GEOTECHNICAL DATA REPORT
LAS GALLINAS LEVEE SYSTEM
SAN RAFAEL, CALIFORNIA

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.

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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

[1]Non-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 (NVGD29). 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., Mw>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 Mw7 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

		Permeability (ft/day)			Anisotropy		Varied Parameters			
Case	Water elevation	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	Gradient at Station 120
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

		Permeability (ft/day)			Anisotropy		Varied Parameters			
Case	Water elevation	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	Gradient at Station 120
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

		Permeability (ft/day)			Anisotropy		Varied Parameters			
Case	Water elevation	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	Gradient at Station 120
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

		Permeability (ft/day)			Anisotropy		Varied Parameters			
Case	Water elevation	Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket K_V/K_H	Bay Mud K_V/K_H	Ks/Kb	Blanket Thickness, z (ft)	SC K_V/K_H	Gradient at Station 120
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

		Permeability (ft/day)			Anisotropy		Varied Parameters			
Case	Water elevation	Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket K_V/K_H	Bay Mud K_V/K_H	Ks/Kb	Blanket Thickness, z (ft)	SC K_V/K_H	Gradient at Station 120
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

		Permeability (ft/day)			Anisotropy		Varied Parameters			
Case	Water elevation	Perm of SC	Perm of Blanket	Perm of Bay Mud	Blanket K_V/K_H	Bay Mud K_V/K_H	Ks/Kb	Blanket Thickness, z (ft)	SC K_V/K_H	Gradient at Station 120
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 (phi 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 Unit Weight (pcf)	Bay Mud Cohesion (psf)	Bay Mud Friction Angle, Phi	Clayey Sand (SC) Unit Weight (pcf)	Clayey Sand (SC) Cohesion (psf)	Clayey Sand (SC) Friction Angle, Phi	Blanket Layer (Clay/Silt) Unit Weight (pcf)	Blanket Layer (Clay/Silt) Cohesion (psf)	Blanket Layer (Clay/Silt) Friction Angle, Phi	Factor Safety
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 Unit Weight (pcf)	Bay Mud Cohesion (psf)	Bay Mud Friction Angle, Phi	Clayey Sand (SC) Unit Weight (pcf)	Clayey Sand (SC) Cohesion (psf)	Clayey Sand (SC) Friction Angle, Phi	Blanket Layer (Clay/Silt) Unit Weight (pcf)	Blanket Layer (Clay/Silt) Cohesion (psf)	Blanket Layer (Clay/Silt) Friction Angle, Phi	Factor Safety
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 Unit Weight (pcf)	Bay Mud Cohesion (psf)	Bay Mud Friction Angle, Phi	Clayey Sand (SC) Unit Weight (pcf)	Clayey Sand (SC) Cohesion (psf)	Clayey Sand (SC) Friction Angle, Phi	Blanket Layer (Clay/Silt) Unit Weight (pcf)	Blanket Layer (Clay/Silt) Cohesion (psf)	Blanket Layer (Clay/Silt) Friction Angle, Phi	Factor Safety
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 Unit Weight (pcf)	Bay Mud Cohesion (psf)	Bay Mud Friction Angle, Phi	Clayey Sand (SC) Unit Weight (pcf)	Clayey Sand (SC) Cohesion (psf)	Clayey Sand (SC) Friction Angle, Phi	Blanket Layer (Clay/Silt) Unit Weight (pcf)	Blanket Layer (Clay/Silt) Cohesion (psf)	Blanket Layer (Clay/Silt) Friction Angle, Phi	Factor Safety
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 Unit Weight (pcf)	Bay Mud Cohesion (psf)	Bay Mud Friction Angle, Phi	Clayey Sand (SC) Unit Weight (pcf)	Clayey Sand (SC) Cohesion (psf)	Clayey Sand (SC) Friction Angle, Phi	Blanket Layer (Clay/Silt) Unit Weight (pcf)	Blanket Layer (Clay/Silt) Cohesion (psf)	Blanket Layer (Clay/Silt) Friction Angle, Phi	Factor Safety
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 Unit Weight (pcf)	Bay Mud Cohesion (psf)	Bay Mud Friction Angle, Phi	Clayey Sand (SC) Unit Weight (pcf)	Clayey Sand (SC) Cohesion (psf)	Clayey Sand (SC) Friction Angle, Phi	Blanket Layer (Clay/Silt) Unit Weight (pcf)	Blanket Layer (Clay/Silt) Cohesion (psf)	Blanket Layer (Clay/Silt) Friction Angle, Phi	Factor Safety
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', Kv/KH = 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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