Additional fill was placed in the 1950s as part of the Santa Venetia residential development. The development, containing approximately 800 residences, was protected along its northern, western, and eastern boundaries by approximately two miles of earthen levee. The levee under consideration begins at high ground at the original, pre-development shoreline adjacent to San Pablo Bay, extends northwest along the border of Santa Venetia marsh, then parallels the right bank of the South Fork of Las Gallinas Creek for about one mile, then extends southeast to the southern end of Santa Margarita Island.

In response to flooding in 1983, the levee in Reach 1 was raised by placing earthen material on the existing levee crown, which resulted in steepened side slopes. At the same time, about 70 percent of Reach 2 received a redwood box-type floodwall at the top of the levee to raise the level of protection (Wood Rodgers, 2013).

In late 2008 a survey was conducted of the residents adjacent to the levee. The survey included questions regarding observed seepage and settlement, existing drainage improvements at the residents' properties, burrowing animals, vegetation, and sedimentation along the Las Gallinas Creek channel.

Subsurface Investigation

Based on the results of our subsurface investigation, which are detailed in Section 6, the levee lies on a foundation consisting of marsh deposits and a thick sequence (up to approximately 65 feet in depth) of soft, compressible bay sediments (locally referred to as Bay Mud), which has consolidated significantly since the levee's construction.

Water Surface Elevations

The United States Army Corps of Engineers (USACE) recently developed water surface elevations (WSEs) based on current hydraulic and hydrologic (H&H) modeling. Values provided by USACE (USACE-SPN, 2012) indicate that 100-year WSEs near Station 80+75 range from about Elevation 6.4 to 8.5 feet (NGVD29 datum). These WSEs model four different rates of sea level rise, from zero rise (present WSE) to about 2 feet of sea level rise (NRC Curve III).

For coastal levees, as required by FEMA criteria identified in the Code of Federal Regulations, Chapter 44, Section 65.10, minimum freeboard is determined by either one foot above the height of the one percent wave, or the maximum wave run-up (whichever is greater) associated with the 100-year stillwater surge elevation at the site. However, at a minimum, the freeboard must be at least two feet above the 100-year stillwater surge elevation.

Portions of the LGLS facing the bay can be exposed to wave run-up. According to the recent H&H study performed for USACE (USACE-SPN, 2012), areas of the inner marsh levee (known herein as the project levee) are susceptible to wave run-up from approximately Station 0+00 to 30+00. The maximum wave run-up calculated for the 2012 USACE report was for the NRC Curve III wave, at the 100-year return period, and was 2.6 feet in height. The mean wave height for year zero due to both San Pablo Bay waves and waves generated in the marsh is 0.9 feet (USACE-SPN, 2012). By FEMA criteria, the maximum wave run-up of 2.6 feet (NRC Curve III) is greater than the year zero condition and should be used as minimum freeboard.

For simplicity, we have applied the wave run-up freeboard criteria to the entirety of Reach 1, extending from Station 0+00 to Station 32+00. Consequently, in Reach 1, minimum freeboard will be the maximum wave run-up of 2.6 feet.

The remainder of LGLS in Reach 2 is not subject to any significant wave action, due to its distance away from the wave run-up. In Reach 2, total freeboard would be two feet above the stillwater tide elevation based on the guidelines in CFR Chapter 44, Section 65.10, as previously stated.

Geotechnical Engineering Assessment

Based on the results of our field investigation and laboratory testing, we have reviewed the existing conditions with respect to seepage, static stability, overtopping, future settlement and seismic hazards.

Based on the WSEs and minimum freeboard requirements, the existing levee in both Reaches 1 and 2 do not meet freeboard requirements.

The District has monitored settlement on the levee and residential streets within Santa Venetia since the early 1960s. Results of the monitoring data indicate that about two feet of settlement has occurred along the Reach 2 levee system over the past 40 years. Based on the settlement information provided by the District, settlement appears to be ongoing and no clear trend in a decrease in settlement is apparent at this time. We anticipate that future settlement will continue to be about 3 to 4 inches every 10 years for the next several decades. This assumes that no new loading, such as placement of new levee fill or construction of floodwalls, will occur. If new loads are placed on the levee, significantly greater settlement will occur.

During a major earthquake occurring on one of the nearby faults, strong to very strong ground shaking is expected to occur at the project site. Strong ground shaking during an earthquake can result in ground failure caused by soil liquefaction, lateral spreading, lurching or cyclic densification. Based on the results of our geotechnical investigation, it is our opinion that the potential for liquefaction to occur within the levee fill, landside fill, and alluvium underlying the Young Bay Mud is high where loose saturated granular deposits were encountered.

Settlement during a large seismic event is a result of the dissipation of excess pore water pressure generated by ground shaking. Based on the results of our engineering analyses, we anticipate the sites to experience up to four inches of liquefaction settlement with about two inches calculated in the overlying levee and landside fill, and two inches within the alluvial deposits underlying the Young Bay Mud. Because the relatively low permeable Young Bay Mud will likely reduce the potential for the dissipation of pore water pressure of the granular deposits underlying the Young Bay Mud from reaching the surface and thereby causing settlement, we anticipate that the settlement reaching the surface will be a maximum of approximately two inches due to the liquefaction of the fill material.

Lateral spreading is defined as the mostly horizontal movement of gently sloping ground (less than 5% surface slope) due to elevated pore pressures or liquefaction in underlying, saturated soils. Because the LGLS is situated adjacent to Las Gallinas Creek, the potential for lateral spreading is high and should be analyzed during the mitigation alternative assessment.

Steep slopes underlain by soft soils can deform laterally or lurch during an earthquake that can lead to cracking and slope failure depending on the height of the exposed slope. Because the existing levee system overlies the soft Young Bay Mud deposits, the potential for ground lurching is high, especially within Reach 1 where the levees are up to seven feet in height (compared to surrounding existing grade) and within Reach 2 where the existing levees are adjacent to Las Gallinas Creek.

Seismically induced compaction or densification of non-saturated sand or silt above the groundwater table due to earthquake vibrations may cause settlement. Although we anticipate the potential for cyclic densification beneath these areas to be high, we estimate settlement associated with cyclic densification in this relatively thin surface layer to be less than ¼ inch.

Fragility Analysis

Kleinfelder has performed the fragility (risk) analysis under guidance of the USACE assuming existing conditions of one (1) index point along the Las Gallinas Levee System along Reach 2, Station 55+50 (601 Vendola Drive). The analysis considered components of seepage, slope stability and engineering judgment to produce a combined fragility curve that represents the anticipated probability of failure at various water surface elevations for existing conditions.

This location was selected due to the subsurface conditions encountered in our borings, the topography in cross sections provided by Wood Rodgers, and prior evidence of water ponding on the landside of the levee at this location. Station 55+50 is about 3,000 feet from the location of the H&H analysis provided by USACE at Station 80+75 (USACE-SPN, 2012.) However, the WSEs at these two stations are not expected to vary significantly. Per communications with USACE, coastal stillwater elevations dominate the combined stage-probability curves, which may be applied equally to both stations.

The purpose of a risk based analysis is to assist in capturing and quantifying the uncertainty and risk inherent in the data used to formulate the individual analysis. The product developed in the geotechnical portion of the analysis is used in conjunction with the hydraulic risk data and economic consequence data to evaluate the relative effectiveness of possible alternatives. The fragility curve developed from the geotechnical analysis provides a combined (seepage, stability and geotechnical judgment) probability of failure. The analysis performed in this study consists of the existing condition analysis. Once alternatives are developed, another analysis is completed for the “improved” condition and the evaluation of the relative effectiveness in reducing the economic consequences can be made.

The total conditional probability of failure as a function of WSE has been developed by combining the probability of failure for three main failure modes; seepage, slope stability, and judgment. The reliability is the probability of no failure due to each mode considered in the calculations. The total probabilities of failure computed for the index point (i.e. cross section) are indicated in Plate 14.

The transition between the 15 percent probability of failure (PNP), marked with a dashed line on the graphs and the 85 percent probability of failure (PFP), marked with a solid black line, occurs over a change in water elevation of approximately two feet (from WSE 4.5 ft. to WSE 6.65 ft.). The primary drivers for the steepness of the combined fragility curve are the judgment factors. Appendix E provides a more detailed summary of the combined probabilities of the seepage, stability, and judgment conditional assessments for the various WSEs.

1. INTRODUCTION

1.1 GENERAL

Kleinfelder is pleased to present this Geotechnical Data Report summarizing our field investigation and geotechnical analyses of the Las Gallinas Creek Levee System (LGLS) in San Rafael, California. Specifically the levees addressed in this report are the levees protecting the community of Santa Venetia that is located adjacent to (south bank of) the Las Gallinas Creek South Fork. This report includes conclusions regarding surface and subsurface conditions of the levees, potential failure modes for the levees, and the results of risk-based analysis (fragility analysis).

1.2 PURPOSE OF PROJECT

The purpose of our geotechnical engineering investigation and analyses is to provide to the Marin County Flood Control and Water Conservation District (District) a summary of the project background information, subsurface conditions, geotechnical assessment of existing conditions of the LGLS, Reaches 1 and 2 (as defined below), and risk-based analyses of existing conditions. The results of our investigation and analyses are important to the community as they will be used to assess the integrity of the existing levee system surrounding Santa Venetia and will help provide a basis for assessing some of the various available alternatives to mitigate the levee system for flood protection. This information will also be made available to USACE for their use in assessing potential flood damages of the existing levee system.

1.3 DATUM

Elevation references in this report are in feet and are based on the National Geodetic Vertical Datum of 1929 (NGVD 29). Northing and Easting coordinates, where given, are based on the California Coordinate System Zone II and the 1983 North American Datum (NAD83).

1.4 WATER SURFACE ELEVATION (WSE)

1.4.1 General

The United States Army Corps of Engineers (USACE) recently developed water surface elevations (WSEs) based on current hydraulic and hydrologic (H&H) modeling. Values provided by USACE (USACE-SPN, 2012) indicate that 100-year WSEs near Station 80+75 range from about Elevation 6.4 to 8.5 feet (NGVD29 datum). These four WSEs include the current 100-year event WSE as well as modeling three different rates of sea level rise, from 0.5 feet (presented as historic sea level rise in USACE-SPN report) to about 2 feet of sea level rise (presented as NRC Curve III in USACE-SPN report). Various WSEs used in our analyses are further discussed in Section 11 of this report.

1.5 FREEBOARD AND WAVE RUN-UP

For coastal levees, as required by FEMA criteria identified in the Code of Federal Regulations, Chapter 44, Section 65.10, minimum freeboard is determined by either one foot above the height of the one percent wave, or the maximum wave run-up (whichever is greater) associated with the 100-year stillwater surge elevation at the site. However, at a minimum, the freeboard must also be at least two feet above the 100-year stillwater surge elevation.

FEMA follows the USACE guidance when it relates to the certification of levees, so the criteria established by FEMA governing the methodology to calculate the freeboard are based on the USACE wave run-up calculations. For this project, one percent wave run-up was calculated in previous USACE and USACE-sponsored studies.

Portions of the LGLS facing the Bay can be exposed to wave run-up. According to the recent H&H study performed for USACE (USACE-SPN, 2012), areas of the inner marsh levee (known herein as the project levee) are susceptible to wave run-up from approximately Station 0+00 to 30+00. The mean wave height anticipated along the project levees is due to waves generated in San Pablo Bay and waves generated in the marsh between the outer and inner levees. The maximum wave run-up calculated for the 2012 USACE report was for the NRC Curve III wave, at the 100-year return period, and is 2.6 feet in height. The mean wave height for year zero due to both San Pablo Bay waves and waves generated in the marsh is 0.9 feet

(USACE-SPN, 2012). By FEMA criteria, the maximum wave run-up of 2.6 feet (NRC Curve III) is greater than the year zero condition and should be used as minimum freeboard.

The USACE study on the water surface elevations (WSEs) for the Las Gallinas levees (USACE-SPN, 2013) develops the probabilistic WSEs for two approaches, time series and event based. Cumulative distribution functions (CDF) and probability density functions (PDF) were developed for both year zero and year 50 condition and considered climate change according to USACE Engineering Circular (EC) 1165-2-212 October 2011, Sea-Level Change Considerations for Civil Works Programs. Further discussions with USACE’s San Francisco District will be needed to be able to compare these CDF and PDF WSEs as they relate to freeboard and crest elevation requirements listed in Table 1.1.

For simplicity, we have applied the wave run-up freeboard criteria to the entirety of Reach 1, extending from Station 0+00 to Station 32+00. Consequently, in Reach 1, minimum freeboard will be the maximum wave run-up of 2.6 feet.

The remainder of LGLS in Reach 2 is not subject to any significant wave action, as these areas are not facing the Bay and will not be subjected to wave run-up. In Reach 2, total freeboard will be two feet above the stillwater tide elevation based on the guidelines in CFR Chapter 44, Section 65.10, as previously stated.

Table 1.1 summarizes minimum required crest elevations for both Reach 1 and Reach 2. The minimum required crest elevations shown in Table 1.1 are based on a stillwater tide elevation of about 6.4 feet (USACE-SPN, 2013).

Table 1.1 – Minimum Required Crest Elevations

Reach	Stillwater Tide Elevation (feet)	Maximum Wave Run-up (feet)	Freeboard (feet)	Total Crest Elevation (feet, NGVD29)
1	6.4	2.6	N/A	9.0
2	6.4	N/A ¹	2	8.4

¹Non-tidal/wave run-up reach

2. PROJECT DESCRIPTION

2.1 PROJECT LOCATION AND DESCRIPTION

The LGLS, as defined for the purposes of this study, includes levees along the southern and eastern bank of the south fork of Las Gallinas Creek. LGLS partially surrounds the community of Santa Venetia located north and east of the City of San Rafael in eastern Marin County, California. A site vicinity map is shown on Plate 1.

Stationing begins with Station 0+00 at the eastern end of the levee system near E. Vendola Drive near Pump Station #4 and increases westward to Station 32+00 near Pump Station Number 5 at the north end of Vendola Drive. The levee then extends to the south along Las Gallinas Creek to Station 108+00 at the south end of Vendola Drive. The levee system with the project stationing is shown on Plate 2.

The stationing limits for LGLS Reaches 1 and 2 are summarized in Table 2.1.

Table 2.1 – Summary of LGLS Reaches

Reach	Station
1	Station 0+00 to Station 32+00
2	Station 32+00 to Station 108+00

The Santa Venetia Marsh Preserve pathway traverses the levee crown over the length of the levee in Reach 1. In Reach 2, the levee extends along the outside edge of existing residences' backyards along Vendola Drive.

3. BACKGROUND INFORMATION

3.1 HISTORY OF LEVEE CONSTRUCTION

The LGLS was initially constructed by placing fill on tidal marshland. From historical topographic map research, it appears that initial fill was placed sometime between 1914 and 1942 (US Geological Survey, 1914 and US Army Corps of Engineers, 1942.) Additional fill was placed in the 1950s as part of the Santa Venetia residential development. The development, containing approximately 800 residences, was protected along its northern, western, and eastern boundaries by approximately two miles of earthen levee. The levee under consideration begins at high ground at the original, pre-development shoreline adjacent to San Pablo Bay, extends northwest along the border of Santa Venetia marsh, then parallels the right bank of the south Fork of Las Gallinas Creek for about one mile, then extends southeast to the southern end of Santa Margarita Island.

Based on the results of our subsurface investigation, which are detailed in Section 6, the levee lies on a foundation consisting of marsh deposits and a thick sequence (up to approximately 65 feet in depth) of soft, compressible bay sediments (locally referred to as Bay Mud), which has consolidated significantly since the levee's construction. The District, which maintains the levee, has monitored settlement points within the development periodically since 1962. Settlement data collected by the District is shown on Plate 3. The results of the monitoring indicate that cumulative settlement of approximately two feet has occurred in some areas. The average rate of settlement in the 1960's and 1970's was approximately six inches every ten years. A slight decrease is evident in the settlement rate over time; the average rate of settlement from the period 1990 to 2012 is approximately three to five inches every ten years.

3.1.1 Reach 1

In 1983, in response to flooding resulting from extreme tidal events, the levee in Reach 1 was raised by placing earthen material on the existing levee crown. Placing additional fill on the levee has resulted in steepened levee side slopes through this reach. Since completing these improvements, the levee has continued to settle due to continuing consolidation of the underlying foundation materials.

3.1.2 Reach 2

The levee in Reach 2 extends behind private residences. In 1983, in response to flooding resulting from extreme tidal events, a redwood box-type temporary floodwall was installed atop about 5,549 lineal feet of the levee in Reach 2 (Wood Rodgers, 2013). The redwood boxes are about 2.5 to 3.2 feet wide, measured perpendicular to levee crest, and rise about 1 to 2 feet above the earthen levee crown. These redwood box structures were intended to raise the level of protection for the areas landside of the Reach 2 levees, and have provided some protection during high water events since their installation. These previously-constructed temporary levee improvements have been in place for almost 30 years and show signs of distress. It is our understanding that the District maintains, repairs and replaces these redwood boxes on an ongoing and as-needed basis with funds from its limited operations and maintenance budget. See Section 5 for site reconnaissance details.

3.2 TYPICAL LEVEE GEOMETRY

The LGLS is 10,800 feet long and between 2 and 6 feet in height, with an average height of 5 feet between the top of the levee and the landside toe in Reach 1, and 2 to 3 feet between the top of the redwood box portion of the levee and the landside toe in Reach 2.

Ground surveys were conducted at selected cross section locations by Wood Rodgers in 2008. Based on survey data, crown elevations of the earthen portion of the levee vary between approximately +8.2 and +8.5 feet (NGVD29) in Reach 1, and between approximately +5.4 and +7.7 feet (NGVD29) in Reach 2. Elevations of the top of the redwood box in Reach 2 vary between approximately +7.4 and +8.7 feet (NGVD29).

Landside toe elevations range from approximately +0.5 to +3.0 feet (NGVD29) in Reach 1, and from approximately +4.3 to +6.5 feet (NGVD29) in Reach 2. Waterside toe elevations range from approximately +3.4 to +4.5 feet (NGVD29) in both reaches. The levee crown width (for the earthen portion of the levee) varies from about 1.5 feet to 10 feet with average widths of 8 feet in Reach 1 and 3 feet in Reach 2. The earthen levee landside slope inclinations are approximately 2 horizontal (H) to 1 vertical (V) in Reach 1 and between approximately 6H:1V and 9H:1V in Reach 2. The earthen levee waterside slope inclinations are approximately 1.7H:1V in Reach 1 and between approximately 1.5H:1V and 3H:1V in Reach 2.

A summary of the elevations at the levee crest and landside toe at the beginning and ending of each reach are presented in Table 3.1.

Table 3.1 – Summary of Las Gallinas Levee Elevations

Reach	Station	Centerline Elevation of Earthen Levee ^{1,2}	Redwood Box Elevation ¹	Landside Toe Elevation ¹
1	4+00	8.4	N/A	2.9
1	18+00	8.2	N/A	1.9
2	36+00	6.7	7.5	4.5
2	55+50	5.5	7.4	4.3
2	79+50	7.7	8.7	6.2

1) Elevations presented are NVGD29

2) In Reach 2, centerline elevations are referenced to the landside levee crest below the redwood box.

3.3 PAST LEVEE PERFORMANCE

Periodic flooding has occurred in the LGLS area since its construction in the early- to mid-1900s. Extensive flooding in the 1940's and 1950's led to the creation of Zone 7 of the District. Further flooding was recorded in 1969. Three times in the past 20 years, the levee has been overtopped by high tide conditions and wind generated waves. This overtopping occurred once in 1982 and twice in 1983. During the January 1982 event, 50 homes were flooded. In January 1983, 160 homes were flooded, and in December 1983, 100 homes were flooded (Wood Rodgers, 2013.)

In late 2008 the District distributed a survey to residents of the Santa Venetia area whose homes are situated along the levee. The survey included questions regarding observed seepage and settlement, existing drainage improvements at the residents' properties, burrowing animals, vegetation, and sedimentation along the Las Gallinas Creek channel.

The general results of the surveys were as follows:

Landside Slope Instabilities – No significant failures reported; minor exceptions included:

- “Slumping” on the water side (39 Vendola Dr.).

- “Disappearance of landside slope beyond planter box due to subsidence, levee top has subsided.” (31 Vendola Dr.)

Drainage, ponding and possible seepage through levee:

- Ponding water noted in backyards, attributed to rainwater and poor drainage.
- Ponding used to be an issue, but corrected after mitigation occurred (e.g. drainage/re-grading/sump pump).
- Approximately half of the homeowners surveyed indicated that they had drainage systems, sump pumps, or other water mitigation measures installed on their property.
- General drainage problems were reported between 55 and 627 Vendola Dr.

Burrowing Animals:

- Reported for homeowners between 117 and 211 Vendola Dr. (in levee and backyards).

Vegetation Issue:

- Large eucalyptus tree on the downslope of the levee fell over. Reported to have occurred on January 4th, 2008. Roots are still in place (39 Vendola Dr.).
- Almost all homeowners report vegetation on the landside of the levee, ranging from grasses and ice plants to a few small trees.

Subsidence/Settlement:

- Many homeowners report settlement and cracking of their homes and yards.

Sedimentation:

- Many homeowners report increasing sediment deposition along the creek channel.
- Homeowners attribute sedimentation to a lack of dredging.

Previous mitigation measures implemented along Reaches 1 and 2 after flooding events have been discussed above and in previous sections. In summary, the following measures have been implemented:

- Increase in levee height by placing fill on top of existing levee along Reach 1 (1983).

- Increase in levee height by the construction of redwood box along top of existing levee along Reach 2 (1983). Redwood box was constructed as a temporary mitigation measure (1983).
- Installation of drainage systems, sump pumps, or other water mitigation measures installed by individual homeowners on their properties.

4. REVIEW OF EXISTING INFORMATION

Kleinfelder has completed previous studies and analyses of LGLS and has reviewed the following documents:

- Miller Pacific Engineering Group (Miller Pacific 2009), "Geotechnical Investigation, Marin County Flood Control, Zone 7 Pump Station #2, Vendola Drive, San Rafael, California," June 23, 2009.
- Wood Rodgers (Wood Rodgers 2013), "County of Marin, Las Gallinas Levee, Evaluation Study, Santa Venetia, CA, Marin County – Proposed Interim Design Water Surface and Top of Levee Elevation Based Upon Previous Hydraulic Studies," June 3, 2013.
- Kleinfelder, Inc. (Kleinfelder 2006a), "Geotechnical Investigation, Pump Plant No. 1, San Rafael, Marin County, California," February 1, 2006.

5. SITE RECONNAISSANCE

In January 2006 the USACE conducted site reconnaissance of the Las Gallinas Levee System. Their observations and interpretations are detailed in their Site Observation report (USACE 2006). In general, their observations are consistent with our observations discussed below.

Prior to our investigation at the project site, the District, Kleinfelder and Wood Rodgers developed and mailed written surveys/questionnaires to the residents of Santa Venetia to gain insight on the current and past performance of the levee system. These surveys included questions regarding occurrence of through seepage, poor drainage, presence of vegetation and animal burrows on/in the levee, current encroachments, and overall past performance of the levee system. The results of these surveys are discussed in Section 3.

Based on the answers provided in these surveys, Kleinfelder prepared a site reconnaissance and subsurface investigation program to further characterize areas of interest. Kleinfelder, Wood Rodgers and County personnel performed a site reconnaissance of the LGLS on October 21, 2008. Information from this site reconnaissance was used to plan and coordinate the subsurface investigation program and was also used to develop the judgment curves for fragility analysis, discussed in Section 11 of this report. In the month leading up to our site reconnaissance, the Las Gallinas area received less than about ½ inch of precipitation. No storm events occurred in the week leading up to our visit or during our visit. To assist in our assessment of the existing conditions, we noted the following elements, if present, during our reconnaissance:

- Levee surfacing and surface geology;
- Evidence of levee settlement;
- Presence and condition of redwood boxes (installed as an interim measure in 1980s to raise levee crest and provide some measure of freeboard during high tide);
- Evidence of filling and/or excavation, tilling, piping, placement of utility poles, and other encroachments;
- Vegetation on the levee;

- Evidence of breaching, cracking, ruts, or depressions; and
- Evidence of ponded water, seepage or sand boils at landside levee toe.

Kleinfelder and Wood Rodgers personnel photographed and tabulated areas of the levee where these items were noted, as described below.

For the Las Gallinas Creek Levee, items of geotechnical and civil engineering significance included the following:

- The redwood box was present from approximately 15 Vendola Drive to 35 Vendola Drive, from approximately 51 Vendola Drive to 79 Vendola Drive, and from approximately 101 Vendola Drive to 623 Vendola Drive;
- The redwood box showed signs of distress including some areas of rotted wood;
- The existing concrete floodwall had rotated outward south of the bridge on Meadow Drive, near 57 Meadow Drive. Based on our discussion with the homeowner, we understand flooding of the front (landside) yard occurred during episodes of high water. Flow was greatest on the northern side of the yard;
- From 5 Vendola Drive to 15 Vendola Drive the landside slope was relatively steep at approximately 2H:1V;
- At 7 Vendola Drive private property owners had installed an approximately 3-foot-tall block retaining wall on the landside levee slope, and concrete riprap on the waterside slope;
- At 15 Vendola Drive the waterside slope was very steep, approaching 1H:1V;
- A remnant concrete wall was observed in the vicinity of 19, 21 and 23 Vendola Drive;
- At 29 Vendola Drive the waterside slope was inclined at about 2H:1V. An additional 12 inches of height had been added to the original redwood box. The box has been in-filled with sand;
- At 29 Vendola Drive a small (approximately 12-inch-diameter) tree was observed on the landside levee slope;

- Outbuildings and decks were observed on the crest and waterside of the levee at 55 and 57 Vendola Drive;
- Outbuildings were observed on the levee crest at 69 and 71 Vendola Drive;
- At 79 Vendola Drive the homeowner indicated the measured high water level during high tide with a storm event was 10 to 12 inches below the crest of the redwood box;
- Pump Station #2 was located between 79 and 101 Vendola Drive;
- According to the homeowner at 101 Vendola Drive, the western edge of the property had settled approximately 18 inches;
- At 119 Vendola Drive a large (approximately 24-inch-diameter) tree was adjacent to the levee on the landside;
- According to discussions with the homeowner at 209 Vendola Drive, rodents had tunneled through the levee allowing water to flow in at high tide;
- Standing water at the landside toe was visible at 313 Vendola Drive;
- Standing water and salt evaporites at the landside toe were observed at 505 Vendola Drive;
- Seepage at the landside toe was observed at 601 Vendola Drive;
- A 1- to 3-inch-diameter drainage pipe was observed extending over the levee crest at 601 Vendola Drive. No flow was observed during our site visit. Evidence of previous erosion was observed on the waterside slope near the drainage pipe outlet. The observed eroded waterside slope appears to have occurred from past flows from the drainage pipe;
- District-maintained portions of the levee extended from Pump Station No. 5 to the southwest, near the intersection of Vendola Drive and Adrian Way;
- A gravel roadway was observed along the District-maintained levee crest ;
- Public access levee (Stations 0+00 to 32+00) waterside and landside slopes were approximately 2H:1V;

- Ponded water was observed along the existing ditch at the landside toe of the public access portions of the levee (Stations 0+00 to 32+00). It is not clear if this water was the result of seepage, tidal influence, or irrigation runoff collecting at the levee toe.
- Four pump stations had pipelines which discharge through the levee midway down the waterside slope. These could provide flow paths for backflow or seepage around the pipelines during high water events.
- A street drainage pipe outfall located at the end of La Playa Way discharged at the landside levee toe. A portable pump was staged here to carry the flow over the levee.
- At 15 Vendola Drive, a 3- to 4- inch diameter drainage pipe was observed extending over the levee crest and down the waterside face of the levee.

In summary, based on this field reconnaissance, Kleinfelder believes the observed existing conditions that are pertinent to geotechnical analysis of these levees (further detailed in Sections 10 and 11 of this report) include:

- Seepage
- Poor Drainage
- Animal burrows and vegetation penetrating the existing levee
- Condition and maintenance of the redwood boxes
- Utilities through the existing levee
- Utilities crossing the crest of the existing levee
- Water flow along inboard toe of existing levee

6. SUBSURFACE INVESTIGATION

Based on the results of Kleinfelder's review of available information, the information provided in responses to the questionnaire, and existing conditions assessed during the site reconnaissance, Kleinfelder developed and conducted a field exploration program to evaluate subsurface conditions along the subject levee.

The geotechnical field exploration program consisted of eleven (11) borings and five (5) CPTs, as shown on Plate 1. Borings were conducted primarily to assess the condition of the levee fill materials and the soils immediately underlying the levee fill. CPTs were used to determine the landside field subsurface conditions, specifically the thickness of fill and Bay Mud.

In addition to this investigation a separate investigation was conducted on the County's property to the northeast of San Rafael Airport, as shown on Plate 1. This separate investigation included two (2) borings and two (2) CPTs which have been included in Appendix A of this report for reference.

6.1 EXPLORATORY BORINGS

The site was explored by drilling 11 soil borings (KC-1 through KC-10 and KT-3) along the subject levee alignment to a maximum depth of 19.5 feet below existing ground surface (bgs).

- Borings KC-1, KC-2, KC-3, KC-6 and KC-7 were drilled on the levee crest along Vendola Drive in the backyards of private residences in Reach 2. These borings were drilled between November 3 and November 17, 2008 by Access Soil Drilling of San Mateo, California (Access).
- Boring KT-3 was drilled at the landside levee toe in the backyard of the private residence at 813 Vendola Drive. This boring was drilled on November 11, 2008 by Access.
- Borings KC-4, KC-5, KC-8, KC-9 and KC-10 were drilled on the levee crest of the public access portion of the levee (Stations 0+00 to 32+00) in Reach 1. These borings were drilled on November 6, 2008 by Exploration Geoservices of San Jose, California (EGS).

In addition, two borings (K-AP1 and K-AP2) were drilled on the levee crest northeast of the San Rafael Airport runway. These borings were drilled on November 4, 2008 by Access and are included in this report (Appendix A) for reference.

Access drilled Borings KC-1, KC-2, KC-3, KT-3, KC-6, KC-7, K-AP1 and K-AP2 using a portable tripod drill rig equipped with a 4-inch-diameter solid stem auger. EGS drilled Borings KC-4, KC-5, KC-8, KC-9 and KC-10 using a truck-mounted Mobile B53 drill rig equipped with a 6-inch-diameter hollow stem auger.

Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance and transported to Kleinfelder's Oakland office for testing and storage. Selected samples were transported to Kleinfelder's Pleasanton laboratory for additional testing.

A key to the Logs of Borings is presented on Plate A-1. Logs of Borings are presented on Plates A-2 through A-14.

6.2 EXPLORATORY CPTS

Cone penetration tests (CPTs) were performed at five (5) locations landward of the subject levee alignment, as shown on Plate 1. The CPTs were performed between November 3 and November 5, 2008 by Gregg Drilling of Martinez, California (Gregg).

- CPTs KCPT-1 through KCPT-3, ranging in depth from 65 to 70 feet, were performed along Vendola Drive approximately 100 to 150 feet from the landside levee toe.
- CPT KCPT-4, extending to a depth of 75 feet, was performed at the end of Rosal Way approximately 50 feet from the landside levee toe.
- CPT KCPT-5, extending to a depth of 82 feet, was performed at the end of La Playa Way approximately 50 feet from the landside levee toe.

In addition, CPTs KCPT-A1 and KCPT-A2, both extending to a depth of 60 feet, were performed at the landside levee toe at the northeast end of the San Rafael Airport runway, as shown on

Plate 1. These CPTs are included in this report for reference as they provide continuity to the northern shore of Las Gallinas Creek.

Dissipation testing was performed in CPTs KCPT-2, KCPT-4, KCPT-5, and KCPT-A2. The results of these tests, with a guide to their interpretation, are presented in Appendix B.

CPT logs are presented in Appendix B and include soil behavior type (SBT), SPT N60 energy ratio, undrained shear strength, and unit weights. These elements are calculated and/or are interpreted based on algorithms presented in Robertson et al. (1986), Robertson (1990), and Lunne et al. (1997). Information regarding interpretation of CPT logs is also included in Appendix B.

6.3 PIEZOMETER INSTALLATION AND MONITORING

Upon completion of soil borings KC-3 and KT-3, piezometers were installed in the hole to measure tidally influenced groundwater depths. Piezometer development details for piezometers KC-3 and KT-3 are shown on Plates 4-A and 4-B, respectively.

Piezometer KC-3 was installed to a depth of 20.0 feet below ground surface. The piezometer consisted of 2"-diameter PVC pipe, solid for the upper 10 feet and slotted from 10 to 20 feet depth. The annulus was filled with #3 sand from a depth of approximately 8 feet to 20 feet bgs, bentonite chips from approximately 3 feet to 8 feet bgs, and neat cement from approximately 0 feet to 3 feet bgs.

Piezometer KT-3 was installed to a depth of 15.0 feet below ground surface. The piezometer consisted of 2"-diameter PVC pipe, solid for the upper 5 feet and slotted from 5 to 15 ft. depth. The annulus was filled with #3 sand from a depth of approximately 2.5 to 15 feet bgs, bentonite chips from approximately 3 inches to 2.5 feet bgs, and neat cement from approximately 0 to 3 inches bgs.

Following installation of the piezometers, water level probes were installed in KC-3 and KT-3 to collect tide-influenced groundwater level measurements. Water level measurements were collected for the period from November 11, 2008 through November 21, 2008.

The results of the water level monitoring indicate tidally influenced fluctuations in the groundwater table of up to 0.4 feet beneath the levee crest (KC-3) and up to about 0.2 feet beneath the levee landside toe (KT-3). Due to difficulties developing the piezometers and the potential for Young Bay Mud residue clouding the water column in both piezometers, it is possible that tidal fluctuations may have a more pronounced effect (i.e. greater tidal fluctuations) on the groundwater table than the variations indicated by our monitoring program.

7. GEOTECHNICAL LABORATORY TESTING

Representative samples obtained from the exploration boring program were tested at Kleinfelder's laboratory in Pleasanton, California. Testing included moisture content and unit weight, Atterberg limits, sieve analyses, unconsolidated-undrained triaxial shear strength (TXUU) and consolidation tests. Specifically, the following tests with their respective ASTM designations were performed.

- Moisture Content and Unit Weight (ASTM D 2937)
- Atterberg Limits (ASTM D 4318)
- Sieve (ASTM D 422)
- Unconsolidated-undrained triaxial shear (TXUU) (ASTM D 2850)
- Consolidation (ASTM D 2435)

The results of the geotechnical laboratory tests are shown on the boring logs in Appendix A. Detailed laboratory results of the tests are presented in Appendix C.

8. SEISMICITY

The site is located within the seismically active North Bay/North Coast region of California and is subject to seismically induced ground shaking from nearby and distant faults. Several faults have been mapped in the general site vicinity. The San Andreas fault zone, located southwest of the site, is the boundary between two tectonic plates: the Pacific Plate (west of the fault) and the North American Plate (east of the fault). At this boundary, the Pacific Plate is moving north relative to the North American Plate. In the North Coast region of California, this movement is distributed across a complex system of predominantly strike-slip, right-lateral, parallel, and sub-parallel faults that include the San Andreas, Hayward, and Calaveras, among others.

The site is not located within an Earthquake Fault Zone as defined by the California Geological Survey (CGS) in accordance with the Alquist-Priolo Earthquake Fault Zone Act of 1972. The nearest known active fault is the Hayward fault, located approximately 10 kilometers (km) southeast of the site, which is capable of producing a maximum earthquake magnitude event of M7.2. Moderate to major earthquakes generated on the Hayward fault can be expected to cause strong ground shaking at the site. Strong ground shaking can also be expected from moderate to major earthquakes generated on other faults in the region such as the Healdsburg-Rogers Creek fault (located 16.7 km northeast of the site), and San Andreas fault (located 18.5 km west of the site).

A number of large earthquakes have occurred within this region in the historic past. Some of the significant nearby events include two 1969 Santa Rosa earthquakes (M5.6, 5.7), the 2000 Yountville earthquake (M5.2), and the 1906 San Francisco earthquake (M8+). Future seismic events in this region can be expected to produce strong seismic ground shaking at this site. The intensity of future shaking will depend on the distance from the site to the earthquake focus, magnitude of the earthquake, and the response of the underlying soil and bedrock.

9. SITE AND SUBSURFACE CONDITIONS

9.1 SITE CONDITIONS

9.1.1 Reach 1

Reach 1 (Stations 0+00 to 32+00) extends from the intersection of the levee with East Vendola Drive northwest toward Pump Station No. 5. The levee height in Reach 1 is typically about 5 to 7 feet above the landside toe with the levee crown ranging between approximately Elevation +8 and +9.5 feet (NGVD29). The landside toe ranges between about Elevation +2 and +3 feet (NGVD29). The levee crown width ranges between about 8 and 12 feet. The landside and waterside slopes are typically 2H:1V.

A ditch extends on the landside toe from Pump Station No. 5 to approximately the intersection of the levee with the end of Palmera Way. Water flowing out of the ground at the landside toe of the levee was been observed in the landside ditch in the vicinity of the end of Descanso Way and the end of Estancio Way, as documented by the District in early 2009. This flow followed a period of winter rains. The source of this flow was not determined at the time, but it may have been due to seepage or piping through the levee or erosion or flow around utility pipelines.

The surface condition of the levee in Reach 1 is generally gravel roadway and serves the public as a well-used walking trail. The slopes of the levee in Reach 1 are well maintained, with short grass and wetlands vegetation growing on the slopes. Vehicle access is possible along the levee crest for the entire reach.

9.1.2 Reach 2

Reach 2 (Stations 32+00 to 108+00) extends the length of Vendola Drive from Pump Station No. 5 southwest to the bridge at Meadow Drive. The total levee height (including redwood boxes) in Reach 2 is typically about 3 to 5 feet above the landside toe. The earthen levee crown ranges between approximately Elevation +5.5 and +7.7 feet (NGVD29). Elevations for the top of the redwood box in Reach 2 vary between approximately +7.4 and +8.7 feet (NVD29). The redwood boxes are about 2.5 to 3.2 feet wide, measured perpendicular to levee crest, and rise about 1 to 2 feet above the earthen levee crown. The landside toe ranges between about

Elevation +3 and +5 feet. The landside and waterside slopes are typically between 1.5H:1V and 2H:1V.

Approximately 80 percent of the levee within this reach has been modified by installing redwood box improvements to raise their crest elevation. The redwood boxes measure approximately 18 inches in width and 12 to 24 inches in height and have been backfilled with a mixture of gravel, sand, silt, and clay soils. Many homeowners have added stairs, boat ramps, fencing, and outbuildings on the slopes of the levee; planted vegetation in the redwood boxes; and otherwise modified the original levee geometry and character.

The waterside and landside slopes of the levees in Reach 2 are oversteepened and exhibit localized slumping. Wet areas and ponded water have been observed landside of the levee. Gopher holes and trees have penetrated the levee, and the slopes are significantly vegetated with plants and occasional trees.

Access to the levees in Reach 2 is limited due to the development of private residences in the area. Residences are within 20 to 100 feet of the levee in many locations, and vehicular access along the levee is not currently possible.

9.2 SUBSURFACE CONDITIONS

In general, the subsurface conditions consist of levee fill material overlying soft Young Bay Mud and other alluvial deposits consisting of varying thicknesses of clay, silt, sand, and gravel layers. Detailed subsurface conditions along the two Reaches are described below; detailed boring and CPT logs are presented in Appendices A and B, respectively.

9.2.1 Reach 1

Based on the soils encountered in Kleinfelder's subsurface exploration programs, the levee fill in Reach 1 is between about 7.5 and 14 feet thick and generally consists of layers of medium stiff to hard clay and silt with up to about 30 percent sand and layers of loose to very dense sand and gravel with clay. Underlying the levee embankment fill is 40 to 45 feet of soft, compressible Young Bay Mud. Underlying the Young Bay Mud is stiff clay and dense sand to the depths

explored. Groundwater was observed at depths of 4.5 to 7.5 feet below the existing ground surface at the location explored.

9.2.2 Reach 2

Based on the soils encountered in Kleinfelder's subsurface exploration programs, the levee fill in Reach 2 is between about 5 and 17 feet thick and generally consists of layers of soft to stiff lean clay and silt with up to about 30 percent sand and layers of very loose to medium dense sands and gravels with clay. Underlying the levee embankment fill is between 45 and 50 feet of Young Bay Mud. Underlying the Young Bay Mud is stiff clay and dense sand to the depths explored. Groundwater was observed at depths ranging from 2.0 to 5.5 feet below existing ground surface (bgs) at the time of drilling.

10. GEOTECHNICAL ENGINEERING ASSESSMENT

Based on the results of our field investigation and laboratory testing, we have reviewed the existing conditions with respect to seepage, static stability, overtopping, future settlement and seismic hazards. This section addresses overtopping, future settlement, and seismic hazards; seepage and stability are discussed in greater detail within the context of the risk-based assessment in Section 11.

10.1 OVERTOPPING

Based on the WSEs and minimum freeboard requirement discussed in Sections 1.3 and 1.4, the existing levees in both Reaches 1 and 2 do not meet freeboard requirements. Table 10.1 presents the minimum levee elevations for the two reaches with the corresponding minimum freeboard requirements.

Table 10.1 – Freeboard Requirements and Overtopping

Reach	Levee Crest Elevations Encountered in this Reach	Minimum Levee Crest Elevation ¹
1	+8.0 to +9.0 ²	+9.0
2	+6.5 to +8.5 ³	+8.4

1. See Table 1.1 for Minimum Required Crest Elevation calculations. Minimum Levee Crest Elevation = Stillwater Tide Elevation plus 2.6 feet in Reach 1, two feet in Reach 2.
2. Isolated survey points in Reach 1 indicate crest elevations of +9.1 to +9.4 feet, but these are not considered representative.
3. Including redwood boxes, where present.

10.2 FUTURE SETTLEMENT

The District has monitored settlement on the levee and residential streets within Santa Venetia since the early 1960s. Results of the monitoring data, as shown in Wood Rodgers (2013), indicate that about two feet of settlement has occurred along the Reach 2 levee system over the past 40 years. Based on the settlement information provided by the District and discussed in Section 3, settlement appears to be ongoing. The average rate of settlement in the 1960's and 1970's was approximately six inches every ten years. A slight decrease is evident in the settlement rate over time; the average rate of settlement from the period 1990 to 2012 is

approximately three to five inches every ten years. We anticipate that future settlement will continue to be about three to five inches every 10 years for the next several decades. This assumes that no new loading, such as placement of new levee fill or construction of floodwalls, will occur. If new loads are placed on the levee, significantly greater settlement will occur.

10.3 SEISMIC HAZARD

During a major earthquake occurring on one of the nearby faults, strong to very strong ground shaking is expected to occur at the project site. Sites with thick deposits of soft Bay mud (like this site) may amplify motions from low magnitude earthquake events, resulting in greater surface ground shaking. Strong ground shaking during an earthquake can result in ground failure caused by soil liquefaction, lateral spreading, lurching or cyclic densification.

10.3.1 Liquefaction

Liquefaction is a soil behavior phenomenon in which a saturated soil loses a substantial amount of strength due to high excess pore pressure generated by strong ground shaking. Soils located below the groundwater table that are geologically recent and relatively unconsolidated, such as uncompacted artificial fills, typically have high to very high susceptibility to liquefaction (Youd and Perkins, 1978). Sands and silty sands are particularly susceptible to liquefaction; silts and gravels may also be susceptible to liquefaction; and some sensitive clays have exhibited liquefaction-type strength losses (Updike et al., 1988). In general, compressible soils such as plastic silts or clays do not generate excess pore water pressure as quickly or to as great an extent as less compressible soils such as sands. Silty and clayey soils, therefore, tend to be less susceptible than sandy soils to liquefaction-type behaviors; even within sandy soils, the presence of finer-grained materials lessens susceptibility to liquefaction.

The potential for generation of excess pore water pressure and for loss of strength are also highly dependent on the density of soils. Density characteristics of soils in a deposit, notably sandy and silty soils, are reflected in the penetration resistance (N) measured during sampling in an exploratory boring. Using penetration resistance data to help assess liquefaction hazards due to an earthquake is considered a reasonable engineering approach (Seed and Idriss, 1982; Seed et al., 1985; National Research Council, 1985), because many of the factors that affect penetration resistance affect the liquefaction resistance of sandy and silty soils in a similar way.

The borings drilled at the site encountered saturated very loose to medium dense clayey sands, clayey gravels, and gravels within the levee fill zone. Outside of the levee area, in the landside field, saturated, very loose to medium dense clayey sands, clayey gravels, and gravels were encountered in the upper 10 feet of the fill and in the alluvium underlying the Young Bay Mud. The penetration resistance data measured in the soil borings and cone penetration tests indicate that the loose to medium dense, saturated sands, clayey sands and gravels within the levee fill can be expected to experience liquefaction under design-level ground shaking conditions.

Based on the results of our geotechnical investigation, it is our opinion that the potential for liquefaction to occur within the levee fill, landside fill, and alluvium underlying the Young Bay Mud is high where loose saturated granular deposits were encountered.

10.3.2 Liquefaction-Induced Settlement

Settlement during a large seismic event is a result of the dissipation of excess pore water pressure generated by ground shaking. Such dissipation produces consolidation within the soil that is manifested at the ground surface as settlement. Typically, the settlements occur after the ground shaking has ceased and as the pore water pressures dissipate (generally minutes to hours; however, some manifestations take months to occur). Volume changes may occur in both liquefied and non-liquefied zones, with significantly larger contributions to settlement expected from liquefied soils.

The amount of settlement at a given location will depend primarily on the thickness of liquefiable soils. We judge the liquefaction potential to be primarily within the saturated levee fill material. Levee fill materials may become saturated due to tidal fluctuations or flooding events. Based on the results of our engineering analyses, we anticipate the sites to experience up to four inches of liquefaction settlement with about two inches calculated in the overlying levee and landside fill, and two inches within the alluvial deposits underlying the Young Bay Mud. Because the relatively low permeable Young Bay Mud will likely reduce the potential for the dissipation of pore water pressure of the granular deposits underlying the Young Bay Mud from reaching the surface and thereby causing settlement, we anticipate that the settlement reaching the surface will be a maximum of approximately two inches due to the liquefaction of the fill material.

10.3.3 Lateral Spreading

Lateral spreading is defined as the mostly horizontal movement of gently sloping ground (less than 5% surface slope) due to elevated pore pressures or liquefaction in underlying, saturated soils. Structures at the head of the slide are sometimes pulled apart while those at the toe are subjected to buckling or compression of the foundation soil. Linear infrastructure, such as utility lines and roadways, are particularly susceptible to earthquake damage from lateral spreads at multiple locations (A. F. Rauch, 1997). Lateral spreading movements typically are greatest near a free-face (such as a levee adjacent to a creek) and diminish with distance from the free-face. Because the LGLS is situated adjacent to Las Gallinas Creek, the potential for lateral spreading is high and should be analyzed during the mitigation alternative assessment.

10.3.4 Ground Lurching

Steep slopes underlain by soft soils can deform laterally or lurch during an earthquake that can lead to cracking and slope failure depending on the height of the exposed slope. Because the existing levee system overlies the soft Young Bay Mud deposits, the potential for ground lurching is high, especially within Reach 1 where the levees are up to seven feet in height (compared to surrounding existing grade) and within Reach 2 where the existing levees are adjacent to Las Gallinas Creek.

10.3.5 Cyclic Densification

Seismically induced compaction or densification of non-saturated sand or silt above the groundwater table due to earthquake vibrations may cause settlement. The groundwater table is at approximately 2 to 7 feet, bgs; therefore the potential for cyclic densification should be confined to these shallow, near surface fills. Most of the soil above the groundwater table at the site consists of clay, silt and sand fill materials. Although we anticipate the potential for cyclic densification beneath these areas to be high, we estimate settlement associated with cyclic densification in this relatively thin surface layer to be less than ¼ inch.

10.3.6 Ground Rupture

No active or potentially active faults have been identified in the immediate vicinity of the project sites, according to the California Geologic Survey (formerly known as California Division of Mines and Geology), and the project area is not located within a State of California Earthquake Fault Zone. The nearest active fault is the Hayward fault, situated approximately 10 km southwest of the project site. Based on this information, it is our opinion that surface fault rupture hazard to proposed improvements at the site is nil.

10.3.7 Strong Ground Shaking

As discussed in Section 8, we expect the site will experience strong ground shaking during a major earthquake on any of the nearby faults.

The project site vicinity has experienced ground shaking from numerous small-magnitude and at least 15 moderate to large-magnitude (i.e., $M_w > 6$) earthquakes that have occurred in the greater San Francisco Bay region during the historic time period (approximately 190 years). Since the levee construction in the 1950s, the site has experienced ground shaking from only a few moderate-magnitude earthquakes, most recently the $M_w 7$ Loma Prieta earthquake in 1989. Significantly stronger ground shaking than that previously experienced at the site is expected to occur during anticipated future larger earthquakes. We calculated the peak ground acceleration of the underlying bedrock at the site to be about 0.34g. However, the soft and loose soils within the fill and Young Bay Mud will likely affect the characteristics of ground motions propagated to the ground surface from the top of the underlying deep, firm soils or bedrock. This amplification effect should be analyzed under future studies.

11. RISK BASED ANALYSIS

Kleinfelder has performed the fragility (risk) analysis under guidance of the USACE assuming existing conditions of one (1) index point along the Las Gallinas Levee System along Reach 2, Station 55+50 (601 Vendola Drive). The analysis considered components of seepage, slope stability and engineering judgment to produce a combined fragility curve that represents the anticipated probability of failure at various water surface elevations for existing conditions.

This location was selected due to the subsurface conditions encountered in our borings, the topography in cross sections provided by Wood Rodgers, and prior evidence of water ponding on the landside of the levee at this location. Station 55+50 is about 3,000 feet from the location of the H&H analysis provided by USACE at Station 80+75 (USACE-SPN, 2012.) However, the WSEs at these two stations are not expected to vary significantly. Per communications with USACE, coastal stillwater elevations dominate the combined stage-probability curves, which may be applied equally to both stations.

The purpose of a risk based analysis is to assist in capturing and quantifying the uncertainty and risk inherent in the data used to formulate the individual analysis. The product developed in the geotechnical portion of the analysis is used in conjunction with the hydraulic risk data and economic consequence data to evaluate the relative effectiveness of planned alternatives. The fragility curve developed from the geotechnical analysis provides a combined (seepage, stability and geotechnical judgment) probability of failure. The analysis performed in this study consists of the existing condition analysis. Once alternatives are developed, another analysis is completed for the “improved” condition and the evaluation of the relative effectiveness in reducing the economic consequences can be made.

The references used for risk based analyses are as follows:

- a. ETL 1110-2-547 “Introduction to Probability and Reliability Methods for Use in Geotechnical Engineering,” USCAE, 30 September 1995
- b. ETL 1110-2-556 “Risk Based Analysis in Geotechnical Engineering for Support of Planning Studies,” USACE, 28 May 1999 with Errata Sheet in 5 March 2003.

- c. ER 1105-2-101 “Risk Analysis for Flood Damage Reduction Studies,” USACE, 3 January 2006
- d. EM 1110-2-1619 “Risk-Based Analysis for Flood Damage Reduction Studies,” USACE, 1 August 1996.
- e. “Reliability and Statistics in Geotechnical Engineering,” G. Baecher and J. T. Christian, Wiley, 2003
- f. “Factors of Safety and Reliability in Geotechnical Engineering,” Duncan Michael J., ASCE, Journal of Geotechnical and Environmental Engineering, April 2000
- g. “Characterization of geotechnical variability,” Phoon, K.K. and Kulhawy, Fred, NRC, Canadian Geotechnical Journal, 36(4) 1999
- h. “Evaluation of geotechnical property variability,” Phoon, K.K. and Kulhawy, Fred, NRC, Canadian Geotechnical Journal, 36(4) 1999

11.1 METHODOLOGY AND APPROACH

This fragility study explicitly considered the effects of water level on seepage and slope stability. Only seepage and slope stability under steady state seepage conditions were analyzed. The USACE guidance on fragility analysis typically uses blanket theory for the evaluation of seepage, however the use of blanket theory implies a number of assumptions that don't apply with the stratigraphy found along this levee system and in addition blanket theory is limited to the analysis of underseepage and does not evaluate through seepage.

In our analysis we used Seep/W to analyze the seepage, both through and underseepage, which is allowed in the USACE guidance, however no explicit examples or guidance are provided. The methodology used for seepage and stability analyses are detailed in Appendix D.

The analyses presented in this report are based on six different WSEs at Station 55+50 (601 Vendola Drive). Our lowest WSE approximately corresponds to the elevation of the waterside levee toe (4.3 ft.), one of our intermediate WSEs approximately corresponds to the top of the earthen levee / bottom of the redwood box (6.0 ft.), and our highest WSE approximately corresponds to the top of the redwood box (7.4 ft.). In order to produce smooth fragility curves, a minimum of six points or water level elevations were used to produce the curves. Water level

elevations of +5, +5.5, 6, and 6.6 [between the maximum and minimum levels] were used in the analyses.

The effects of seepage and stability at the various water levels were considered independently and combined with a judgment curve to produce a combined curve to estimate the probability of failure, based on these independent and explicit analyses (seepage and slope stability) and various implicit factors (judgment). The methods used to develop the fragility curves are outlined below in a general discussion.

11.1.1 Theoretical Underpinnings of Fragility Analysis

The probability of failure was evaluated by assessing the foundation and embankment materials and assigning values for the probability moments of the random variables considered in the analyses. The First-Order-Second-Moment (FOSM) method, as recommended in ETL 1110-2-556, "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies" dated 28 May 1999, was generally followed during the evaluation of the existing conditions of each levee unit. In this approach, the uncertainty in performance is taken to be a function of the uncertainty in model parameters. The standard deviations of a performance function were estimated based on the expected values (means) and some form of standard deviation of the random variable means. Due to the limited number of data points with which to characterize each layer for permeability, shear strength and spatial variability, the "standard deviation" for each of the parameters was developed using the referenced data sources and engineering judgment. Two methodologies were available to estimate the standard deviation, first common coefficients of variation as given in Duncan (2000) can be used or as also given in Duncan (2000) the six sigma rule can be used. The six-sigma rule uses the difference between the highest conceivable value (HCV) and the lowest conceivable value (LCV) that is divided by 6, which is the number of standard deviations encompassing 99.7 percent of a normal distribution. For purposes of our study we used the six-sigma rule to estimate the standard deviations of our parameters.

The performance functions considered were embankment slope stability and seepage. The risk of failure due to internal erosion via seepage through the levee embankment was not considered as a performance function; however, it was considered as one of the judgment elements.

External erosion is not typically considered in the fragility analyses; however, this factor was considered in the context of the judgment curve.

The final result of the FOSM is a reliability index, Beta (β), representing the amount of standard deviation of the performance function by which the expected value exceeds the limit state. The limit state for the slope stability was defined using a factor of safety of 1.0, while the limit state for seepage was a critical gradient $i_{critical} = 0.8$. The standard deviation and variance of the performance function are calculated from the standard deviation and variance of the foundation and embankment parameters using the Taylor's series method based on a Taylor's series expansion of the performance function about the expected values. The partial derivatives were calculated numerically using an increment of plus and minus one standard deviation centered on the expected value. The variance of the performance function was obtained by summing the products of the partial derivatives of the performance function considering the variance of the corresponding parameters. For the existing condition of the levee, the probability of slope or underseepage failure ($Pr(f)$) was expressed as a function of Las Gallinas Creek water elevation and other factors including soil strengths, permeabilities, and subsurface stratification. Reliability (R) is defined as:

$$R = (1 - Pr(f))$$

The combined geotechnical conditional probability of failure, considering the probability of failure due to seepage failure, slope stability and judgment probability is

$$Pr(f) = 1 - ((1 - Pr(f)_{us}) \times (1 - Pr(f)_{st}) \times (1 - Pr(f)_{jd}))$$

Where: $Pr(f)$ = combined probability of failure

$Pr(f)_{seepage}$ = probability of failure due to seepage

$Pr(f)_{stability}$ = probability of failure due to slope stability (in steady state condition)

$Pr(f)_{judgement}$ = judgment probability of failure

A set of conditional-probability-of-failure versus floodwater-elevation graphs were developed as related to seepage, stability and judgment. The probability of geotechnical failure of a levee is conditional on the uncertainties associated with hydrologic and hydraulic aspects of determining

the water surface profile during a flood. This is generally accomplished for economic purposes through estimation of two index elevations for each levee reach within the study area. These index elevations are defined as follows:

The Probable Non-Failure Point (PNP) is the water elevation below which it is highly likely that the levee would not fail.

The Probable Failure Point (PFP) is the water elevation above which it is highly likely that the levee would fail.

The term “highly likely that the levee would fail” is defined by the USACE ETL as having 85% probability of occurrence. Therefore, the probability of failure at the PNP is 15% and the probability of failure at the PFP is 85%. A linear distribution is assumed for an economic model between the PNP and PFP.

11.2 SEEPAGE

11.2.1 Seepage Reliability

Seepage analyses were performed for the index point selected. Simplified subsurface stratigraphies were developed based on our current geotechnical investigations and are shown on Plates 5 through 7 with various blanket thicknesses up to four feet. Plate 7 shows the anticipated or “expected” case (i.e. a low permeable blanket thickness of one foot). The other two conditions shown (i.e. zero and four-foot blanket conditions) are presented to provide a range of conditions for the fragility analyses as described above. The lower permeability blanket thickness, soil type (for determination of the estimated permeability ratio), and anisotropy ratio were estimated based on subsurface information and engineering experience with similar levees. The parameters and ranges for the blanket thickness, hydraulic conductivity (i.e. permeability) ratio, and blanket layer anisotropy are provided in Table 11.1.

Table 11.1 – Seepage Parameters and Ranges

Permeability Value Range (feet per day)

	Low	Expected	High
Bay Mud	-	0.0028	-
SC (Levee fill & underlying layer)	-	0.28	-
Blanket (CL / ML)	0.0028	0.028	0.28

Anisotropy (Kv/Kh) Value Range

	Low	Expected	High
Bay Mud	-	0.1	-
SC (Levee fill & underlying layer)	0.01	0.1	1
Blanket (CL / ML)	-	0.25	-

Blanket Thickness (feet)

	Blanket Thickness (in feet)		
Blanket (CL / ML)	4	1	0 (No blanket)

Blanket thickness, which is the thickness of a comparatively low-permeability “blanket” layer beneath the levee (typically fine grained material such as clay or silt) that reduces upward seepage pressures, ranged from “no blanket” to four feet, with the “expected” case of one foot. Figure 7 presents the “expected” case.

Seepage analysis was performed using the finite element program Seep/W by Geo-Slope International, Ltd.

Finite element analyses within the SEEP/W program was used to provide input pore pressures for the steady state seepage stability analysis of the landside slopes. A critical gradient $i_c = 0.80$ was used. The phreatic line obtained by the finite element method (SEEP/W) was also used in the stability analysis for each water elevation (for the “expected” case).

Reliability analysis was performed using Taylor’s Series Method. In the Taylor method, random variables are quantified by their expected values, “standard deviations,” and correlation coefficients. The variations of the properties, such as horizontal permeability, permeability ratio and “blanket” thickness were used to generate the analysis data for the probability of failure

where the calculated vertical or local gradient, depending on whether there was a “blanket” present or not, was compared to the critical gradient.

$$\Pr(f) = P(i_{critical} < i_0)$$

Where,

$\Pr(f)$ – is the probability that the calculated gradient is lower than the critical gradient.

11.2.2 Seepage Results

A baseline seepage analysis, using the “expected” (or anticipated) values, was performed using SEEP/W, the results of which were used in the stability analysis (see Section 11.3). The variables used for seepage analysis input are, as previously described, the blanket thickness, the permeability ratio between the blanket and permeable layers, and the anisotropy of the blanket material. Table 11.1 presents the “expected” values of the soil properties, and the maximum and minimum values used in each of the reliability analyses. The results of the seepage analysis are presented in Table 11.2 below, where each case presented indicates a change in one of the three variables for the various WSEs (Elevations +4.3 through +7.4). Plate 8 presents a graphical result of the analysis for the four-foot blanket assuming a permeability ratio of 10 and a blanket anisotropy (k_v/k_h) of 0.25, assuming a WSE of +7.4.

The landside slope in this location is fairly gentle and there is not a pronounced levee toe. For this reason, in all models (both with and without blanket layers) the gradient was calculated at a point 20 feet from the levee crest where the gradients were found to be highest in the base case models.

The results of risk based analyses for seepage are graphically represented in Plate 9a by the probability of failure for each water surface elevation assumed. The results indicate that the probability of failure for seepage for existing conditions is very low. Since the probability of failure, as shown on Plate 9a, is relatively low when plotted on a full range probability graph, the scale was adjusted to between 0 and three percent probability in order to observe the change of probability with WSE (Plate 9b). This curve is combined with the stability and judgment curves (discussed below) to produce the combined probability of failure curve (See Section 11.5).

**Table 11.2 – Seepage Analysis Results
Risk Based Analysis**

Seepage with 7.4' WSE

Case	Water elevation	Permeability (ft/day)			Anisotropy		Varied Parameters			Gradient at Station 120
		Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket Kv/K _H	Bay Mud Kv/K _H	Ks/Kb	Blanket Thickness, z (ft)	SC Kv/K _H	
1A	7.4	0.28	0.028	0.0028	0.25	0.1	10	4	0.10	0.3047
2A	7.4	0.28	0.28	0.0028	0.25	0.1	1	4	0.10	0.0932
3A	7.4	0.28	0.0028	0.0028	0.25	0.1	100	4	0.10	0.5057
4A	7.4	0.28	0.028	0.0028	0.25	0.1	10	0	0.10	0.4731
5A	7.4	0.28	0.028	0.0028	0.25	0.1	10	1	0.10	0.1768
6A	7.4	0.28	0.028	0.0028	0.25	0.1	10	4	1.00	0.4168
7A	7.4	0.28	0.028	0.0028	0.25	0.1	10	4	0.01	0.1112

Seepage with 6.6' WSE

Case	Water elevation	Permeability (ft/day)			Anisotropy		Varied Parameters			Gradient at Station 120
		Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket Kv/K _H	Bay Mud Kv/K _H	Ks/Kb	Blanket Thickness, z (ft)	SC Kv/K _H	
1B	6.6	0.28	0.028	0.0028	0.25	0.1	10	4	0.10	0.2289
2B	6.6	0.28	0.28	0.0028	0.25	0.1	1	4	0.10	0.0767
3B	6.6	0.28	0.0028	0.0028	0.25	0.1	100	4	0.10	0.3787
4B	6.6	0.28	0.028	0.0028	0.25	0.1	10	0	0.10	0.4063
5B	6.6	0.28	0.028	0.0028	0.25	0.1	10	1	0.10	0.5455
6B	6.6	0.28	0.028	0.0028	0.25	0.1	10	4	1.00	0.3061
7B	6.6	0.28	0.028	0.0028	0.25	0.1	10	4	0.01	0.079

Seepage with 6.0' WSE

Case	Water elevation	Permeability (ft/day)			Anisotropy		Varied Parameters			Gradient at Station 120
		Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket Kv/K _H	Bay Mud Kv/K _H	Ks/Kb	Blanket Thickness, z (ft)	SC Kv/K _H	
1C	6	0.28	0.028	0.0028	0.25	0.1	10	4	0.10	0.1723
2C	6	0.28	0.28	0.0028	0.25	0.1	1	4	0.10	0.0612
3C	6	0.28	0.0028	0.0028	0.25	0.1	100	4	0.10	0.2826
4C	6	0.28	0.028	0.0028	0.25	0.1	10	0	0.10	0.3551
5C	6	0.28	0.028	0.0028	0.25	0.1	10	1	0.10	0.4257
6C	6	0.28	0.028	0.0028	0.25	0.1	10	4	1.00	0.2419
7C	6	0.28	0.028	0.0028	0.25	0.1	10	4	0.01	0.0549

Seepage with 5.5' WSE

Case	Water elevation	Permeability (ft/day)			Anisotropy		Varied Parameters			Gradient at Station 120
		Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket Kv/K _H	Bay Mud Kv/K _H	Ks/Kb	Blanket Thickness, z (ft)	SC Kv/K _H	
1D	5.5	0.28	0.028	0.0028	0.25	0.1	10	4	0.10	0.1215
2D	5.5	0.28	0.28	0.0028	0.25	0.1	1	4	0.10	0.0404
3D	5.5	0.28	0.0028	0.0028	0.25	0.1	100	4	0.10	0.1985
4D	5.5	0.28	0.028	0.0028	0.25	0.1	10	0	0.10	0.2727
5D	5.5	0.28	0.028	0.0028	0.25	0.1	10	1	0.10	0.3032
6D	5.5	0.28	0.028	0.0028	0.25	0.1	10	4	1.00	0.1796
7D	5.5	0.28	0.028	0.0028	0.25	0.1	10	4	0.01	0.0322

Seepage with 5.0' WSE

Case	Water elevation	Permeability (ft/day)			Anisotropy		Varied Parameters			Gradient at Station 120
		Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket Kv/K _H	Bay Mud Kv/K _H	Ks/Kb	Blanket Thickness, z (ft)	SC Kv/K _H	
1E	5	0.28	0.028	0.0028	0.25	0.1	10	4	0.10	0.058
2E	5	0.28	0.28	0.0028	0.25	0.1	1	4	0.10	0.015
3E	5	0.28	0.0028	0.0028	0.25	0.1	100	4	0.10	0.0985
4E	5	0.28	0.028	0.0028	0.25	0.1	10	0	0.10	0.1374
5E	5	0.28	0.028	0.0028	0.25	0.1	10	1	0.10	0.1429
6E	5	0.28	0.028	0.0028	0.25	0.1	10	4	1.00	0.0962
7E	5	0.28	0.028	0.0028	0.25	0.1	10	4	0.01	0

Seepage with 4.3' WSE

Case	Water elevation	Permeability (ft/day)			Anisotropy		Varied Parameters			Gradient at Station 120
		Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket Kv/K _H	Bay Mud Kv/K _H	Ks/Kb	Blanket Thickness, z (ft)	SC Kv/K _H	
1F	4.3	0.28	0.028	0.0028	0.25	0.1	10	4	0.10	0
2F	4.3	0.28	0.28	0.0028	0.25	0.1	1	4	0.10	0
3F	4.3	0.28	0.0028	0.0028	0.25	0.1	100	4	0.10	0
4F	4.3	0.28	0.028	0.0028	0.25	0.1	10	0	0.10	0.0202
5F	4.3	0.28	0.028	0.0028	0.25	0.1	10	1	0.10	0
6F	4.3	0.28	0.028	0.0028	0.25	0.1	10	4	1.00	0
7F	4.3	0.28	0.028	0.0028	0.25	0.1	10	4	0.01	0

Seepage Gradient Summary

WSE	7.4	6.6	6	5.5	5	4.3
Case 1	0.30	0.23	0.17	0.12	0.06	0.00
Case 2	0.09	0.08	0.06	0.04	0.02	0.00
Case 3	0.51	0.38	0.28	0.20	0.10	0.00
Case 4	0.47	0.41	0.36	0.27	0.14	0.02
Case 5	0.18	0.55	0.43	0.30	0.14	0.00
Case 6	0.42	0.31	0.24	0.18	0.10	0.00
Case 7	0.11	0.08	0.05	0.03	0.00	0.00

11.3 SLOPE STABILITY

11.3.1 Slope Stability Reliability

Stability risk analyses were performed on the same index point as the previously discussed seepage analyses. In general, slope stability is sensitive to changes in strength parameters, phi (the soil's angle of internal friction) and c (cohesion) of the Mohr-Coulomb soil model and the unit weight of the soil. These are the parameters that were varied in the slope stability reliability analysis. A maximum of two layers of the stratigraphy were chosen as the critical layers and their strength and unit weights were varied. The two layers were typically the existing levee material and the uppermost foundation/blanket material. The available subsurface data from CPTs and borings, both historical and current, does not provide enough laboratory data or reliable SPT blow count data to perform a statistical analysis of the soil strength and unit weight. Soil strength and unit weight values were estimated based on our knowledge of the properties of Young Bay Mud in the San Pablo Bay area, and on our engineering knowledge of typical sandy fill material. An expected value for statistical analysis is often taken as the mean value. However, in the case where a statistical variation cannot be calculated (i.e. due to a lack of data for proper statistical analysis) the modal should be taken as the expected value. The modal value was used as the base case and a variation in the properties were made.

For the cohesion parameter, the variations were not equal on both sides of the "expected" value, as the "expected" value was assumed to be toward the lower end of the range for the cohesion, thus an equal variation on either side of the "expected" value would have resulted in a negative value. A similar methodology was used for the drained friction angle and unit weight although the variations would not have resulted in negative values; however, the expected values for each were skewed to the lower end range of possible values.

11.3.2 Cases Analyzed and Methods Used

The cases analyzed for stability risk analyses considered long-term (drained) conditions with steady state seepage along the landside slope of the levee, as per USACE methodology (ETL 1110-2-556). Other conditions typically analyzed for the design and construction of levees including end-of-construction, rapid drawdown, and earthquake conditions, were not considered in the fragility analyses as these are not part of the USACE methodology. Note that all slope stability analyses for all cases will need to be analyzed during final design of this project.

The phreatic surface was developed for the steady state condition using the finite element program SEEP/W (See Section 11.2). The limit equilibrium computer program SLOPE/W was used to perform the stability analyses. Pore pressure distributions from the SEEP/W models were imported into the SLOPE/W models and served as the steady state seepage basis for the limit equilibrium analysis. Circular failure surfaces initiating through the embankment were assumed to be the dominant method of failure and both shallow and deep-seated rotational failures through the embankment and foundation soils were analyzed.

The analyses consisted of performing a search routine to identify the critical failure surface using Spencer's Method. The expected values for each of the materials were used in the initial (base case) stability analysis for the assumed stratigraphy as shown on Plate 10. The material properties were then varied sequentially, one variable at a time. The factor of safety for each of the material property variations was entered into an Excel spreadsheet and the probability of failure calculated taking into account the variation in factor of safety with variation in material parameters. The same sequence was performed for each of the assumed WSEs (i.e. creek stages) from Elevation +7.4 (top of levee) to Elevation +4.3 (the elevation where the probability of failure, for landside slope stability, is essentially zero or negligible).

11.3.3 Soil Strength

Soil strength parameters used in the stability analyses were the drained soil parameters as determined from the current and previous studies. The variations in the soil strength parameters and unit weight of the soils were obtained using methodologies described in the previous paragraphs, outlined in ETL 1110-2-556, and those proposed by G. Baecher and J. T. Christian in "Reliability and Statistics in Geotechnical Engineering" and engineering judgment based on

past experience with these soils. The values of the drained soil parameters (ϕ and c) and unit weight are presented in Table 11.3 below.

**Table 11.3 – Judgment Based Geotechnical Parameters
for Stability Analyses Fragility Curve
Risk Based Analysis**

	Friction Angle (degrees) Value Range		
	Low	Expected	High
Bay Mud	-	16	-
SC (Levee fill & underlying layer)	22	28	32
Blanket (CL / ML)	18	26	30

	Cohesion (psf) Value Range		
	Low	Expected	High
Bay Mud	-	200	-
SC (Levee fill & underlying layer)	0	50	300
Blanket (CL / ML)	0	50	200

	Unit Weight (pcf) Value Range		
	Low	Expected	High
Bay Mud	-	92	-
SC (Levee fill & underlying layer)	110	125	130
Blanket (CL / ML)	110	125	130

11.3.4 Independence of Material Properties

The Probability of Failure of a slope ($Pr(F)$) is defined as the probability that the critical failure surface could be loaded to the limit equilibrium state. This infers the slope is loaded to its maximum capacity. For this study, the variables for slope stability were considered independent and not assumed to be correlated to the parameters for seepage analyses.

11.3.5 Result of Stability Analyses

The results of the stability analysis are presented in Table 11.4. Plate 11 presents a graphical representation of the slope stability results for the “expected” case. Each case analyzed has an associated graphical result. Due to the number of analyses involved, we have not included graphical results of each analysis in this report.

**Table 11.4 – Slope Stability Analysis Results
Risk Based Analysis**

Static Slope Stability with 7.4' WSE

File Name	Case	Water elevation (feet)	Bay Mud			Clayey Sand (SC)			Blanket Layer (Clay/Silt)			Factor Safety
			Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	
SLOPE_Cases 1A-1F_2010-06-18	1A	7.4	92	200	16	125	50	28	120	50	26	1.8389
SLOPE_Cases 2A-2F_2010-06-18	2A	7.4	92	200	16	110	50	28	120	50	26	1.9478
SLOPE_Cases 3A-3F_2010-06-18	3A	7.4	92	200	16	130	50	28	120	50	26	1.8057
SLOPE_Cases 4A-4F_2010-06-18	4A	7.4	92	200	16	125	0	28	120	50	26	1.2972
SLOPE_Cases 5A-5F_2010-06-22	5A	7.4	92	200	16	125	300	28	120	50	26	4.1147
SLOPE_Cases 6A-6F_2010-06-22	6A	7.4	92	200	16	125	50	22	120	50	26	1.6794
SLOPE_Cases 7A-7F_2010-06-22	7A	7.4	92	200	16	125	50	32	120	50	26	1.9462
SLOPE_Cases 8A-8F_2010-06-22	8A	7.4	92	200	16	125	50	28	100	50	26	1.6984
SLOPE_Cases 9A-9F_2010-06-22	9A	7.4	92	200	16	125	50	28	125	50	26	1.8661
SLOPE_Cases 10A-10F_2010-06-22	10A	7.4	92	200	16	125	50	28	120	0	26	1.2212
SLOPE_Cases 11A-11F_2010-06-22	11A	7.4	92	200	16	125	50	28	120	200	26	2.5422
SLOPE_Cases 12A-12F_2010-06-22	12A	7.4	92	200	16	125	50	28	120	50	18	1.7653
SLOPE_Cases 13A-13F_2010-06-22	13A	7.4	92	200	16	125	50	28	120	50	30	1.8741

Static Slope Stability with 6.6' WSE

File Name	Case	Water elevation (feet)	Bay Mud			Clayey Sand (SC)			Blanket Layer (Clay/Silt)			Factor Safety
			Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	
SLOPE_Cases 1A-1F_2010-06-18	1B	6.6	92	200	16	125	50	28	120	50	26	2.0660
SLOPE_Cases 2A-2F_2010-06-18	2B	6.6	92	200	16	110	50	28	120	50	26	2.1980
SLOPE_Cases 3A-3F_2010-06-18	3B	6.6	92	200	16	130	50	28	120	50	26	2.0221
SLOPE_Cases 4A-4F_2010-06-18	4B	6.6	92	200	16	125	0	28	120	50	26	1.4755
SLOPE_Cases 5A-5F_2010-06-22	5B	6.6	92	200	16	125	300	28	120	50	26	4.2273
SLOPE_Cases 6A-6F_2010-06-22	6B	6.6	92	200	16	125	50	22	120	50	26	1.8608
SLOPE_Cases 7A-7F_2010-06-22	7B	6.6	92	200	16	125	50	32	120	50	26	2.2015
SLOPE_Cases 8A-8F_2010-06-22	8B	6.6	92	200	16	125	50	28	100	50	26	1.9015
SLOPE_Cases 9A-9F_2010-06-22	9B	6.6	92	200	16	125	50	28	125	50	26	2.0931
SLOPE_Cases 10A-10F_2010-06-22	10B	6.6	92	200	16	125	50	28	120	0	26	1.4703
SLOPE_Cases 11A-11F_2010-06-22	11B	6.6	92	200	16	125	50	28	120	200	26	3.4118
SLOPE_Cases 12A-12F_2010-06-22	12B	6.6	92	200	16	125	50	28	120	50	18	1.9667
SLOPE_Cases 13A-13F_2010-06-22	13B	6.6	92	200	16	125	50	28	120	50	30	2.1090

Static Slope Stability with 6.0' WSE

File Name	Case	Water elevation (feet)	Bay Mud			Clayey Sand (SC)			Blanket Layer (Clay/Silt)			Factor Safety
			Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	
SLOPE_Cases 1A-1F_2010-06-18	1C	6	92	200	16	125	50	28	120	50	26	2.2404
SLOPE_Cases 2A-2F_2010-06-18	2C	6	92	200	16	110	50	28	120	50	26	2.3649
SLOPE_Cases 3A-3F_2010-06-18	3C	6	92	200	16	130	50	28	120	50	26	2.1877
SLOPE_Cases 4A-4F_2010-06-18	4C	6	92	200	16	125	0	28	120	50	26	1.6902
SLOPE_Cases 5A-5F_2010-06-22	5C	6	92	200	16	125	300	28	120	50	26	4.4004
SLOPE_Cases 6A-6F_2010-06-22	6C	6	92	200	16	125	50	22	120	50	26	1.9834
SLOPE_Cases 7A-7F_2010-06-22	7C	6	92	200	16	125	50	32	120	50	26	2.3967
SLOPE_Cases 8A-8F_2010-06-22	8C	6	92	200	16	125	50	28	100	50	26	2.0425
SLOPE_Cases 9A-9F_2010-06-22	9C	6	92	200	16	125	50	28	125	50	26	2.2673
SLOPE_Cases 10A-10F_2010-06-22	10C	6	92	200	16	125	50	28	120	0	26	1.6241
SLOPE_Cases 11A-11F_2010-06-22	11C	6	92	200	16	125	50	28	120	200	26	3.5273
SLOPE_Cases 12A-12F_2010-06-22	12C	6	92	200	16	125	50	28	120	50	18	2.0980
SLOPE_Cases 13A-13F_2010-06-22	13C	6	92	200	16	125	50	28	120	50	30	2.2886

Static Slope Stability with 5.5' WSE

File Name	Case	Water elevation (feet)	Bay Mud			Clayey Sand (SC)			Blanket Layer (Clay/Silt)			Factor Safety
			Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	
SLOPE_Cases 1A-1F_2010-06-18	1D	5.5	92	200	16	125	50	28	120	50	26	2.3341
SLOPE_Cases 2A-2F_2010-06-18	2D	5.5	92	200	16	110	50	28	120	50	26	2.5044
SLOPE_Cases 3A-3F_2010-06-18	3D	5.5	92	200	16	130	50	28	120	50	26	2.2866
SLOPE_Cases 4A-4F_2010-06-18	4D	5.5	92	200	16	125	0	28	120	50	26	1.7254
SLOPE_Cases 5A-5F_2010-06-22	5D	5.5	92	200	16	125	300	28	120	50	26	4.5141
SLOPE_Cases 6A-6F_2010-06-22	6D	5.5	92	200	16	125	50	22	120	50	26	2.1152
SLOPE_Cases 7A-7F_2010-06-22	7D	5.5	92	200	16	125	50	32	120	50	26	2.5073
SLOPE_Cases 8A-8F_2010-06-22	8D	5.5	92	200	16	125	50	28	100	50	26	2.2088
SLOPE_Cases 9A-9F_2010-06-22	9D	5.5	92	200	16	125	50	28	125	50	26	2.3946
SLOPE_Cases 10A-10F_2010-06-22	10D	5.5	92	200	16	125	50	28	120	0	26	1.7601
SLOPE_Cases 11A-11F_2010-06-22	11D	5.5	92	200	16	125	50	28	120	200	26	3.5546
SLOPE_Cases 12A-12F_2010-06-22	12D	5.5	92	200	16	125	50	28	120	50	18	2.2454
SLOPE_Cases 13A-13F_2010-06-22	13D	5.5	92	200	16	125	50	28	120	50	30	2.4235

Static Slope Stability with 5.0' WSE

File Name	Case	Water elevation (feet)	Bay Mud			Clayey Sand (SC)			Blanket Layer (Clay/Silt)			Factor Safety
			Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	
SLOPE_Cases 1A-1F_2010-06-18	1E	5	92	200	16	125	50	28	120	50	26	2.5532
SLOPE_Cases 2A-2F_2010-06-18	2E	5	92	200	16	110	50	28	120	50	26	2.7641
SLOPE_Cases 3A-3F_2010-06-18	3E	5	92	200	16	130	50	28	120	50	26	2.4904
SLOPE_Cases 4A-4F_2010-06-18	4E	5	92	200	16	125	0	28	120	50	26	1.5072
SLOPE_Cases 5A-5F_2010-06-22	5E	5	92	200	16	125	300	28	120	50	26	4.7121
SLOPE_Cases 6A-6F_2010-06-22	6E	5	92	200	16	125	50	22	120	50	26	2.4522
SLOPE_Cases 7A-7F_2010-06-22	7E	5	92	200	16	125	50	32	120	50	26	2.8717
SLOPE_Cases 8A-8F_2010-06-22	8E	5	92	200	16	125	50	28	100	50	26	2.3903
SLOPE_Cases 9A-9F_2010-06-22	9E	5	92	200	16	125	50	28	125	50	26	2.7811
SLOPE_Cases 10A-10F_2010-06-22	10E	5	92	200	16	125	50	28	120	0	26	1.9846
SLOPE_Cases 11A-11F_2010-06-22	11E	5	92	200	16	125	50	28	120	200	26	3.6360
SLOPE_Cases 12A-12F_2010-06-22	12E	5	92	200	16	125	50	28	120	50	18	2.4068
SLOPE_Cases 13A-13F_2010-06-22	13E	5	92	200	16	125	50	28	120	50	30	2.8073

Static Slope Stability with 4.3' WSE

File Name	Case	Water elevation (feet)	Bay Mud			Clayey Sand (SC)			Blanket Layer (Clay/Silt)			Factor Safety
			Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	Unit Weight (pcf)	Cohesion (psf)	Friction Angle, Phi	
SLOPE_Cases 1A-1F_2010-06-18	1F	4.3	92	200	16	125	50	28	120	50	26	2.8781
SLOPE_Cases 2A-2F_2010-06-18	2F	4.3	92	200	16	110	50	28	120	50	26	3.0672
SLOPE_Cases 3A-3F_2010-06-18	3F	4.3	92	200	16	130	50	28	120	50	26	2.7576
SLOPE_Cases 4A-4F_2010-06-18	4F	4.3	92	200	16	125	0	28	120	50	26	1.5072
SLOPE_Cases 5A-5F_2010-06-22	5F	4.3	92	200	16	125	300	28	120	50	26	4.9901
SLOPE_Cases 6A-6F_2010-06-22	6F	4.3	92	200	16	125	50	22	120	50	26	2.5894
SLOPE_Cases 7A-7F_2010-06-22	7F	4.3	92	200	16	125	50	32	120	50	26	3.1181
SLOPE_Cases 8A-8F_2010-06-22	8F	4.3	92	200	16	125	50	28	100	50	26	2.7619
SLOPE_Cases 9A-9F_2010-06-22	9F	4.3	92	200	16	125	50	28	125	50	26	2.8483
SLOPE_Cases 10A-10F_2010-06-22	10F	4.3	92	200	16	125	50	28	120	0	26	2.1178
SLOPE_Cases 11A-11F_2010-06-22	11F	4.3	92	200	16	125	50	28	120	200	26	3.6556
SLOPE_Cases 12A-12F_2010-06-22	12F	4.3	92	200	16	125	50	28	120	50	18	2.7681
SLOPE_Cases 13A-13F_2010-06-22	13F	4.3	92	200	16	125	50	28	120	50	30	2.9257

Static Slope Stability Factor of Safety Summary

	A	B	C	D	E	F
Case	7.4	6.6	6.0	5.5	5	4.3
1	1.84	2.07	2.24	2.33	2.55	2.88
2	1.95	2.20	2.36	2.50	2.76	3.07
3	1.81	2.02	2.19	2.29	2.49	2.76
4	1.30	1.48	1.69	1.73	1.51	1.51
5	4.11	4.23	4.40	4.51	4.71	4.99
6	1.68	1.86	1.98	2.12	2.45	2.59
7	1.95	2.20	2.40	2.51	2.87	3.12
8	1.70	1.90	2.04	2.21	2.39	2.76
9	1.87	2.09	2.27	2.39	2.78	2.85
10	1.22	1.47	1.62	1.76	1.98	2.12
11	2.54	3.41	3.53	3.55	3.64	3.66
12	1.77	1.97	2.10	2.25	2.41	2.77
13	1.87	2.11	2.29	2.42	2.81	2.93

Note: Water surface was imported into model from SEEP/W using the base case of $K_{ratio} = 10$, Blanket of 4', $K_v/K_H = 0.1$

The results of risk based analyses for stability are graphically represented by the probability of failure for each water surface elevation assumed. Plate 12a presents the results of our stability analyses. The results indicate that the probability of failure for stability for existing conditions is very low. Because the probability of failure, as shown on Plate 12a, is relatively low when plotted on a full range probability graph, we increased the scale to be between 0 and 30 percent probability in order to observe the change of probability with WSE (Plate 12b). This curve will be combined with the seepage and judgment curves (discussed below) to produce the combined probability of failure curve (See Section 11.5).

11.4 JUDGMENT CURVES

This section presents the conditional probability of failure as a function of floodwater elevation for the risk-based assessment. A judgment-based conditional probability was based on historical characteristics, such as deterioration of the redwood box improvements, landside standing water/seepage, vegetation, animal burrows, and any other significant factors. In general, the judgment aspect tends to moderate the influence of one or more of the other factors explicitly considered in the analysis. The judgment assessment can provide a reality check of the results that do not, or appear to not, capture the consequences in an adequate

manner, concerning the levee, such as encroachments on the levee slopes, erosion, vegetation on the levee slopes, existing cracks and holes due to animal burrows, number/condition of levee penetrations, anticipated performance of the redwood box improvements, and on the past history of seepage/standing water on the landside of the existing levees.

Judgment curves can be developed by a variety of methods, ranging from a single individual using their experience with either the particular system or similar levee systems, to a panel of experts. With the panel of experts, a moderator or facilitator is tasked with developing the questions in a manner which will help reduce bias due to a myriad of factors (Vick, 2002). Whether an experienced individual or an expert panel is used to develop the judgment curve, the amount of data available regarding the factors to be considered in the judgment curve will influence the variability and results of the fragility analysis. For this report, the factors and reliability/probability of failure for the various factors are based on our collective experience and knowledge.

The factors we have assumed to be significant for the judgment assessment are briefly discussed below:

11.4.1 Redwood Boxes

As discussed in Section 3.1.2, redwood box temporary floodwalls were installed atop about 5,550 lineal feet of the levee in Reach 2. The current top of the redwood box varies from about Elevation 7.3 to 8.7 (NGVD29). These structures have been in place since the early to mid-1980s and are exhibiting signs of distress. It is our understanding that the District maintains, repairs and replaces these redwood boxes on an ongoing basis, with an average of about two to three properties assessed/maintained each year. The condition of the boxes may have a significant impact on the performance of the levee during flooding events. The use of redwood material or synthetic wood material (such as “Trex”), the long-term integrity or rotting of the redwood material, the presence of lateral reinforcement bars within the boxes, vegetation within the boxes, installation of fencing or other penetrations through the boxes, and compaction of soil within the boxes are all factors that can vary significantly from one property to the next and can impact the performance of the boxes. We estimated that the reliability of the redwood box will vary from approximately a 40% probability of failure (POF) for a WSE of 6.0 feet (near the base of the redwood box) and/or where annual maintenance occurs on the boxes to approximately a

80% POF for a WSE of 7.4 feet (at the top of the redwood box) and/or where maintenance of the boxes do not occur on an annual basis. Additionally we estimate that for a WSE of 5.5 feet, just below the base of the redwood box, the POF is 5% due to the possibility that such a WSE could weaken the embedded wooden foundation of the box resulting in failure.

11.4.2 Encroachments

With only minor exception, the District or County does not possess right-of way for levee maintenance on either the landside or waterside of the existing levees along Reach 2. An easement for the levee is not in place. Hence the possibility exists for the homeowner to significantly modify the existing levee without regard to the integrity of the existing system. During our site reconnaissance (see Section 5), we observed stairs, decks, and out-structures on top of and penetrating into the existing levee, vegetation planted by the homeowner and drainage lines through the levee. The ability for the homeowner to reduce the height of the existing levee is a possibility. The damage or destruction of such encroachments due to high water events could cause failure or lead to a breach that precipitates progressive failure of the surrounding levees. We estimate the effects of encroachment will vary from approximately 1% POF for a WSE of 4.3 ft. to 25% POF for a WSE of 7.4 ft.

11.4.3 Animal Burrows

Animal burrows, whether by gophers, squirrels, or other animals, were observed during our site reconnaissance within the existing levee system (See Section 5) and were reported by homeowners (Section 3.3). Animal burrows that progress across the entire levee width near the base of the existing levee represent the worst case for potential animal-burrow-induced flooding as the burrows provide an access point for flood waters to enter into the landside property, and with subsequent erosion of the levee by floodwater scour, greater flooding will occur. The number and size of the burrows that exist along the levee alignment are not known. We estimate the effects of animal burrows will vary from approximately 0% POF for a WSE of 4.3 ft. to 20% POF for a WSE of 7.4 ft.

11.4.4 Erosion

For this assessment, we have defined erosion as processes that occur due to wave action and/or creek currents that have the ability to undercut the levee and cause failure or precipitate

a progressive failure of the levee. For this assessment, erosional distress due to animal burrows, encroachments or vegetation has been considered as part of those separate judgment curve elements and is not included in the “Erosion” judgment curve. Water surface elevations of +5.0, +5.5 and +6.0 would result in wave action along the earthen embankment portion of the levee below the redwood box, and thus the probability of failure at these WSEs was judged to be the highest. As the wave action rises up to the midpoint or top of the redwood box, erosion effects are expected to be less severe. We estimate the effects of erosion will vary from approximately 2.5% POF for a WSE of 4.3 ft. to 15% POF for a WSE of 6 ft.

11.4.5 Vegetation

Vegetation exists on both the land and water sides of the existing levee system. Observed vegetation ranged from ankle high grasses and pickleweed to trees up to 20+ feet in height. Of the various vegetation encountered during our site reconnaissance, trees can cause the most damage to a levee system as the tree’s relatively large and extensive root system can provide a pathway for water to travel from the waterside to the landside when the WSE is at or above the elevations of the roots. In addition, trees on the levee that die and fall may cause the soil around its trunk and root system to break-up and be disturbed resulting in the area being exposed to potential erosion during high water event. Vegetation on the waterside slopes of the levee may also contribute to erosional scour around root balls and clumps of grasses. We estimate the effects of vegetation will vary from approximately 0 % POF for a WSE of 4.3 ft. to 10% POF for a WSE of 7.4 ft.

11.4.6 Utilities

During our site reconnaissance, utility lines, whether installed for homeowner’s use or by local municipalities, were observed through the existing levee. The only utility lines observed during our reconnaissance were drainage pipes, but it is possible that additional utility lines such as electrical conduits carrying power to docks or outbuildings may penetrate the existing levee. WSEs at or higher than the waterside exit points of the utilities can cause flood waters to travel around and possibly through these existing utility lines allowing flooding of landside property. The elevation of existing utility lines will influence the relative probability of failure, which would be zero percent below the elevation of the lowest occurring utility. We estimate the effects of

utilities will vary from approximately 0% POF for a WSE of 4.3 ft. to 15% POF for a WSE of 7.4 ft.

Plate 13 presents the judgment based curve for the analyzed levee cross section and is used in the combined probability of failure curve discussed above. It can be seen that the transition between the 15 percent probability of failure (PNP), marked with a dashed line on the graphs and the 85 percent probability of failure (PFP), marked with a solid black line, occurs over a change in water elevation of approximately one foot. The primary drivers for the steepness of the combined fragility curve are redwood boxes, encroachments and erosion. Appendix E provides a more detailed summary of the probabilities of the various components and WSEs.

11.5 RESULTS OF RISK-BASED ANALYSES (SEEPAGE, STABILITY, AND JUDGMENT)

The total conditional probability of failure as a function of WSE has been developed by combining the probability of failure for three main failure modes; seepage, slope stability, and judgment. The reliability is the probability of no failure due to each mode considered in the calculations. The total probabilities of failure computed for the index point (i.e. cross section) are indicated in Plate 14.

It can be seen that the transition between the 15 percent probability of failure (PNP), marked with a dashed line on the graphs and the 85 percent probability of failure (PFP), marked with a solid black line, occurs over a change in water elevation of approximately two feet (from WSE 4.5 ft. to WSE 6.65 ft.). The primary drivers for the steepness of the combined fragility curve are the judgment factors. Appendix E provides a more detailed summary of the combined probabilities of the seepage, stability, and judgment conditional assessments for the various WSEs.

12. LIMITATIONS

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

This report may be used only by County of Marin, Marin County Flood Control and Water Conservation District, Wood Rodgers, and the registered design professional in responsible charge and only for the purposes stated for this specific engagement within a reasonable time from its issuance, but in no event later than two (2) years from the date of the report.

The scope of services was limited to eleven geotechnical borings, five CPTs, and laboratory testing of selected soil samples. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on subsurface exploration including borings drilled to a maximum depth of 19.5 feet, CPTs conducted to a maximum depth of 82 feet, groundwater level measurements in soil boring holes during drilling, laboratory testing of soil plasticity, gradation, and engineering analyses.

Kleinfelder offers various levels of investigative and engineering services to suit the varying needs of different clients. Although risk can never be eliminated, more detailed and extensive studies yield more information, which may help with the understanding and management of the level of risk. Since detailed study and analysis involves greater expense, our clients participate in determining levels of service, which provide information for their purposes at acceptable levels of risk. The client and key members of the design team should discuss the issues covered in this report with Kleinfelder, in order that the issues are understood and applied in a manner consistent with the owner's budget, tolerance of risk, and expectations for future performance and maintenance.

Conclusions and/or recommendations contained in this report are based on our field observations and subsurface explorations, limited laboratory tests, and our present knowledge of the proposed project. It is possible that soil or groundwater conditions could vary between or beyond the points explored. If soil or groundwater conditions are encountered during construction that differ from those described herein, the client is responsible for ensuring that Kleinfelder is notified immediately in order that we may reevaluate the recommendations of this report. If the scope of the proposed construction, including the estimated additional height of the levee, changes from that described in this report, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed and the conclusions of this report are modified or approved in writing by Kleinfelder.

Kleinfelder cannot be responsible for interpretation by others of this report or the conditions encountered in the field.

The scope of services for this subsurface exploration and geotechnical report did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous substances in the soil, surface water, or groundwater at this site. The scope of services did not include topographic survey of the levee system nor preparation of a topographic base plan for the project.

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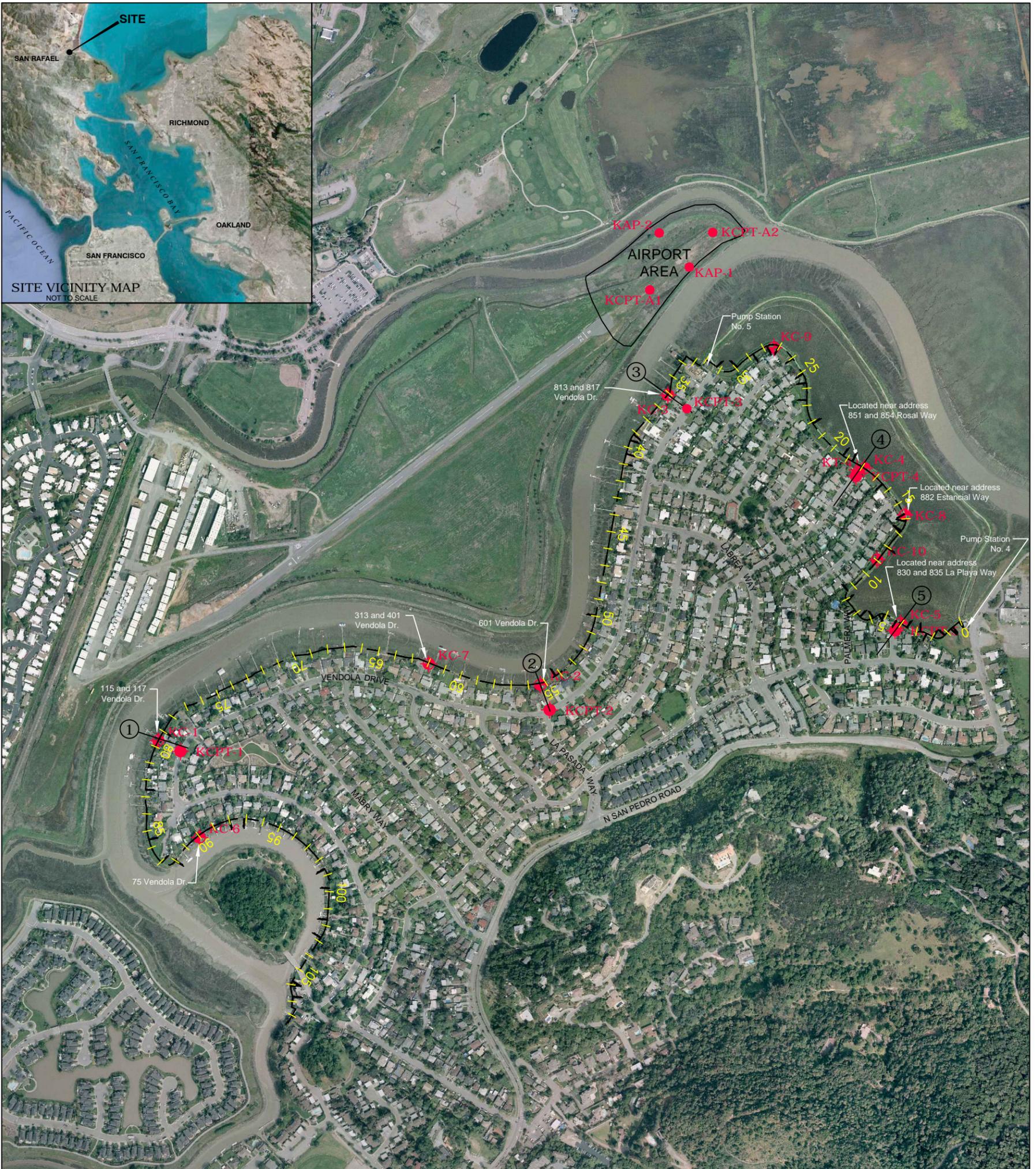
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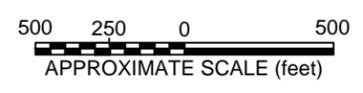
PLATES

SITE AND BORING LOCATION PLAN



LEGEND

- ① — PROPOSED INVESTIGATION SECTIONS
- PROPOSED BORINGS
- ◆ KC-1 LEVEE CROWN BORINGS
- ▲ KT-4 LEVEE TOE BORING/OBSERVATION WELL
- KCPT-1 CONE PENETRATION TEST (CPT)
- KAP-1 AIRPORT BORING
- KCPT-A1 AIRPORT CPT



LAS GALLINAS LEVEE INVESTIGATION
 GEOTECHNICAL DATA REPORT
 SAN RAFAEL, CALIFORNIA

LEVEE ALIGNMENT

LAS GALLINAS LEVEE SYSTEM

MARIN CO. FLOOD CONTROL & WATER CONSERVATION DISTRICT

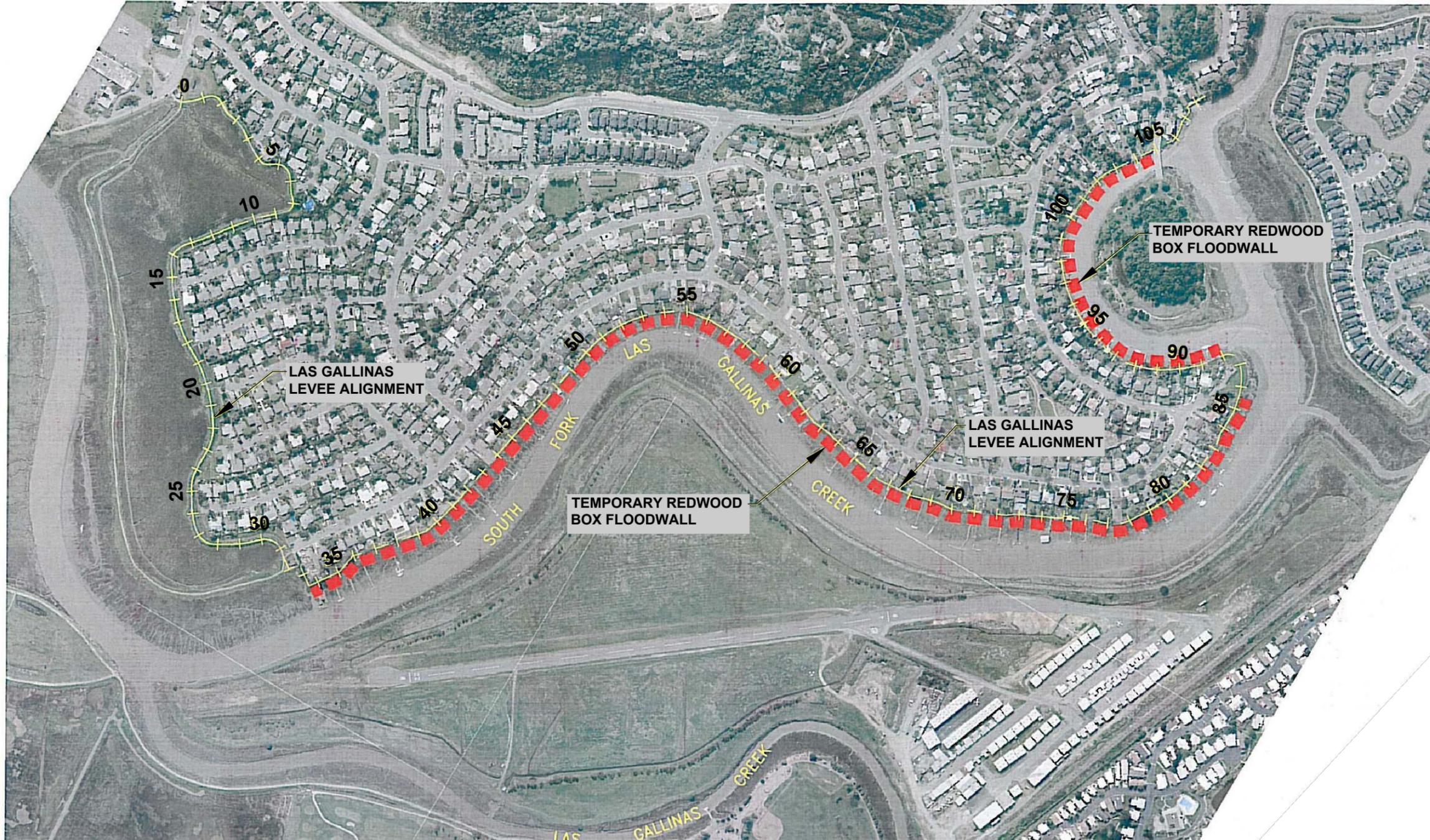
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CALIFORNIA

DECEMBER, 2008



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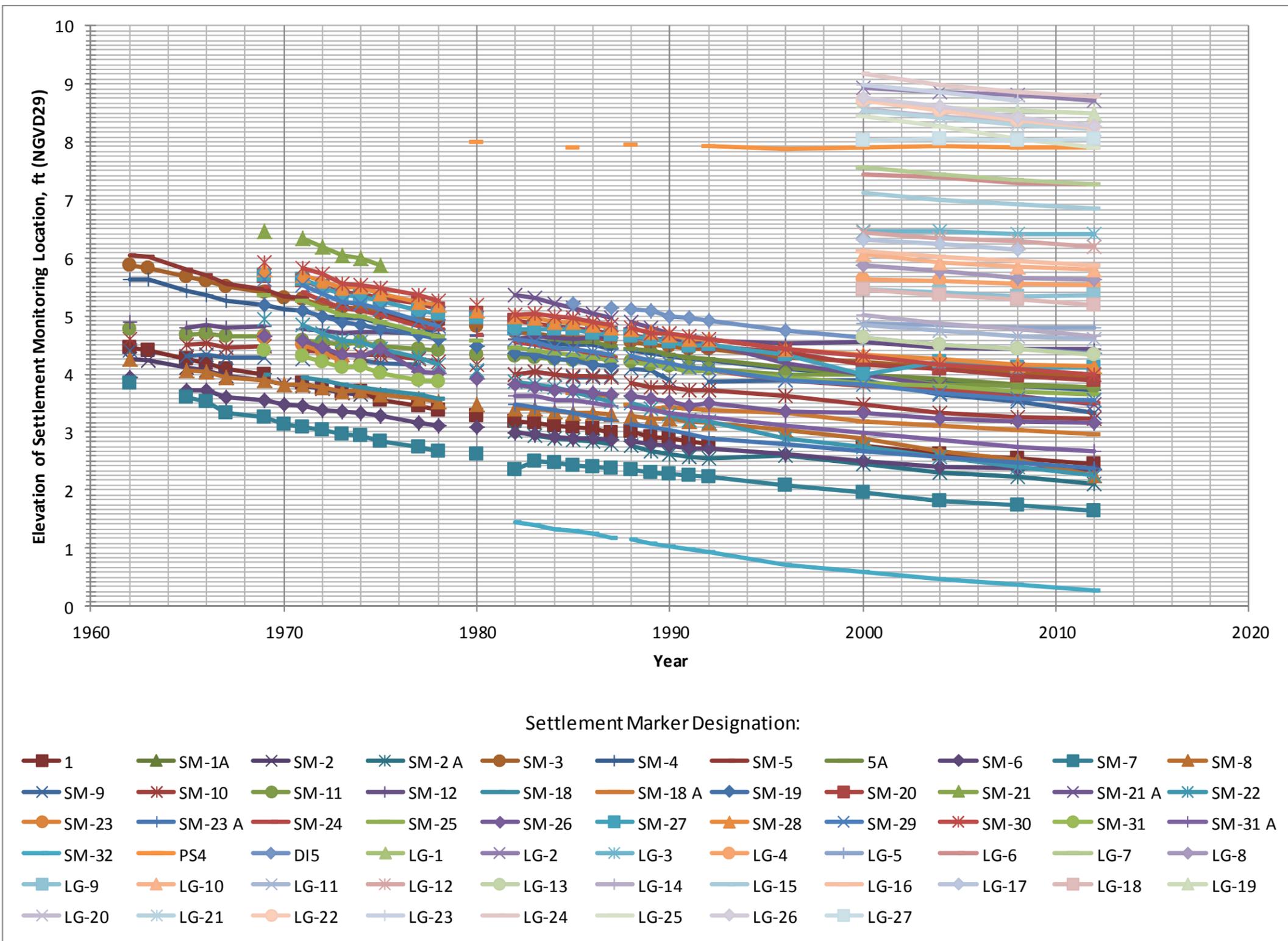
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 dated December 2008



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FILE NAME:	LEVEE ALIGNMENT.dwg

LEVEE ALIGNMENT WITH STATIONING

LAS GALLINAS LEVEE INVESTIGATION
 GEOTECHNICAL DATA REPORT
 SAN RAFAEL, CALIFORNIA



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Settlement data was provided by Marin County Flood Control and Water Conservation District (the District) on March 25, 2013.



PROJECT NO.	96670
DATE:	6.12.2013
DRAWN BY:	M. John
CHECKED BY:	C. Hall
FILE NAME:	

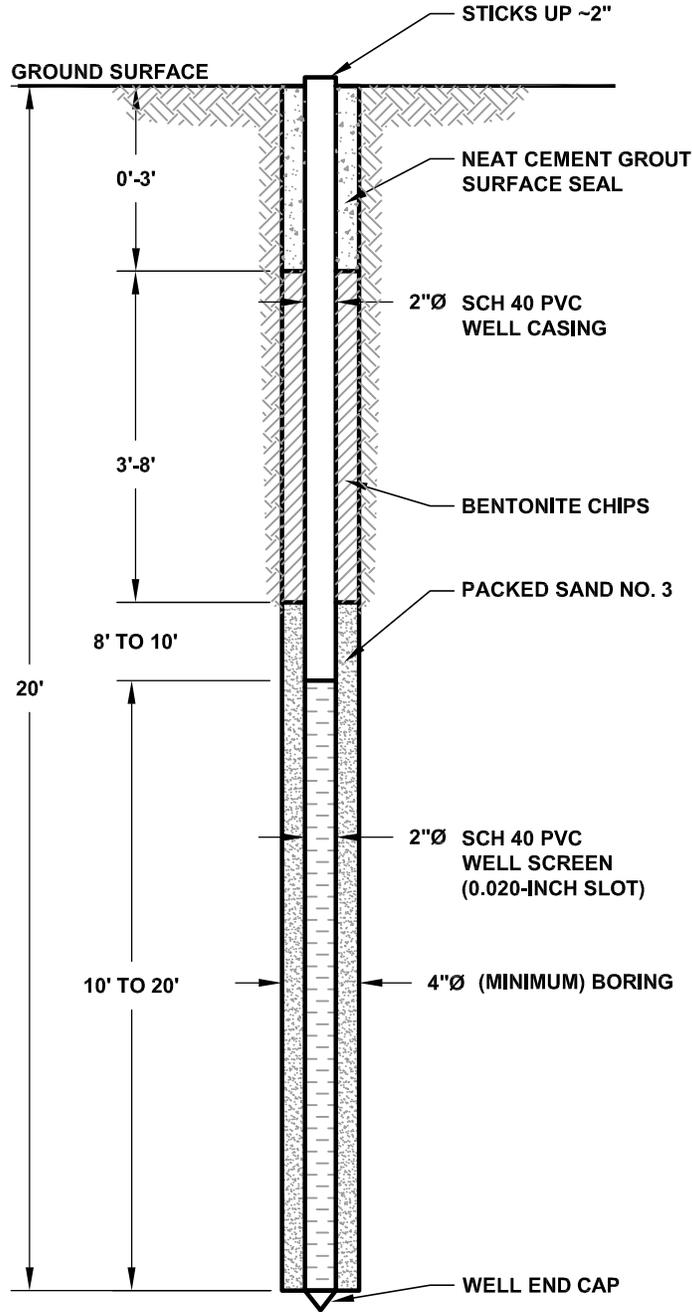
**SETTLEMENT MONITORING DATA
1962 TO 2012**

GEOTECHNICAL DATA REPORT
LAS GALLINAS LEVEE SYSTEM
SAN RAFAEL, CALIFORNIA

PLOTTED: 06 Jul 2010, 3:52pm, AGekas

LAYOUT: KC-3

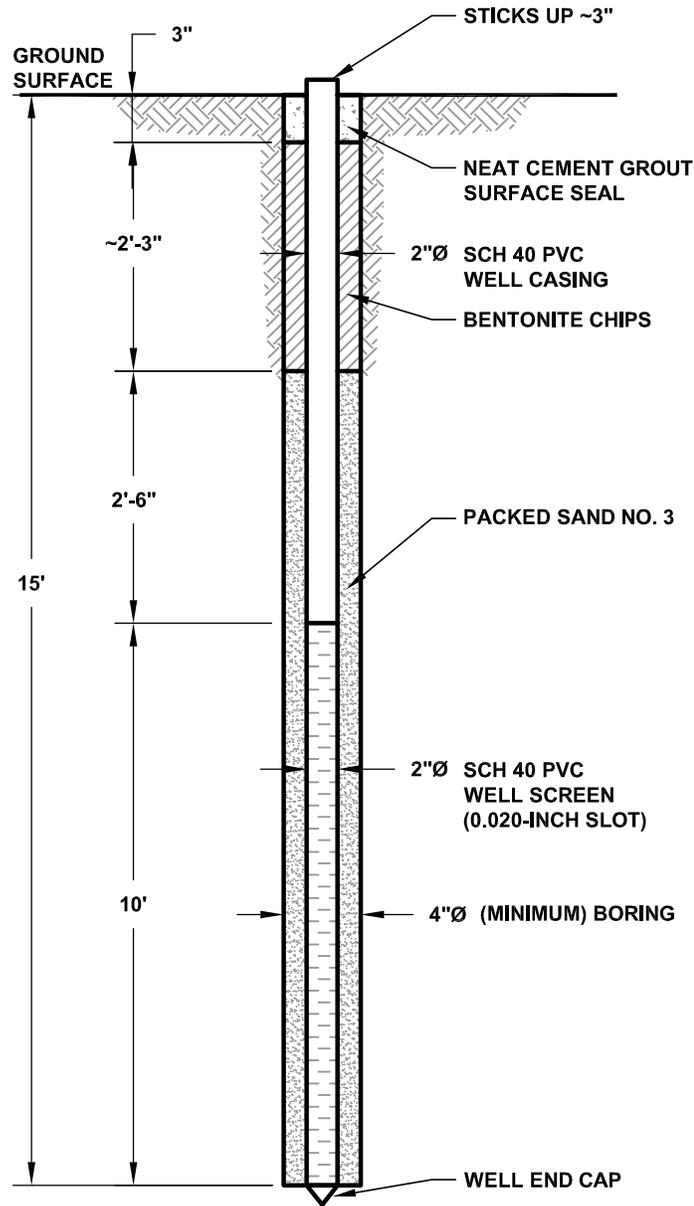
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 PLEASANTON, CA



NOT TO SCALE

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	PROJECT NO. 96670	PIEZOMETER KC-3 DETAILS	PLATE
	DRAWN: JULY 2010		
	DRAWN BY: JDS/AG	LAS GALLINAS LEVEE INVESTIGATION GEOTECHNICAL DATA REPORT SAN RAFAEL, CALIFORNIA	4a
	CHECKED BY: EMJ		
	FILE NAME: PIEZOMETER.dwg		



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	PROJECT NO. 96670	PIEZOMETER KT-3 DETAILS	PLATE
	DRAWN: JULY 2010		
	DRAWN BY: JDS/AG	LAS GALLINAS LEVEE INVESTIGATION GEOTECHNICAL DATA REPORT SAN RAFAEL, CALIFORNIA	4b
	CHECKED BY: EMJ		
	FILE NAME: PIEZOMETER.dwg		

Location: 601 Vendola Dr.
 Last Saved Date: 4/5/2013
 Analysis Type: Steady-State
 View: 2D

Horizontal Hydraulic Conductivity (Kh) and Anisotropy Ratio (Kv:Kh = K-Ratio)

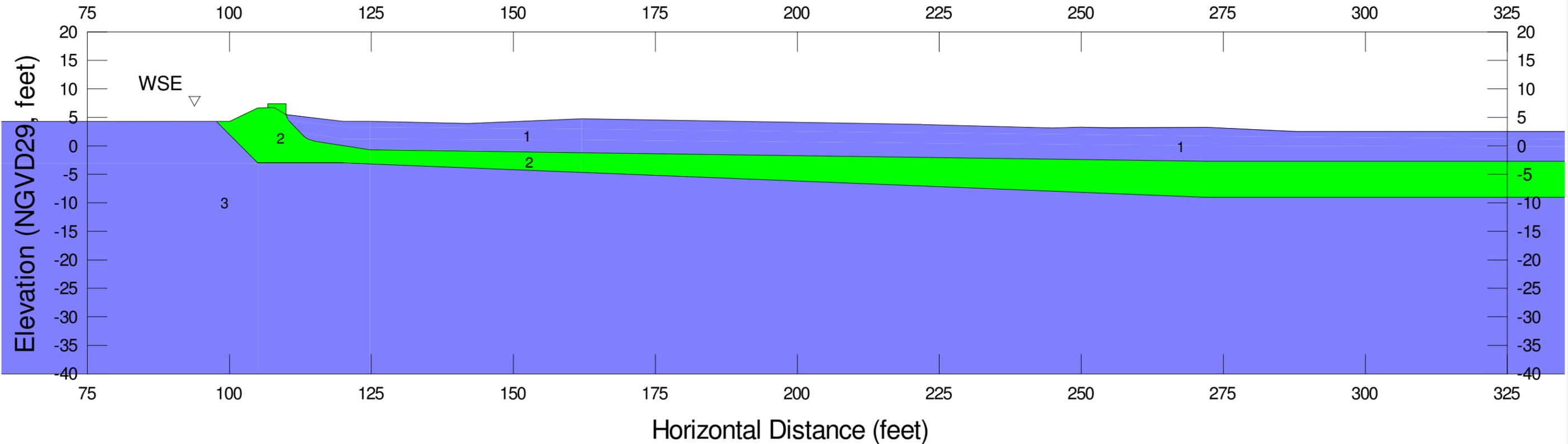
Material #1: Clay (CL, Fill)
 Material #2: Clayey Sand (SC)
 Material #3: Clay (CH, Native)

Model Geometries and Inputs

Top of Levee Elevation: 7.4 ft (Timber Box)
 Landside Toe Elevation: ~5 ft

Landside Edge Boundary Condition: No Flow (Q=0)
 Waterside Edge Boundary Condition: H=WSE

Approximate Blanket Thickness at Levee Toe: 4 ft



Notes:

- 1) Explorations CPT KCPT-2 and boring KC-2 were primarily used in developing the soil stratigraphy and material properties for the cross section shown.
- 2) Topography based on WR field survey data provided by WR 2008.
- 3) Subsurface conditions depicted on this plot are generalized and modeled for analysis purposes only. The cross section shown does not represent actual subsurface conditions.

Analyzed for 5 Water Surface Elevations (WSE):
 7.4', 6.6', 6.0', 5.5', 5.0', 4.3'

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 Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 96670	SEEPAGE FRAGILITY ANALYSIS EXISTING CONDITIONS: 4 FT. BLANKET	PLATE 5
	DATE: 5.10.2013		
	DRAWN BY: R. Yuen	GEOTECHNICAL DATA REPORT LAS GALLINAS LEVEE SYSTEM SAN RAFAEL, CALIFORNIA	
	CHECKED BY: C. Hall		
FILE NAME:			

Location: 601 Vendola Dr.
 Last Saved Date: 4/5/2013
 Analysis Type: Steady-State
 View: 2D

Horizontal Hydraulic Conductivity (Kh) and Anisotropy Ratio (Kv:Kh = K-Ratio)

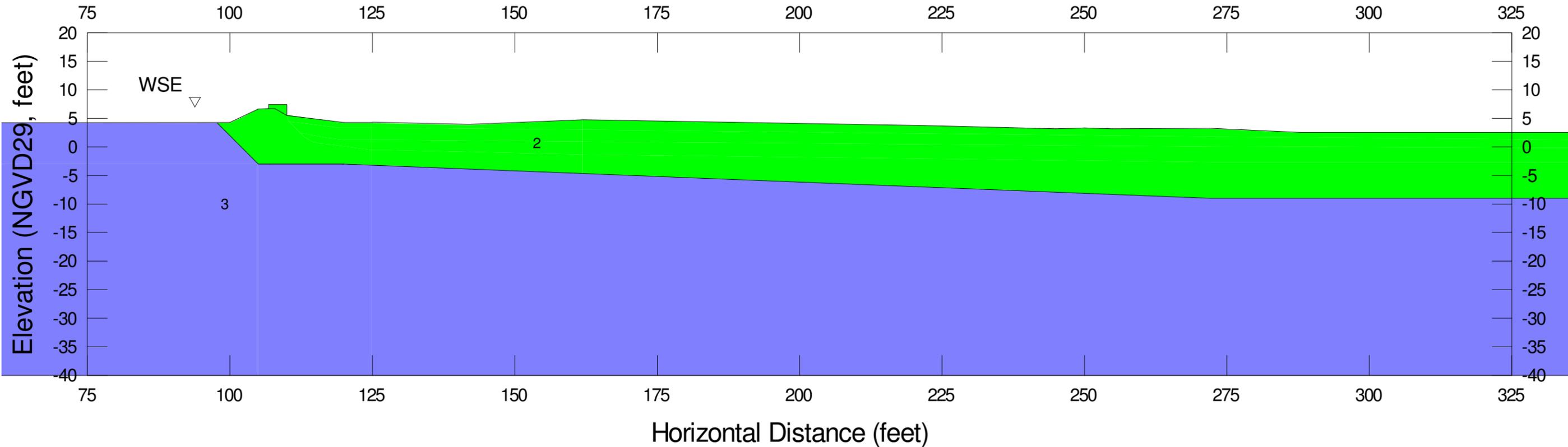
Material #1: Clay (CL, Fill) (Not present in this model)
 Material #2: Clayey Sand (SC)
 Material #3: Clay (CH, Native)

Model Geometries and Inputs

Top of Levee Elevation: 7.4 ft (Timber Box)
 Landside Toe Elevation: ~5 ft

Landside Edge Boundary Condition: No Flow (Q=0)
 Waterside Edge Boundary Condition: H=WSE

Approximate Blanket Thickness at Levee Toe: 0 ft



Notes:

- 1) Explorations CPT KCPT-2 and boring KC-2 were primarily used in developing the soil stratigraphy and material properties for the cross section shown.
- 2) Topography based on WR field survey data provided by WR 2008.
- 3) Subsurface conditions depicted on this plot are generalized and modeled for analysis purposes only. The cross section shown does not represent actual subsurface conditions.

Analyzed for 5 Water Surface Elevations (WSE):
 7.4', 6.6', 6.0', 5.5', 5.0', 4.3'

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 Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 96670	SEEPAGE FRAGILITY ANALYSIS EXISTING CONDITIONS: NO BLANKET	PLATE 6
	DATE: 5.10.2013		
	DRAWN BY: R. Yuen	GEOTECHNICAL DATA REPORT LAS GALLINAS LEVEE SYSTEM SAN RAFAEL, CALIFORNIA	
	CHECKED BY: C. Hall		
FILE NAME:			

Location: 601 Vendola Dr.
 Last Saved Date: 4/5/2013
 Analysis Type: Steady-State
 View: 2D

Horizontal Hydraulic Conductivity (Kh) and Anisotropy Ratio (Kv:Kh = K-Ratio)

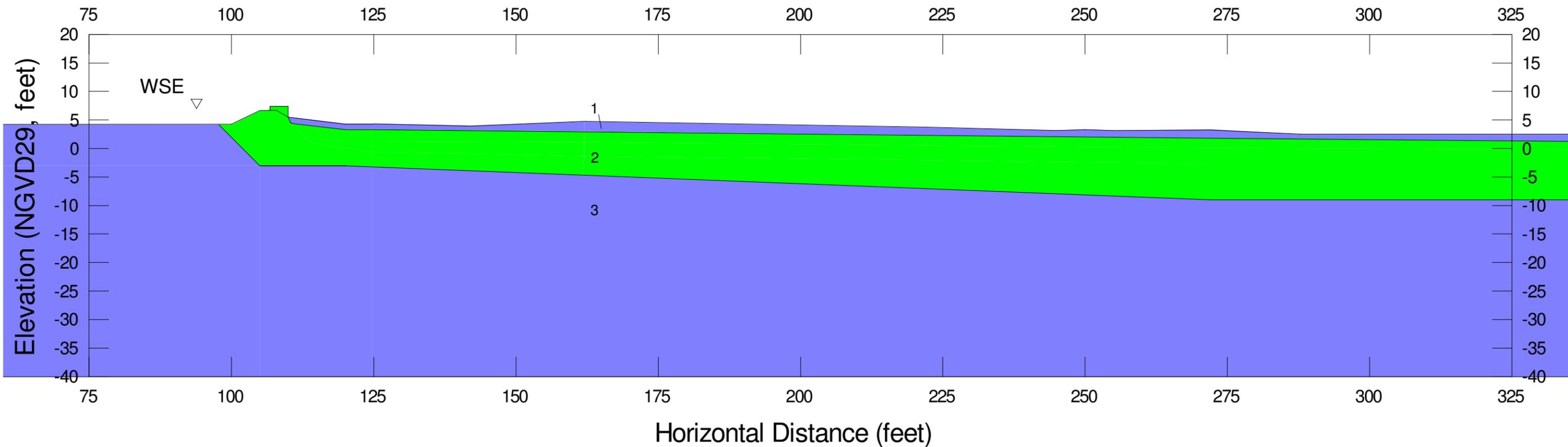
Material #1: Clay (CL, Fill)
 Material #2: Clayey Sand (SC)
 Material #3: Clay (CH, Native)

Model Geometries and Inputs

Top of Levee Elevation: 7.4 ft (Timber Box)
 Landside Toe Elevation: ~5 ft

Landside Edge Boundary Condition: No Flow (Q=0)
 Waterside Edge Boundary Condition: H=WSE

Approximate Blanket Thickness at Levee Toe: 1 ft



Notes:

- 1) Explorations CPT KCPT-2 and boring KC-2 were primarily used in developing the soil stratigraphy and material properties for the cross section shown.
- 2) Topography based on WR field survey data provided by WR 2008.
- 3) Subsurface conditions depicted on this plot are generalized and modeled for analysis purposes only. The cross section shown does not represent actual subsurface conditions.

Analyzed for 5 Water Surface Elevations (WSE):
 7.4', 6.6', 6.0', 5.5', 5.0', 4.3'

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 Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 96670	SEEPAGE FRAGILITY ANALYSIS EXISTING CONDITIONS: 1 FT. BLANKET	PLATE 7
	DATE: 5.10.2013		
	DRAWN BY: R. Yuen	GEOTECHNICAL DATA REPORT LAS GALLINAS LEVEE SYSTEM SAN RAFAEL, CALIFORNIA	
	CHECKED BY: C. Hall		
FILE NAME:			

Location: 601 Vendola Dr.
 File Name: SEEP_Kratio_10_Blkt_4_KvKhRatio_0.1_2013-03-26_TimberBox.gsz
 Last Saved Date: 4/8/2013
 Analysis Type: Steady-State
 View: 2D

Model Geometries and Inputs

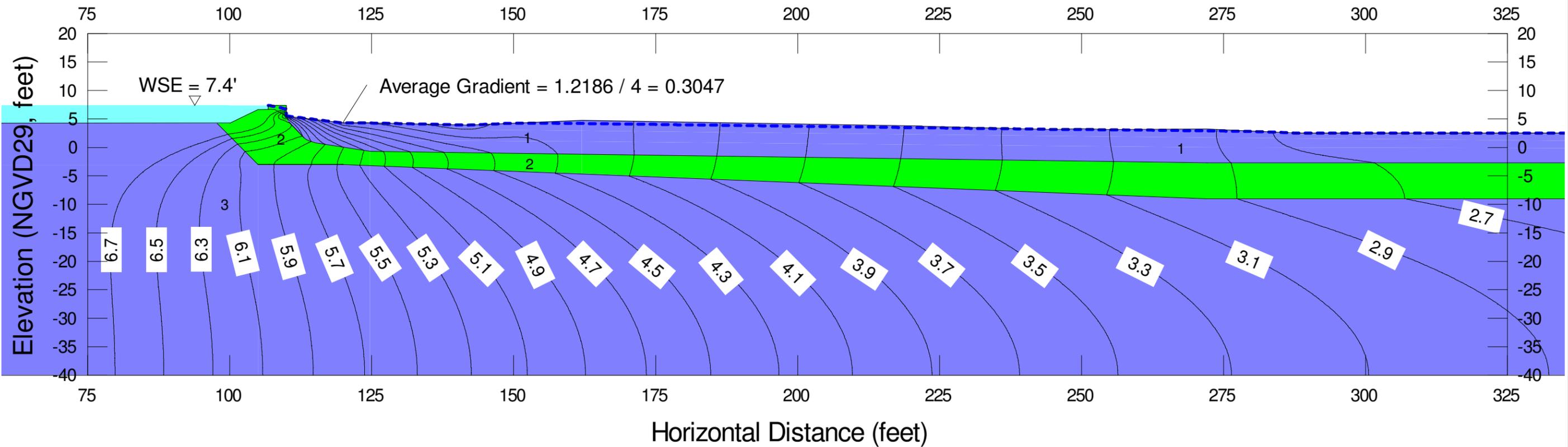
Top of Levee Elevation: 7.4 ft (Timber Box)
 Landside Toe Elevation: ~5 ft

Landside Edge Boundary Condition: No Flow (Q=0)
 Waterside Edge Boundary Condition: H=WSE

Approximate Blanket Thickness at Levee Toe: 4 ft

Horizontal Hydraulic Conductivity (Kh) and Anisotropy Ratio (Kv:Kh = K-Ratio)

Material #1: Clay (CL, Fill) Kh = 0.028 ft/day Kv/Kh = 0.25
 Material #2: Clayey Sand (SC) Kh = 0.28 ft/day Kv/Kh = 0.1
 Material #3: Clay (CH, Native) Kh = 0.0028 ft/day Kv/Kh = 0.1



Notes:

- 1) Explorations CPT KCPT-2 and boring KC-2 were primarily used in developing the soil stratigraphy and material properties for the cross section shown.
- 2) Topography based on WR field survey data provided by WR 2008.
- 3) Subsurface conditions depicted on this plot are generalized and modeled for analysis purposes only. The cross section shown does not represent actual subsurface conditions.

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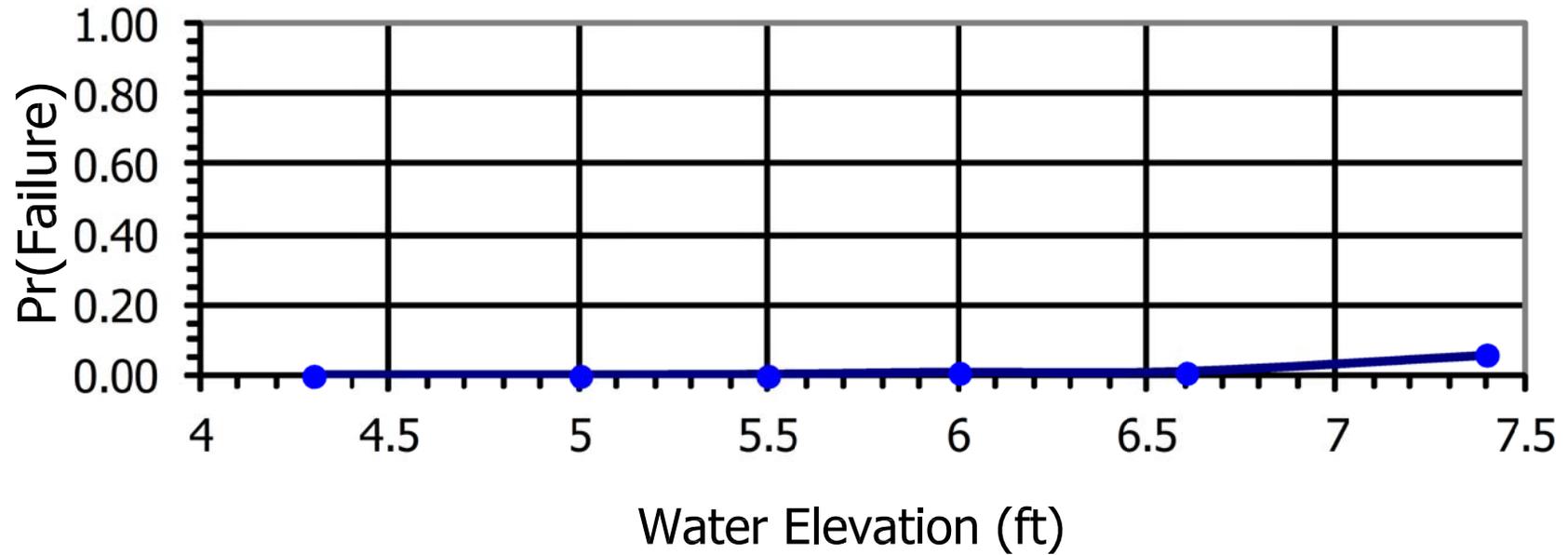


PROJECT NO.	96670
DATE:	5.10.2013
DRAWN BY:	R. Yuen
CHECKED BY:	C. Hall
FILE NAME:	

RESULTS OF SEEPAGE ANALYSIS EXISTING CONDITIONS: CASE 1A	GEOTECHNICAL DATA REPORT LAS GALLINAS LEVEE SYSTEM SAN RAFAEL, CALIFORNIA

PLATE
8

Underseepage Probability of Failure



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PROJECT NO.	96670
DRAWN:	APRIL 2013
DRAWN BY:	AG
CHECKED BY:	CH
FILE NAME:	April-2013-8a-13.dwg

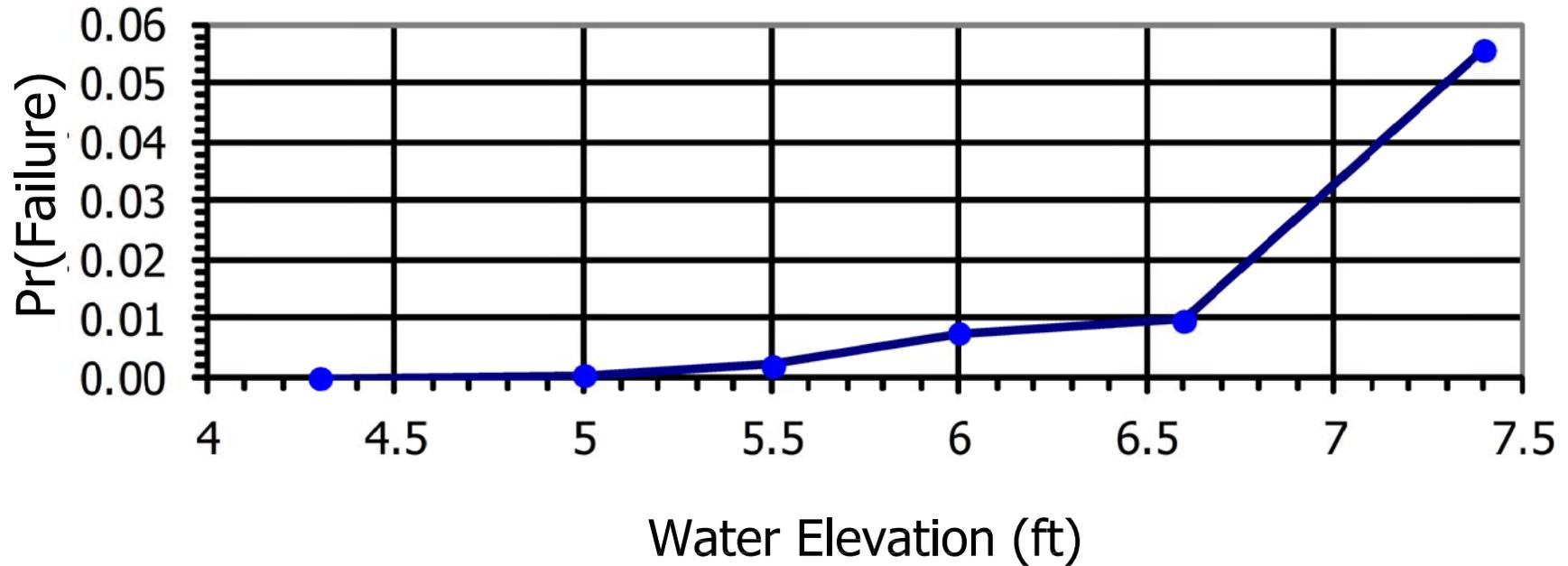
**UNDERSEEPAGE
 PROBABILITY OF FAILURE
 (FULL SCALE)**

LAS GALLINAS LEVEE INVESTIGATION
 GEOTECHNICAL DATA REPORT
 SAN RAFAEL, CALIFORNIA

PLATE

9a

Underseepage Probability of Failure



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PROJECT NO.	96670
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DRAWN BY:	AG
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FILE NAME:	April-2013-8a-13.dwg

**UNDERSEEPAGE
 PROBABILITY OF FAILURE
 (0 TO 6 PERCENT SCALE)**

LAS GALLINAS LEVEE INVESTIGATION
 GEOTECHNICAL DATA REPORT
 SAN RAFAEL, CALIFORNIA

PLATE

9b

Location: 601 Vendola Dr.
 Last Saved Date: 4/5/2013
 Analysis Type: Spencer

Piezometric Surface Imported from SEEP model base case with the following parameters:
 Material #1: Clay (CL, Fill) $K_h = 0.028$ ft/day $K_v/K_h = 0.25$
 Material #2: Clayey Sand (SC) $K_h = 0.28$ ft/day $K_v/K_h = 0.1$
 Material #3: Clay (CH, Native) $K_h = 0.0028$ ft/day $K_v/K_h = 0.1$

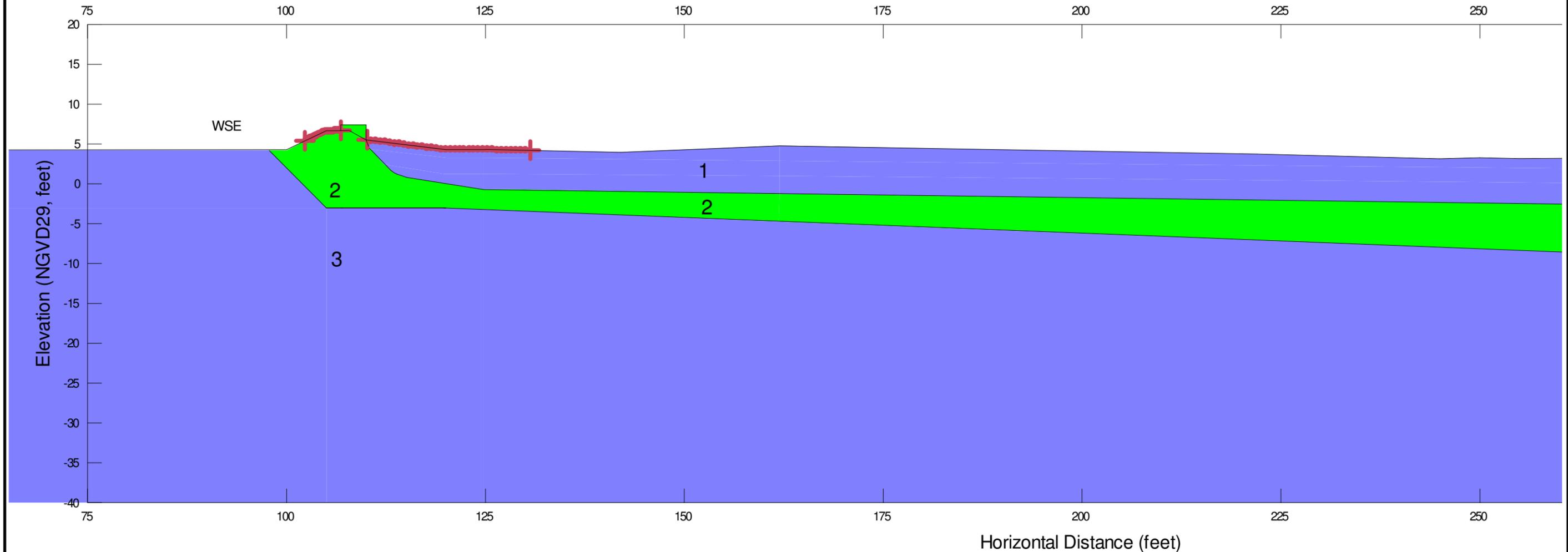
Material #1: Clay (CL, Fill) Unit Weight= 120 pcf Cohesion = 50 psf $\Phi = 26^\circ$
 Material #2: Clayey Sand (SC) Unit Weight= 125 pcf Cohesion = 50 psf $\Phi = 28^\circ$
 Material #3: Clay (CH, Native) Unit Weight= 92 pcf Cohesion = 200 psf $\Phi = 16^\circ$

Model Geometries and Inputs

Top of Levee Elevation: 7.4 ft (Timber Box)
 Landside Toe Elevation: ~5 ft

Approximate Blanket Thickness at Levee Toe: 4 ft

Search for Tension Crack, Tension Crack Fluid Unit Weight: 62.4 pcf
 Minimum Slip Surface Depth: 1 ft



Notes:

- 1) Explorations CPT KCPT-2 and boring KC-2 were primarily used in developing the soil stratigraphy and material properties for the cross section shown.
- 2) Topography based on WR field survey data provided by WR 2008.
- 3) Subsurface conditions depicted on this plot are generalized and modeled for analysis purposes only. The cross section shown does not represent actual subsurface conditions.

Analyzed for 5 Water Surface Elevations (WSE):
 7.4', 6.6', 6.0', 5.5', 5.0', 4.3'

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PROJECT NO.	96670
DATE:	5.10.2013
DRAWN BY:	R. Yuen
CHECKED BY:	C. Hall
FILE NAME:	

SLOPE STABILITY FRAGILITY ANALYSIS
 EXISTING CONDITIONS: "EXPECTED" CASE

GEOTECHNICAL DATA REPORT
 LAS GALLINAS LEVEE SYSTEM
 SAN RAFAEL, CALIFORNIA

PLATE

10

Location: 601 Vendola Dr.
 File Name: SLOPE_Cases 1A-1F_2013-03-27_TimberBox.gsz
 7.4' WSE Case 1A
 Last Saved Date: 4/8/2013
 Analysis Type: Spencer

Model Geometries and Inputs

Top of Levee Elevation: 7.4 ft (Timber Box)
 Landside Toe Elevation: ~5 ft

Piezometric Surface Imported from SEEP model base case with the following parameters:

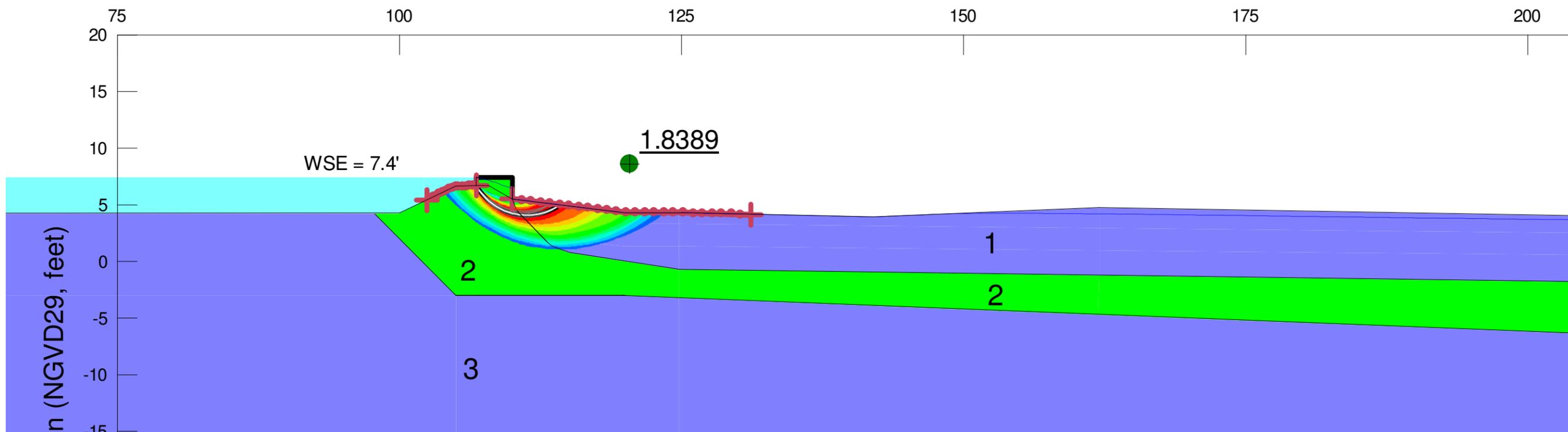
Material #1: Clay (CL, Fill) $K_h = 0.028$ ft/day $K_v/K_h = 0.25$
 Material #2: Clayey Sand (SC) $K_h = 0.28$ ft/day $K_v/K_h = 0.1$
 Material #3: Clay (CH, Native) $K_h = 0.0028$ ft/day $K_v/K_h = 0.1$

Approximate Blanket Thickness at Levee Toe: 4 ft

Search for Tension Crack, Tension Crack Fluid Unit Weight: 62.4 pcf
 Minimum Slip Surface Depth: 1 ft

Material #1: Clay (CL, Fill) Unit Weight= 120 pcf Cohesion = 50 psf $\Phi = 26^\circ$
 Material #2: Clayey Sand (SC) Unit Weight= 125 pcf Cohesion = 50 psf $\Phi = 28^\circ$
 Material #3: Clay (CH, Native) Unit Weight= 92 pcf Cohesion = 200 psf $\Phi = 16^\circ$

Results:
 Factor of Safety = 1.8389



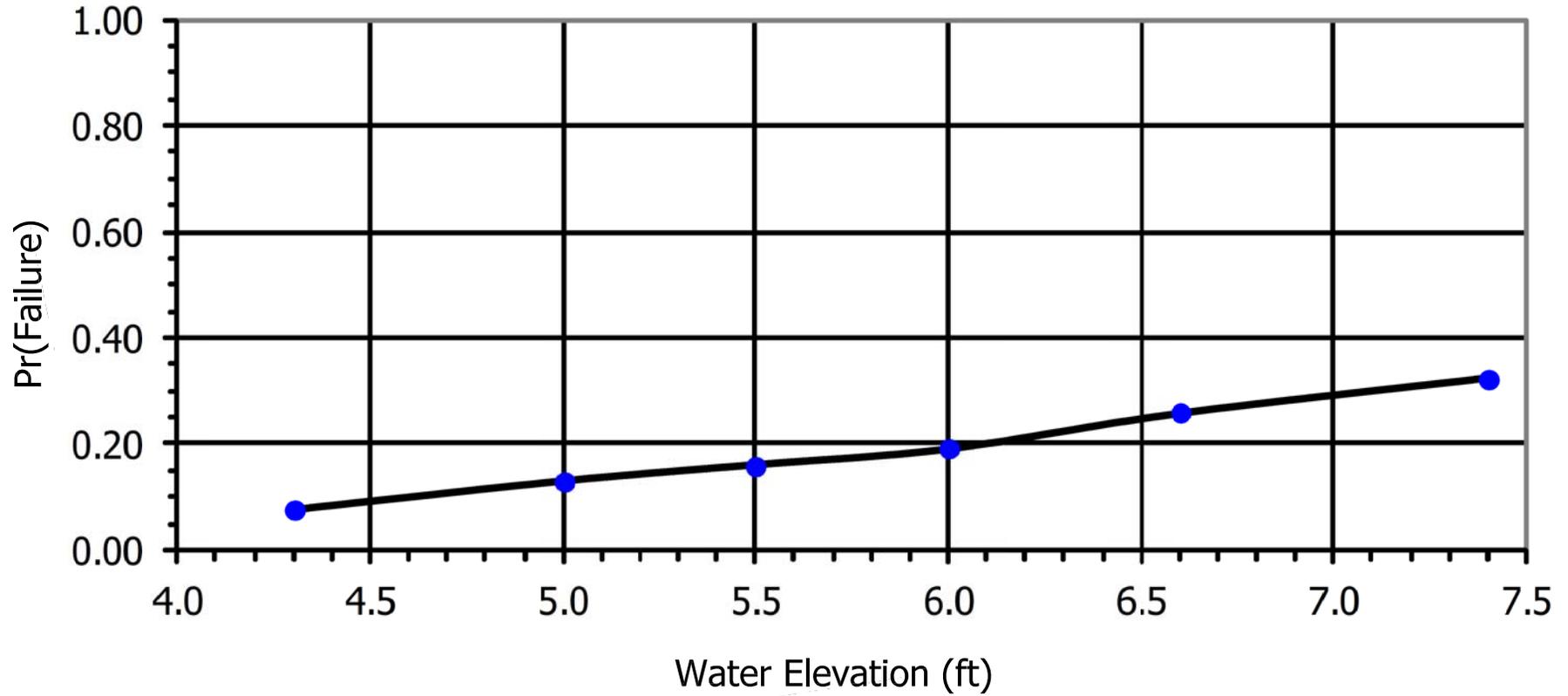
Notes:

- 1) Explorations CPT KCPT-2 and boring KC-2 were primarily used in developing the soil stratigraphy and material properties for the cross section shown.
- 2) Topography based on WR field survey data provided by WR 2008.
- 3) Subsurface conditions depicted on this plot are generalized and modeled for analysis purposes only. The cross section shown does not represent actual subsurface conditions.

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	PROJECT NO.	96670	RESULTS OF SLOPE STABILITY ANALYSIS EXISTING CONDITIONS: CASE 1A "EXPECTED"	PLATE 11
	DATE:	5.10.2013		
	DRAWN BY:	R. Yuen	GEOTECHNICAL DATA REPORT LAS GALLINAS LEVEE SYSTEM SAN RAFAEL, CALIFORNIA	
	CHECKED BY:	C. Hall		
FILE NAME:				

Stability Probability of Failure



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PROJECT NO.	96670
DRAWN:	APRIL 2013
DRAWN BY:	AG
CHECKED BY:	CH
FILE NAME:	April-2013-8a-13.dwg

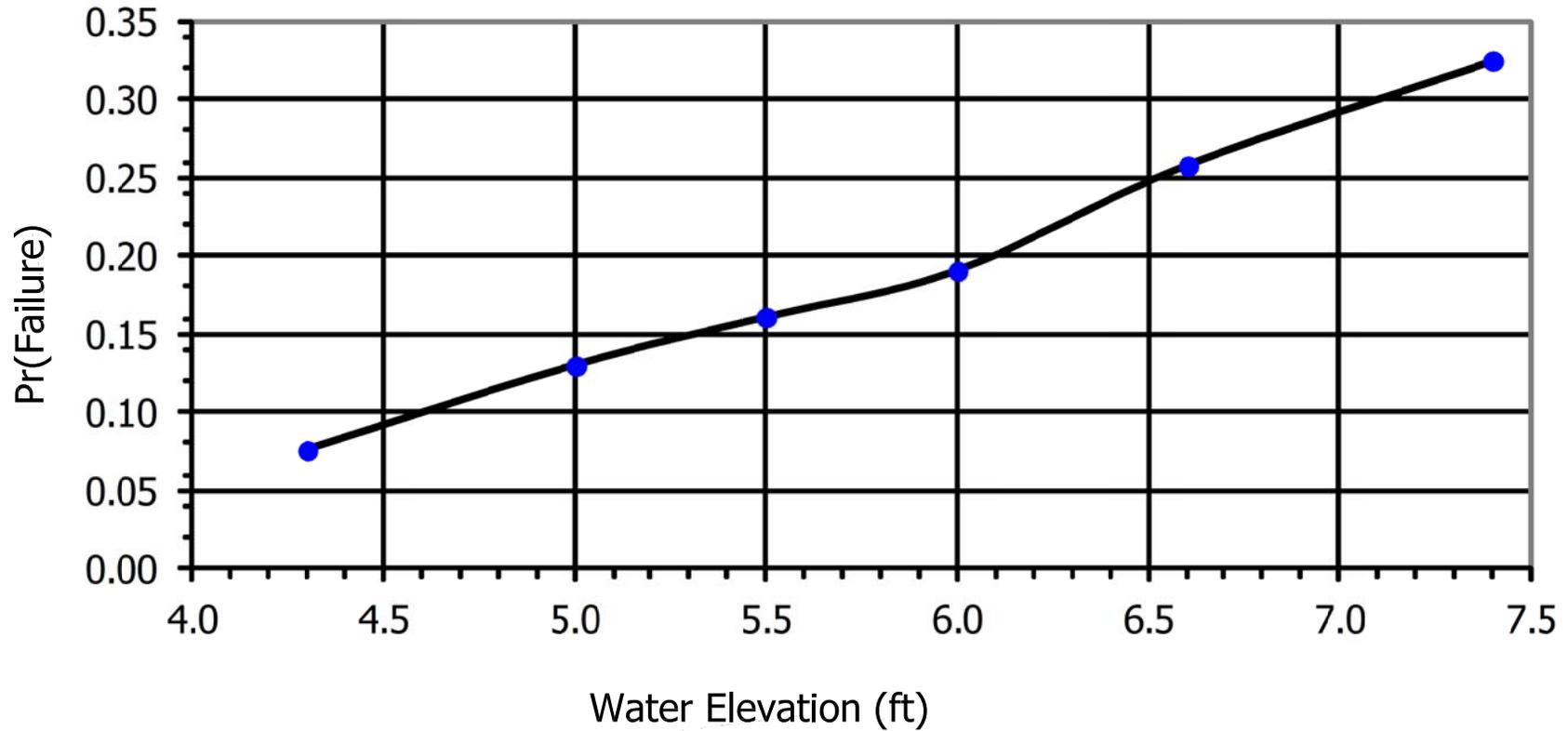
**STABILITY PROBABILITY OF FAILURE
(FULL SCALE)**

LAS GALLINAS LEVEE INVESTIGATION
 GEOTECHNICAL DATA REPORT
 SAN RAFAEL, CALIFORNIA

PLATE

12a

Stability Probability of Failure



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PROJECT NO.	96670
DRAWN:	APRIL 2013
DRAWN BY:	AG
CHECKED BY:	CH
FILE NAME:	April-2013-8a-13.dwg

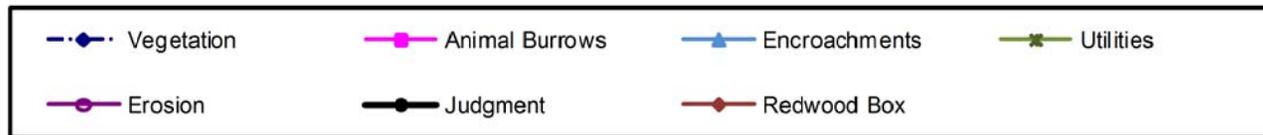
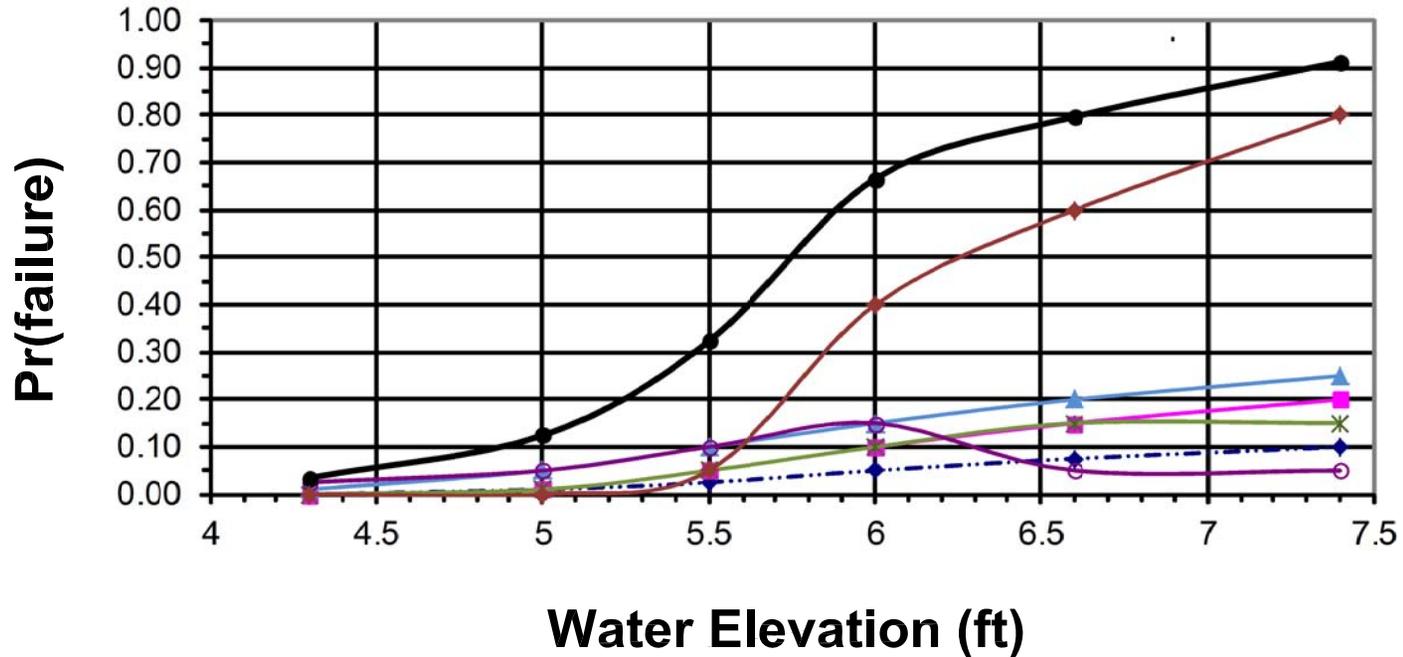
**STABILITY PROBABILITY OF FAILURE
 (0 TO 35 PERCENT SCALE)**

LAS GALLINAS LEVEE INVESTIGATION
 GEOTECHNICAL DATA REPORT
 SAN RAFAEL, CALIFORNIA

PLATE

12b

Judgment Probability of Failure



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PROJECT NO.	96670
DRAWN:	APRIL 2013
DRAWN BY:	AG
CHECKED BY:	CH
FILE NAME:	April-2013-8a-13.dwg

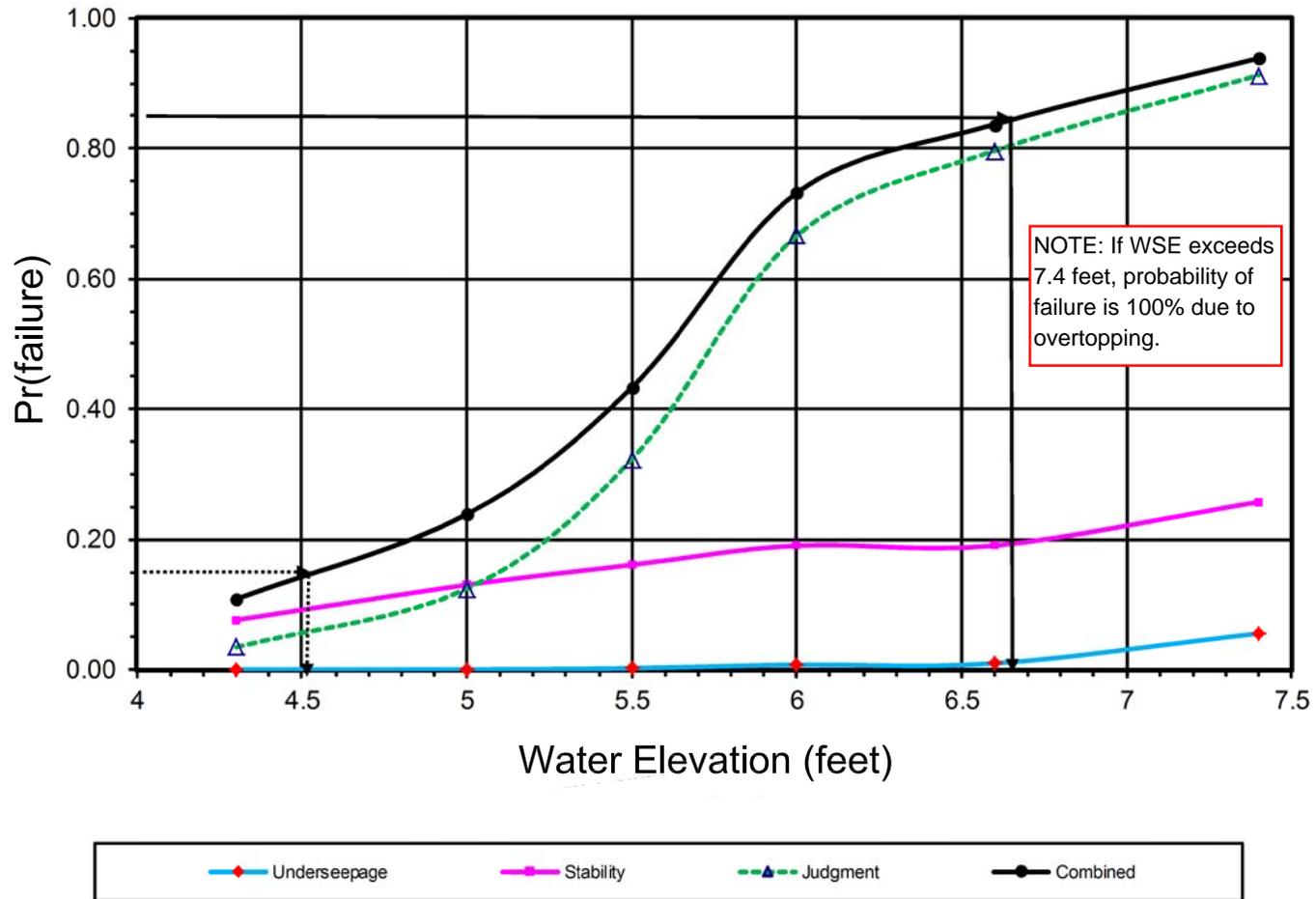
JUDGMENT PROBABILITY OF FAILURE

LAS GALLINAS LEVEE INVESTIGATION
 GEOTECHNICAL DATA REPORT
 SAN RAFAEL, CALIFORNIA

PLATE

13

Combined Probability of Failure Existing Conditions



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PROJECT NO.	96670
DRAWN:	APRIL 2013
DRAWN BY:	AG
CHECKED BY:	CH
FILE NAME:	April-2013-8a-13.dwg

COMBINED PROBABILITY OF FAILURE EXISTING CONDITIONS

LAS GALLINAS LEVEE INVESTIGATION
 GEOTECHNICAL DATA REPORT
 SAN RAFAEL, CALIFORNIA

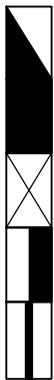
PLATE

14

APPENDIX A
LOGS OF BORINGS

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION	MAJOR DIVISIONS	LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY		GW	Well-graded gravels or gravel with sand, little or no fines.	FINE GRAINED SOILS		ML	Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
			GP	Poorly-graded gravels or gravel with sand, little or no fines.			CL	Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
			GM	Silty gravels, silty gravel with sand mixture.			OL	Organic silts and organic silt-clays of low plasticity.
			GC	Clayey gravels, clayey gravel with sand mixture.			MH	Inorganic elastic silts, micaceous or diatomaceous or silty soils.
	SAND AND SANDY		SW	Well-graded sands or gravelly sands, little or no fines.			CH	Inorganic fat clays (high plasticity).
			SP	Poorly-graded sands or gravelly sands, little or no fines.			OH	Organic clays of medium high to high plasticity.
			SM	Silty sand.			Pt	Peat and other highly organic soils.
			SC	Clayey sand.		HIGHLY ORGANIC SOILS		



Standard Penetration Split Spoon Sampler 2.0 inch O.D., 1.4 inch I.D.

Modified California Sampler 2.5 inch O.D., 2.0 inch I.D.

Bulk Sample

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Shelby Tube 3.0 inch O.D.



Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.



Approximate water level observed in boring following drilling

PEN Pocket Penetrometer reading, in tsf

TV:Su Torvane shear strength, in ksf

LL Liquid Limit

PI Plasticity Index

%-#200 Sieve Analysis (#200 Screen)

DS Direct Shear

C Cohesion (psf)

PHI Friction Angle

UC Unconfined Compression

TxUU Triaxial Shear

CONSOL Consolidation

R-Value Resistance Value

SE Sand Equivalent

EI Expansion Index

FS Free Swell (U.S.B.R.)

Notes: Blow counts represent the number of blows a 140-pound hammer falling 30 inches required to drive a sampler through the last 12 inches of an 18 inch penetration, unless otherwise noted.

The lines separating strata on the logs represent approximate boundaries only. The actual transition may be gradual. No warranty is provided as to the continuity of soil strata between borings. Logs represent the soil section observed at the boring location on the date of drilling only.



PROJECT NO. 96670

BORING LOG LEGEND

Las Gallinas Levee Evaluation
Santa Venetia
San Rafael, California

PLATE

A-1

Date Completed: **11/3/08**

Drilling method: **4" Solid Stem Auger**

Logged By: **EMJ**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **Approximately 10.5 ft**

Notes: **Rope and cathead hammer, 4" diameter hole.**

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
								Surface Elevation: Estimated 8.4 feet (MSL)
8								SANDY LEAN CLAY (CL) , dark brown, soft, wet, with roots. [TOPSOIL]
52						4.5		CLAYEY SAND with GRAVEL (SC) , reddish brown, dense, dry, fine to medium sand, some rock fragments, trace roots. [FILL].
50								Grades to medium brown
5								CLAYEY SAND (SC) , greenish gray, loose to dense, dry, fine to coarse sand. [FILL]
6								Asphalt and brick fragments from 7.0 to 7.5 ft.
5								POORLY GRADED SAND (SP) , greenish gray, loose, wet, trace clay, trace small roots. Clay lens from 7.5 to 7.7 ft. [FILL]
10								FAT CLAY (CH) , greenish gray, soft, wet, trace roots and grass. [NATIVE - DISTURBED YOUNG BAY MUD]
								NOTES: 1. Boring terminated at approximately 10.5 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
15								
20								
25								
30								

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PROJECT NO. 96670

LOG OF BORING NO. KC-1

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

A-2

7/14/2009 12:16:26 PM

Date Completed: **11/17/08**

Drilling method: **4" Solid Stem Auger**

Logged By: **EMJ**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **Approximately 10.0 ft**

Notes: **Rope and cathead hammer, 4" diameter hole.**

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
14								<p>Surface Elevation: Estimated 7.3 feet (MSL)</p> <p>SANDY LEAN CLAY (CL), reddish brown, stiff, moist, fine to medium sand, trace fine gravel, trace roots. Upper 4" topsoil and ice plant roots. [FILL]</p>
7								
17			10:33 am 11/17/08					<p>CLAYEY SAND with GRAVEL (SC), yellowish brown, loose to medium dense, wet, fine to medium grained sand, low to medium plasticity clay, trace roots. [FILL]</p> <p>Some loose gravel in sampler tip.</p>
5						LL=26; PI=10 Passing -#200=19%		
7								
3				17.8				<p>CLAYEY SAND with GRAVEL (SC), olive brown, very loose, wet, fine to coarse sand, fine to coarse gravel, rounded sand, angular to subangular gravel, low plasticity clay. [FILL]</p>
10						LL=25; PI=11 Passing -#200=23%		
3								<p>FAT CLAY (CH), dark gray, very soft, wet, trace black organics/roots. [NATIVE - YOUNG BAY MUD]</p>
15			65	100.0		LL=97; PI=64 TV:Su = 0.26 ksf		
15								<p>NOTES:</p> <ol style="list-style-type: none"> Boring terminated at approximately 14.0 feet below ground surface. Boring backfilled with cement-bentonite grout. Blow counts shown are raw, uncorrected values.
20								
25								
30								

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PROJECT NO. **96670**

LOG OF BORING NO. KC-2

Las Gallinas Levee Evaluation
Santa Venetia
San Rafael, California

PLATE

A-3

7/14/2009 12:16:28 PM

Date Completed: 11/11/08

Drilling method: 4" Solid Stem Auger

Logged By: EMJ

Hammer Wt: 140 lbs., 30" drop

Total Depth: Approximately 20.0 ft

Notes: Rope and cathead hammer, 4" diameter hole.

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
7								LEAN CLAY (CL) , medium brown, very stiff, moist, some fine to medium grained sand, trace gravel, trace small roots. [FILL]
53							3.0	POORLY GRADED SAND with CLAY (SP-SC) , light tan, dense, dry. [FILL]
20			115	7.3		Passing #200=32%	2.2	CLAYEY SAND with GRAVEL (SC) , dark yellowish brown, medium dense, moist, trace roots and grass, fine to coarse sand, fine to medium gravel, angular sands and gravels. [FILL] Loose, wet.
11								Increasing sand and gravel content from 5.5 to 7.5 ft.
4								CLAYEY SAND with GRAVEL (SC) , reddish brown, very loose, wet, trace roots, fine to coarse sand, fine to medium gravel, angular sands and gravels. [FILL]
3								POORLY GRADED GRAVEL (GP) , dark gray, medium dense, wet, trace clay. [FILL]
16								FAT CLAY (CH) , dark gray and black, very soft, wet, some gravel, with organics (roots, grass, bark). [FILL]
10								POORLY GRADED SAND with CLAY and GRAVEL (SP-SC) , dark brownish gray, loose, wet, fine to coarse sand, fine to medium gravel, angular sands and gravels. [FILL]
8								CLAYEY GRAVEL with SAND (GC) , olive brown, medium dense, wet, trace organics, subangular gravels. [FILL]
16						Passing #200=12%		Loose at 15.0 ft.
12								FAT CLAY (CH) , olive gray, very soft, wet, trace sand and roots. [NATIVE - YOUNG BAY MUD]
15								
7								
11								
4								
20								
25								
30								

10:00 am
11/11/08

Surface Elevation: **Estimated 5.5 feet (MSL)**

- NOTES:**
- Boring terminated at approximately 19.5 feet below ground surface.
 - Boring backfilled with cement-bentonite grout.
 - Blow counts shown are raw, uncorrected values.

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PROJECT NO. 96670

LOG OF BORING NO. KC-3

Las Gallinas Levee Evaluation
Santa Venetia
San Rafael, California

PLATE

A-4

7/14/2009 12:16:30 PM

Date Completed: **11/11/08**

Drilling method: **4" Solid Stem Auger**

Logged By: **EMJ**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **Approximately 15.0 ft**

Notes: **Rope and cathead hammer, 4" diameter hole.**

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
12								<p>Surface Elevation: Estimated 5.0 feet (MSL)</p>
23								
27								<p>LEAN CLAY with GRAVEL (CL), dark brown, stiff, moist, some sand, trace roots and grass, angular fine to medium gravel. [FILL]</p>
5	6							<p>POORLY GRADED GRAVEL with SAND (GP), reddish brown, medium dense, wet, some clay, trace organics. [FILL] Grades to trace clay</p>
5	5							<p>POORLY GRADED GRAVEL with CLAY and SAND (GP-GC), brown and gray, loose, wet, trace organics. [FILL]</p>
4								<p>FAT CLAY (CH), olive gray and black mottled, very soft, wet, trace organics. [NATIVE - YOUNG BAY MUD]</p>
10								
15	4							
15								<p>NOTES:</p> <ol style="list-style-type: none"> Boring terminated at approximately 15.0 feet below ground surface. Boring backfilled with cement-bentonite grout. Blow counts shown are raw, uncorrected values.
20								
25								
30								

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PROJECT NO. **96670**

LOG OF BORING NO. KT-3

Las Gallinas Levee Evaluation
Santa Venetia
San Rafael, California

PLATE

A-5

7/14/2009 12:16:36 PM

Date Completed: **11/6/08**

Drilling method: **6" Hollow Stem Auger**

Logged By: **EMJ**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **Approximately 20.0 ft**

Notes: **Cable winch**

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
25								LEAN CLAY WITH SAND (CL) , gray and brown, medium stiff to very stiff, moist, some fine to medium gravel, fine to coarse sand. [FILL]
12								
35								LEAN CLAY (CL) , medium brown and gray, very stiff, moist, trace sand and gravel. [FILL]
24								
9								CLAYEY SAND with GRAVEL (SC) , reddish brown, loose, wet, fine to medium sand, fine to medium gravel. [FILL] Becoming dense at 8.0 ft.
56								
50/6"			122	12.5	3.44 @	TXUU:Su = 3.4 ksf		WELL GRADED GRAVEL with CLAY and SAND (GW) , gray, very dense, wet, fine to coarse sand, fine to medium angular gravel. [FILL] Becoming dense at 11.0 ft. Becoming medium dense at 12.5 ft.
36						Passing #200=25%		
17						Passing #200=9%		FAT CLAY (CH) , dark olive gray, soft, wet, trace organics, trace sand. [NATIVE - YOUNG BAY MUD]
15						TV:Su = 0.56 ksf		
NOTES: 1. Boring terminated at approximately 15.5 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.								

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PROJECT NO. 96670

LOG OF BORING NO. KC-4

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

A-6

7/14/2009 12:16:31 PM

Date Completed: **11/6/08**

Drilling method: **6" Hollow Stem Auger**

Logged By: **EMJ**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **Approximately 15.5 ft**

Notes: **Cable winch**

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
								Surface Elevation: Estimated 8.4 feet (MSL)
23								SANDY SILT (ML) , medium brown, hard, dry, trace gravel, trace organics. [FILL]
66								LEAN CLAY with SILT (CL) , grayish brown, hard, moist, trace organics, trace sand and gravel, low to medium plasticity. [FILL]
37								Some sand and gravel.
5								
32								
55/6"								CLAYEY GRAVEL (GC) , gray and brown, very dense, wet, some sand, fine to medium gravel, medium to coarse sand, low to medium plasticity clay. [FILL]
77								Decreasing clay content from 9.0 to 11.0 ft.
10								
61								
50/2"								POORLY GRADED GRAVEL with SAND (GP) , gray, very dense, wet, trace clay. [FILL]
27								Becoming medium dense at 12.5 ft.
15								FAT CLAY (CH) , olive gray and black, medium stiff, wet, trace silt, seams up to 3" thick of 90% organic material. [NATIVE - YOUNG BAY MUD]
								NOTES: 1. Boring terminated at approximately 15.5 feet below ground surface. 2. Boring backfilled cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
20								
25								
30								

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PROJECT NO. 96670

LOG OF BORING NO. KC-5

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

A-7

7/14/2009 12:16:32 PM

Date Completed: 11/3/08

Drilling method: 4" Solid Stem Auger

Logged By: EMJ

Hammer Wt: 140 lbs., 30" drop

Total Depth: Approximately 9.0 ft

Notes: Rope and cathead hammer, 4" diameter hole.

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
								Surface Elevation: Estimated 8.2 feet (MSL)
49								 <p>SILTY SAND (SM), light brown, dense, dry, fine to medium sand, organic topsoil and roots upper 4". [FILL] Becoming medium dense at 2.0 ft.</p> <p>CLAYEY SAND (SC), reddish brown, medium dense, moist, fine to coarse sand, some orange oxidation. [FILL] Roots from 4.5 to 5.0 ft. Wet at 4.5 ft. Transition to clay at 5.0 ft.</p> <p>FAT CLAY (CH) with organics, olive and brownish gray, soft, wet, some roots and grass. [NATIVE - YOUNG BAY MUD]</p>
24								
10								
5								
3								
							TV:Su = 0.34 ksf	
10								NOTES: 1. Boring terminated at approximately 9.0 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
15								
20								
25								
30								

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PROJECT NO. 96670

LOG OF BORING NO. KC-6

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

A-8

7/14/2009 12:16:33 PM

Date Completed: **11/3/08**

Drilling method: **4" Solid Stem Auger**

Logged By: **EMJ**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **Approximately 10.5 ft**

Notes: **Rope and cathead hammer, 4" diameter hole.**

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
								Surface Elevation: Estimated 6.6 feet (MSL)
8								SILTY CLAY (CL) , brown, stiff, moist, with roots, some sand. [TOPSOIL]
14			104	10.0		Passing #200=28%		CLAYEY SAND (SC) , gray, loose to medium dense, wet, trace clay. [FILL]
14								CLAYEY SAND (SC) , dark yellowish brown, loose to medium dense, moist, fine to medium sand, trace roots, trace rock fragments. [FILL]
5	15		121	14.5		Passing #200=24%		
			13:40 pm 11/3/08					
	9							
	7							LEAN CLAY with GRAVEL (CL) , grayish brown, soft, wet, some sand, trace roots and grass. [FILL]
10								FAT CLAY (CH) , gray to olive gray, soft, wet, trace roots and grass. [NATIVE - YOUNG BAY MUD]
								NOTES: 1. Boring terminated at approximately 10.5 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
15								
20								
25								
30								

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PROJECT NO. 96670

LOG OF BORING NO. KC-7

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

A-9

7/14/2009 12:16:34 PM

Date Completed: **11/6/08**

Drilling method: **6" Hollow Stem Auger**

Logged By: **EMJ**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **Approximately 14.0 ft**

Notes: **Cable winch**

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
								Surface Elevation: Estimated 8.6 feet (MSL)
32	X							LEAN CLAY with SAND and GRAVEL (CL) , medium brown, very stiff, dry, trace organics. [FILL]
29								
22								Becoming hard at 4.0 ft.
5								At 4.0 to 4.5 ft, transitions to SC.
23			11:40 am 11/6/08					CLAYEY SAND (SC) , medium brown, medium dense, wet, with angular gravel up to 2", fine to medium sand, trace roots and grass.
10								
74								Becoming dense at 8.0 ft. High blow counts due to 2 1/2" gravel cobble at bottom of sampler.
10								SANDY GRAVEL (GP) , medium brown and gray, medium dense, wet, trace clay, fine to coarse sand, fine to medium angular gravel. [FILL]
10								
9								Becoming loose at 11.0 ft.
15								FAT CLAY (CH) , dark olive gray and black mottled, medium stiff, wet, trace sand, up to 20% organic material. [NATIVE - YOUNG BAY MUD]
								NOTES: 1. Boring terminated at approximately 14.0 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
20								
25								
30								

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PROJECT NO. **96670**

LOG OF BORING NO. KC-8

Las Gallinas Levee Evaluation
Santa Venetia
San Rafael, California

PLATE

A-10

7/14/2009 12:16:34 PM

Date Completed: 11/6/08 Drilling method: 6" Hollow Stem Auger

Logged By: EMJ

Total Depth: Approximately 9.6 ft Hammer Wt: 140 lbs., 30" drop
 Notes: Cable winch

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
								Surface Elevation: Estimated 9.4 feet (MSL)
75	X							 CLAYEY GRAVEL with SAND (GC) , dark yellowish brown, hard, moist. [FILL]
34								
55								
5								
23								
7						Passing -#200=25% LL=39; PI=22		
4								 FAT CLAY with GRAVEL (CH) , dark olive gray and black mottled, very soft, wet, trace organics. [NATIVE - YOUNG BAY MUD]
10		50/1"						 Hard dark gray shale rock (cobble) from 9.5 to 9.6 ft.
								NOTES: 1. Boring terminated at approximately 9.6 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
15								
20								
25								
30								



LOG OF BORING NO. KC-9

PLATE

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

A-11

PROJECT NO. 96670

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7/14/2009 12:16:35 PM

Date Completed: 11/6/08 Drilling method: 6" Hollow Stem Auger

Logged By: EMJ

Total Depth: Approximately 14.0 ft Hammer Wt: 140 lbs., 30" drop
 Notes: Cable winch

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
	Surface Elevation: Estimated 9.0 feet (MSL)							
22							4.5	FAY CLAY with SAND (CH) , dark grayish brown, hard, moist, approximately 20% fine grained sand. [FILL]
26								
25			76	34.1	2.72 @ 3.5%	TXUU:Su = 2.7 ksf LL=73; PI=40		
25								
28								CLAYEY SAND with GRAVEL (SC) , reddish brown, dense, wet, some rock fragments, fine to medium sand. [FILL]
59								
10								
13								FAT CLAY (CH) , dark olive and black mottled, stiff, wet, up to 60% organics in 1" to 2" seams, isolated roots. [NATIVE - YOUNG BAY MUD]
						TV:Su = 0.48 ksf		
15								NOTES: 1. Boring terminated at approximately 14.0 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
20								
25								
30								

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PROJECT NO. 96670

LOG OF BORING NO. KC-10

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

A-12

7/14/2009 12:16:27 PM

Date Completed: **11/4/08**

Drilling method: **4" Solid Stem Auger**

Logged By: **EMJ**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **Approximately 13.5 ft**

Notes: **Rope and cathead hammer, 4" diameter hole.**

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
6								SANDY CLAY (CL) , brownish gray, medium stiff to very stiff, moist, low to medium plasticity clay, fine sand. [FILL]
7								
11								
5	13							
11			96	20.0	1.51 @ 15.0%	TXUU:Su = 1.5 ksf Passing #200=49%		CLAYEY GRAVEL with SAND (GC) , very dark grayish brown, loose, moist, fine to medium sand, fine angular gravel, low to medium plasticity clay. [FILL]
9								LEAN CLAY (CL) , gray with brown staining, medium stiff to stiff, moist, trace fine sand, trace organics. [FILL]
10	4							FAT CLAY (CH) , dark olive gray, soft, wet, trace small roots. [NATIVE - YOUNG BAY MUD]
3						TV:Su = 0.20 ksf		
15								NOTES: 1. Boring terminated at approximately 13.5 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
20								
25								
30								



LOG OF BORING NO. K-AP1

PLATE

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

A-13

PROJECT NO. 96670

Date Completed: 11/4/08

Drilling method: 4" Solid Stem Auger

Logged By: EMJ

Hammer Wt: 140 lbs., 30" drop

Total Depth: Approximately 9.0 ft

Notes: Rope and cathead hammer, 4" diameter hole.

Depth, ft	FIELD		LABORATORY				Pen, tsf	DESCRIPTION
	Sample	Blows/ft	Dry Density pcf	Moisture Content %	Compress. Strength tsf	Other Tests		
5								SILT (ML) , light brown, medium stiff, dry, trace roots, trace sand and gravel. [FILL]
9								LEAN CLAY (CL) , olive gray, hard, moist, trace roots, some orange weathered zones, some silt, low to medium plasticity. [FILL]
7								
5								FAT CLAY (CH) , dark gray and olive, soft, moist, some grass and roots, trace sand. [DISTURBED YOUNG BAY MUD FILL] Becoming wet, very soft at 6.0 ft.
2								
0								
10								NOTES: 1. Boring terminated at approximately 9.0 feet below ground surface. 2. Boring backfilled with cement-bentonite grout. 3. Blow counts shown are raw, uncorrected values.
15								
20								
25								
30								

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PROJECT NO. 96670

LOG OF BORING NO. K-AP2

Las Gallinas Levee Evaluation
Santa Venetia
San Rafael, California

PLATE

A-14

7/14/2009 12:16:25 PM

APPENDIX B

LOGS OF CONE PENETRATION TESTS



GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

November 7, 2008

Kleinfelder
Attn: Craig Hall
1970 Broadway St., Suite 710
Oakland, California 94612

Subject: CPT Site Investigation
Las Gallinas Creek Levee
San Rafael, California
GREGG Project Number: 08-282MA

Dear Mr. Hall:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	<input checked="" type="checkbox"/>
2	Pore Pressure Dissipation Tests	(PPD)	<input checked="" type="checkbox"/>
3	Seismic Cone Penetration Tests	(SCPTU)	<input type="checkbox"/>
4	Resistivity Cone Penetration Tests	(RCPTU)	<input type="checkbox"/>
5	UVOST Laser Induced Fluorescence	(UVOST)	<input type="checkbox"/>
6	Groundwater Sampling	(GWS)	<input type="checkbox"/>
7	Soil Sampling	(SS)	<input type="checkbox"/>
8	Vapor Sampling	(VS)	<input type="checkbox"/>
9	Vane Shear Testing	(VST)	<input type="checkbox"/>
10	SPT Energy Calibration	(SPTC)	<input type="checkbox"/>

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager



Bibliography

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Site Characterization, Robertson & Mayne (editors), 1998 Balkema, Rotterdam, ISBN 90 5410 939 4 pp 35-47.

Campanella, R.G. and I. Weemeees, "Development and Use of An Electrical Resistivity Cone for Groundwater
Contamination Studies", Canadian Geotechnical Journal, Vol. 27 No. 5, 1990 pp. 557-567.

DeGroot, D.J. and A.J. Lutenegeger, "Reliability of Soil Gas Sampling and Characterization Techniques", International
Site Characterization Conference - Atlanta, 1998.

Woeller, D.J., P.K. Robertson, T.J. Boyd and Dave Thomas, "Detection of Polyaromatic Hydrocarbon Contaminants
Using the UVIF-CPT", 53rd Canadian Geotechnical Conference Montreal, QC October pp. 733-739, 2000.

Zemo, D.A., T.A. Delfino, J.D. Gallinatti, V.A. Baker and L.R. Hilpert, "Field Comparison of Analytical Results from
Discrete-Depth Groundwater Samplers" BAT EnviroProbe and OED HydroPunch, Sixth national Outdoor Action
Conference, Las Vegas, Nevada Proceedings, 1992, pp 299-312.

Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure (CPT)

Gregg Drilling & Testing, Inc. carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, Figure CPT. The soundings are conducted using a 20 ton capacity cone with a tip area of 15 cm² and a friction sleeve area of 225 cm². The cone is designed with an equal end area friction sleeve and a tip end area ratio of 0.80.

The cone takes measurements of cone bearing (q_c), sleeve friction (f_s) and dynamic pore water pressure (u_2) at 5-cm intervals during penetration to provide a nearly continuous hydrogeologic log. CPT data reduction and interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored on disk for further analysis and reference. All CPT soundings are performed in accordance with revised (2002) ASTM standards (D 5778-95).

The cone also contains a porous filter element located directly behind the cone tip (u_2), Figure CPT. It consists of porous plastic and is 5.0mm thick. The filter element is used to obtain dynamic pore pressure as the cone is advanced as well as Pore Pressure Dissipation Tests (PPDT's) during appropriate pauses in penetration. It should be noted that prior to penetration, the element is fully saturated with silicon oil under vacuum pressure to ensure accurate and fast dissipation.

When the soundings are complete, the test holes are grouted using a Gregg Drilling support rig. The grouting procedure consists of pushing a hollow CPT rod with a "knock out" plug to the termination depth of the test hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

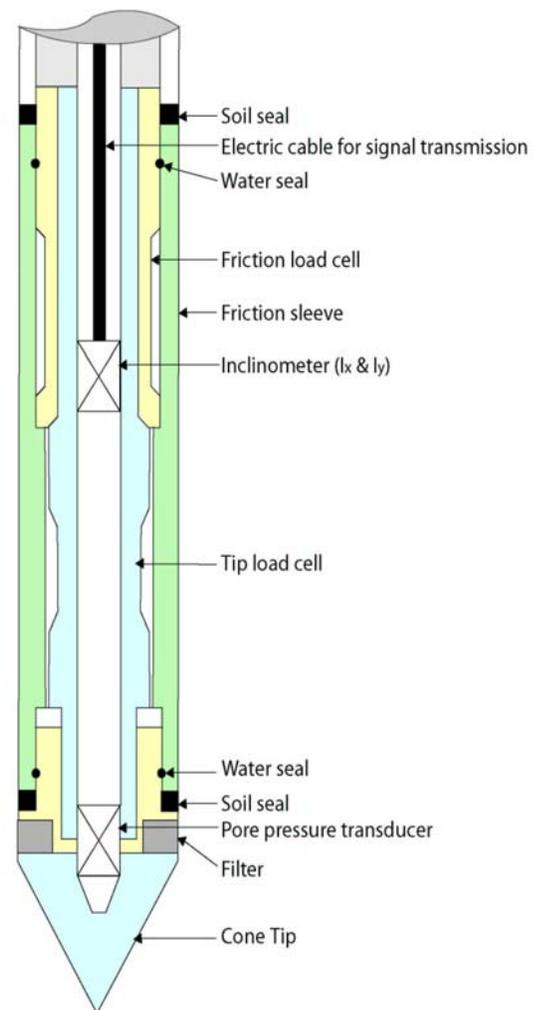


Figure CPT



Cone Penetration Test (CPT) Interpretation

Gregg have recently updated their CPT interpretation and plotting software (2007). The software takes the CPT data and performs basic interpretation in terms of soil behavior type (SBT) and various geotechnical parameters using current published empirical correlations based on the comprehensive review by Lunne, Robertson and Powell (1997). The interpretation is presented in tabular format using MS Excel. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

The following provides a summary of the methods used for the interpretation. Many of the empirical correlations to estimate geotechnical parameters have constants that have a range of values depending on soil type, geologic origin and other factors. The software uses 'default' values that have been selected to provide, in general, conservatively low estimates of the various geotechnical parameters.

Input:

- 1 Units for display (Imperial or metric) (atm. pressure, $p_a = 0.96$ tsf or 0.1 MPa)
- 2 Depth interval to average results, (ft or m). Data are collected at either 0.02 or 0.05m and can be averaged every 1, 3 or 5 intervals.
- 3 Elevation of ground surface (ft or m)
- 4 Depth to water table, z_w (ft or m) – input required
- 5 Net area ratio for cone, a (default to 0.80)
- 6 Relative Density constant, C_{Dr} (default to 350)
- 7 Young's modulus number for sands, α (default to 5)
- 8 Small strain shear modulus number
 - a. for sands, S_G (default to 180 for SBT_n 5, 6, 7)
 - b. for clays, C_G (default to 50 for SBT_n 1, 2, 3 & 4)
- 9 Undrained shear strength cone factor for clays, N_{kt} (default to 15)
- 10 Over Consolidation ratio number, k_{ocr} (default to 0.3)
- 11 Unit weight of water, (default to $\gamma_w = 62.4$ lb/ft³ or 9.81 kN/m³)

Column

- 1 Depth, z , (m) – CPT data is collected in meters
- 2 Depth (ft)
- 3 Cone resistance, q_c (tsf or MPa)
- 4 Sleeve friction, f_s (tsf or MPa)
- 5 Penetration pore pressure, u (psi or MPa), measured behind the cone (i.e. u_2)
- 6 Other – any additional data, if collected, e.g. electrical resistivity or UVIF
- 7 Total cone resistance, q_t (tsf or MPa) $q_t = q_c + u(1-a)$
- 8 Friction Ratio, R_f (%) $R_f = (f_s/q_t) \times 100\%$
- 9 Soil Behavior Type (non-normalized), SBT see note
- 10 Unit weight, γ (pcf or kN/m³) based on SBT, see note

11	Total overburden stress, σ_v (tsf)	$\sigma_{vo} = \gamma z$
12	Insitu pore pressure, u_o (tsf)	$u_o = \gamma_w (z - z_w)$
13	Effective overburden stress, σ'_{vo} (tsf)	$\sigma'_{vo} = \sigma_{vo} - u_o$
14	Normalized cone resistance, Q_{t1}	$Q_{t1} = (q_t - \sigma_{vo}) / \sigma'_{vo}$
15	Normalized friction ratio, F_r (%)	$F_r = f_s / (q_t - \sigma_{vo}) \times 100\%$
16	Normalized Pore Pressure ratio, B_q	$B_q = u - u_o / (q_t - \sigma_{vo})$
17	Soil Behavior Type (normalized), SBT_n	see note
18	SBT_n Index, I_c	see note
19	Normalized Cone resistance, Q_{tn} (n varies with I_c)	see note
20	Estimated permeability, k_{SBT} (cm/sec or ft/sec)	see note
21	Equivalent SPT N_{60} , blows/ft	see note
22	Equivalent SPT $(N_1)_{60}$ blows/ft	see note
23	Estimated Relative Density, D_r , (%)	see note
24	Estimated Friction Angle, ϕ' , (degrees)	see note
25	Estimated Young's modulus, E_s (tsf)	see note
26	Estimated small strain Shear modulus, G_o (tsf)	see note
27	Estimated Undrained shear strength, s_u (tsf)	see note
28	Estimated Undrained strength ratio	s_u / σ_v'
29	Estimated Over Consolidation ratio, OCR	see note

Notes:

- Soil Behavior Type (non-normalized), SBT listed below Lunne et al. (1997)
- Unit weight, γ either constant at 119 pcf or based on Non-normalized SBT (Lunne et al., 1997 and table below)
- Soil Behavior Type (Normalized), SBT_n Lunne et al. (1997)
- SBT_n Index, I_c $I_c = ((3.47 - \log Q_{t1})^2 + (\log F_r + 1.22)^2)^{0.5}$
- Normalized Cone resistance, Q_{tn} (n varies with I_c)
 $Q_{tn} = ((q_t - \sigma_{vo})/pa) (pa/(\sigma'_{vo})^n$ and recalculate I_c , then iterate:
 When $I_c < 1.64$, $n = 0.5$ (clean sand)
 When $I_c > 3.30$, $n = 1.0$ (clays)
 When $1.64 < I_c < 3.30$, $n = (I_c - 1.64)0.3 + 0.5$
 Iterate until the change in n, $\Delta n < 0.01$
- Estimated permeability, k_{SBT} (based on Normalized SBT_n) (Lunne et al., 1997 and table below)

- 7 Equivalent SPT N_{60} , blows/ft Lunne et al. (1997)
- $$\frac{(q_t/p_a)}{N_{60}} = 8.5 \left(1 - \frac{I_c}{4.6} \right)$$
- 8 Equivalent SPT $(N_1)_{60}$ blows/ft where $C_N = (p_a/\sigma'_{vo})^{0.5}$ $(N_1)_{60} = N_{60} C_N$
- 9 Relative Density, D_r , (%) $D_r^2 = Q_{tn} / C_{Dr}$
Only SBT_n 5, 6, 7 & 8 Show 'N/A' in zones 1, 2, 3, 4 & 9
- 10 Friction Angle, ϕ' , (degrees) $\tan \phi' = \frac{1}{2.68} \left[\log \left(\frac{q_c}{\sigma'_{vo}} \right) + 0.29 \right]$
Only SBT_n 5, 6, 7 & 8 Show 'N/A' in zones 1, 2, 3, 4 & 9
- 11 Young's modulus, E_s $E_s = \alpha q_t$
Only SBT_n 5, 6, 7 & 8 Show 'N/A' in zones 1, 2, 3, 4 & 9
- 12 Small strain shear modulus, G_o
a. $G_o = S_G (q_t \sigma'_{vo} p_a)^{1/3}$ For SBT_n 5, 6, 7
b. $G_o = C_G q_t$ For SBT_n 1, 2, 3 & 4
Show 'N/A' in zones 8 & 9
- 13 Undrained shear strength, s_u $s_u = (q_t - \sigma_{vo}) / N_{kt}$
Only SBT_n 1, 2, 3, 4 & 9 Show 'N/A' in zones 5, 6, 7 & 8
- 14 Over Consolidation ratio, OCR $OCR = k_{ocr} Q_{t1}$
Only SBT_n 1, 2, 3, 4 & 9 Show 'N/A' in zones 5, 6, 7 & 8

SBT Zones**SBT_n Zones**

The following updated and simplified SBT descriptions have been used in the software:

1	sensitive fine grained	1	sensitive fine grained
2	organic soil	2	organic soil
3	clay	3	clay
4	clay & silty clay	4	clay & silty clay
5	clay & silty clay		
6	sandy silt & clayey silt		
7	silty sand & sandy silt	5	silty sand & sandy silt
8	sand & silty sand	6	sand & silty sand
9	sand		
10	sand	7	sand
11	very dense/stiff soil*	8	very dense/stiff soil*
12	very dense/stiff soil*	9	very dense/stiff soil*

* heavily overconsolidated and/or cemented

Track when soils fall with zones of same description and print that description (i.e. if soils fall only within SBT zones 4 & 5, print 'clays & silty clays')

Estimated Permeability (see Lunne et al., 1997)

SBT _n	Permeability (ft/sec)	(m/sec)
1	3×10^{-8}	1×10^{-8}
2	3×10^{-7}	1×10^{-7}
3	1×10^{-9}	3×10^{-10}
4	3×10^{-8}	1×10^{-8}
5	3×10^{-6}	1×10^{-6}
6	3×10^{-4}	1×10^{-4}
7	3×10^{-2}	1×10^{-2}
8	3×10^{-6}	1×10^{-6}
9	1×10^{-8}	3×10^{-9}

Estimated Unit Weight (see Lunne et al., 1997)

SBT	Approximate Unit Weight (lb/ft ³)	(kN/m ³)
1	111.4	17.5
2	79.6	12.5
3	111.4	17.5
4	114.6	18.0
5	114.6	18.0
6	114.6	18.0
7	117.8	18.5
8	120.9	19.0
9	124.1	19.5
10	127.3	20.0
11	130.5	20.5
12	120.9	19.0



Cone Penetration Test Data & Interpretation

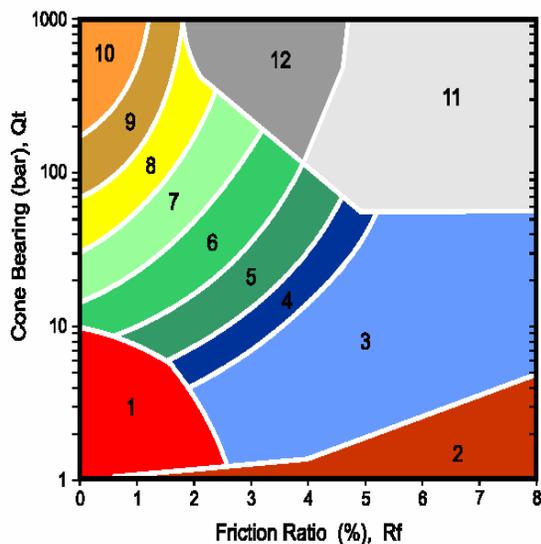
The Cone Penetration Test (CPT) data collected from your site are presented in graphical form in the attached report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings extending greater than 50 feet, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson. The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg InSitu and Gregg Drilling & Testing Inc. do not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and do not assume any liability for any use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software.

Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t , f_s , and u_2 . In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.

(After Robertson, et al., 1986)

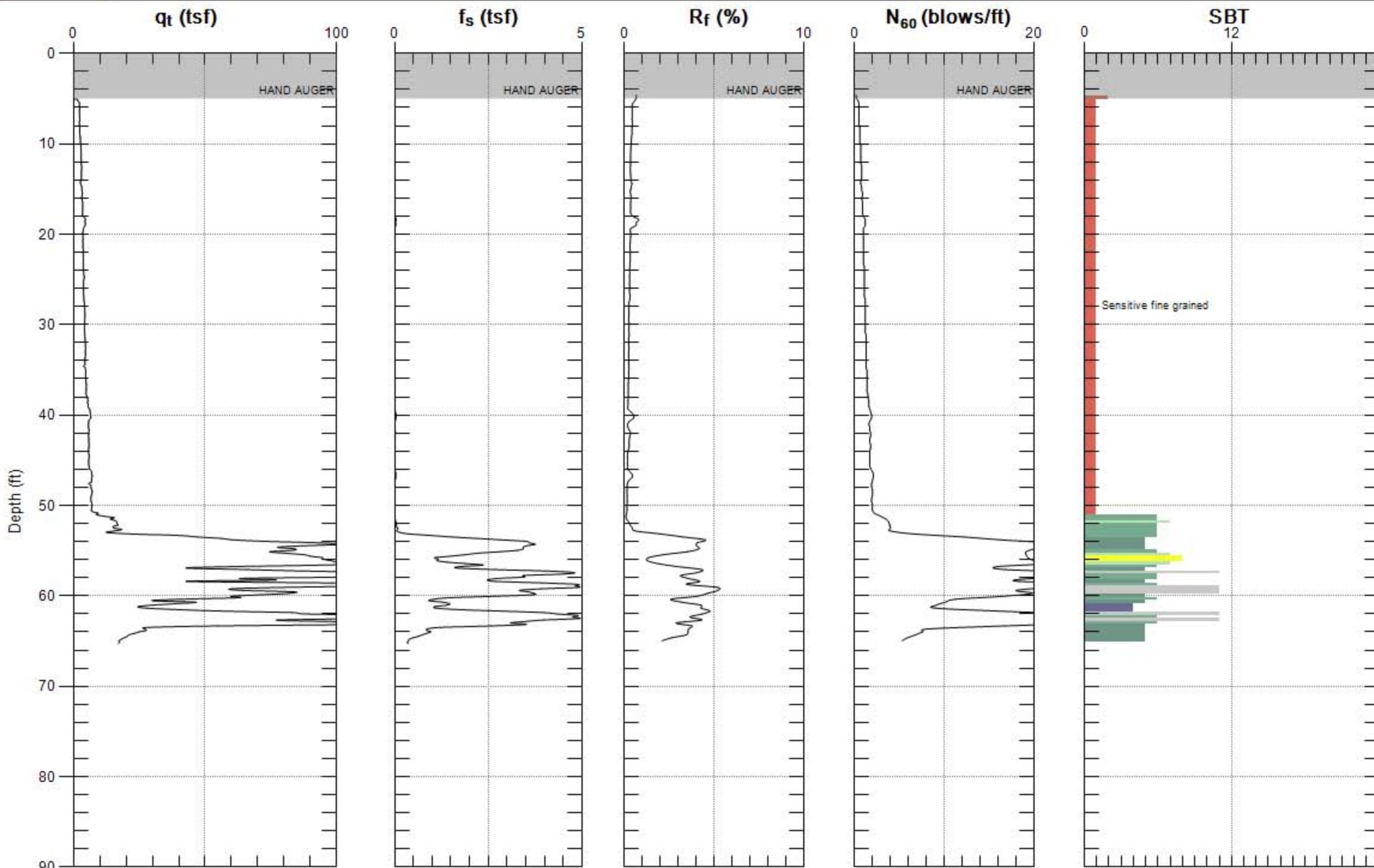


ZONE	SBT
1	Sensitive, fine grained
2	Organic materials
3	Clay
4	Silty clay to clay
5	Clayey silt to silty clay
6	Sandy silt to clayey silt
7	Silty sand to sandy silt
8	Sand to silty sand
9	Sand
10	Gravelly sand to sand
11	Very stiff fine grained*
12	Sand to clayey sand*

*over consolidated or cemented

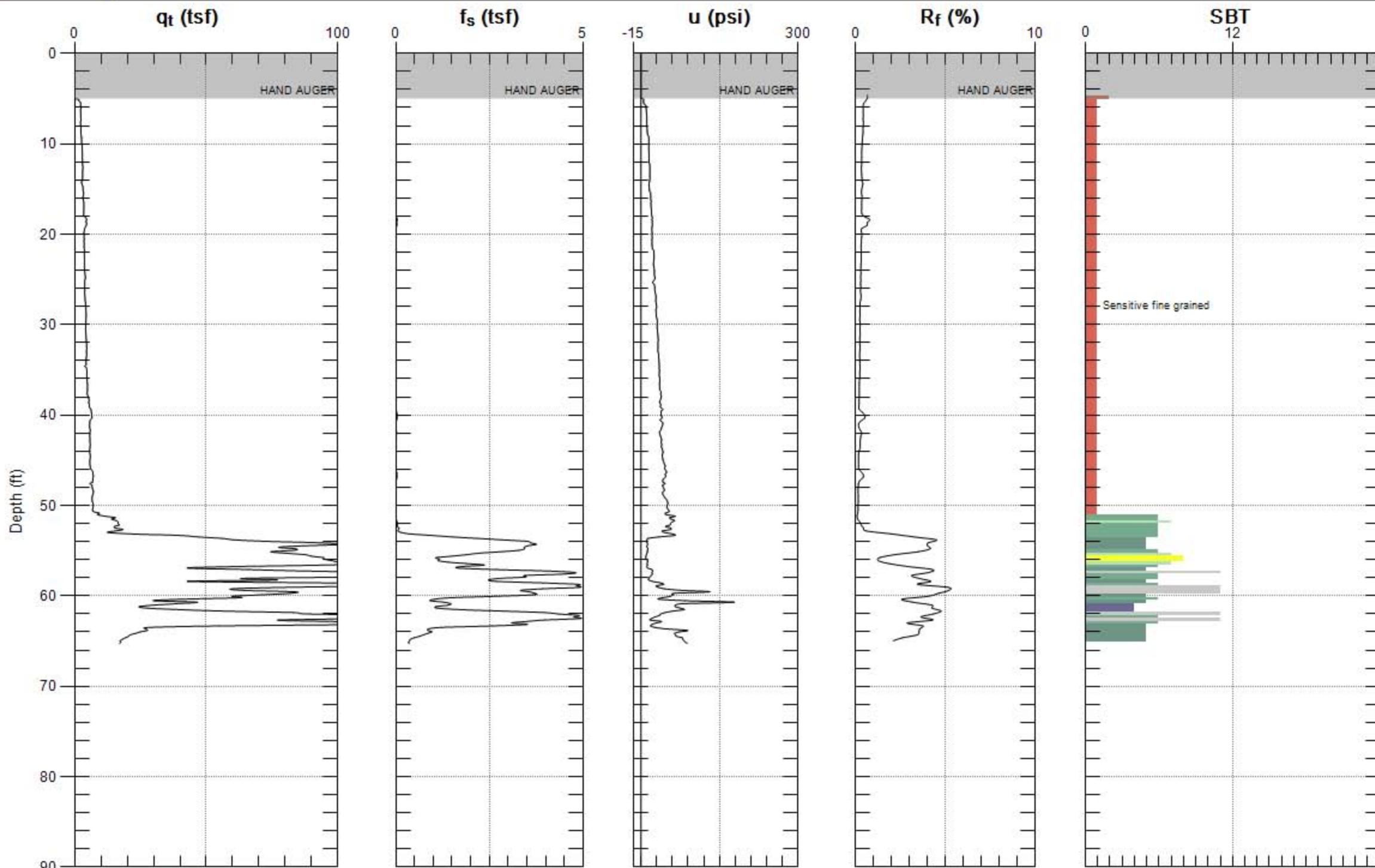
Figure SBT





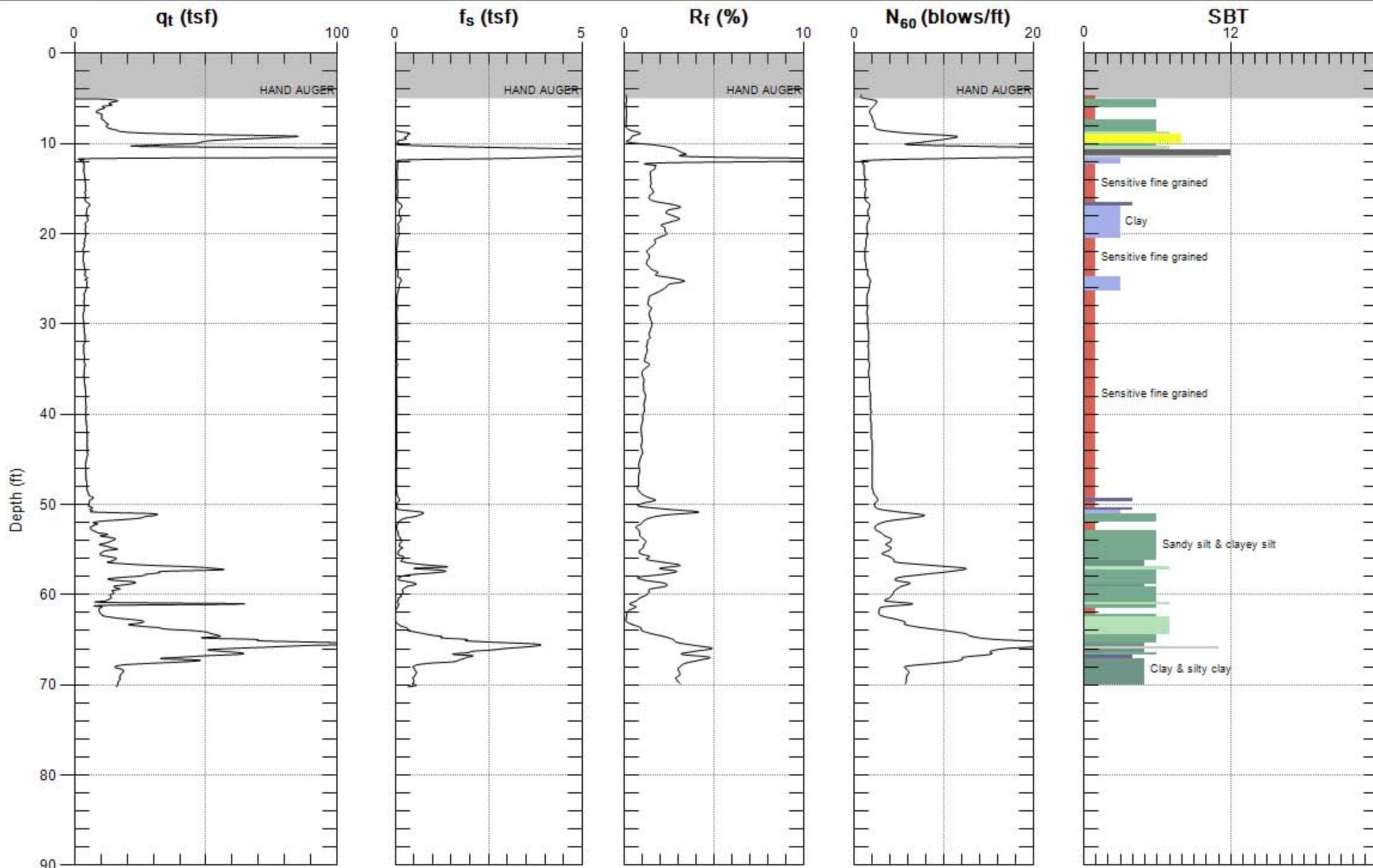
Max. Depth: 65.289 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



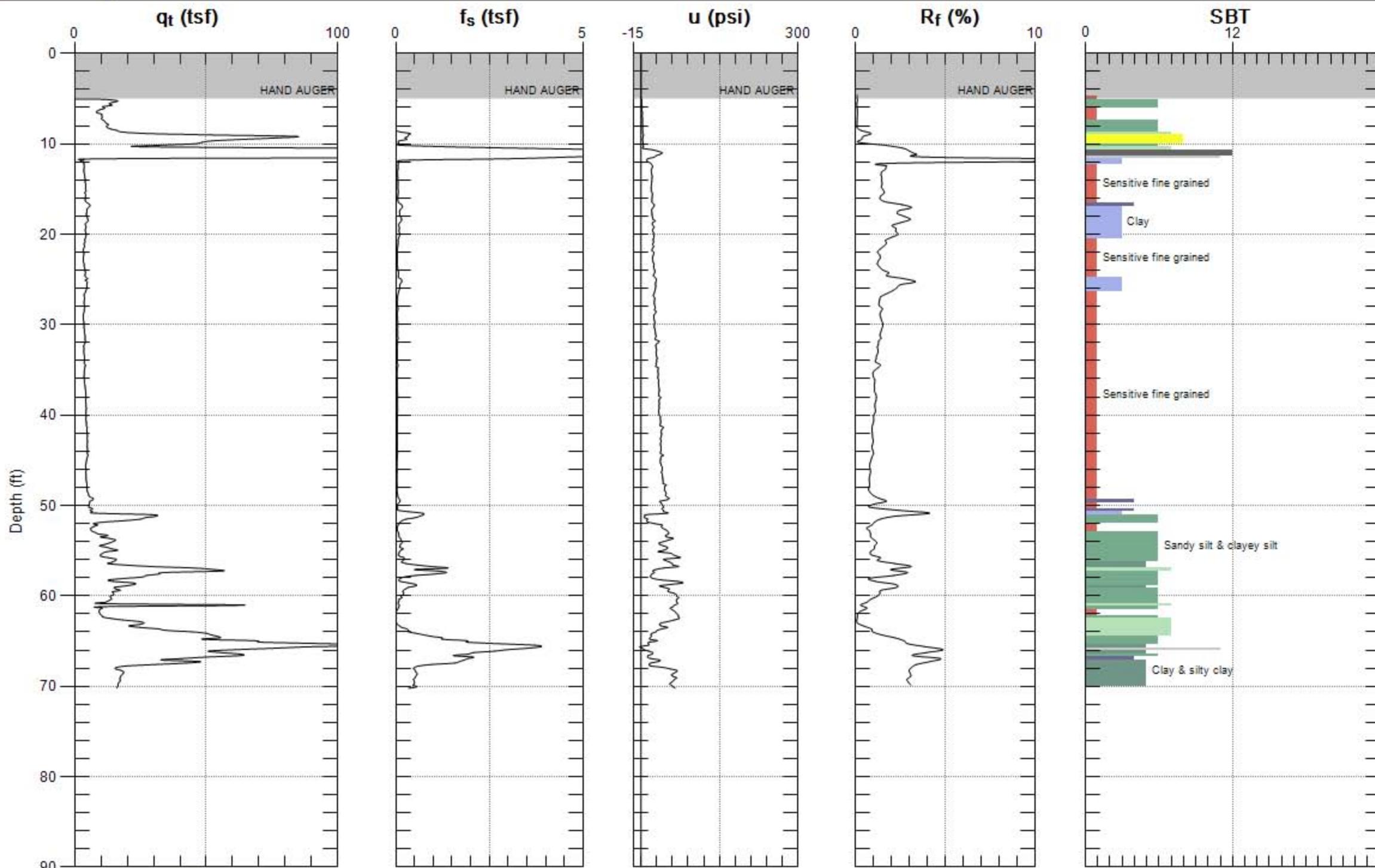
Max. Depth: 65.289 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



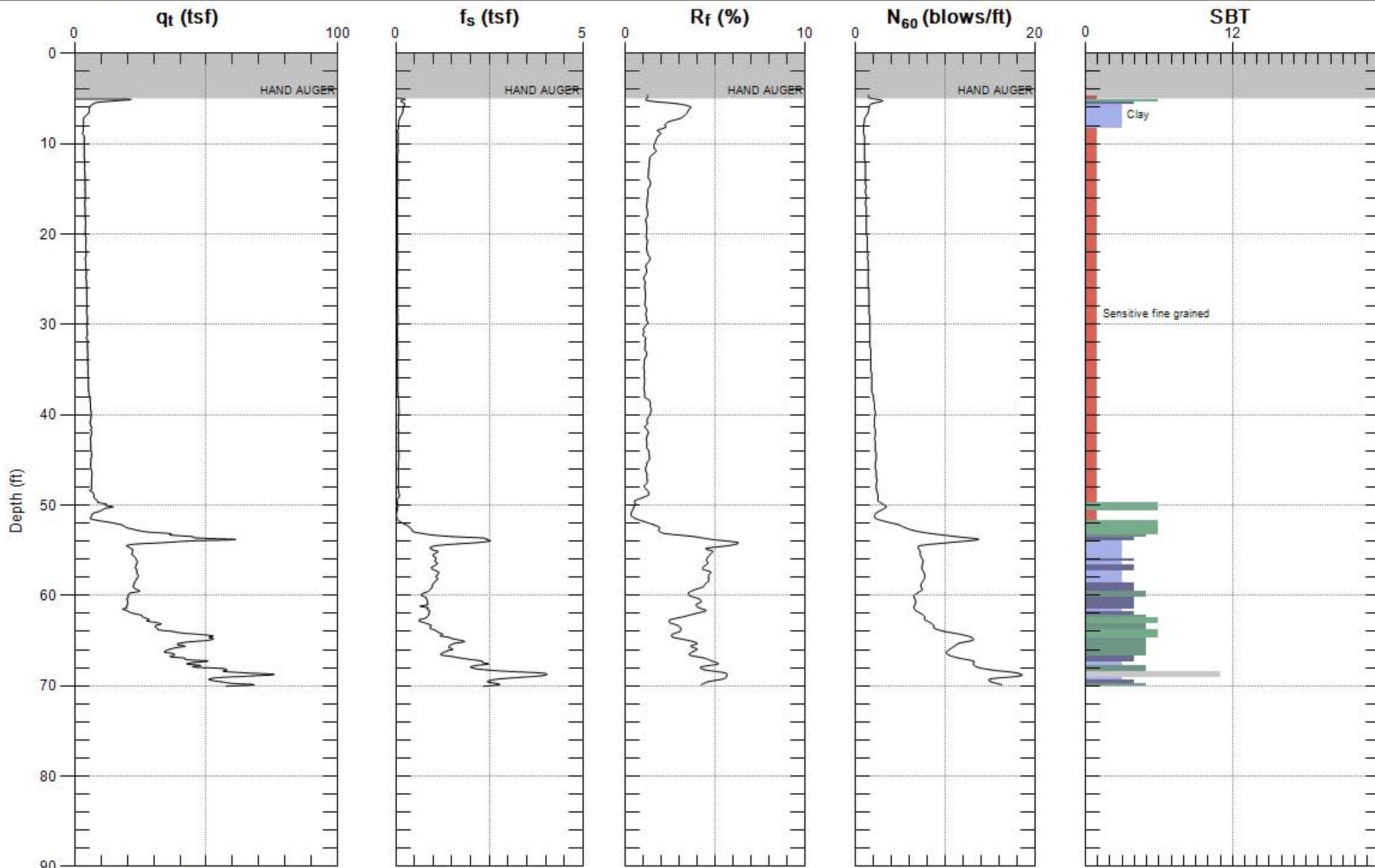
Max. Depth: 70.210 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



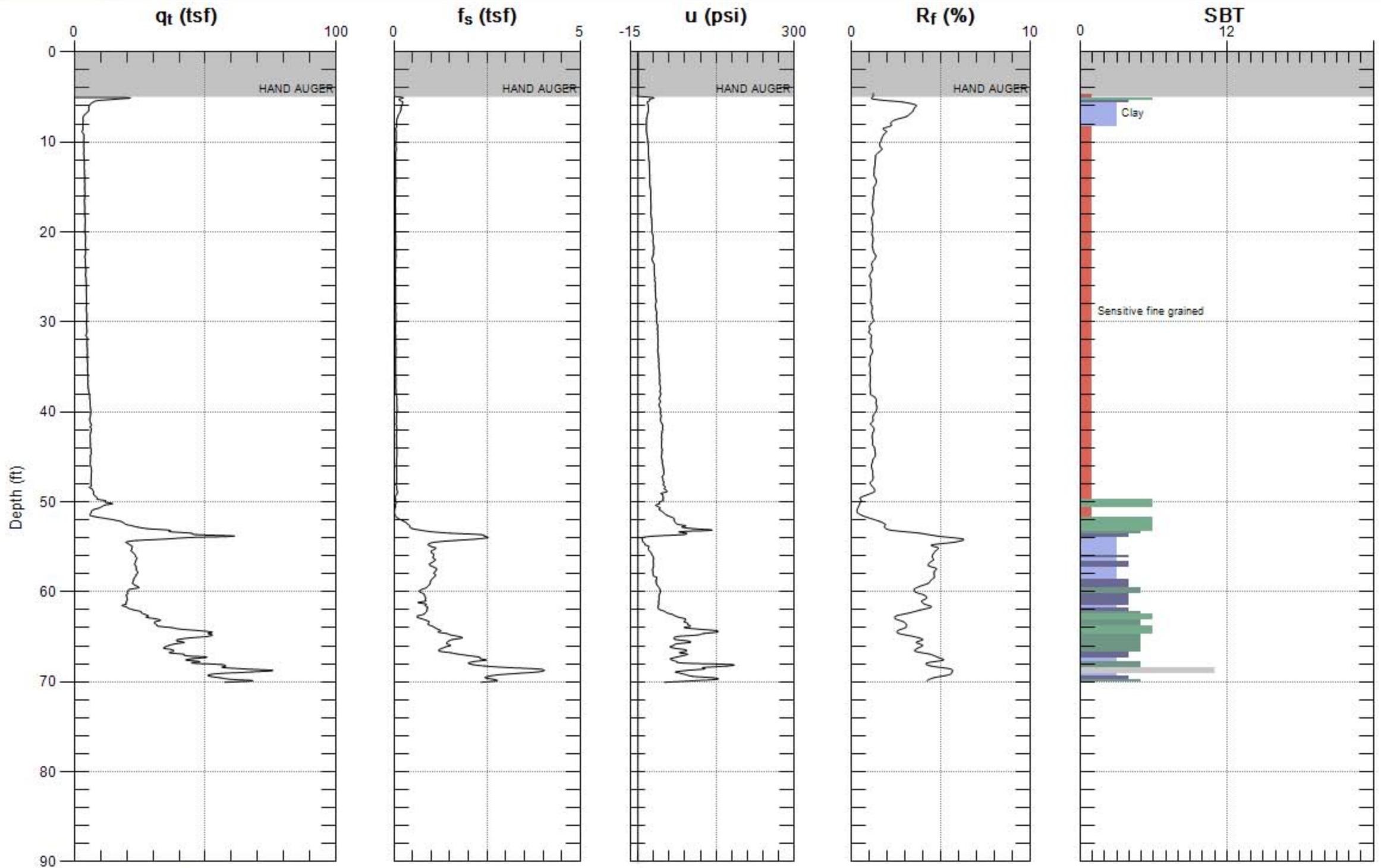
Max. Depth: 70.210 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



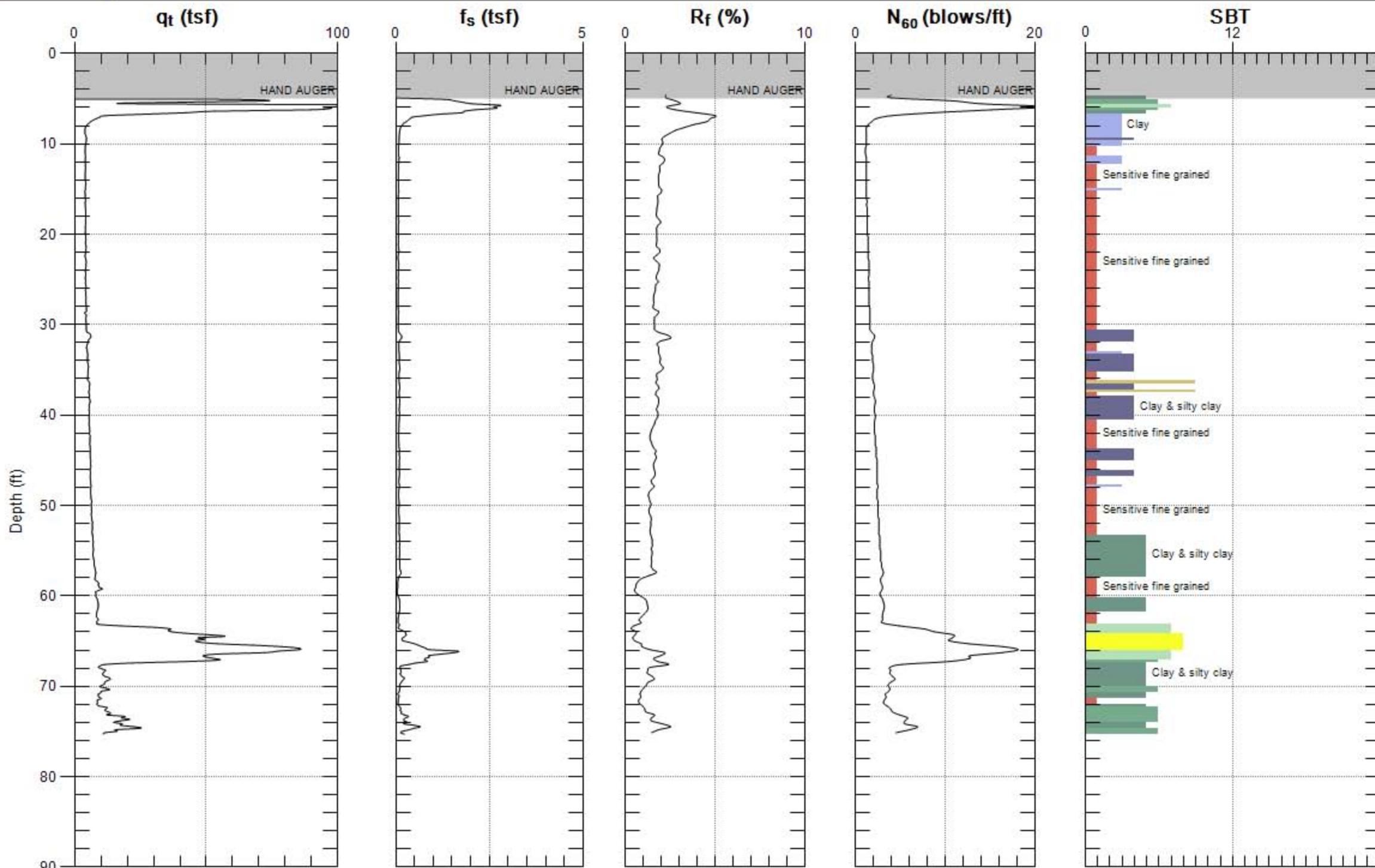
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 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



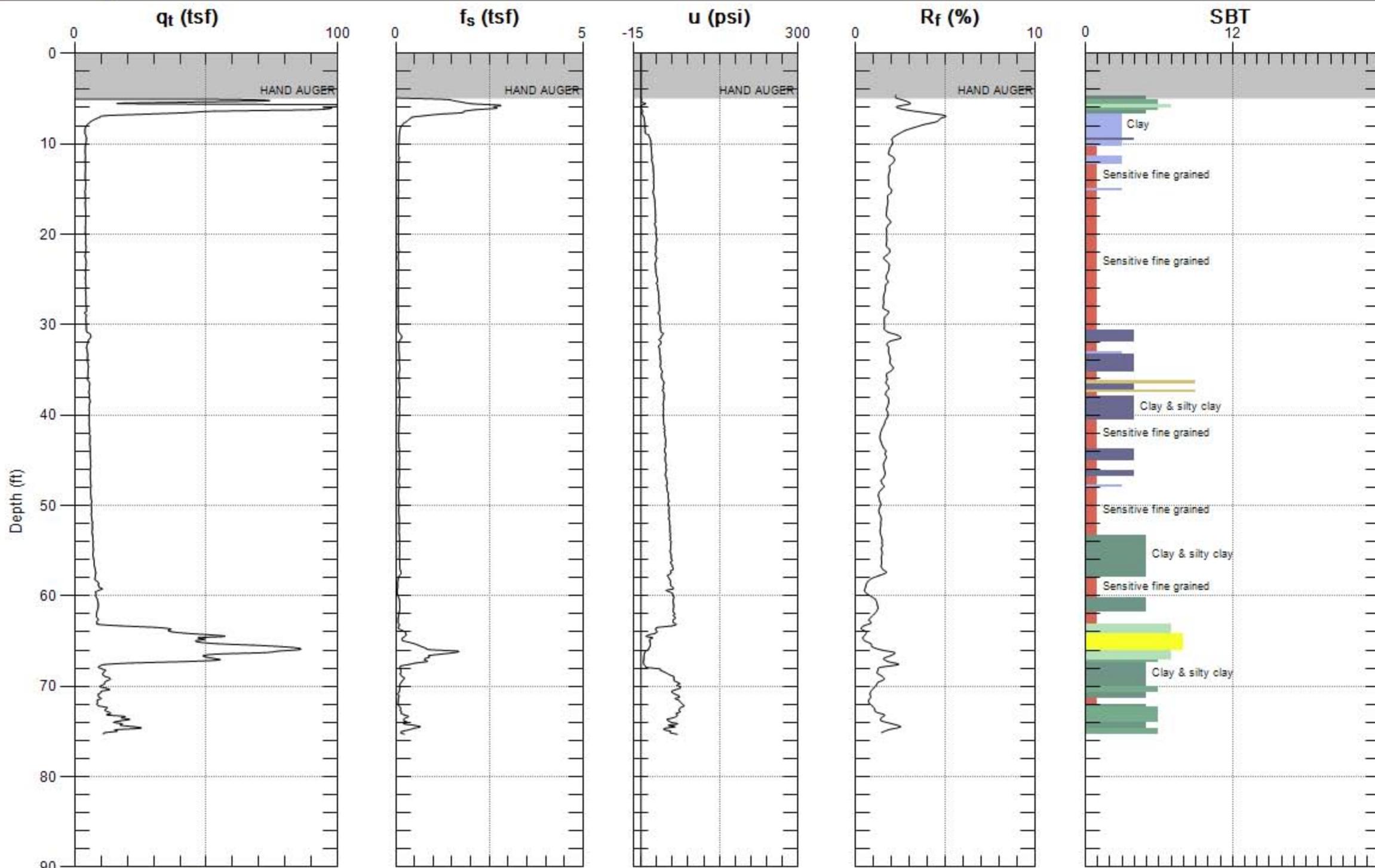
Max. Depth: 70.046 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



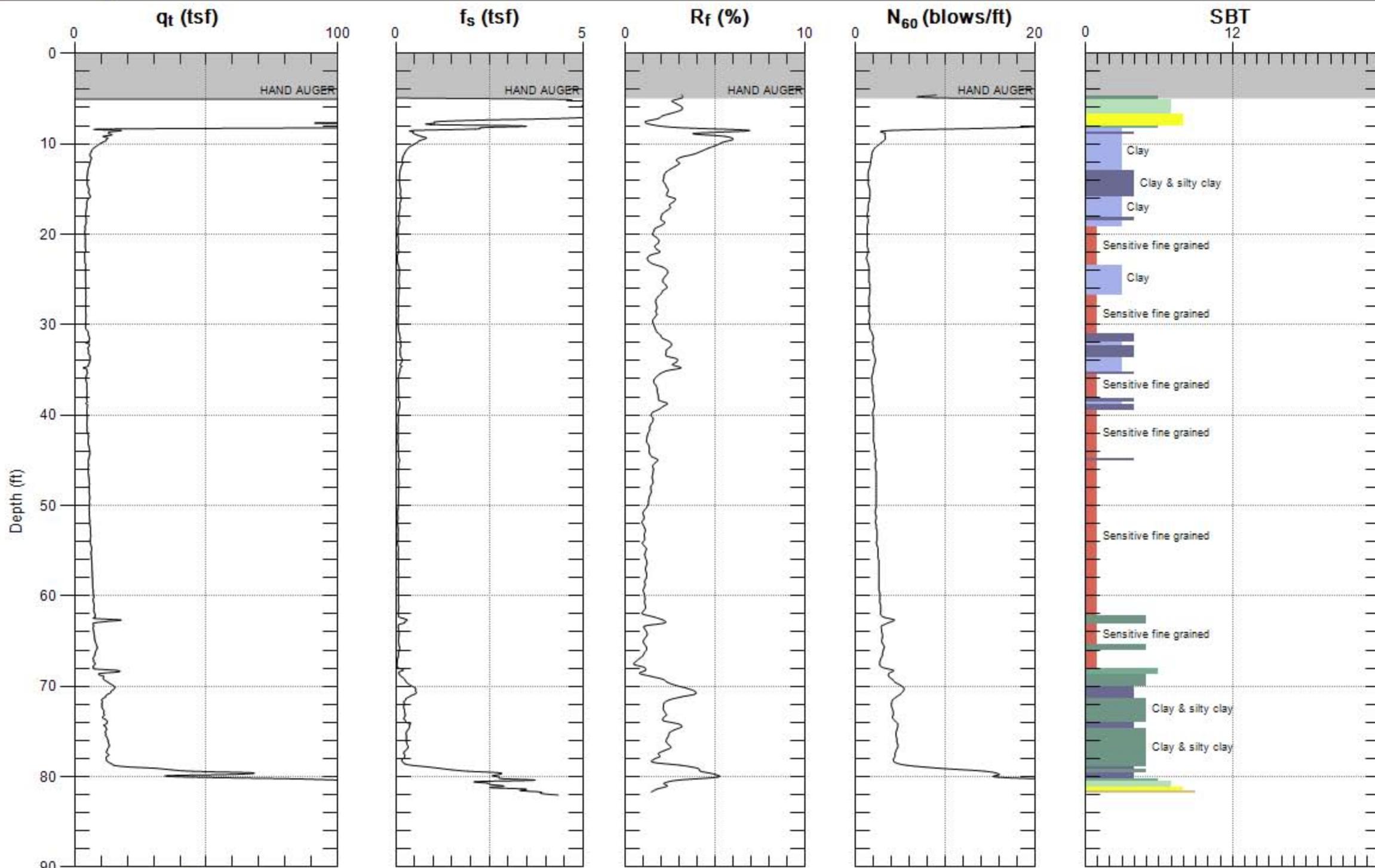
Max. Depth: 75.295 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



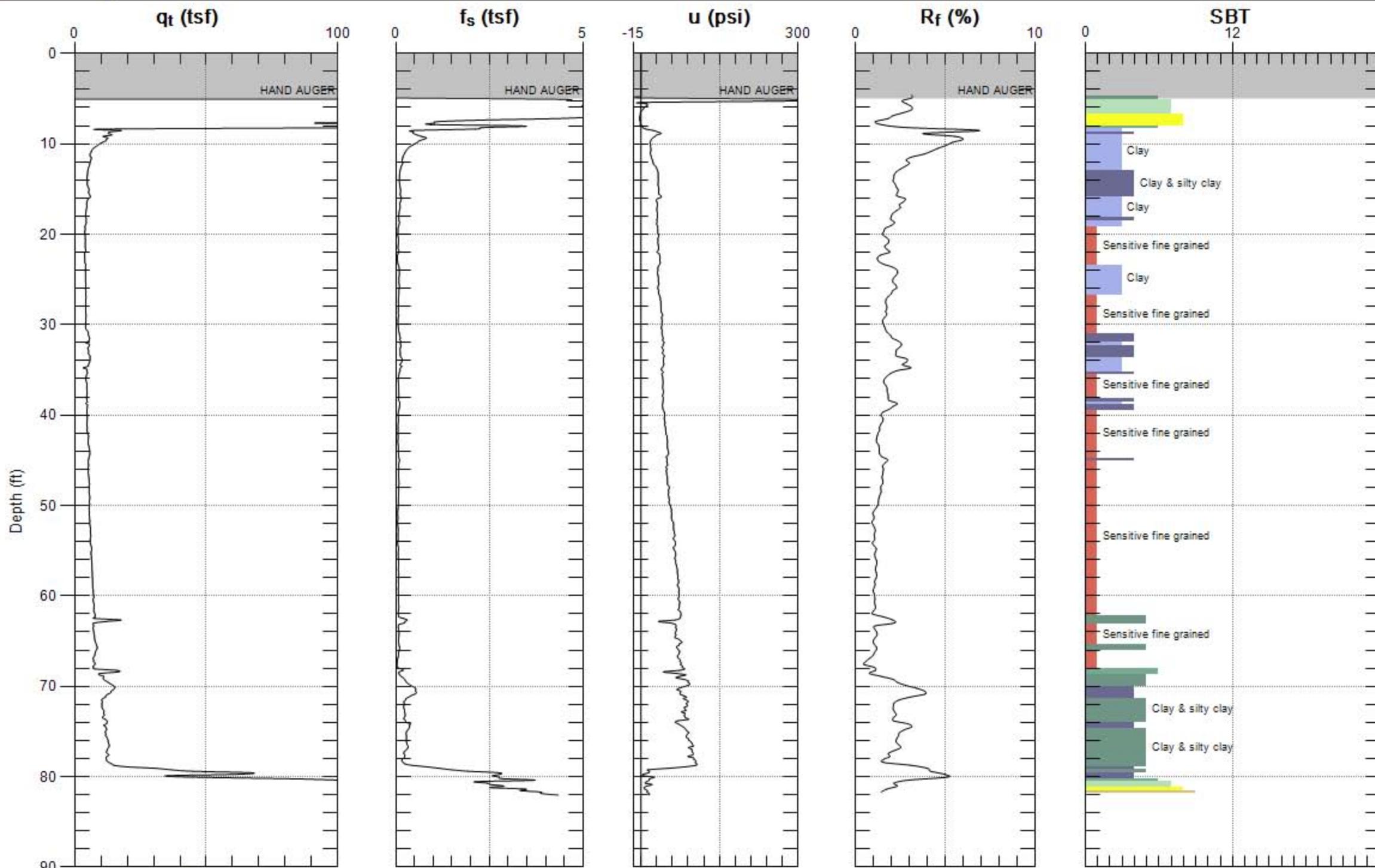
Max. Depth: 75.295 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



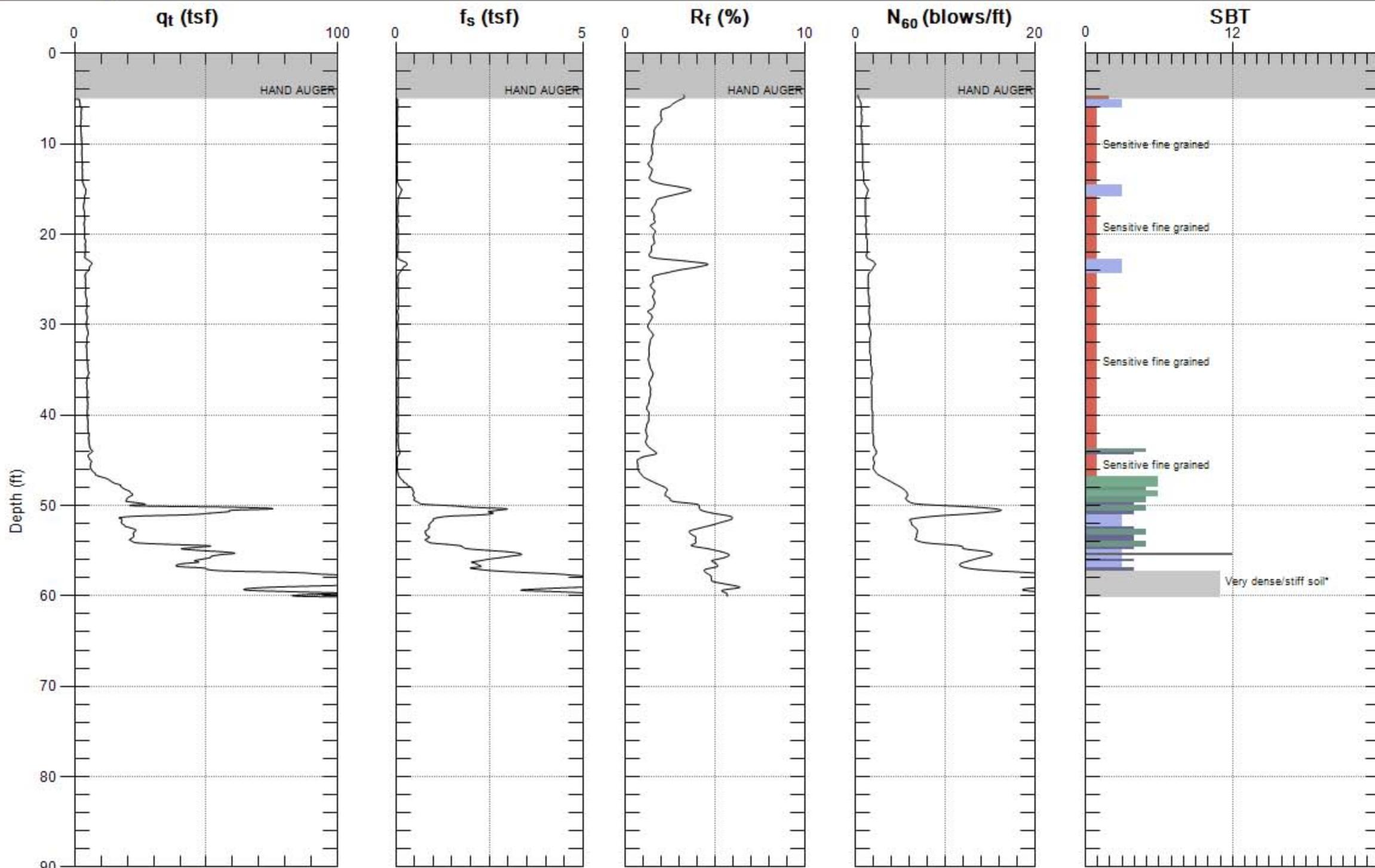
Max. Depth: 82.021 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



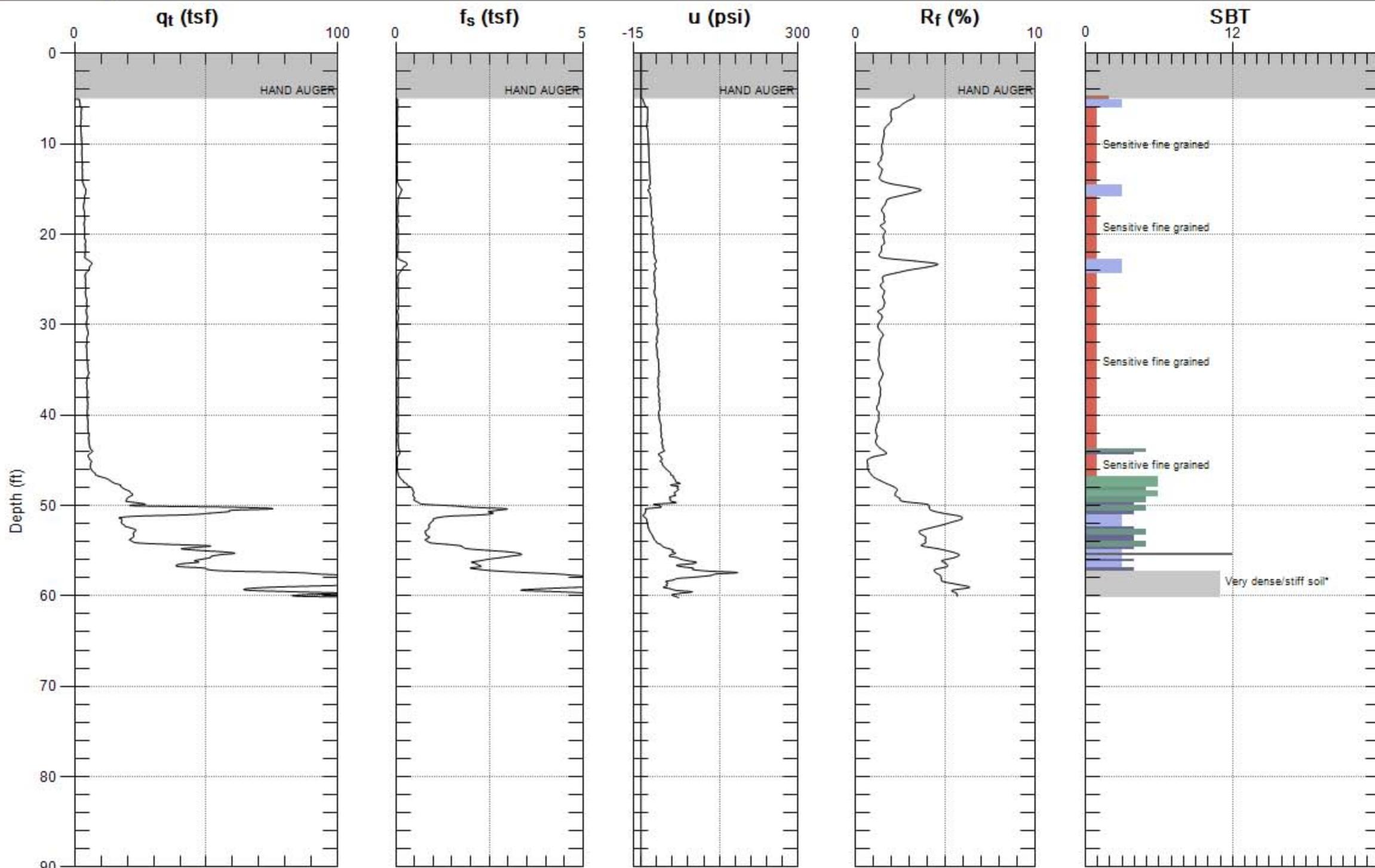
Max. Depth: 82.021 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



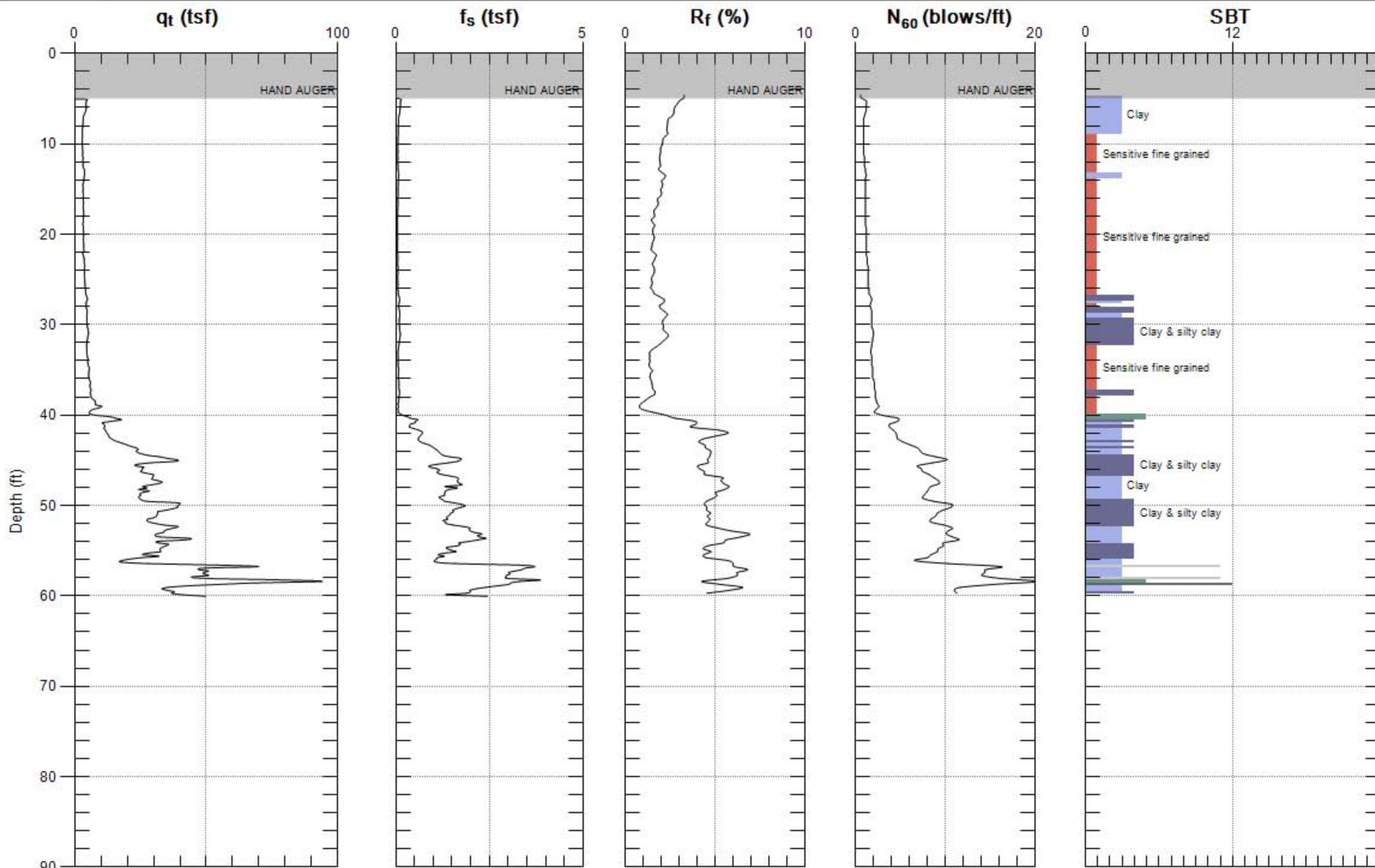
Max. Depth: 60.203 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



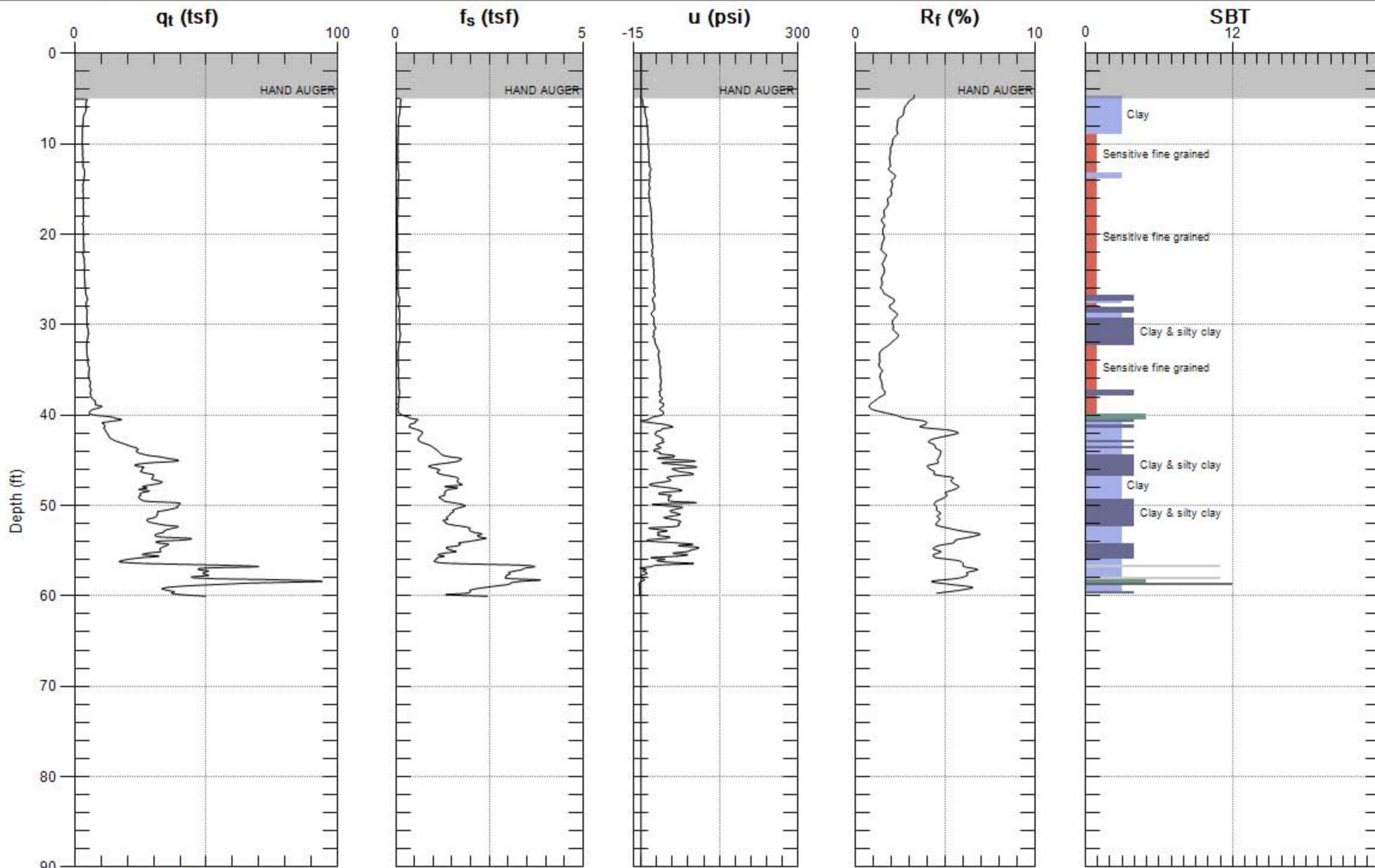
Max. Depth: 60.203 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



Max. Depth: 60.039 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)



Max. Depth: 60.039 (ft)
 Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

Pore Pressure Dissipation Tests (PPDT)

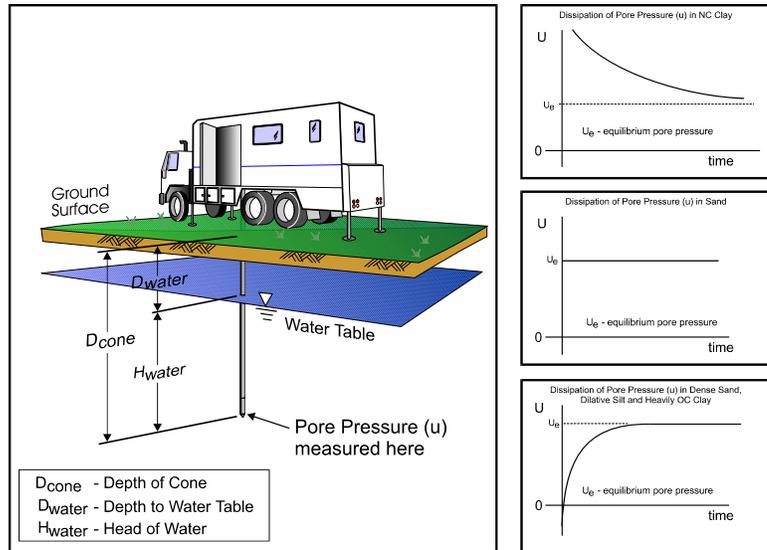
Pore Pressure Dissipation Tests (PPDT's) conducted at various intervals measured hydrostatic water pressures and determined the approximate depth of the ground water table. A PPDT is conducted when the cone is halted at specific intervals determined by the field representative. The variation of the penetration pore pressure (u) with time is measured behind the tip of the cone and recorded by a computer system.

Pore pressure dissipation data can be interpreted to provide estimates of:

- Equilibrium piezometric pressure
- Phreatic Surface
- In situ horizontal coefficient of consolidation (c_h)
- In situ horizontal coefficient of permeability (k_h)

In order to correctly interpret the equilibrium piezometric pressure and/or the phreatic surface, the pore pressure must be monitored until such time as there is no variation in pore pressure with time, *Figure PPDT*. This time is commonly referred to as t_{100} , the point at which 100% of the excess pore pressure has dissipated.

A complete reference on pore pressure dissipation tests is presented by Robertson et al. 1991.



Water Table Calculation

$$D_{\text{water}} = D_{\text{cone}} - H_{\text{water}}$$

where $H_{\text{water}} = U_e$ (depth units)

Useful Conversion Factors: 1psi = 0.704m = 2.31 feet (water)
 1tsf = 0.958 bar = 13.9 psi
 1m = 3.28 feet

Figure PPDT

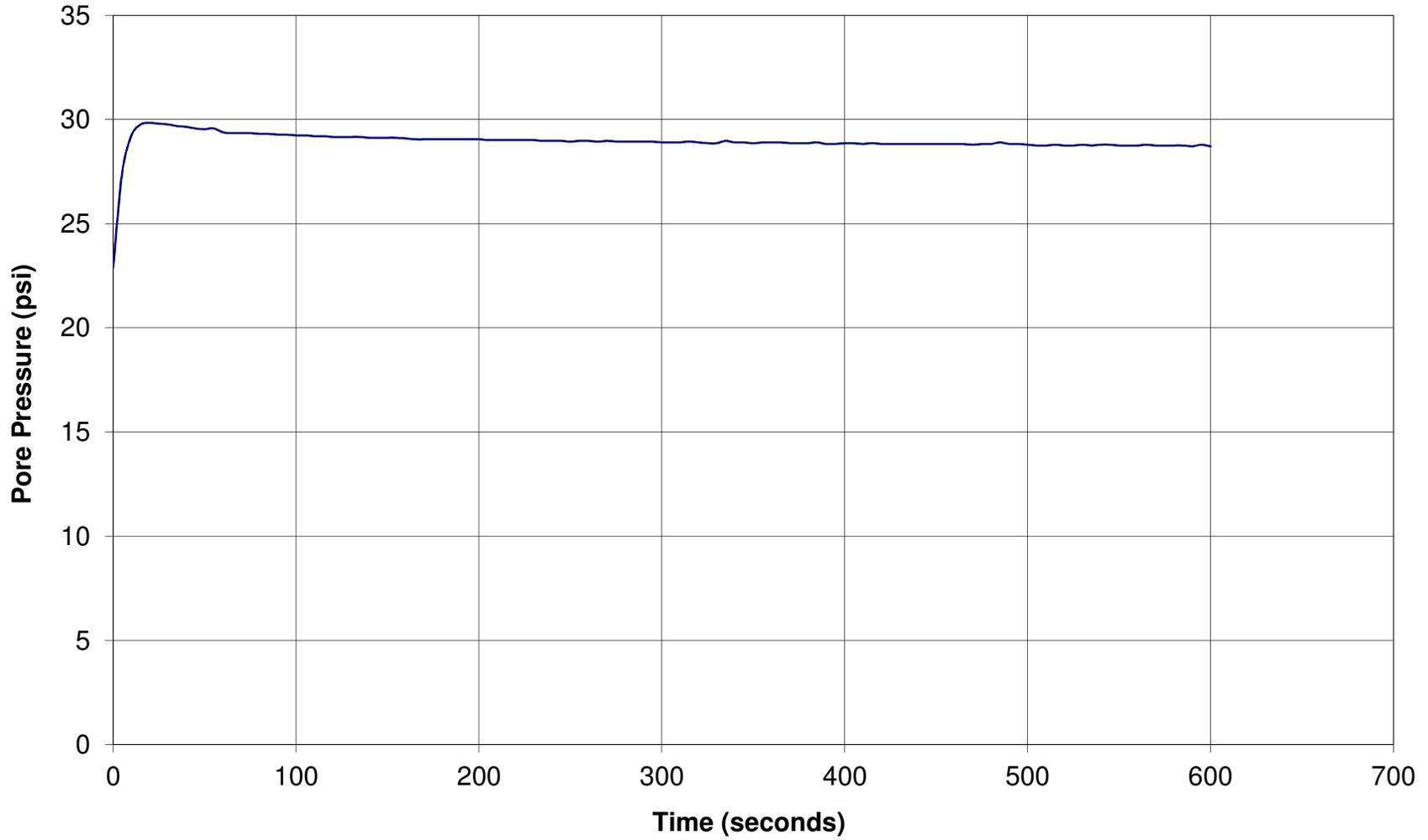




GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: KCPT-2
Depth: 64.304
Site: LAS GALLINAS CREEK
Engineer: E.MORLEY JOHN

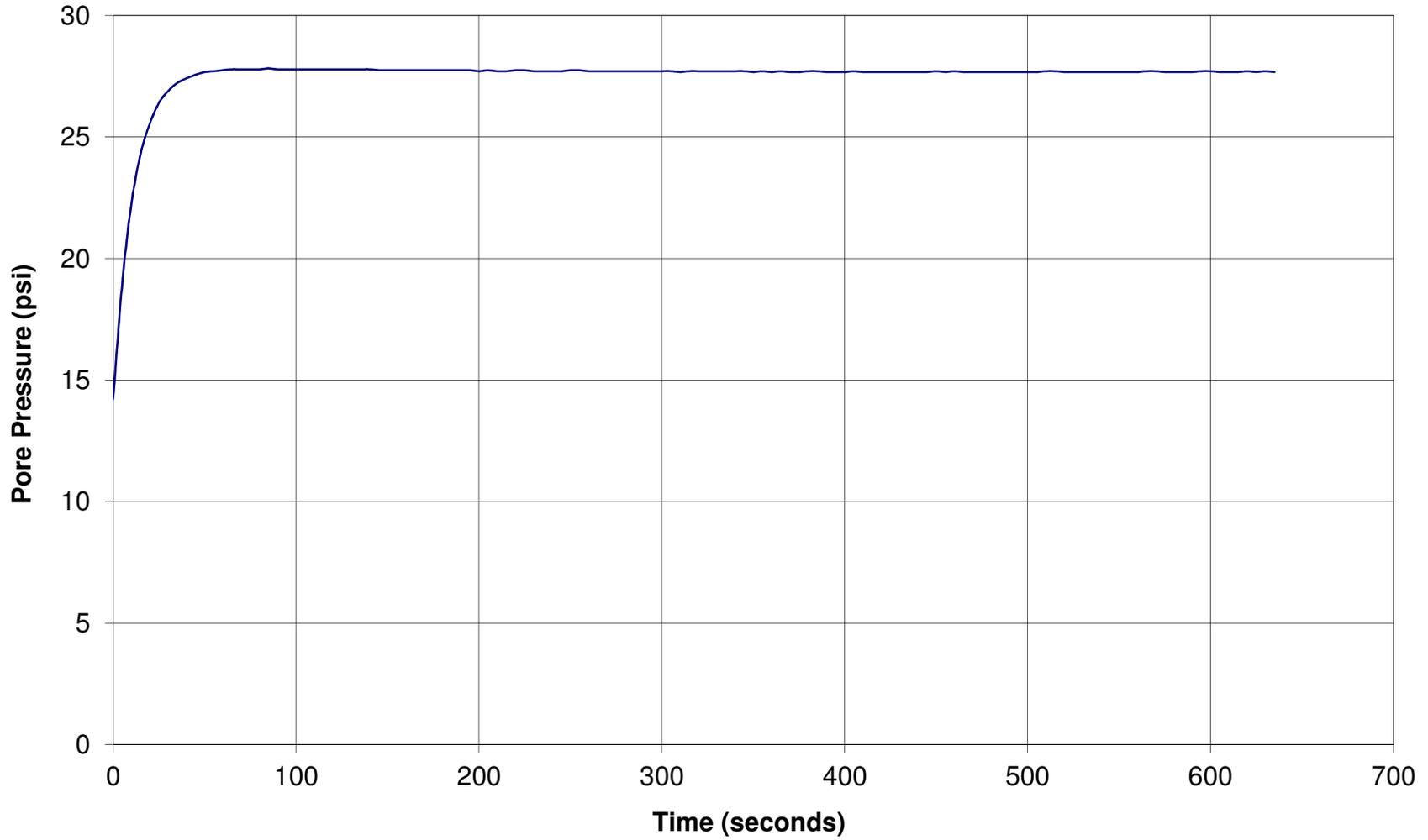




GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: KCPT-4
Depth: 64.468
Site: LAS GALLINAS CREEK
Engineer: E.MORLEY JOHN

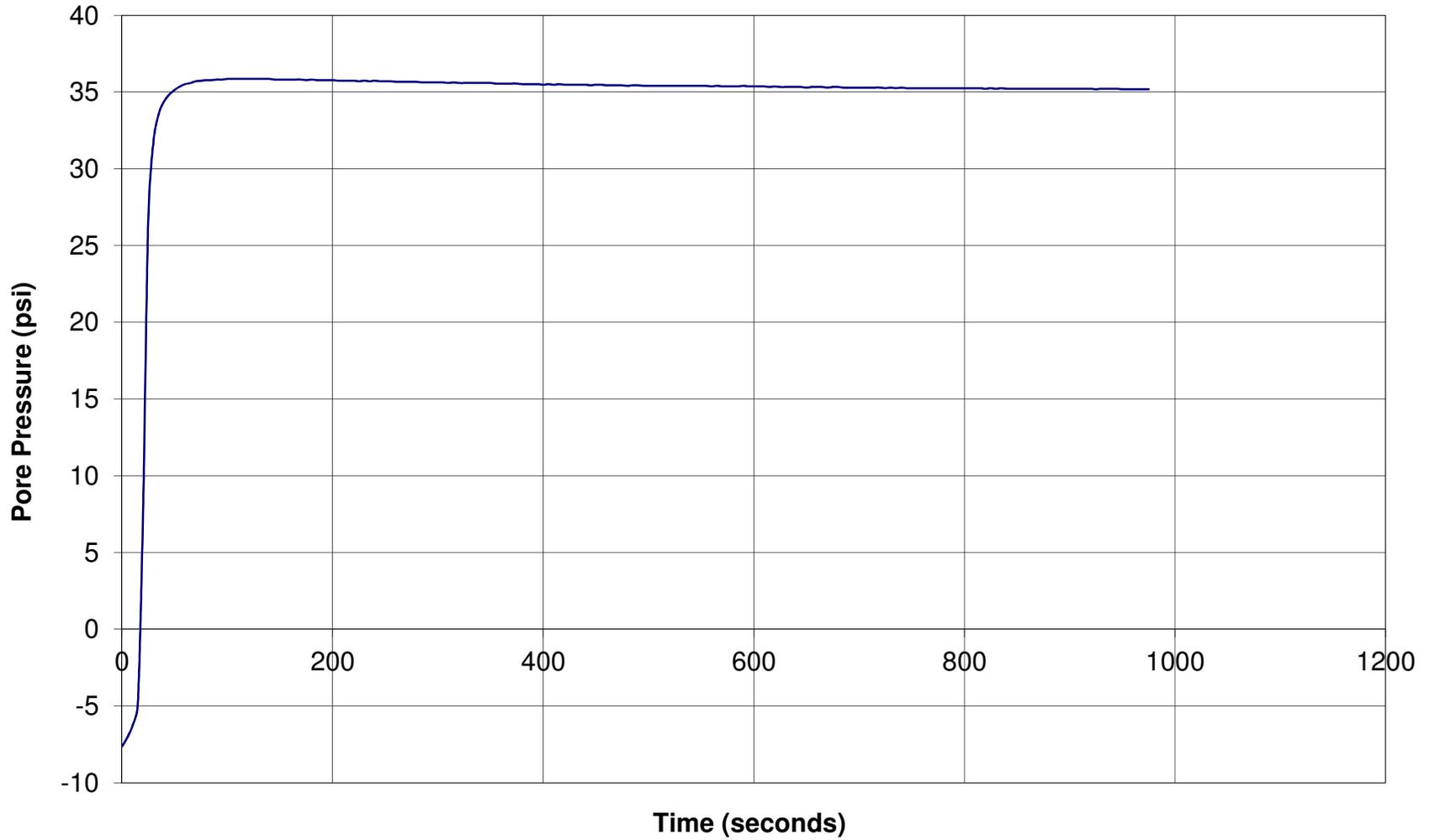




GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: KCPT-5
Depth: 81.857
Site: LAS GALLINAS CREEK
Engineer: E.MORLEY JOHN

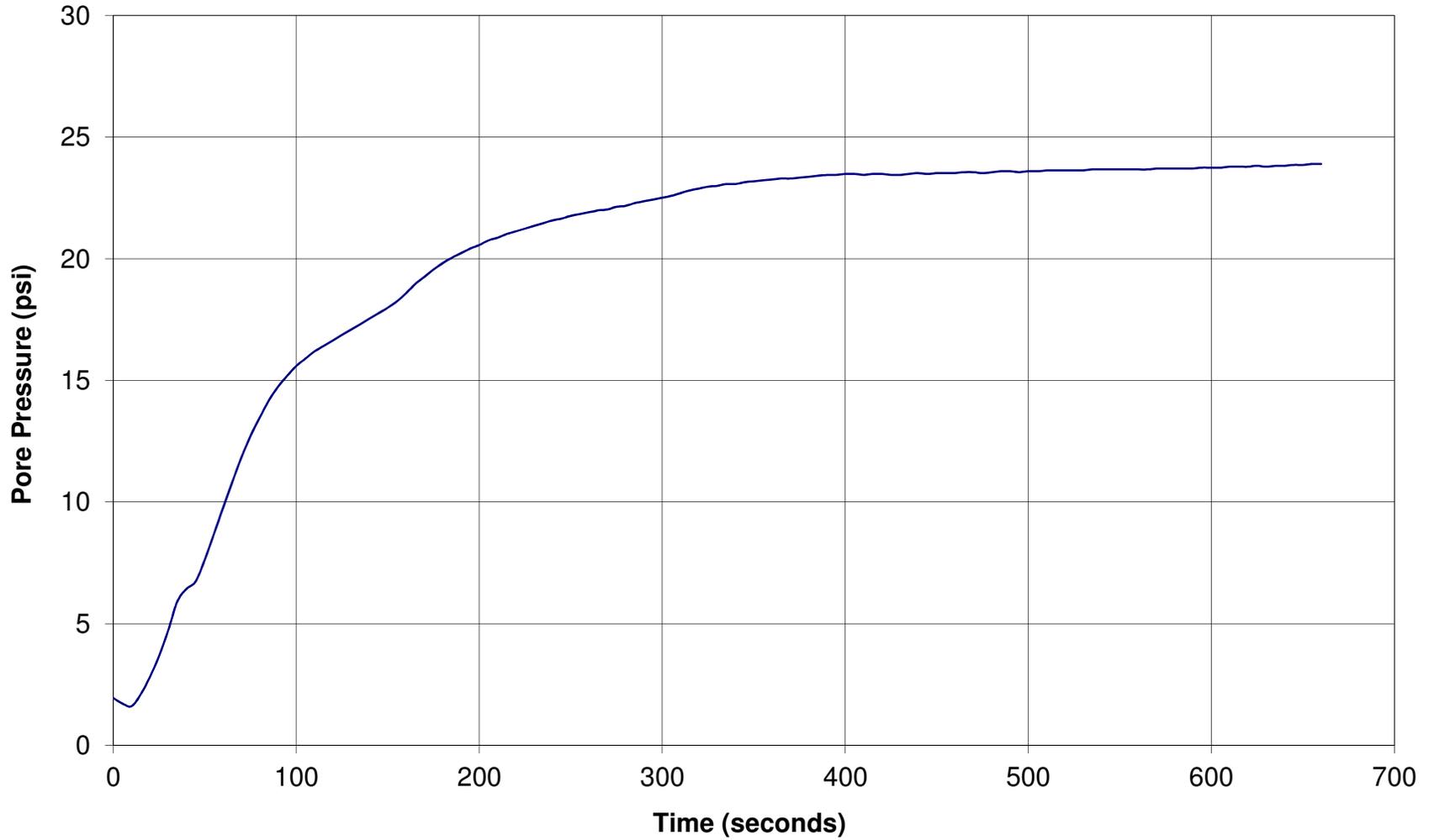




GREGG DRILLING & TESTING

Pore Pressure Dissipation Test

Sounding: KCPT-A2
Depth: 56.758
Site: LAS GALLINAS CREEK
Engineer: E.MORLEY JOHN



APPENDIX C

LABORATORY TEST RESULTS

BORING NO.	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	% PASSING #200 SIEVE	WATER CONTENT (%)	DRY DENSITY (pcf)	MAXIMUM DEVIATOR STRESS (ksf)	AXIAL STRAIN (%)	CONFINING STRESS (ksf)
KC- 2	3.5	26	15	10						
KC- 2	4.0				19					
KC- 2	8.0	25	14	11						
KC- 2	8.5				23	17.8				
KC- 2	12.5	97	33	64						
KC- 2	12.5					100.0	64.9			
KC- 3	2.5				32	7.3	114.9			
KC- 3	13.0				12					
KC- 4	9.0				25	12.5	122.0	6.88	14.9	1.0
KC- 4	12.0				9					
KC- 7	1.0				28	10.0	103.6			
KC- 7	4.0				24	14.5	120.8			
KC- 9	5.0									
KC- 9	5.5				25					
KC- 9	6.0	39	17	22						
KC-10	3.0	73	33	40		34.1	75.7	5.44	3.5	0.5
K-AP1	5.5				49	20.0	95.5	3.03	15.0	0.6



SUMMARY OF LABORATORY RESULTS

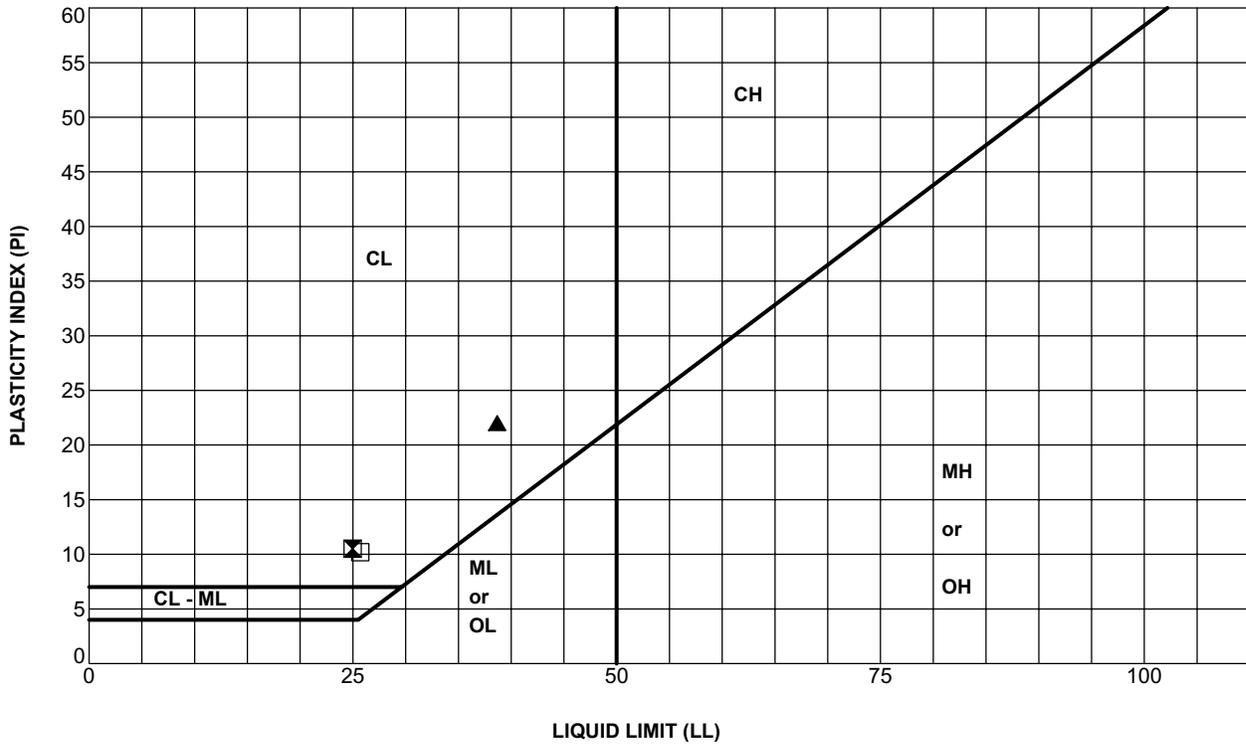
PLATE

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

C-1

PROJECT NO. 96670

L:\2009\PROJECTS\96670\96670_LABONLY.GPJ



SYMBOL	BORING	DEPTH, ft	LL	PL	PI	SAMPLE DESCRIPTION
□	KC-2	3.5	26	15	10	Yellowish Brown Clayey Sand with Gravel (SC)
☒	KC-2	8.0	25	14	11	Mottled Light Olive Brown Clayey Sand with Gravel (SC)
▲	KC-9	6.0	39	17	22	Dark Yellowish Brown Clayey Gravel with Sand (GC)

Unified Soil Classification
Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine sands of slight plasticity	MH	Inorganic silts and clayey silts of high plasticity
CL	Inorganic clays of low to medium plasticity	CH	Inorganic clays of high plasticity
OL	Organic silts and organic silty clays of low plasticity	OH	Organic clays of medium to high plasticity, organic silts

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318 (WET PREP)

Pursuant to 2006 IBC Section 1704, the results presented in this report are for the exclusive use of the client and the registered design professional in responsible charge. The results apply only to the samples tested. If changes to the specifications were made and not communicated to Kleinfelder, Kleinfelder assumes no responsibility for pass/fail (meets/does not meet), if provided.

As the samples tested were sampled and/or transported to our laboratory by parties other than Kleinfelder staff, this report makes no representation of whether the samples are representative of the material onsite.



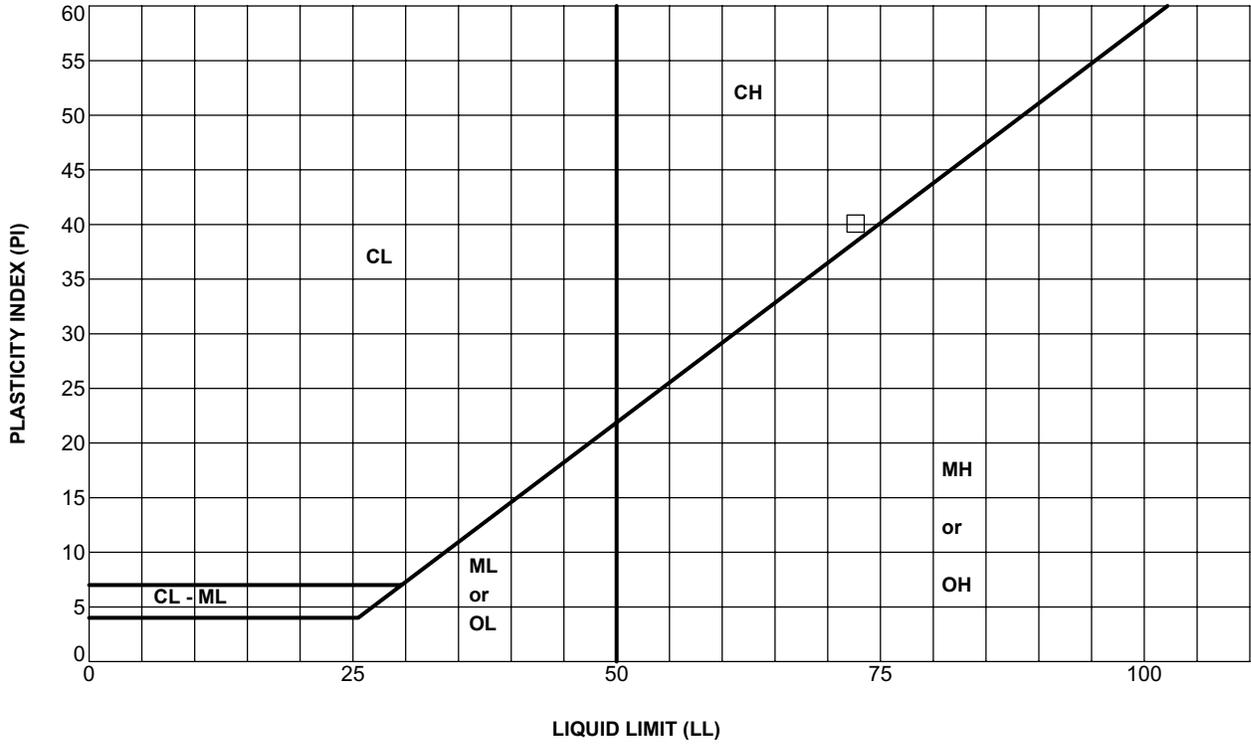
ATTERBERG LIMITS*

Las Gallinas Levee Evaluation
Santa Venetia
San Rafael, California

PLATE

C-2

PROJECT NO. 96670



SYMBOL	BORING	DEPTH, ft	LL	PL	PI	SAMPLE DESCRIPTION
□	KC-10	3.0	73	33	40	Dark Grayish Brown Fat Clay with Sand (CH)

Unified Soil Classification
Fine Grained Soil Groups

Symbol	LL < 50	Symbol	LL > 50
ML	Inorganic clayey silts to very fine sands of slight plasticity	MH	Inorganic silts and clayey silts of high plasticity
CL	Inorganic clays of low to medium plasticity	CH	Inorganic clays of high plasticity
OL	Organic silts and organic silty clays of low plasticity	OH	Organic clays of medium to high plasticity, organic silts

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318 (DRY PREP)



PROJECT NO. 96670

ATTERBERG LIMITS*

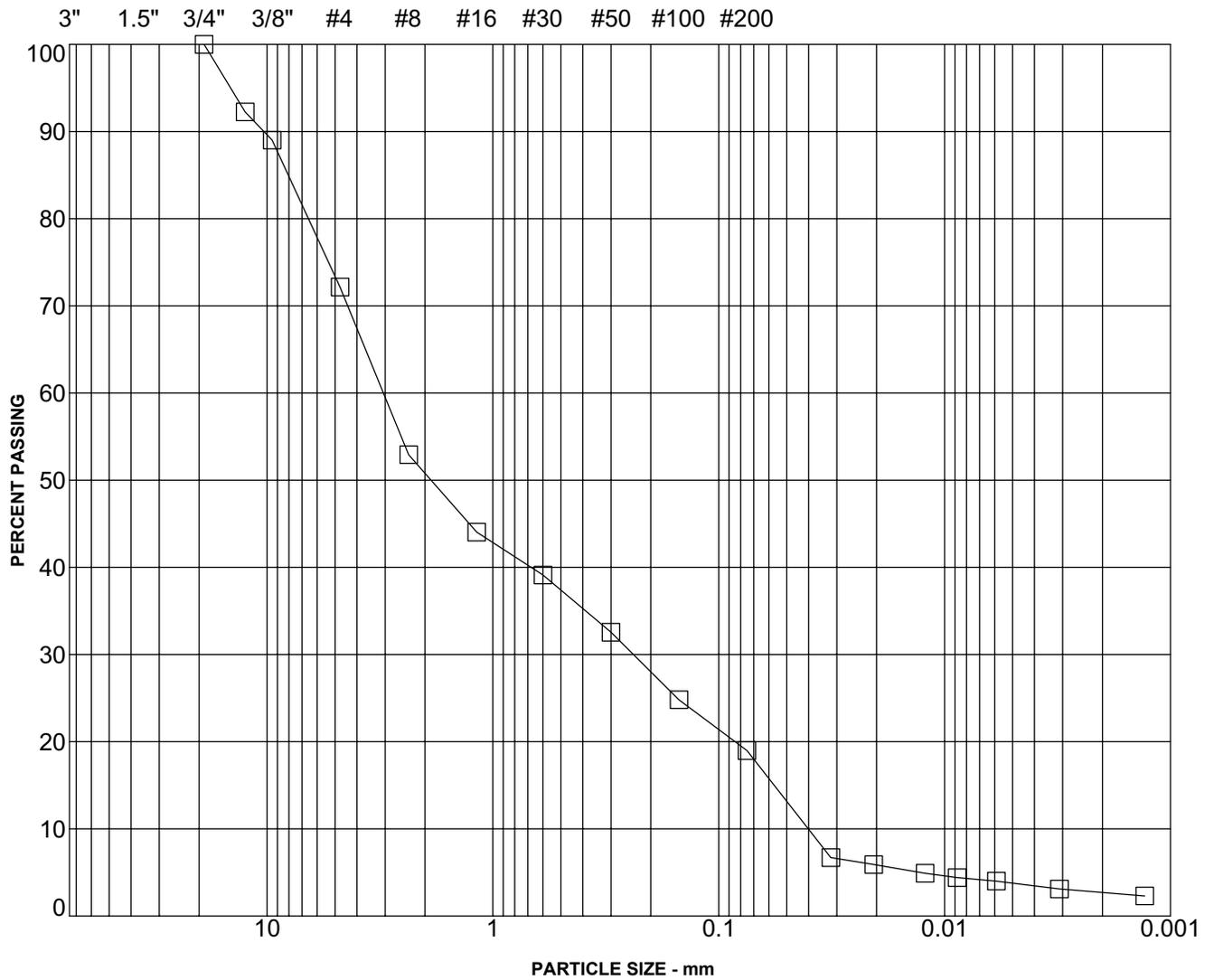
Las Gallinas Levee Evaluation
Santa Venetia
San Rafael, California

PLATE

C-3

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SIEVE ANALYSIS	HYDROMETER
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GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	KC- 2	4.0	Yellowish Brown Clayey Sand with Gravel (SC)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422



PROJECT NO. 96670

PARTICLE SIZE ANALYSIS*

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

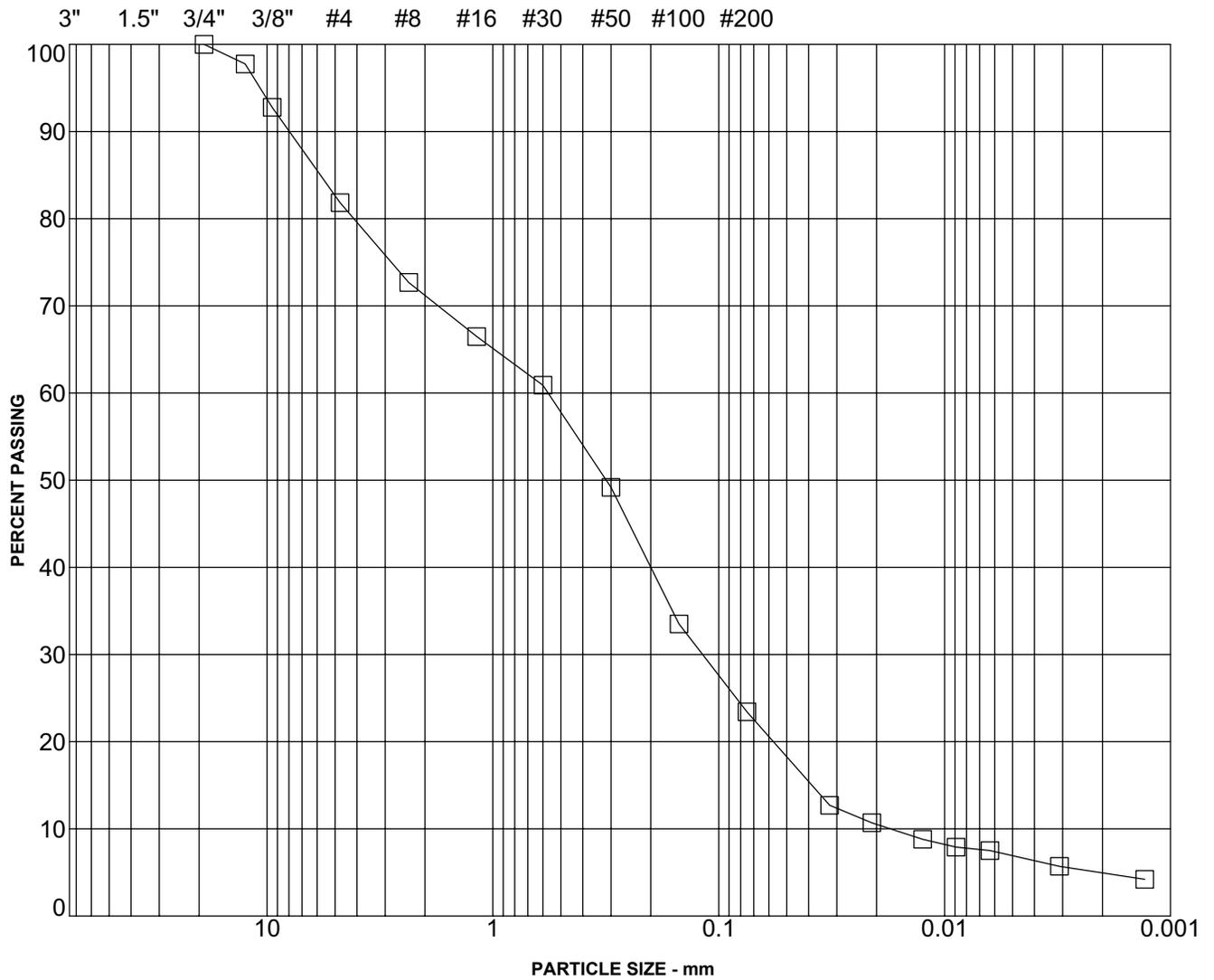
PLATE

C-4

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SIEVE ANALYSIS

HYDROMETER



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	KC- 2	8.5	Olive Brown Clayey Sand with Gravel (SC)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422



PARTICLE SIZE ANALYSIS*

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

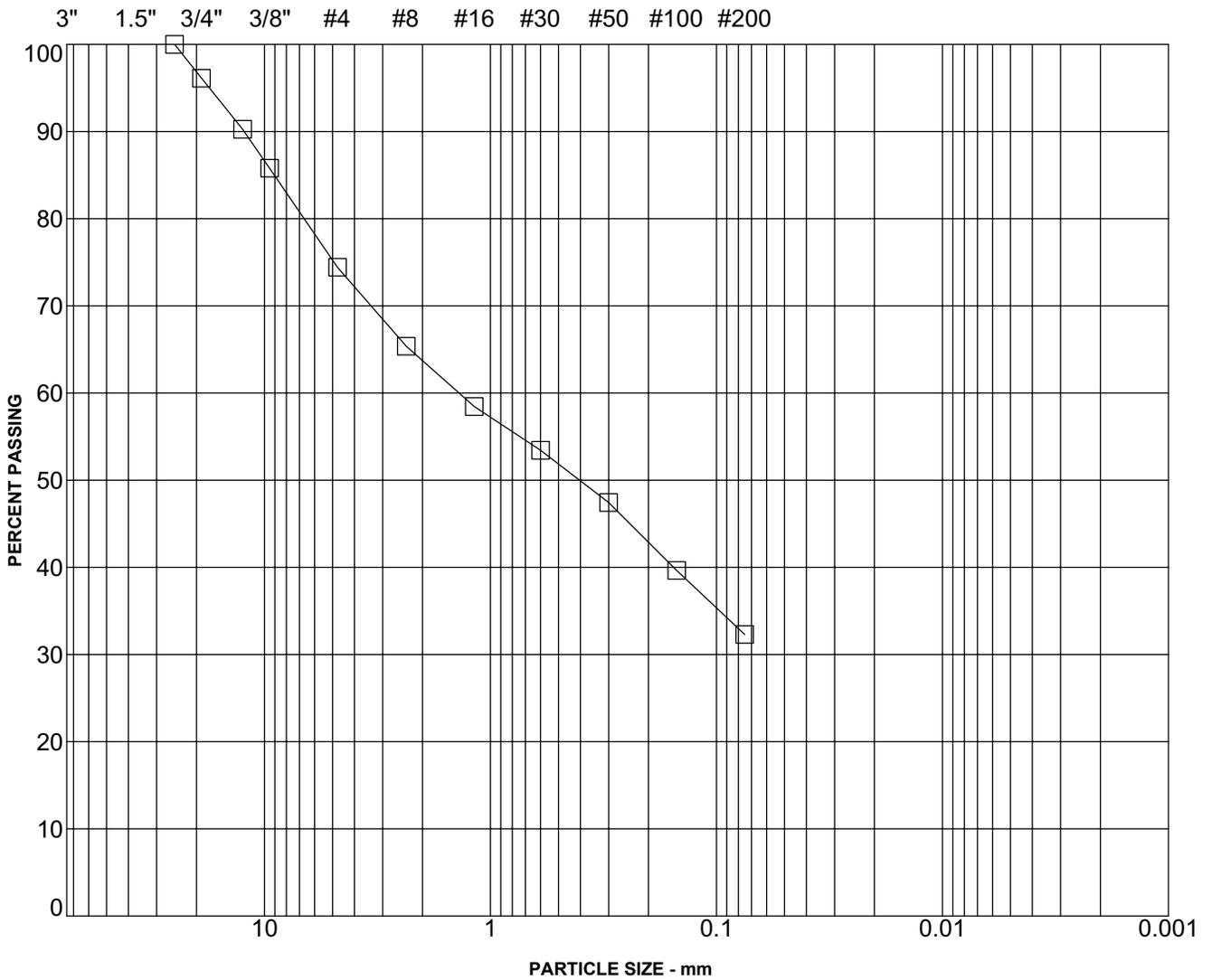
C-5

PROJECT NO. 96670

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SIEVE ANALYSIS

HYDROMETER



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	KC- 3	2.5	Dark Yellowish Brown Clayey Sand with Gravel (SC)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422



PROJECT NO. 96670

PARTICLE SIZE ANALYSIS*

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

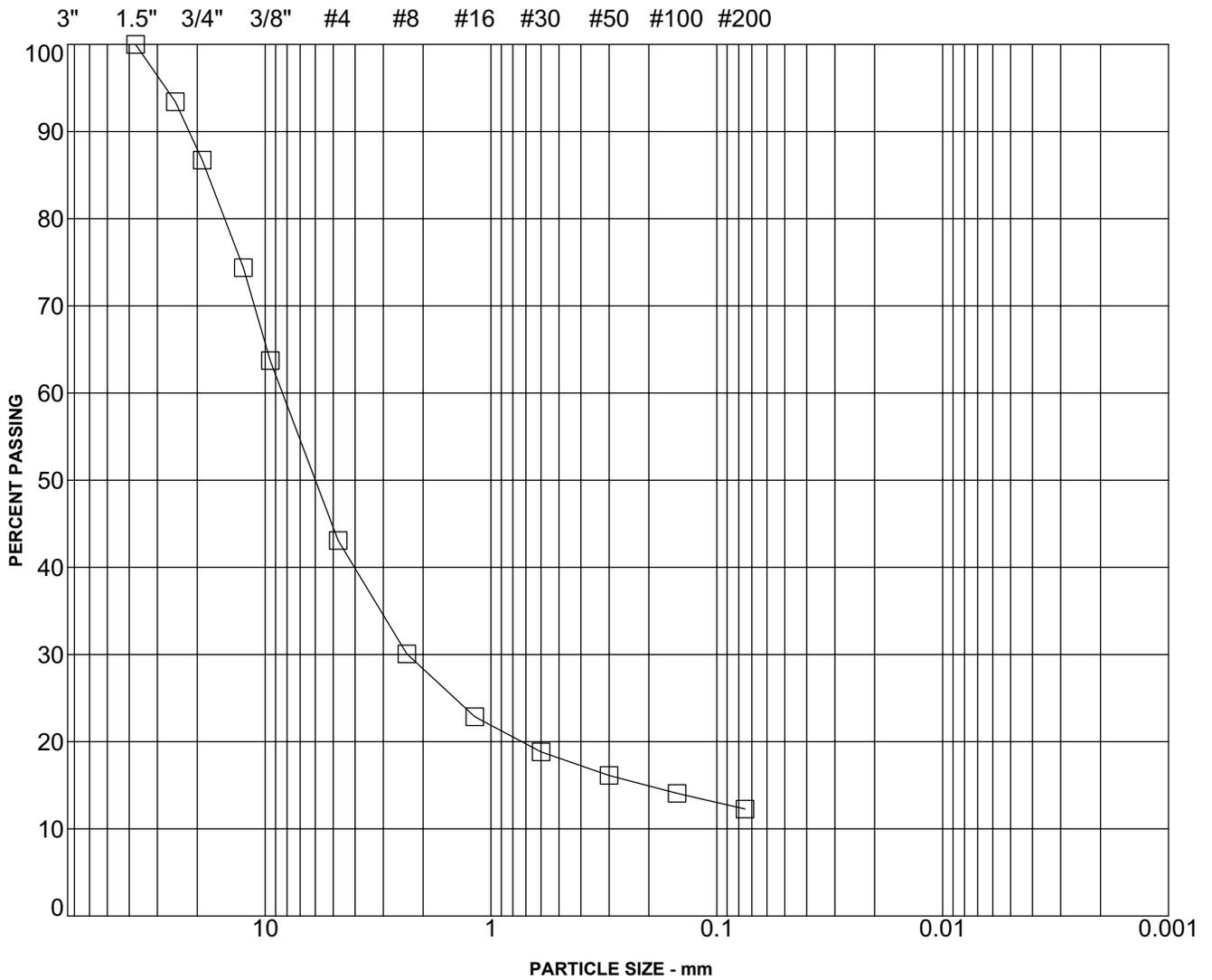
PLATE

C-6

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SIEVE ANALYSIS

HYDROMETER



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	KC- 3	13.0	Olive Brown Clayey Gravel with Sand (GC)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422



PARTICLE SIZE ANALYSIS*

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

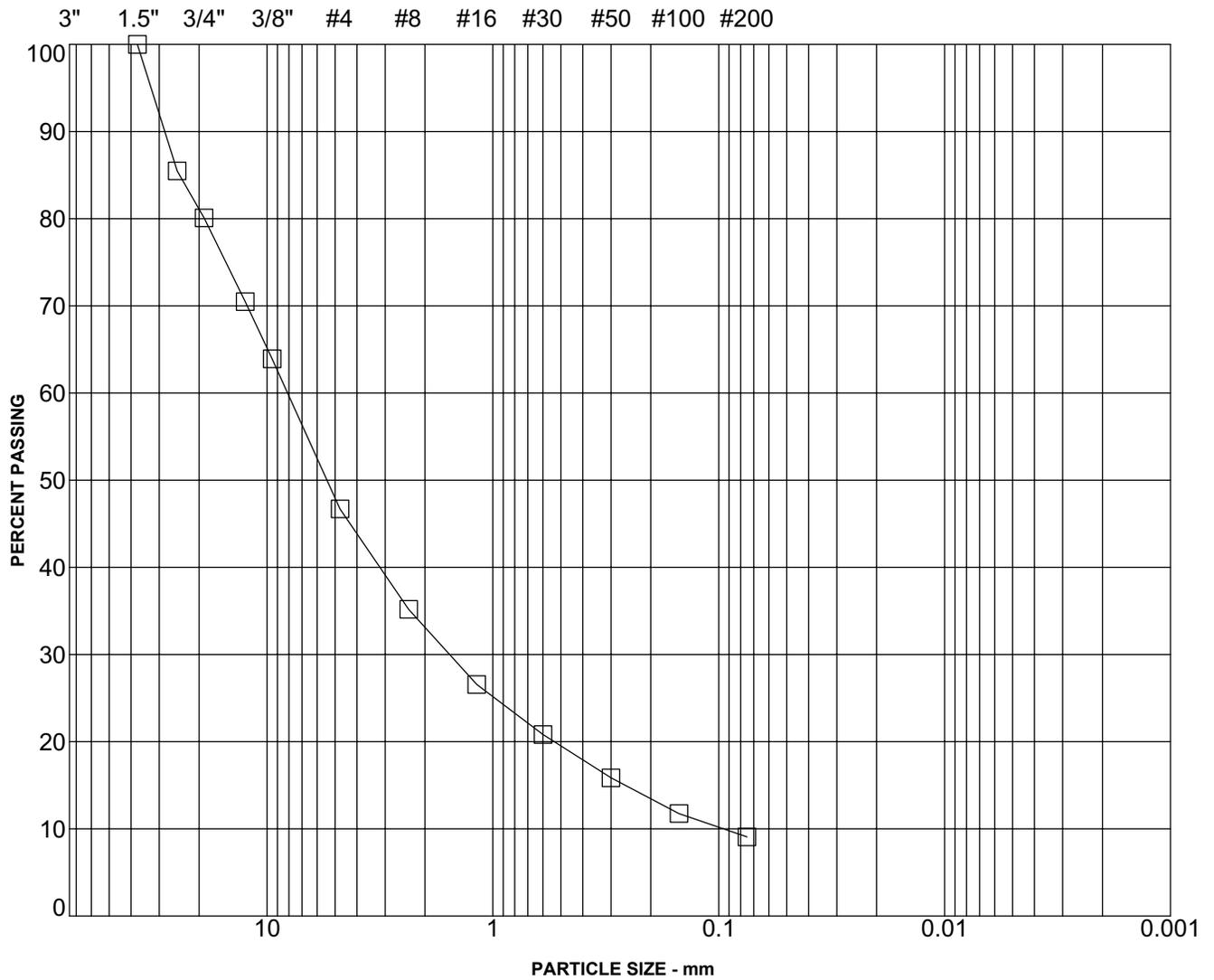
C-7

PROJECT NO. 96670

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SIEVE ANALYSIS

HYDROMETER



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	KC- 4	12.0	Olive Gray Well Graded Gravel with Clay and Sand (GW)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422



PARTICLE SIZE ANALYSIS*

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

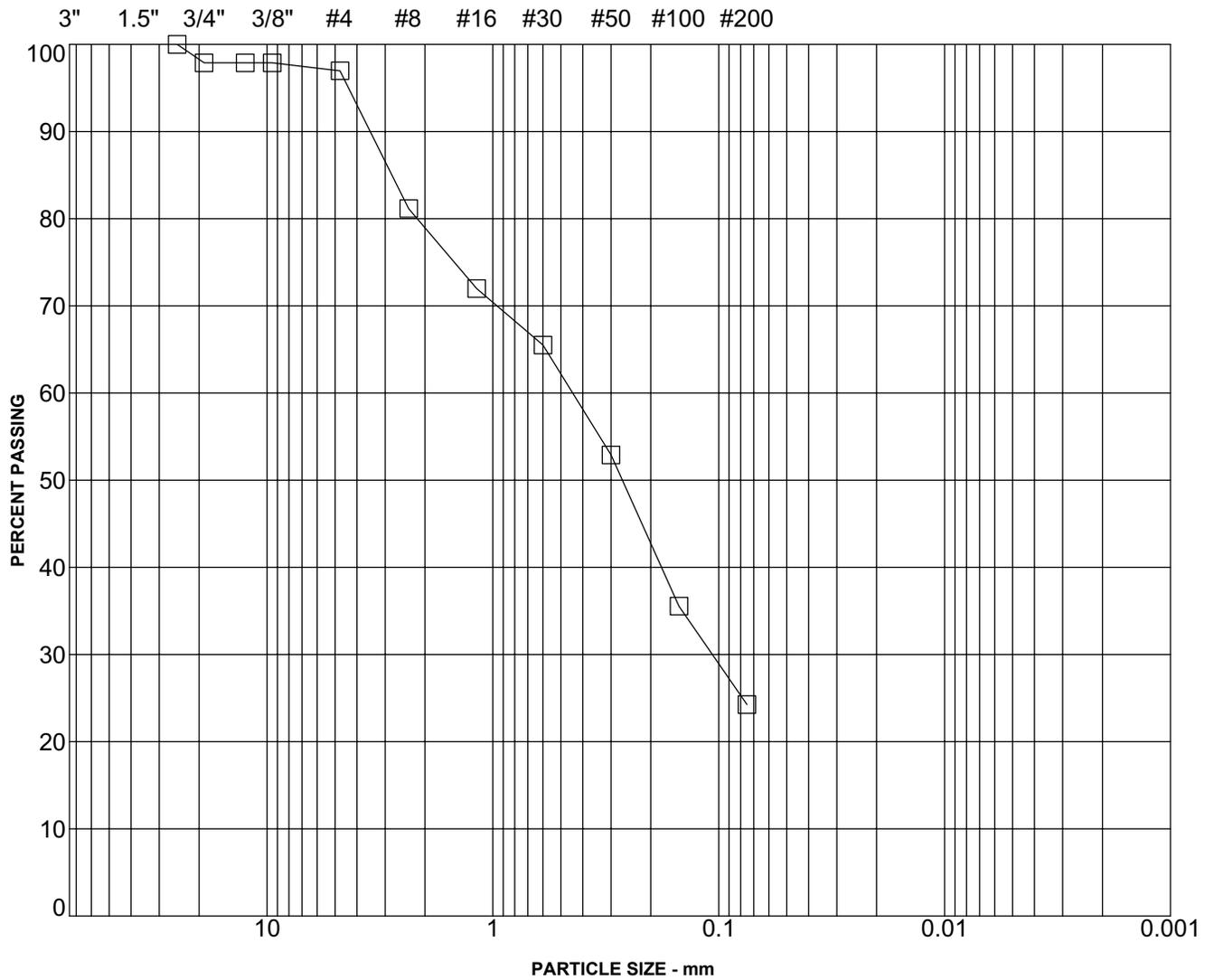
PLATE

C-8

PROJECT NO. 96670

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SIEVE ANALYSIS	HYDROMETER
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GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	KC- 7	4.0	Dark Yellowish Brown Clayey Sand (SC)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422



PROJECT NO. 96670

PARTICLE SIZE ANALYSIS*

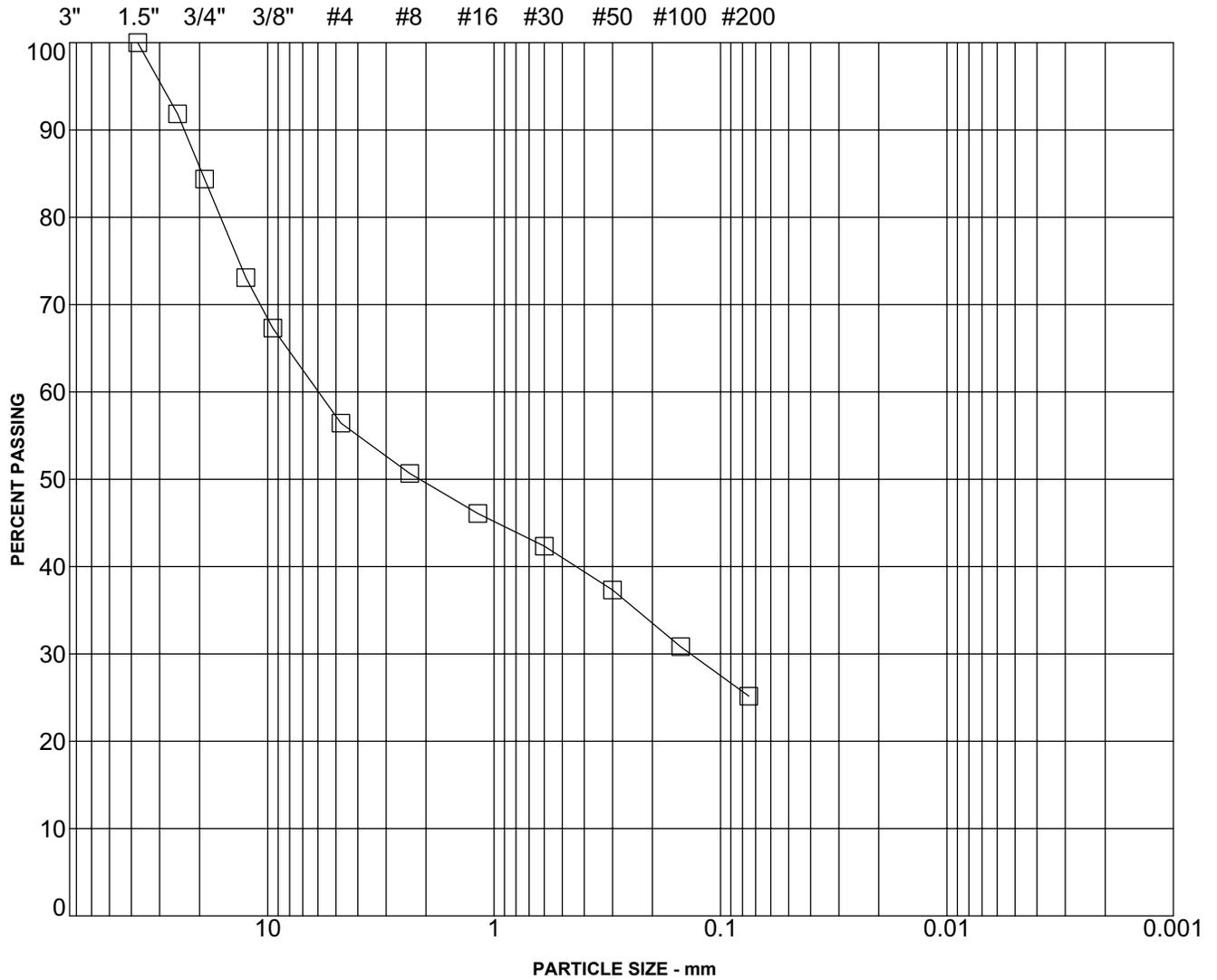
Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

C-9

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SIEVE ANALYSIS	HYDROMETER
----------------	------------



GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	KC- 9	6.0	Dark Yellowish Brown Clayey Gravel with Sand (GC)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422



PROJECT NO. 96670

PARTICLE SIZE ANALYSIS*

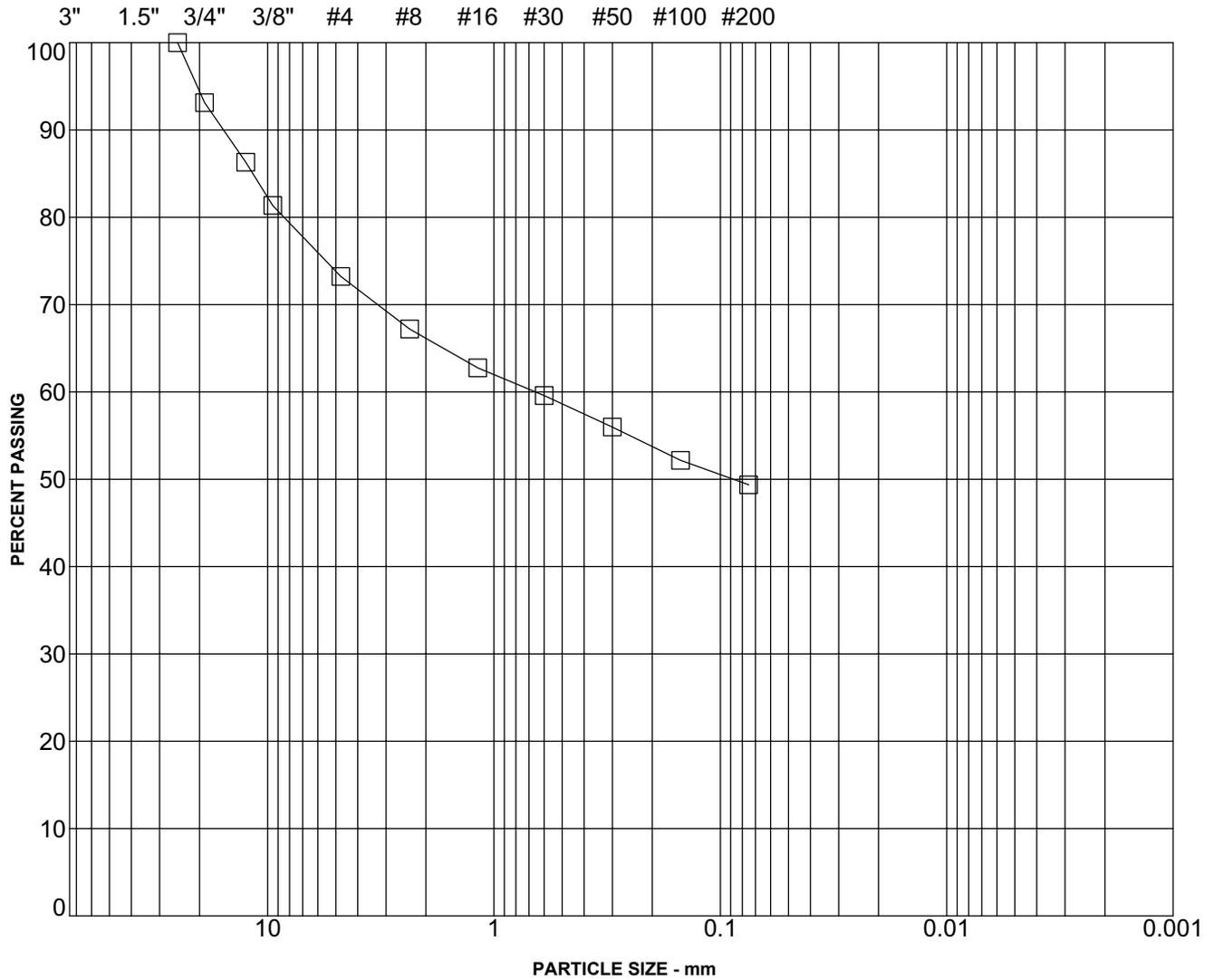
Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

C-10

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SIEVE ANALYSIS	HYDROMETER
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GRAVEL		SAND			SILT	CLAY
coarse	fine	coarse	medium	fine		

SYMBOL	BORING	DEPTH, ft	SAMPLE DESCRIPTION
□	K-AP1	5.5	Very Dark Grayish Brown Clayey Gravel with Sand (GC)

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422



PROJECT NO. 96670

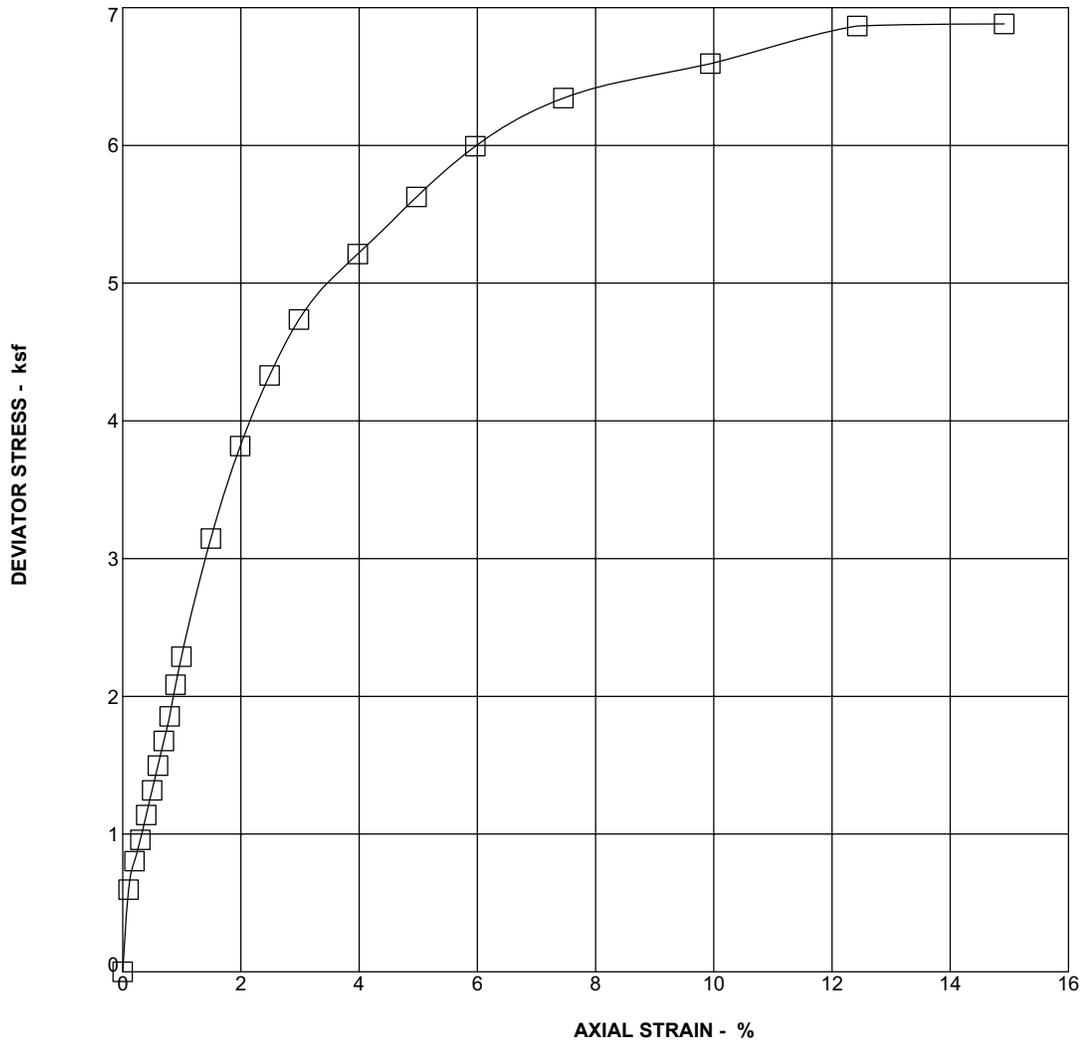
PARTICLE SIZE ANALYSIS*

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

PLATE

C-11

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BORING NO. KC- 4

DRY DENSITY - pcf 122

DEPTH - ft 9.0

WATER CONTENT - % 12.5

SAMPLE DESCRIPTION Light Olive Brown Clayey Sand with Gravel (SC)

CONFINING STRESS - ksf 1.0

MAXIMUM DEVIATOR STRESS= 6.88 ksf at 14.9 % STRAIN

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850



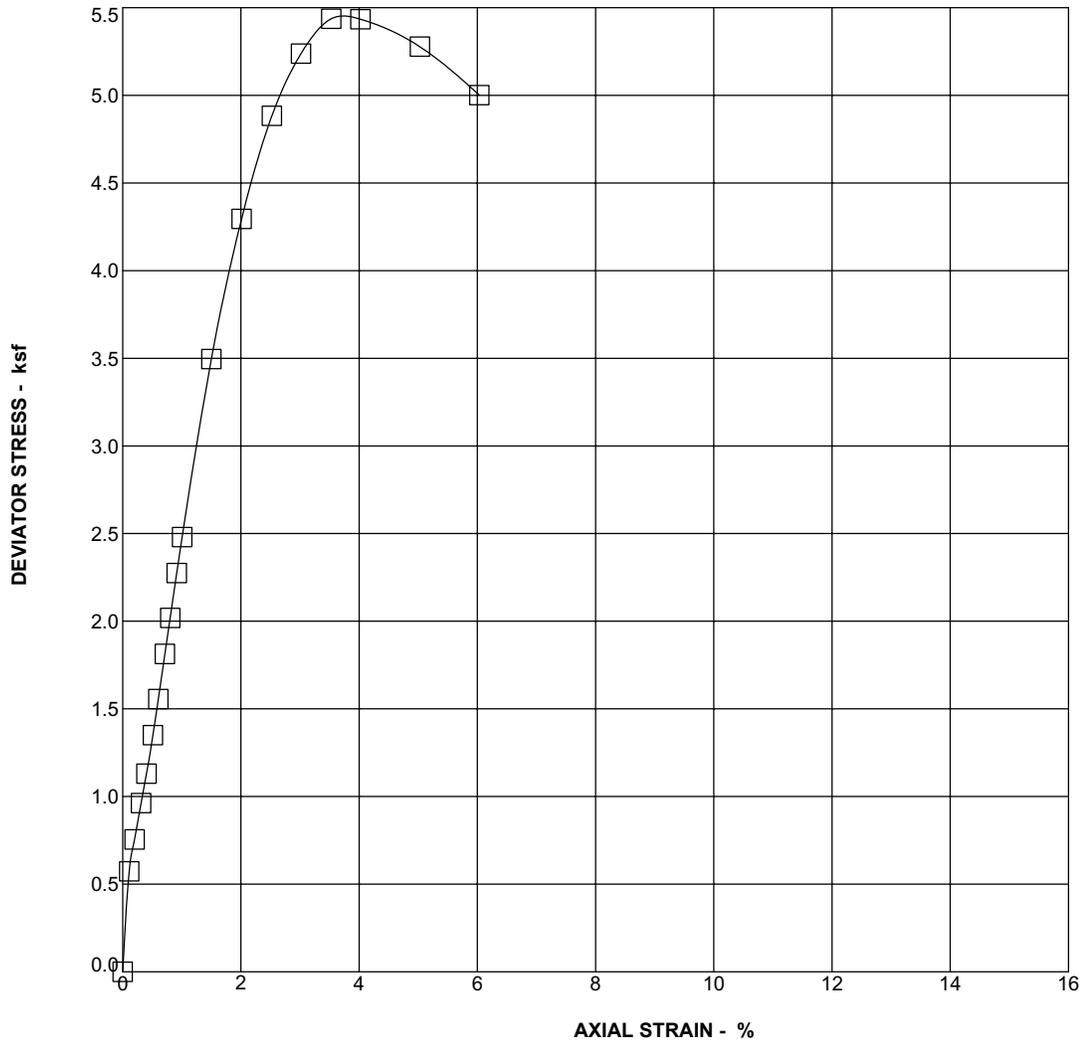
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION*

PLATE

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

C-12

PROJECT NO. 96670



BORING NO. KC-10
DEPTH - ft 3.0
SAMPLE DESCRIPTION Dark Grayish Brown Fat Clay with Sand (CH)
CONFINING STRESS - ksf 0.5

DRY DENSITY - pcf 76
WATER CONTENT - % 34.1

MAXIMUM DEVIATOR STRESS= 5.44 ksf at 3.5 % STRAIN

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850



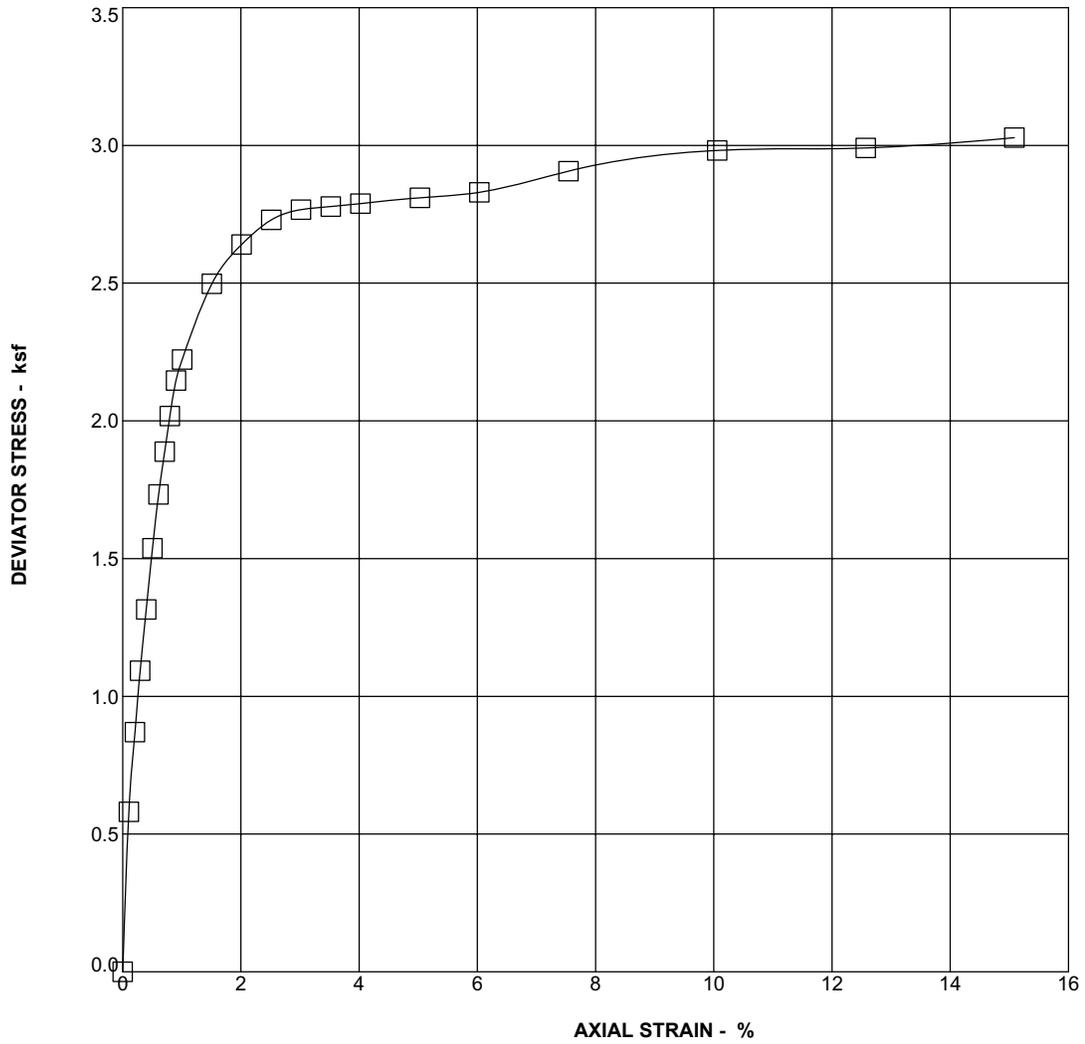
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION*

PLATE

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

C-13

PROJECT NO. 96670



BORING NO. K-AP1

DRY DENSITY - pcf 96

DEPTH - ft 5.5

WATER CONTENT - % 20.0

SAMPLE DESCRIPTION Very Dark Grayish Brown Clayey Gravel with Sand (GC)

CONFINING STRESS - ksf 0.6

MAXIMUM DEVIATOR STRESS= 3.03 ksf at 15.0 % STRAIN

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2850



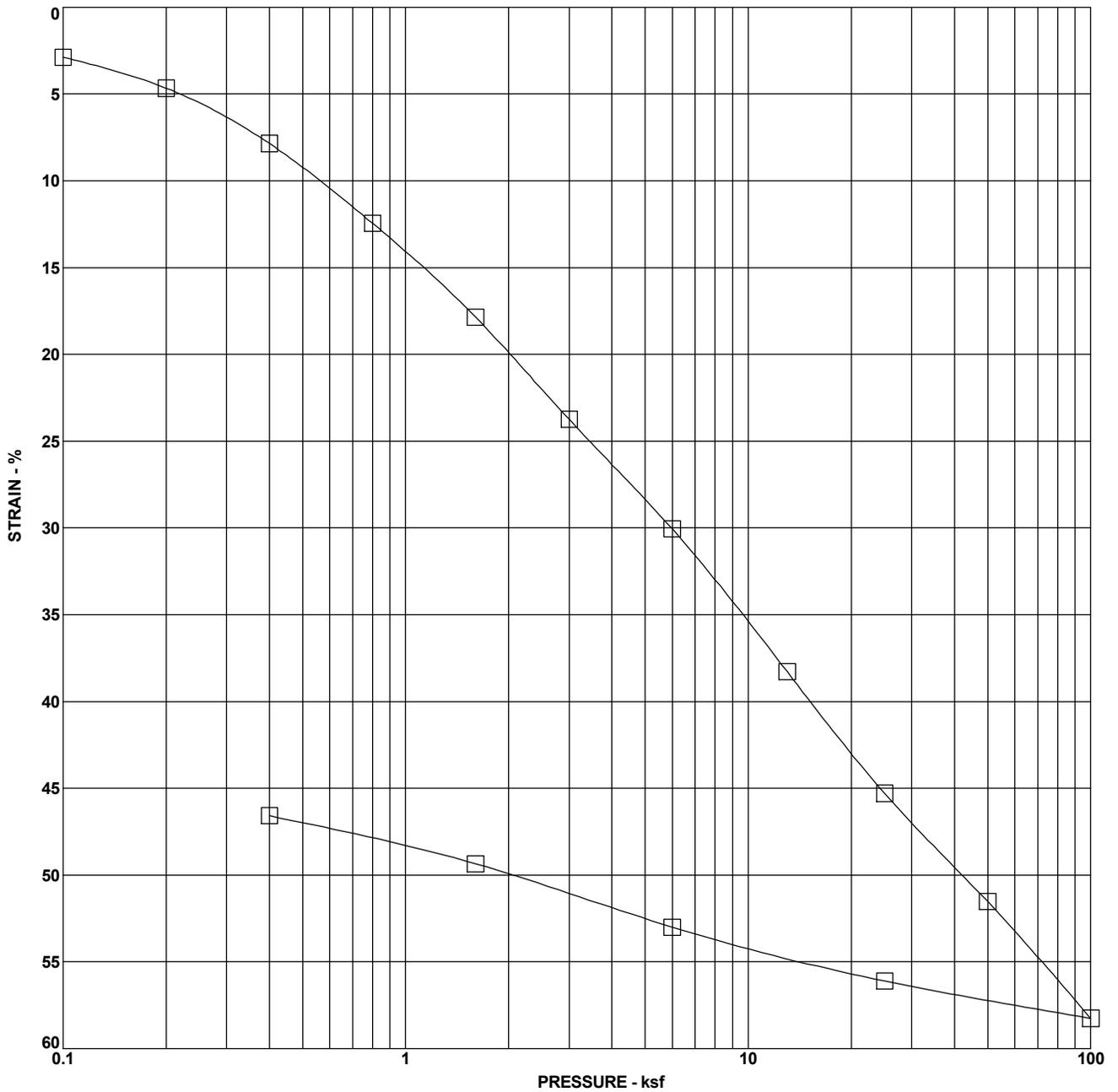
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION*

PLATE

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

C-14

PROJECT NO. 96670



BORING NO. KC-2
DEPTH, ft 12.5
SAMPLE DESCRIPTION Very Dark Gray Fat Clay with Shells (CH)
PRECONSOLIDATION PRESSURE ksf
COMPRESSION RATIO= $C_c/1+e_0$ _____
RECOMPRESSION RATIO= $C_r/1+e_0$ _____
LL _____ **PL** _____

	INITIAL	FINAL
DRY DENSITY, pcf	46.6	79.9
WATER CONTENT, %	96.1	45.7
SAMPLE HEIGHT, in.	.7840	.4189

*PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435



ONE DIMENSIONAL CONSOLIDATION*

PLATE

Las Gallinas Levee Evaluation
 Santa Venetia
 San Rafael, California

C-15

PROJECT NO. 96670

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APPENDIX D

METHODOLOGIES OF SEEPAGE AND STABILITY ANALYSIS

APPENDIX D – SEEPAGE AND STABILITY MODELING

D.1 Underseepage

Levees constructed on low permeability foundation soil (silt and/or clay) underlain by a higher permeability layer (sand and/or gravel) may be susceptible to piping and landside failure due to underseepage during high river stages. Under these conditions, seepage travels horizontally under the levee through the pervious layers with relatively little head loss. At the landside levee toe, seepage is driven vertically upward through the low-permeability foundation (blanket) layer due to the relatively higher total head at the bottom of the blanket. Failure can occur by either uplift of the blanket materials (if the blanket materials are nearly impervious and do not have enough weight to resist the upward pressure head), or by piping (if the blanket consists of low- to non-plastic erodible soils). As either condition may develop, the results of the seepage analyses are checked for both conditions.

The risk of uncontrolled underseepage that could lead to failure of a levee increases as the vertical seepage gradient across the landside blanket layer increases. It is customary (EM-1110-2-1913, USACE 2000) to calculate the exit gradient (average vertical gradient) through the blanket layer as the head loss through the blanket divided by the blanket layer thickness. Gradient contours can also be plotted directly by many finite element method (FEM) programs either as individual x (horizontal) and y (vertical) components, or as resultant vectors. The FEM gradient contours can be used to identify local maximum gradients and may reveal potential boil locations. The USACE blanket layer methodology (USACE, 2000) was used for this analysis.

The USACE acceptance criteria for underseepage exit gradients are summarized in Table D-1. Where existing conditions did not meet these acceptance criteria, these values were used as design values during the evaluation of alternatives.

Table D.1 – Allowable Exit Gradients (after USACE).

Location	Allowable Exit Gradients ³
Landside toe of levee	≤ 0.5
Toe of seepage berm ¹	≤ 0.8
Bottom of empty ditch at landside toe ²	≤ 0.5
Bottom of empty ditch 150 feet landward of toe ²	≤ 0.8
Ditch between landside toe and 150 feet landward of toe ²	Interpolate

- Notes:
1. For the maximum width seepage berm (300 to 400 feet), current USACE criteria allow gradients greater than 0.8. However, as outlined in ETL 1110-2-569, judgment should be used in those cases where the gradient exceeds 0.8.
 2. Reference USACE EM 1110-2-1913, Section 8-16, 'Ditches Landside of Levee'.
 3. Gradients are only applicable for 100-year and 200-year +1 foot WSEs. Higher WSEs may have slightly higher allowable exit gradients based on current discussions with USACE. Assumes a minimum saturated soil unit weight = 112 pcf.

The factor of safety (FOS) in this table represents the ratio of the effective overburden stress divided by the pore water pressure for the soil at the bottom of the blanket layer.

D.2 Through Seepage

Seepage through a levee embankment can occur during periods of high water. Depending on the duration of the high water stage and the permeability of the levee materials, seepage may exit on the landside slope of the levee (referred to as 'through seepage'). There are three potential through seepage related impacts on levee performance, stability, and safety:

- Piping (or transport by water flow) of fine-grained, erodible materials from within the levee embankment,
- Concentrated seepage conditions through more permeable layers within the levee embankment, and
- Increased soil pore pressures resulting in reduced slope stability of the landside slope.

The USACE acceptance criteria for through seepage are 1) no seepage on the landside slope of the levee, and 2) a FOS of at least 1.4 against slope failure under steady-state seepage conditions.

D.3 Steady-State Slope Stability

Steady-State seepage occurs when the water level remains at or near a high elevation (such as near the top of the levee) for an extended period of time. This condition fully saturates the levee embankment soils and a steady-state seepage phreatic surface develops. This case requires a minimum FOS of 1.4.

APPENDIX E
RISK BASED ANALYSIS

UNDERSEEPAGE ANALYSIS
 Las Gallinas Levee Recertification Project
 Reach 1: 601 Vendola Dr, San Rafael, existing conditions

Crest width (feet)
 Horizontal to vertical slope ratio

K_s/K_b Permeability ratio of clayey sand over silt/clay blanket
 K_v/K_{vt} Anisotropy ratio of layer
 z Blanket thickness

Head =

Mean	Ks/Kb	z	Kv/Kvt	I	Variance Component	Percent of Variance
	10	4	0.1	0.3047		
	1	4	0.1	0.0932		
	100	4	0.1	0.5057	0.0425391	48.4305
	10	0.0	0.1	0.4731		
	10	1	0.1	0.1768	0.0219484	24.988
	10	4	1.0	0.4168		
	10	4	0.01	0.1112	0.0233478	26.5814

Total 0.0878353 100

$E[I] = 0.3047$ $E[\ln I] = -1.52133$
 $Var[I] = 0.08784$
 $\sigma[I] = 0.29637$ $\sigma[\ln I] = 0.815974$
 $V(I) = 0.9727$

I crit = $\ln(I \text{ crit}) = -0.22314$

$F(z) = 0.94419$
 $Pr(f) = 5.58080$

Head =

Mean	Ks/Kb	z	Kv/Kvt	I	Variance Component	Percent of Variance
	10	4	0.1	0.2289		
	1	4	0.1	0.0767		
	100	4	0.1	0.3787	0.0228010	56.2449
	10	0.0	0.1	0.4063		
	10	1	0.1	0.5455	0.0048442	11.949
	10	4	1.0	0.3061		
	10	4	0.01	0.079	0.0128936	31.8056

Total 0.0405388 100

$E[I] = 0.2289$ $E[\ln I] = -1.76101$
 $Var[I] = 0.04054$
 $\sigma[I] = 0.20134$ $\sigma[\ln I] = 0.757017$
 $V(I) = 0.8796$

I crit = $\ln(I \text{ crit}) = -0.22314$

$F(z) = 0.97890$
 $Pr(f) = 2.11032$

Head =

Mean	Kf/Kb	z	Kv/Kvt	I	Variance Component	Percent of Variance
	10	4	0	0.1723		
	10	4	0	0.0612		
	10	4	0	0.2826	0.0122545	55.0941
	10	8	0	0.3551		
	10	0	0	0.4257	0.0012461	5.602
	10	4	0	0.2419		
	10	4	0	0.0549	0.0087423	39.3037

Total 0.0222428 100

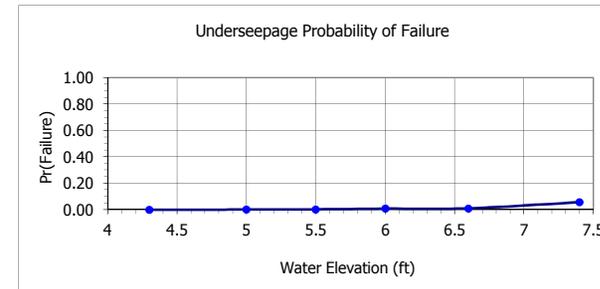
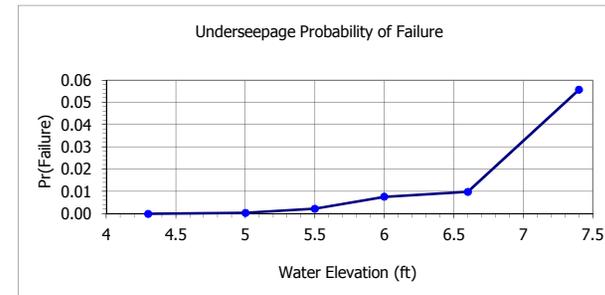
$E[I] = 0.1723$ $E[\ln I] = -2.03811$
 $Var[I] = 0.02224$
 $\sigma[I] = 0.14914$ $\sigma[\ln I] = 0.747783$
 $V(I) = 0.8656$

I crit = $\ln(I \text{ crit}) = -0.22314$

$F(z) = 0.99239$
 $Pr(f) = 0.76095$

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket z	4	4	100
Perm Ratio Ks/Kb	10		0
SC Anisotropy Kv/Kvt	0.1		0

Head	Elevation	Pr(f)
4.3	4.3	0.00000
5	5	0.00035
5.5	5.5	0.00229
6	6	0.00761
6.6	6.6	0.00979
7.4	7.4	0.05581



Head = 5.50

Kf/Kb	z	Kv/K _{st}	I	Variance Component	Percent of Variance
10	4	0	0.1215		
10	4	0	0.0404	0.0062489	52.4538
10	4	0	0.1985		
10	8	0	0.2727	0.0002326	1.952
10	0	0	0.3032		
10	4	0	0.1796	0.0054317	45.5941
10	4	0	0.0322		
Total				0.0119132	100

E[I] = 0.1215 E[ln I] = -2.40368
 Var[I] = 0.01191
 sigma[I] = 0.10915 sigma [ln I] = 0.769200
 V(I) = 0.8983

I crit = 0.800 ln(I crit) = -0.22314

F(z) = 0.99771
 Pr(f) = 0.22927

Head = 5.00

Kf/Kb	z	Kv/K _{st}	I	Variance Component	Percent of Variance
10	4	0	0.058		
10	4	0	0.015	0.0017431	42.8878
10	4	0	0.0985		
10	8	0	0.1374	0.0000076	0.186
10	0	0	0.1429		
10	4	0	0.0962	0.0023136	56.9261
10	4	0	0		
Total				0.0040642	100

E[I] = 0.0580 E[ln I] = -3.24339
 Var[I] = 0.00406
 sigma[I] = 0.06375 sigma [ln I] = 0.890032
 V(I) = 1.0992

I crit = 0.800 ln(I crit) = -0.22314

F(z) = 0.99965
 Pr(f) = 0.03451

Head = 4.30

Kf/Kb	z	Kv/K _{st}	I	Variance Component	Percent of Variance
10	4	0	0		
10	4	0	0	0.0000000	0.0000
10	4	0	0		
10	8	0	0.0202	0.0001020	100.000
10	0	0	0		
10	4	0	0	0.0000000	0.0000
10	4	0	0		
Total				0.0001020	100

E[I] = 0.0000 E[ln I] = #NUM!
 Var[I] = 0.00010
 sigma[I] = 0.01010 sigma [ln I] = #DIV/0!
 V(I) = #DIV/0!

I crit = 0.800 ln(I crit) = -0.22314

F(z) = #NUM!
 Pr(f) = #NUM!

STABILITY ANALYSIS

Las Gallinas Levee Recertification Project
 Reach 1: 601 Vendola Dr, San Rafael, existing conditions

Head = 7.4

Reliability Analysis of Critical Slide								Variance Component	Percent of Variance
Run	Clayey Sand γ	Clayey Sand cohesion (psf)	Clayey Sand Phi	Blanket γ	Blanket cohesion (psf)	Blanket Phi	FS		
1	125	50	28	120	50	26	1.8389		
2	110	50	28	120	50	26	1.9478	0.0050481	0.2057
3	130	50	28	120	50	26	1.8057		
4	125	0	28	120	50	26	1.2972		
5	125	300	28	120	50	26	4.1147	1.9845766	80.8819
6	125	50	22	120	50	26	1.6794		
7	125	50	32	120	50	26	1.9462	0.0177956	0.7253
8	125	50	28	100	50	26	1.6984		
9	125	50	28	125	50	26	1.8661	0.0070308	0.2865
10	125	50	28	120	0	26	1.2212		
11	125	50	28	120	200	26	2.5422	0.4362603	17.7799
12	125	50	28	120	50	18	1.7653		
13	125	50	28	120	50	30	1.8741	0.0029594	0.1206
Total							2.4536707		

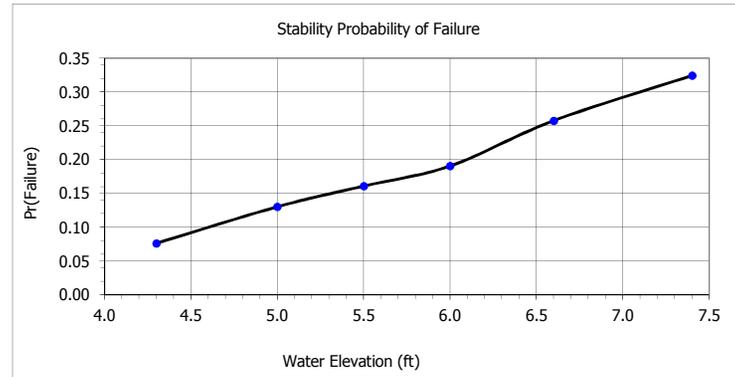
E[FS] = 1.8389 E[ln FS] = 0.33638
 Var[FS] = 2.45367
 sigma[FS] = 1.56642 sigma [ln FS] = 0.738632
 V(FS) = 0.8518

FS req'd = 1.000 ln(FS req'd) = 0.00000

Beta =	0.455408
F(z) =	0.324408
Pr(f) =	32.440796

Elevation	Prf Stability
4.3	0.07638
5	0.13032
5.5	0.16100
6	0.19065
6.6	0.25777
7.4	0.32441

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Clayey Sand ϕ	28		
Blanket (CL/ML) ϕ	26		
Clayey Sand, c	50		
Blanket (CL/ML), c	50		
Clayey Sand, γ	125		
Blanket (CL/ML), γ	120		



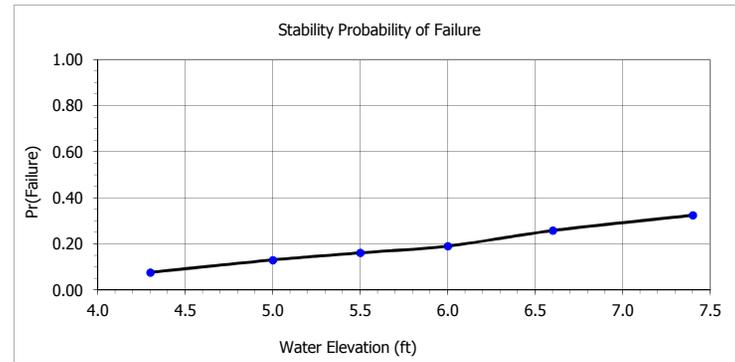
Head = 6.6

Reliability Analysis of Critical Slide								Variance Component	Percent of Variance
Run	Clayey Sand γ	Clayey Sand cohesion (psf)	Clayey Sand Phi	Blanket γ	Blanket cohesion (psf)	Blanket Phi	FS		
1	125	50	28	120	50	26	2.0660		
2	110	50	28	120	50	26	2.1980	0.0077352	0.2680
3	130	50	28	120	50	26	2.0221		
4	125	0	28	120	50	26	1.4755		
5	125	300	28	120	50	26	4.2273	1.8931008	65.5858
6	125	50	22	120	50	26	1.8608		
7	125	50	32	120	50	26	2.2015	0.0290191	1.0054
8	125	50	28	100	50	26	1.9015		
9	125	50	28	125	50	26	2.0931	0.0091776	0.3180
10	125	50	28	120	0	26	1.4703		
11	125	50	28	120	200	26	3.4118	0.9423556	32.6476
12	125	50	28	120	50	18	1.9667		
13	125	50	28	120	50	30	2.1090	0.0050623	0.1754
Total							2.8864507		

E[FS] = 2.0660 E[ln FS] = 0.46734
 Var[FS] = 2.88645
 sigma[FS] = 1.69896 sigma [ln FS] = 0.718718
 V(FS) = 0.8223

FS req'd = 1.000 ln(FS req'd) = 0.00000

Beta =	0.650236
F(z) =	0.257770
Pr(f) =	25.776981



Head = 6.0

Reliability Analysis of Critical Slide								Variance Component	Percent of Variance
Run	Clayey Sand γ	Clayey Sand cohesion (psf)	Clayey Sand Phi	Blanket γ	Blanket cohesion (psf)	Blanket Phi	FS		
1	125	50	28	120	50	26	2.2404		
2	110	50	28	120	50	26	2.3649	0.0078500	0.2790
3	130	50	28	120	50	26	2.1877		
4	125	0	28	120	50	26	1.6902	1.8362960	65.2532
5	125	300	28	120	50	26	4.4004		
6	125	50	22	120	50	26	1.9834	0.0427042	1.5175
7	125	50	32	120	50	26	2.3967		
8	125	50	28	100	50	26	2.0425	0.0126338	0.4489
9	125	50	28	125	50	26	2.2673		
10	125	50	28	120	0	26	1.6241	0.9055426	32.1787
11	125	50	28	120	200	26	3.5273		
12	125	50	28	120	50	18	2.0980	0.0090821	0.3227
13	125	50	28	120	50	30	2.2886		

Total 2.8141086 100

E[FS] = 2.2404 E[ln FS] = 0.58410
 Var[FS] = 2.81411 sigma[ln FS] = 0.667159
 sigma[FS] = 1.67753
 V(FS) = 0.7488
 FS req'd = 1.000 ln(FS req'd) = 0.00000

Beta =	0.875510
F(z) =	0.190648
Pr(f) =	19.064823

Head = 5.5

Reliability Analysis of Critical Slide								Variance Component	Percent of Variance
Run	Clayey Sand γ	Clayey Sand cohesion (psf)	Clayey Sand Phi	Blanket γ	Blanket cohesion (psf)	Blanket Phi	FS		
1	125	50	28	120	50	26	2.3341		
2	110	50	28	120	50	26	2.5044	0.0118592	0.4211
3	130	50	28	120	50	26	2.2866		
4	125	0	28	120	50	26	1.7254	1.9442119	69.0386
5	125	300	28	120	50	26	4.5141		
6	125	50	22	120	50	26	2.1152	0.0384356	1.3648
7	125	50	32	120	50	26	2.5073		
8	125	50	28	100	50	26	2.2088	0.0086304	0.3065
9	125	50	28	125	50	26	2.3946		
10	125	50	28	120	0	26	1.7601	0.8050576	28.5874
11	125	50	28	120	200	26	3.5546		
12	125	50	28	120	50	18	2.2454	0.0079299	0.2816
13	125	50	28	120	50	30	2.4235		

Total 2.8161246 100

E[FS] = 2.3341 E[ln FS] = 0.63929
 Var[FS] = 2.81612 sigma[ln FS] = 0.645503
 sigma[FS] = 1.67813
 V(FS) = 0.7190
 FS req'd = 1.000 ln(FS req'd) = 0.00000

Beta =	0.990374
F(z) =	0.160996
Pr(f) =	16.099555

Head = 5.0

Reliability Analysis of Critical Slide								Variance Component	Percent of Variance
Run	Clayey Sand γ	Clayey Sand cohesion (psf)	Clayey Sand Φ	Blanket γ	Blanket cohesion (psf)	Blanket Φ	FS		
1	125	50	28	120	50	26	2.5532		
2	110	50	28	120	50	26	2.7641	0.0187279	0.5523
3	130	50	28	120	50	26	2.4904		
4	125	0	28	120	50	26	1.5072		
5	125	300	28	120	50	26	4.7121	2.5678460	75.7336
6	125	50	22	120	50	26	2.4522		
7	125	50	32	120	50	26	2.8717	0.0439951	1.2975
8	125	50	28	100	50	26	2.3903		
9	125	50	28	125	50	26	2.7811	0.0381812	1.1261
10	125	50	28	120	0	26	1.9846		
11	125	50	28	120	200	26	3.6360	0.6817805	20.1078
12	125	50	28	120	50	18	2.4068		
13	125	50	28	120	50	30	2.8073	0.0401001	1.1827
Total									

$E[FS] = 2.5532$
 $Var[FS] = 3.39063$
 $\sigma[FS] = 1.84137$
 $V[FS] = 0.7212$

$E[\ln FS] = 0.72795$
 $\sigma[\ln FS] = 0.647144$
 $\ln(FS \text{ req'd}) = 0.00000$

Beta =	1.124866
F(z) =	0.130323
Pr(f) =	13.032291

Head = 4.3

Reliability Analysis of Critical Slide								Variance Component	Percent of Variance
Run	Clayey Sand γ	Clayey Sand cohesion (psf)	Clayey Sand Φ	Blanket γ	Blanket cohesion (psf)	Blanket Φ	FS		
1	125	50	28	120	50	26	2.8781		
2	110	50	28	120	50	26	3.0672	0.0239630	0.6432
3	130	50	28	120	50	26	2.7576		
4	125	0	28	120	50	26	1.5072		
5	125	300	28	120	50	26	4.9901	3.0326481	81.3964
6	125	50	22	120	50	26	2.5894		
7	125	50	32	120	50	26	3.1181	0.0698809	1.8756
8	125	50	28	100	50	26	2.7619		
9	125	50	28	125	50	26	2.8483	0.0018662	0.0501
10	125	50	28	120	0	26	2.1178		
11	125	50	28	120	200	26	3.6556	0.5912072	15.8680
12	125	50	28	120	50	18	2.7681		
13	125	50	28	120	50	30	2.9257	0.0062094	0.1667
Total									

$E[FS] = 2.8781$
 $Var[FS] = 3.72577$
 $\sigma[FS] = 1.93023$
 $V[FS] = 0.6707$

$E[\ln FS] = 0.87142$
 $\sigma[\ln FS] = 0.609438$
 $\ln(FS \text{ req'd}) = 0.00000$

Beta =	1.429878
F(z) =	0.076376
Pr(f) =	7.637598

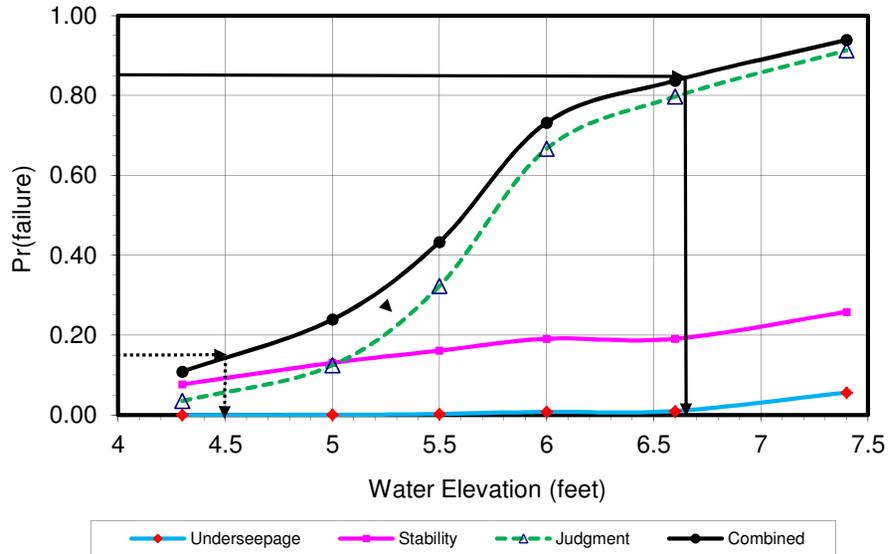
Combined Risk - Poor Performance

Las Gallinas Levee Recertification Project

Reach 1: 601 Vendola Dr, San Rafael, existing conditions

	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
Elevation	Underseepage		Stability		Judgment		Combined	
4.3	0.00000	1.00000	0.07638	0.92362	0.03475	0.96525	0.10847	0.891528
5	0.00035	0.99965	0.13032	0.86968	0.12431	0.87569	0.23869	0.761309
5.5	0.00229	0.99771	0.16100	0.83900	0.32289	0.67711	0.43320	0.566797
6	0.00761	0.99239	0.19065	0.80935	0.66642	0.33358	0.73207	0.267928
6.6	0.00979	0.99021	0.19065	0.80935	0.79683	0.20317	0.83718	0.162824
7.4	0.05581	0.94419	0.25777	0.74223	0.91279	0.08721	0.93888	0.061117

Combined Probability of Failure Existing Conditions



Levee Waterside Toe Elev. 4.3 feet
 Levee Crest Elev. 7.4 feet
 15% Probability of Failure 4.5 feet
 85% Probability of failure 6.65 feet

The elevations are in Datum NGVD 29

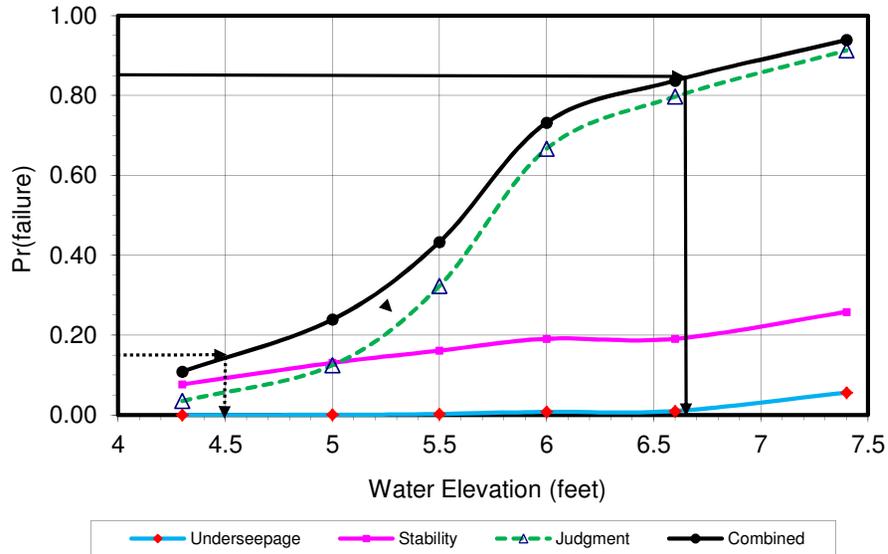
Combined Risk - Poor Performance

Las Gallinas Levee Recertification Project

Reach 1: 601 Vendola Dr, San Rafael, existing conditions

	Pr(f)	R	Pr(f)	R	Pr(f)	R	Pr(f)	R
Elevation	Underseepage		Stability		Judgment		Combined	
4.3	0.00000	1.00000	0.07638	0.92362	0.03475	0.96525	0.10847	0.891528
5	0.00035	0.99965	0.13032	0.86968	0.12431	0.87569	0.23869	0.761309
5.5	0.00229	0.99771	0.16100	0.83900	0.32289	0.67711	0.43320	0.566797
6	0.00761	0.99239	0.19065	0.80935	0.66642	0.33358	0.73207	0.267928
6.6	0.00979	0.99021	0.19065	0.80935	0.79683	0.20317	0.83718	0.162824
7.4	0.05581	0.94419	0.25777	0.74223	0.91279	0.08721	0.93888	0.061117

Combined Probability of Failure Existing Conditions



Levee Waterside Toe Elev. 4.3 feet
 Levee Crest Elev. 7.4 feet
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