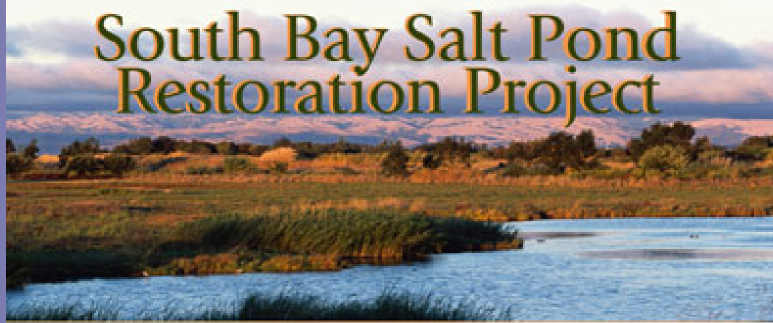
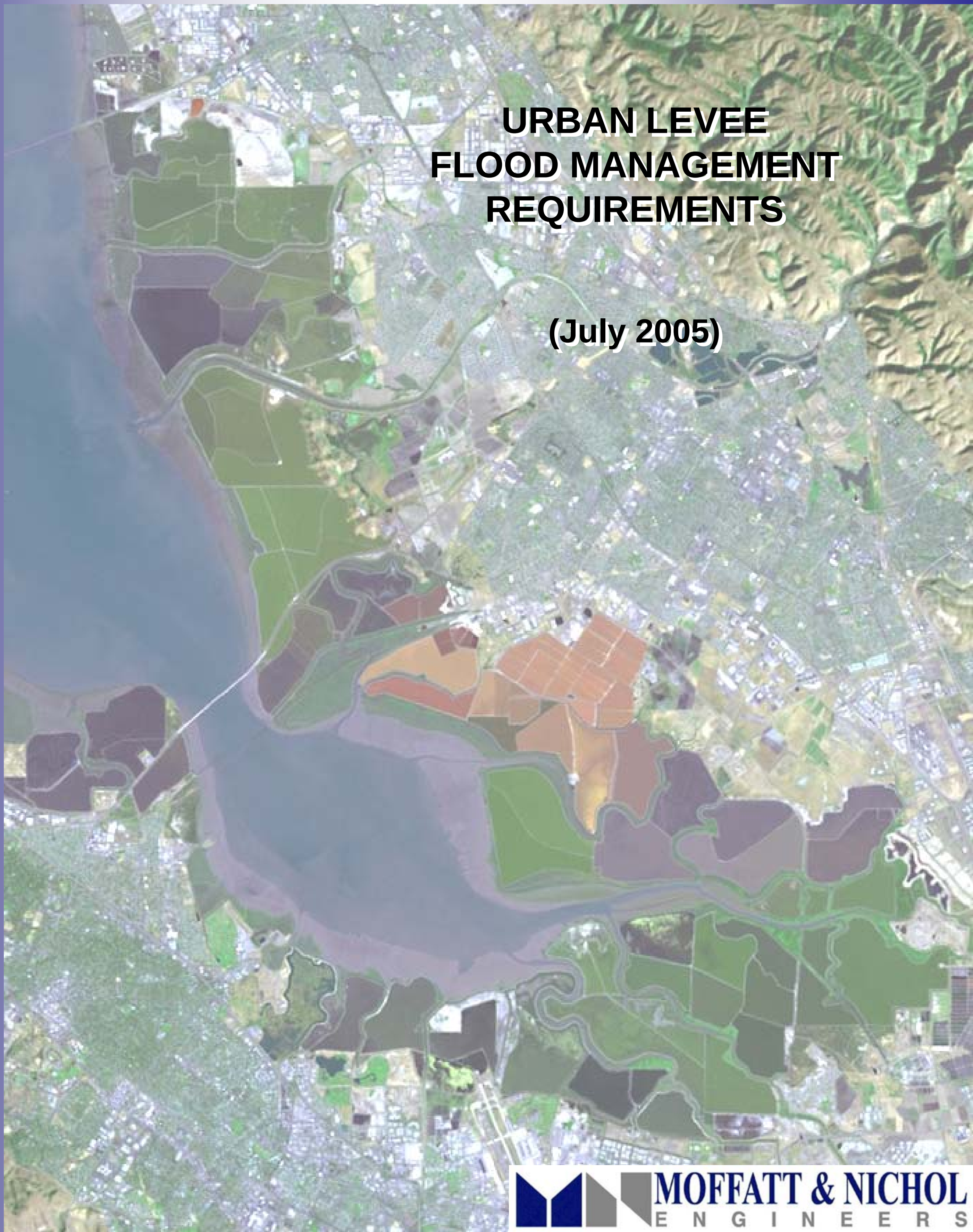


South Bay Salt Pond Restoration Project

Coastal Conservancy

A San Francisco Bay project provided by the California Department of Fish & Game, Coastal Conservancy and U.S. Fish & Wildlife Service



URBAN LEVEE FLOOD MANAGEMENT REQUIREMENTS

(July 2005)

MOFFATT & NICHOL ENGINEERS

URBAN LEVEE FLOOD MANAGEMENT REQUIREMENTS

SOUTH BAY SALT POND RESTORATION PROJECT

Prepared For :

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July 2005

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GLOSSARY

100-year flood	The one-percent annual chance flood. The one-percent annual flood is the flood that has a one-percent chance of being equaled or exceeded in any given year.[†]
Base Flood	The flood which has "regulatory significance". Generally it is considered synonymous with the flood having a one-percent chance of being equaled or exceeded in any given year.[‡]
Coastal Flooding	Flooding that occurs along the Great Lakes, the Atlantic and Pacific Oceans, the Gulf of Mexico, and other estuarine coastlines.[†]
Coastal High Hazard Areas	Special Flood Hazard Areas along the coasts that have additional hazards due to wind and wave action. These areas are identified on Flood Insurance Rate Maps as Zones V, V1-V30, and VE.[‡]
Flood	A general and temporary condition of partial or complete inundation of normally dry land areas from (1) the overflow of inland or tidal waters or (2) the unusual and rapid accumulation or runoff of surface waters from any source.[†]
Flood Hazard Factor	The average weighted difference between the 10- and 100-year water surface elevations rounded to the nearest one-half foot, multiplied by 10, and shown as a three-digit code.[1]
Flood Insurance Rate Map	Official map of a community on which the Mitigation Division Administrator has delineated both the special hazard areas and the risk premium zones applicable to the community.[‡]
Floodplain	Any land area susceptible to being inundated by flood waters from any source.[‡]
Freeboard	An addition to a levee's design height to ensure against overtopping during the design flood.*
Hydraulic Analysis	An engineering analysis of a flooding source carried out to provide estimates of the elevations of floods of selected recurrence intervals.[†]
Hydrologic Analysis	An engineering analysis of a flooding source carried out to establish peak flood discharges and their frequencies of occurrence.[†]
Levee	A manmade structure, usually an earthen embankment, designed and constructed in accordance with sound engineering practices to contain, control, or divert the flow of water so as to provide protection

[†] Glossary of Terms, "Guidelines and Specifications for Flood Hazard Mapping Partners [April 2003]", FEMA, http://www.fema.gov/pdf/fhm/frm_gsgl.pdf

[‡] NFIP Definitions, FEMA website http://www.fema.gov/nfip/19def2.shtm

[*] Appendix A, "Risk Analysis and Uncertainty in Flood Damage Reduction Studies", National Research Council, 2000

[1] Flood Insurance Study, City of San Jose, California, Volume 1, FEMA, 1998, p. 62

	from temporary flooding.[†]
Riverine/Fluvial Flooding	The overbank flooding of rivers and streams.[†]
Special Flood Hazard Area	The area delineated on a National Flood Insurance Program map as being subject to inundation by the base flood. SFHAs are determined using statistical analyses of records of riverflow, storm tides, and rainfall; information obtained through consultation with a community; floodplain topographic surveys; and hydrologic and hydraulic analyses.[†]
Stillwater Flood Elevation	Projected elevation that flood waters would assume in the absence of waves resulting from wind or seismic effects.[†]
Urban Levee	Levees that provide protection from flooding in communities, including their industrial, commercial, and residential facilities.*

[†] Glossary of Terms, FEMA

* EM 1110-2-1913, "Design and Construction of Levees", US Army Corps of Engineers, April 30, 2000, p.1-2

ABBREVIATIONS

ACFCWCD	Alameda County Flood Control and Water Conservation District
BFE	Base Flood Elevation
CFS	cubic feet per second
FEMA	Federal Emergency Management Agency
FHF	Flood Hazard Factor
FIRM	Flood Insurance Rate Map
GIS	Geographic Information System
HOWL	Highest Observed Water Level
LOWL	Lowest Observed Water Level
MHHW	Mean Higher High Water
MHW	Mean High Water
MLLW	Mean Lower Low Water
MLW	Mean Low Water
MSL	Mean Sea Level (incorrectly used as NGVD frequently)
MTL	Mean Tide Level
NFIP	National Flood Insurance Program
NGVD	National Geodetic Vertical Datum, 1929
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
SCVWD	Santa Clara Valley Water District
SFHA	Special Flood Hazard Area
SFRWQCB	San Francisco Regional Water Quality Control Board
USACE	United States Army Corps of Engineers
WSEL	Water Surface Elevation

1 INTRODUCTION

1.1 BACKGROUND AND PURPOSE

The South Bay Salt Ponds Restoration Project consists of enhancing the recently acquired Cargill Salt Ponds, which have been grouped into the following pond complexes:

- Alviso Complex - The 7500-acre complex of ponds between Charleston Slough and Mud Slough in Santa Clara and Alameda Counties, which consists of seven smaller groups of ponds separated by streams. The Alviso complex is shown in Figures 1.1a through 1.1c.
- West Bay Complex - The 1500-acre complex of ponds in southern San Mateo County, which consists of three smaller groups of ponds separated by Hwy 84 and Ravenswood Slough. The West Bay complex is shown in Figure 1.1d.
- Baumberg Complex - The 4800-acre complex of ponds between Hwy 92 and Coyote Hills Slough in Alameda County, which consists of three smaller groups of ponds separated by streams. The Baumberg complex is shown in Figure 1.1e.

Work described in this report was conducted for the California Coastal Conservancy, as part of the data acquisition phase of the restoration project. This report is intended to identify, at a conceptual engineering level, the flood management requirements for the inboard salt pond levees which could function as perimeter (Bayfront) levees after implementation of the proposed restoration project. In addition, it includes a conceptual feasibility analysis which addresses levee improvements and parametric cost estimates that could be used for planning a continuous perimeter flood protection barrier for urban communities.

Prior studies have addressed potential interactions between the restoration project and water conveyance facilities (M&N 2003). The M&N report discussed the possible effects of restoration on San Francisco Bay and on the flood conveyance characteristics of local creeks, rivers, and flood control channels due to reestablishing hydraulic connections to the Bay.

With the expected range of effects in flooding characteristics, an important planning consideration is the preservation or improvement of existing flood protection levels for local communities. The salt ponds currently provide a varying, but substantial, level of flood protection at the bayfront levees fronting the Bay. During the planning phase of the restoration project, and potentially into the future, these levees will not be maintained which transfers flood protection functions to the urban levees that currently function primarily as salt pond perimeter levees.

1.2 SCOPE OF WORK

The scope of work included the following:

Task A - Determine Criteria and Standards For Flood Control Levees

This task included review of existing codes and literature and discussions with local flood control districts to determine the flood control standards that will apply to perimeter levees in the South Bay.

Task B – Urban Levee Condition Report

This task included evaluating the existing urban levees for the purpose of developing a conceptual engineering feasibility study. The following subtasks were performed:

- Identify Urban Levee Segments
- Gather Existing Data
- Additional Field Data – This subtask includes a survey of the existing urban levee crest centerline and establishment of benchmarks for future use.
- Reconnaissance Level Geotechnical Assessment - This subtask includes a field investigation to characterize the existing conditions of the salt pond levees and to identify deficiencies in the urban levees to meet the flood protection requirements as a result of the restoration project.

Task C - Develop Scope For Subsequent Geotechnical Assessment (Provided Separately)

This task included preparing a scope of work for an initial geotechnical assessment of the condition of the urban levees in the project area. The scope of work included a description of required field investigation techniques and equipment type, access requirements, limits and extent of geotechnical sampling, laboratory testing to be performed, and preliminary level costs to conduct the investigation.

Task D - Assess Feasibility Of Providing Continuous Flood Protection Levee

This task included an engineering feasibility analysis to provide continuous flood protection using selected urban levee segments. Each levee segment identified as a potential flood protection levee will be addressed separately, to assist in the environmental documentation process.

Since the flood level and the required levee crest elevation will depend on the restoration alternative selected, a range of water levels was assumed for cost estimates. The improvement concepts and costs could be used in preparing alternatives for environmental documentation.

Task E - Provide Information For GIS (Provided Separately)

In anticipation of a GIS system to be developed by others, this task includes providing a GIS layer(s) for existing the urban levee that will include topographic data and will identify the future flood protection levees alignment.

2 FLOOD PROTECTION REQUIREMENTS

This report focuses on the improvement of the urban levees between the salt ponds and upland communities. For purposes of this report, the urban levee is defined as the salt pond levee which "directly" protects (or will protect after restoration) a community from tidal flooding. In most cases it is not the present Bayfront or Interior (between 2 adjacent ponds) levee, but represents a "last line of defense" against tidal and fluvial flooding. As a common means to reduce flood damage, the construction of levees can also result in revision of floodplain limits and relieve a community from the cost of purchasing federally mandated flood insurance policies. The following sections describe levee design and construction standards that apply to the restoration project.

2.1 HISTORICAL BACKGROUND

The federal government has been involved in flood control since the creation of the U.S. Army Corps of Engineers (USACE) in 1802, and has been explicitly responsible for flood control since the Flood Control Act of 1936. Construction of levees has been an important component of flood control, and over the years the USACE has developed standard levee design methods and criteria for the purpose of providing a uniform level of flood protection.

The 1968 National Flood Insurance Act and the 1973 Flood Disaster Protection Act created and revised the National Flood Insurance Program (NFIP) which sought to accomplish two primary goals:

> "Congress wanted property owners to purchase flood insurance to (1) provide them with financial relief should they suffer losses in a flood and (2) lessen the financial burden on federal, state, and local governments to provide grants and low-interest loans to cover the losses of uninsured property owners. These acts also sought to reduce damage from moderate-sized floods by encouraging construction of levees and other flood damage reduction structures." (National Research Council 2000)

In 1979, the Federal Emergency Management Agency (FEMA) was created to consolidate numerous government agencies into a less complex, centralized agency. FEMA was given responsibility for administration of the NFIP, and adopted USACE flood control standards for certification of levees.

The following sections describe the role of the federal agencies in flood protection, their standards and methods, and the resulting levee certification requirements for the restoration project.

2.2 US ARMY CORPS OF ENGINEERS (USACE)

The USACE provides financial resources, design services, and construction support for various projects designed to improve existing flood protection levels. The USACE also has FEMA-authorized authority to certify that a levee provides 100-year flood protection (Federal Register, 44 CFR Part 65.10e).

2.2.1 General Standards

Flood control standards have been developed to protect communities from various types of flooding. The project area is subject to two main flooding types: coastal flooding and fluvial flooding. Coastal flooding can occur due to high tides in the Bay exceeding the ground elevations or overtopping levees. These tidal flooding events are a result of a combination of astronomical (tidal), atmospheric (low pressure), and local (wind/wave) effects. Fluvial flooding can result from high water levels in waterways during extreme rainfall events, overtopping riverbanks, levees, or floodwalls.

Through the USACE's extensive experience with levees has emerged an acknowledgement of the need for engineering judgement in developing design criteria:

> "Numerous factors must be considered in levee design. These factors may vary from project to project, and no specific step-by-step procedure covering details of a particular project can be established. However, it is possible to present general, logical steps based on successful past projects that can be followed in levee design and can be used as a base for developing more specific procedures for any particular project." (USACE 2000)

<u>100-Year WSEL</u>

The 100-year WSEL is the elevation of the flood with a one-percent chance of exceedance during a single year. This elevation is a site-specific elevation based on existing topography and hydraulic modeling, and typically increases in elevation with increasing distance from the receiving waterbody.

The 100-year WSEL can be based on coastal or fluvial flooding, or a combination of the two. Coastal flooding can be estimated by collecting data on tides, winds, shore geometry, and other data, and comparing with existing ground elevations. Fluvial flooding can be estimated by performing a hydrologic/hydraulic analysis of the waterway based on rainfall data, channel geometry, and other factors.

2.2.2 Coastal Flood Protection

The USACE's approach to coastal levees in the Bay has been described in the San Francisco Bay Shoreline Study (USACE 1988), and can be summarized as follows:

> "The design crest elevations for the levees and other protective structures considered in this study were based on four components: "still-water" tide elevations; tidal flood elevations; wave run-up elevations; and freeboard. An allowance for overbuild above the design crest elevations of levees was made to compensate for post-construction settlement."

For outboard (nearest to the Bay) coastal levees exposed to wind-induced waves, the crest elevation requires 1 foot of freeboard above the wave run-up elevation. For outboard coastal levees not exposed to wind-induced waves and for inboard (landward of the outboard levee) coastal levees, the crest elevation requires 1 foot of freeboard above the still-water tide elevation. Figure 2.2.2 shows these freeboard requirements. While Figure 2.2.2 essentially portrays the levee crest elevation design criteria as contained in the "Office Report, Southern Alameda and Santa Clara Counties Study Area, San Francisco Bay Shoreline Study," U.S. Army Corps of Engineers, San Francisco District, October 1988, it is no longer relevant criteria as the Corps has adopted risk-based analysis as it's criteria.

Wave effects were not included for the inboard levees due to the presence of the outboard levee; however, the USACE acknowledged the following:

> "If an existing outboard levee were severely eroded or breached, the actual water surface elevation at the inboard project levee would approach or equal the tide elevation in the open bay, which could significantly exceed the design water surface elevation…….
>
> ..…. Because additional freeboard was included in the crest elevations of the inboard project levees in response to the threat of failure of the existing outboard levees, no height was added to the inboard levees to address the small amount of wave run-up expected. Although this design could result in wave overtopping of the inboard levees

> under worst-case conditions (i.e., failure of the outboard levees in combination with severe winds during the peak of an extreme high tide), the volume of overtopping is expected to be minor even under those conditions."

For a tidal restoration project, breaching of an existing outboard levee will occur in one of two ways: 1) immediately, as part of the restoration, to restore tidal influence to a specific area; or 2) slowly, as erosion effects are coupled with a lack of maintenance, possibly over a period of years. In both cases the inboard levees will be exposed to tidal waters and most likely wind-wave action, requiring that wave run-up and erosion be considered in the analysis.

It is important to note that the potential of levee overtopping (where permissible) mentioned above highlights the use of site-specific engineering judgement by the USACE. Engineering design criteria for determining the levee crest elevation could thus differ from past or adjacent projects.

2.2.3 Fluvial Flood Protection

The USACE set the original standards for riverine levees as the design flood elevation plus 3 feet of freeboard, which became the widely accepted industry standard:

> "The best estimate has traditionally been based on the expected height of a design flood (e.g., a 100-year flood, the magnitude of which has a 1 percent chance of being equaled or exceeded in any given year, and which is here called the "1% flood"). Freeboard was then added above the expected height. Many Corps flood damage reduction projects used a standard of 3 feet of freeboard. "Three feet of freeboard" became an engineering tradition within the Corps and was employed in hundreds of Corps flood damage reduction studies and projects." (National Research Council, 2000)

This standard was used and continues to be used for FEMA certification. However, the relatively recent development of advanced hydraulic modeling techniques has caused the use of freeboard to be eliminated:

> "The 3-feet-of-freeboard concept was used as a design parameter to account for uncertainties associated with hydrologic and hydraulic analysis (Huffman and Eiker, 1991). If these uncertainties were accounted for, exceptions to the 3-feet-of-freeboard requirement were granted." (National Research Council, 2000)

The uncertainties mentioned have been accounted for in the USACE's current risk-based analysis approach, which establishes the 100-year WSEL with a high degree of reliability and eliminates the freeboard concept. The 100-year WSEL corresponds with the median one-percent event; that is, an elevation with a 50% chance of non-exceedance during a 1% chance annual event (USACE 1996). This higher reliability is very evident in fluvial systems which have a long flow record (for example, rivers such as Mississippi, Colorado, Sacramento, San Joaquin, etc.)

As the USACE adopted this new analysis approach, concerns arose over the non-uniformity of flood protection due to the differences between USACE design methodology and the traditional methods used by FEMA. The following section describes the current status of levee certification by the USACE.

2.2.4 Riverine Levee Certification

Given the differences between the FEMA and USACE methods, an agreement was reached that allows the USACE to certify levees meeting one of two criteria:

No risk-based analysis performed – Use standard FEMA criteria

Use standard criteria of BFE (1% flood) plus 3 feet of freeboard.

Risk-based analysis performed – Use one of the following outcomes

If the standard FEMA criteria elevation results in a conditional non-exceedance probability of **less than 90%** passing the 1% flood, the minimum levee elevation will be brought up to the elevation corresponding to a 90% chance of non-exceedance.

If the standard FEMA criteria elevation results in a conditional non-exceedance probability of **greater than 95%** passing the 1% flood, the minimum levee elevation can be brought down to the elevation corresponding to a 95% chance of standard FEMA criteria.

If the standard FEMA criteria elevation results in a conditional non-exceedance probability of **between 90% and 95%** passing the 1% flood, the minimum levee elevation can correspond to the standard FEMA criteria.

This certification framework provides integration of the two divergent design methodologies used by the USACE and FEMA. Figure 2.2.4 shows a levee certification decision tree graphically depicting this framework. For existing levees formulated with risk-based analysis and levees to be constructed or modified by the U.S. Army Corps of Engineers, the certification procedure will follow the "LEVEE CERTIFICATION DECISION TREE" as shown in Figure 2.2.4.

For certification of an existing levee or those constructed by others, a detailed geotechnical evaluation and review of existing conditions must be made to determine if the levee system can contain the base water surface profile or at what lower elevation the levee is likely to fail. The review of records, inspection and evaluation for certification purposes should address the following guidance as a minimum (Memorandum from the Chief, Engineering Division, Directorate of Civil Works, USACE, June 1997):

a. Review of available data and information:

- Geologic maps, boring logs, and groundwater history
- Aerial photographic records
- Original design and construction records
- Records of utility crossings
- Annual surveys of top of protection and cross sections
- Water surface elevations and duration of previous high water levels
- Levee performance (instrumentation and visual records) before, during and after these previous high water
- Performance and maintenance of underseepage control measures
- Flood fighting records
- Design and construction records of remedial or other project modifications
- Natural drainage, interior drainage and ponding areas
- Operations and Maintenance Manual
- Maintenance records

b. Field inspection:

The field inspection is intended to be the basis for the evaluation and will verify the physical aspects and the maintenance condition of the project levee. This inspection should document all new observations of encroachments, animal burrows, condition of the top of the levee, evidence of erosion on and adjacent to the levee, excavations in or adjacent to the levee for ponding, adequacy of the foundation and tie-in for closure structures, evidence of seepage, piping, sloughs or other instability and overall maintenance. Interviews with local residents and maintenance personnel involved in routine inspections and flood fighting activities are important to obtain eyewitness reports of levee performance. It is important to prepare through documentation to include photographs of the inspection, which will become part of the permanent documentation for the project.

2.2.5 Other Standards

- The USACE has adopted minimum Factors of Safety (FS) for Static and Dynamic Geotechnical Stability. These standards apply to different time periods, durations, and loading conditions as follows:
- Case I – End of construction, FS = 1.3
- Case II – Sudden Drawdown, FS = 1.0 to 1.2
- Case III – Steady seepage from full flood stage, FS = 1.4
- Case IV – Earthquake, conduct standard and site specific studies including seismic analysis for all locations in seismic zones 3 and 4, and those locations in zone 2 where potential for liquefaction exists (USACE 1995)

2.3 FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA)

Among many responsibilities, FEMA sets federal flood management standards, including determination of areas that are in the 100-year floodplain (coastal, fluvial or combination of both). These areas are identified on the Flood Insurance Rate Maps (FIRMs) as Special Flood Hazard Areas (SFHA), and properties within the SFHA are subject to several requirements:

- all federally-backed loans must be protected by flood insurance (most lenders require this whether federal or not);
- all new or substantially improved structures must have the lowest floor elevated above the BFE.

Areas which are designated as SFHA are lower than the BFE and are not protected by a FEMA certified levee. If a non-certified levee fronts an urban development or community, a Flood Hazard Factor (FHF), which is a measure of the flooding risk and extent of potential flooding of the community, is usually determined by FEMA. The SFHA are shown on the FIRMs as zones that start with the letter A or V:

- Zone A – base flood elevation (BFE) and flood hazard factor (FHF) not determined;
- Zone AO – sheet flooding from 1 to 3 feet deep, no FHF determined;
- Zone AH – BFE shown, flooding from 1 to 3 feet deep, no FHF determined;
- Zone AE – BFE shown, depth will vary based on ground elevation, no FHF determined;
- Zone A1-30 – BFE and FHF determined, subdivided by FHF;

- Zone V – coastal flooding areas subject to 3-foot high wave action or greater, no BFE or FHF determined;
- Zone VE – coastal flooding areas subject to 3-foot high wave action or greater, BFE determined, no FHF determined;
- Zone V1-30 – coastal flooding areas subject to 3-foot high wave action or greater, BFE and FHF determined.

Additionally, properties outside of the SFHA but within the 500-year floodplain are defined on the FIRM's as either Zone B or Zone X and are not subject to the SFHA requirements. However, voluntary flood insurance for these zones is available at lower premiums than that for properties within the 100-year floodplain.

Although flooding can be divided into two main categories, coastal flooding and fluvial flooding, estuarine areas will in most cases be exposed to a combination of both types of flooding. Coastal flooding can occur due to high tides in the Bay exceeding the ground elevations or waves overtopping the levees. These tidal flooding events are a result of a combination of astronomical (tidal), atmospheric (low pressure), and local (wind/wave) effects. Fluvial flooding can result from high water levels in waterways during extreme rainfall events, overtopping riverbanks, levees, or floodwalls.

As previously stated, this report focuses on the improvement of the urban levees between the salt ponds and the upland communities. As a common means to reduce flood damages, the construction of levees that are certified by FEMA can also result in revision of floodplain limits and relieve a community from current levels of flood risk, and in most cases from the cost of purchasing federally mandated flood insurance policies. The following sections describe levee design and construction standards that have been adopted by FEMA.

2.3.1 General Standards

The Base Flood Elevation (BFE) is the FEMA equivalent of the USACE 100-year WSEL, and is similarly dependent on whether the flooding is coastal or fluvial. Where determined, the BFE is shown on the FIRM. The BFE can be shown as a single elevation for a region, or can be a typical depth over a specified area, or can be shown with contours where the BFE is more variable.

A freeboard is required by FEMA for levee certification. Similar to the determination of the BFE, the amount of freeboard depends on whether the flooding is coastal or fluvial.

Levee Certification Procedure - Because the certification of a levee effectively revises the floodplain, FEMA requires the submission of application forms entitled "Revisions to NFIP Map, MT-2". The MT-2 package contains forms designed to attest to the levee's adequacy, and to ensure that :

- the data and methodology are based on current conditions;
- qualified professionals have assembled data and performed all necessary computations;
- all individuals and organizations affected by proposed changes are aware of the changes and will have an opportunity to comment on them.

The forms require adequate proof that proper levee analysis, design, and construction were performed before revising the floodplain. The standards adopted by FEMA are described in the following sections.

2.3.2 Coastal Flood Protection

For coastal flood protection, the freeboard "must be established at 1 foot above the height of the one percent wave or the maximum wave runup (whichever is greater) associated with the 100-year stillwater surge elevation at the site." (Federal Register, 44CFR Part 65). Lesser freeboard requirements may be approved if supported with appropriate analysis, but freeboard must be established at a minimum height of 2 feet above the 100-year stillwater surge elevation. Figure 2.3.2 shows these freeboard requirements.

The 100-year stillwater surge elevation incorporates two components of the expected water surface elevation: the *stillwater elevation* is the water elevation based on tides and atmospheric conditions; when adding wave setup,[†] the elevation is defined as the *stillwater surge elevation*.

Wave runup elevations above the stillwater surge elevation are determined through coastal engineering analysis (wave hindcasting and forecasting techniques) taking into account various parameters, including bathymetry, water levels, wind speed and direction, and fetch length.

2.3.3 Riverine/Fluvial Flood Protection

Water surface elevations from fluvial flooding are determined by hydraulic modeling based on rainfall and waterway characteristics. The modeling results are typically shown as a longitudinal profile along the waterway due to the variation of water surface elevations with distance from the mouth.

To certify a riverine levee as providing 100-year protection, FEMA has historically required the levees to meet the following design criteria:

- 3 feet of freeboard above the base flood elevation (BFE). Additional freeboard is required within 100 feet of structures/constrictions and at the upstream levee limit (which is tapered out over the length of the levee).
- Geotechnically stable, including foundation stability, prevention of seepage, settlement and erosion which lowers levee stability;

A maintenance and operation plan adopted by and under the jurisdiction of a Federal or State agency, an agency created by Federal or State law, or an agency of a community participating in the NFIP must assume ultimate responsibility for maintenance. This plan must document the formal procedure that ensures that the stability, height, and overall integrity of the levee and its associated structures and systems are maintained. At a minimum, maintenance plans shall specify the maintenance activities to be performed, the frequency of their performance, and the person by name or title responsible for their performance.

2.4 SUMMARY AND APPLICABILITY TO SBSP PROJECT

It is very likely that Federal funding will be sought for implementation of the restoration project, and the USACE and/or Fish and Wildlife Service may be the lead contracting agency for project construction. The project will also in all likelihood change existing flooding characteristics in the area. This implies that levee construction and eventual certification will be based on requirements of both FEMA and the USACE.

[†] Wave setup is the super-elevation of mean water level caused by wave action.

Since flooding in the study area is influenced by both tides and river flow, site-specific Flood Insurance Studies will be required. Although flood protection benefits are quite likely after implementation of the restoration project, the effect of the restoration project, especially considering the long timeline for implementation, may be significant when interim flood management requirements are factored in.

3 EXISTING LEVEE ASSESSMENT

The existing salt pond levees were constructed for salt processing operations, and were not intended to provide a specific level of flood protection. As flood protection requirements were developed and refined to the current state described in Section 2 of this report, it also becomes clear that the levees do not meet *current* design requirements. However, the levees have nonetheless provided significant flood protection for adjacent urban areas. Coastal flooding has rarely overtopped the levees, and the level of flood protection appears to be consistent due to regular maintenance provided by the salt pond operators.

3.1 APPROACH

All the ponds with levees which would function as a future urban levee were evaluated, but costs were provided for a subset of ponds as explained below.

Ponds A1 to A3W

These ponds encompass the shoreline between Mtn View and Sunnyvale. These ponds are evaluated & costs provided in this report.

Pond east of A3W

This pond is the Sunnyvale WWTP's Oxidation Pond (not part of Refuge). It was assumed that the small pond between the Oxidation ponds and Lockheed Martin which is not part of Refuge property would not be restored to full tidal action. Levee improvements would be necessary along these interior levees to mitigate flooding of the ponds at high water levels if Pond A3W is restored. However, since these costs would not be associated with “urban” levee improvements the interior levees were not surveyed, neither are costs provided in this report. A contingency should be added to the A3W restoration to account for additional levee improvements resulting from the project.

Pond A4 (not part of Refuge, owned by SCVWD)

The District has evaluated A4 in much more detail. So, although the pond was evaluated, costs were not provided (The documentation by the SCVWD for the restoration of Pond A4 will include those).

Pond A8

The pond was considered by SCVWD for flood control. The Lower Guadalupe EIS/R covers the pond in greater detail. The southern part of the pond has interior berms which protect the tern nesting areas. It was assumed that there would be no breaches from Guadalupe Slough to the southern part. Consequently, “urban” levee improvements would not be necessary, so no costs are provided. However, if the northern part of A8 (A8N) is restored, additional costs will be incurred to seal and raise the interior berms to avoid impacts to the southern part of the pond (A8S). These should be added as a contingency to the restoration costs for Pond A8, similar to the discussion above for Pond A3W.

New Chicago Marsh area

The New Chicago Marsh area (along with the SPRR track) constrains Ponds A12, A13, and A16. Originally it was assumed that a flood control levee along the south side of the marsh would be constructed, to protect the Town of Alviso assuming the 3 ponds were restored, and an alignment was surveyed (shown in the attached figures), but that assumes the marsh would be inundated to tidal levels. Based on conversations with the USFWS, this may not be practical, therefore costs for a new levee are not provided. If full tidal restoration of Ponds A12, A3, and A16 is envisioned, a levee would be needed along the boundary with the New Chicago Marsh and costs for this levee should be added to the restoration cost of the ponds.

Ponds A22 and A23

These ponds are surrounded by other ponds/sloughs to the south and west. The only development which would be affected by restoration is Cushing Parkway and refuge property north of A22 and SPRR tracks to the west. Pond A22 was surveyed and quantities for the north side are estimated.

Data Collection

Digital ortho-photographs from USGS for the mid-90's time period were used as the base map for this analysis. The alignment of the Bay Trail was obtained from the Association of Bay Area Governments (ABAG) website. Several documents including feasibility studies and environmental documents prepared by the local flood control agencies were also reviewed.

Tide Gage information was obtained from data and reports published by the U.S. Army Corps of Engineers, the National Ocean Service, and prior reports for the study area.

Flood Insurance Rate Maps (FIRM) and Flood Insurance Studies (FIS) were obtained from FEMA to determine the areal extent and type of flooding in the project areas. Revisions to the FIRM due to Letters of Map Revision (LOMR) or Conditional Letters of Map Revision (CLOMR) as a result of recently constructed Flood Control Projects were not reviewed as part of this report; therefore current flood zone limits may vary slightly from limits shown on the FIRM.

Visual Reconnaissance Surveys

Hultgren-Tillis (HTE) and Moffatt & Nichol (M&N) conducted a feasibility-level visual reconnaissance survey of the existing levees in the Alviso and West Bay units in November and December 2003 to determine the general condition of the levees and adjacent topographic features. For each pond the existing urban levee was accessed and general conditions of the levees, such as crest width, slope geometry, levee condition, and presence of vegetation were recorded at various locations along the levees. Typical cross sections were recorded by pacing and by using a plunging hand level. Using centerline elevations from the ground survey provided by Tucker and Associates, the elevations were corrected to NGVD. The nature of the fill that makes up the existing levee was assessed by observing exposed surfaces. A sample form used for the field assessment is shown in Figure 3.1.1 and the forms themselves are provided in Appendix B.

A reference line was established for this study, for orientation and stationing during field work, and adjusted as needed to match field conditions. Generally, the reference line shown in this report follows established perimeter levees, some of which have been upgraded for pedestrian and vehicle traffic.

Topographic Survey

A topographic survey of existing levee elevations was conducted by Tucker & Associates in December 2003. The survey consisted of a series of spot elevations using kinematic and GPS survey methods taken along the levee crest centerline. Spot elevations were taken at 100-foot spacing (approximate) to provide a general profile of the levee and true positions (x, y, z coordinates) were recorded. Several benchmarks were also established and monumented in the study area to serve as control for future survey efforts. The data are presented in plates in Appendix A, and average, typical elevations are shown in the figures attached to the main body of the report.

Geotechnical Analysis

Numerous soil investigation reports and boring logs were reviewed by HTE, including discussions with staff at Public Works Agencies for the adjoining Cities. The type of information obtained is described in Section 3.2.2.

3.2 EXISTING LEVEES – PHYSICAL CONDITIONS

Weak clays and silts (Bay Mud) are present at the surface of the project area. Since the site is located at the fringes of San Francisco Bay, changing the land use of the salt ponds to a tidal marsh will subject the landward-most salt pond levees to tidal and flood stages at the new margin of the Bay. In the past, these levees retained ponded water that had controlled water surface elevations. The existing levees were not well compacted when they were constructed. These soils are highly compressible and will continue to settle and deform for several years. Additional material will be needed to maintain design crest elevations as the levees settle. Though no shallow seepage was observed during the visual reconnaissance survey, re-compacting the existing levee fills will likely be required for any new levee improvements.

Loose to medium dense sands occur beneath portions of the site, which could liquefy during a strong earthquake. The breadth and freeboard of the final levee configurations may accommodate the expected deformations for most areas. However, ground improvement or modified levee alignments and cross-sections may be needed in some areas.

The SBSP restoration project will require raising the existing levees to reach specified elevations to provide tidal flood protection. Earth embankments used to raise and broaden the existing levees will be constructed over the Bay Mud. In many areas, the Bay Mud is too weak to allow the levees to be raised to their final heights without special considerations. These may include wide stability berms and/or wick drains. Placing the fill in vertical stages and allowing the underlying Bay Mud to consolidate and gain strength can be an effective method for building on weak soils, provided ample time (years) is allowed between filling stages.

3.2.1 Construction Method for Existing Levees

At the salt ponds, the levees were primarily constructed by excavating materials from within the ponds with the use of a dragline or clamshell and casting the excavated material to the side to form the levees. Periodically the levees were raised and widened using the same approach. Most of the salt pond levees consist predominately of cast-up bay mud. The initial heights of most salt pond levees did not need to be very high. Consequently, the underlying Bay Mud foundation materials were not usually overstressed.

Stability failures can occur in soft Bay Mud foundation materials if levee embankment fills are placed too high over a short period. The overloaded ground beneath the levee fill sinks and the adjacent mudflat heaves up. This type of failure is common when filling over soft ground too rapidly. It is commonly referred to as a "mudwave”. Except for eroding levee faces, the existing salt pond levees are typically low to moderate in height and have fairly flat slopes. This configuration results in stable levees. The on-going maintenance filling adds or replaces eroded fill and creates moderately low loadings. These practices are sound for minimizing the risk of mudwaves.

In addition to diking of the ponds, several other areas were diked to create cells for refuse disposal. Solid waste landfills were created on the tidal flats and abut the salt ponds in these areas. The urban levees in the Alviso complex will abut the solid waste landfill dikes in Mountain View. Some dikes were created from imported soil, rock fragments, broken concrete and other predominately inorganic debris (rubble). The rubble-fill dikes typically have steeper slopes than the salt pond levees that are made from Bay Mud. One method for constructing rubble-fill dikes is to mound the fill high with steep side slopes and intentionally fail the ground, creating a mudwave. The rubble-fill displaces the Bay Mud creating a deep section of rubble fill. Rubble fill is pervious and is not well suited for creating levee sections with minimal seepage.

The landfill perimeter levee is likely constructed of clays, derived from imported fill and/or local Bay Mud. These levees should be of low permeability to control leachate seepage. A geometrically intuitive scheme for the urban levee would be to simply buttress the existing landfill slope and provide slope protection. However, the proximity of the new tidal waters to the landfill may be an issue for the landfill. Placement and design of the urban levee will need to be coordinated with the landfill owners and regulators. Leachate recovery wells with pneumatic pumps are located along the landfill perimeter. The geotechnical engineer will need to coordinate planned exploration locations with the landfill operator, checking that the pneumatic service and leachate extraction lines are identified as part of the utility clearances before drilling borings

3.2.2 Available Geotechnical Data

This feasibility study relies on existing geotechnical data. No new geotechnical data was developed as part of this study. Geotechnical characterization was accomplished by a combination of site visits and reviewing available existing data. Selected adjacent landowners and agencies were polled to locate existing geotechnical data near the salt pond levees that had been previously collected for various purposes. This data collection effort focused on data that could readily be obtained. The purpose was to provide data for a broad overview of the sub-surface conditions in the project area. Although additional data exists that was not obtained during this effort, sufficient data was identified to provide an overview appropriate for the feasibility study. This data collection should continue in subsequent preliminary design phases.

<u>Data sources include:</u>

- Town of Menlo Park
- City of Mountain View
- San Jose/Santa Clara Water Pollution Control Plant
- Santa Clara Valley Water District
- City of Fremont
- California Geological Survey
- URS Corporation
- Hultgren-Tillis Engineers internal files

The locations of the borings and cone penetration tests which were collected are shown on Figures 3.2.2a through 3.2.2c. Copies of the boring and CPT logs are contained in Volume 2: Supplemental Data. To identify the borings/CPT's, each data point was assigned a unique name/number. The numbering system consists of three parts. The first part signifies the name (or initials) of the company (or agency) that collected the data; the second part is the company's (agency's) project number, and the third part is the boring number assigned by the company (agency).

The geotechnical explorations primarily included drilled soil borings and pushed cone penetration tests (CPT) that were performed to obtain data for proposed structures or for environmental purposes. Limited data was found along the urban levee alignments or within the salt ponds. However, the data was sufficient to provide the broad overview that was sought for the feasibility study.

3.2.3 Subsurface Conditions

The young (Holocene) geologic unit upon which the existing levees and dikes were constructed is called San Francisco Bay Mud (Bay Mud). Based on a review of available geotechnical data, most of the levees are underlain by Bay Mud. Bay Mud consists of fine sediments that have settled out in the quiet waters of the San Francisco Bay estuary within the last 10,000 years. This material is predominately clay, has low strength and is highly compressible. Figure 3.2.3a shows the subsidence in the Santa Clara Valley between 1934 and 1967, which indicates that the pond levees may have settled as much as 4 to 5 feet due to groundwater extraction (SCVWD 2002). Figure 3.2.3b shows the amount of subsidence and groundwater levels in San Jose over a longer period (SCVWD 2002).

HTE estimated the elevation of the base of Bay Mud, based on selected exploration points from boring records. The data indicates that the Bay Mud extends to depths ranging from about –5 to –15 feet relative to NGVD in the majority of the Alviso and West Bay complexes. Deeper layers of Bay Mud were observed locally at existing creeks that create buried channels or fingers extending from the bay towards the land. The Bay Mud layer contains intermediate sand layers or lenses at various locations.

The Bay Mud typically overlies alluvial deposits, consisting primarily of medium stiff to stiff clays and loose to dense sands. Loose sands may also be present within the levee fills at some locations. In general, the clays and sands are of sufficient strength not to affect the existing static stability of the existing perimeter levees. The clays may be too weak in some areas to allow the new levees to be reliably constructed to final design heights in a single stage. Loose sands below the groundwater table may be at risk of liquefying during a large earthquake. These factors are discussed further herein.

3.2.4 Seismicity and Liquefaction

The project area lies between the San Andreas and Hayward faults. These two faults, the San Andreas and the Hayward, are classified as Type A faults by the California Department of Conservation Division of Mines and Geology (now part of the California Geological Survey). The nearest sections of the San Andreas fault to the various levees range from 10 kilometers (km) in Menlo Park to 26 km in Fremont and Newark. The Hayward fault to the east ranges from 6 kilometers to 19 kilometers from the levees. Four Type B faults lie near the levees on either side of the Santa Clara Valley. On the west side, the Monte Vista-Shannon Fault lies 8 to 22 km from the levees. To the east, the southeast Type B extension of the Hayward fault lies from 5 to 24 km from the levees. In addition, the Calaveras North and South fault segments are located from 11 to 29 km and 15 to 34 km east of the levees, respectively. Other faults with lower potential risk for strong seismic shaking may lie at closer distances than the above referenced faults. However, for seismic design the Type A and Type B faults described above may be used as the primary seismic source hazards for earthquake shaking.

The USGS has undertaken a seismic probability study in which it assesses the risk of occurrence of various levels of groundshaking from known fault sources in the vicinity. For a seismic risk of occurrence of 10 percent within 50 years, the peak acceleration for rock or very stiff soils at depth ranges from 0.44g to 0.64g below the salt pond levees.

Soil liquefaction is a phenomenon in which loose- to medium-dense saturated granular soil undergoes reduction of internal strength because of increased pore water pressure generated by shear strains within the soil mass. This behavior is most commonly induced by strong groundshaking associated with earthquakes. HTE judges that the loose and medium dense sands at the site will be susceptible to liquefaction. Though the levees may deform if sand layers within the foundation liquefy, broad, well-compacted levees with fairly flat slopes and ample freeboard will have minimal risk of overtopping or breaching. Assessing the liquefaction risk, estimating deformation and evaluating the resulting potential change in risk of overtopping/breaching will need to be addressed during subsequent design phases for the levees.

3.3 ALVISO COMPLEX

3.3.1 Existing Flooding

All of the Alviso Complex ponds are within the 100-year floodplain as defined by FEMA, with varying levels of potential flooding depending on the BFE and topography. The 100-year flood plain is shown on Figures 3.3.1a and 3.3.1b. Much of the surrounding area is flat and is thus also in the floodplain, with the exception of high ground associated with landfills in portions of Mountain View and San Jose. For the study area, stillwater elevations as described in various Flood Insurance Studies (FIS) performed by SCVWD and/or others for CLOMR applications, are as follows:

Presently Adopted 100-year Stillwater Elevations

(FEMA Flood Insurance Studies 1997, 1998, 1999)

Location	**Elevation (ft, NGVD)**
Palo Alto	7.7
Mountain View	8.0
Sunnyvale	8.0
Guadalupe Slough & Coyote Creek	8.1
SPRR & Alviso Slough	8.6
Milpitas	8.6

Available tidal benchmark data for tide stations near the Alviso Complex are presented in Table 3.3.1. However, many tide stations in the Alviso area were in operation for limited periods of time in the late 1970's; thus the data may not accurately reflect present conditions (M&N 2003).

Table 3.3.1 : Tidal Benchmark Data For Alviso Complex
(elevations in feet, MLLW)

Tidal Plane	941 4519 Mowry Slough	941 4521 Mud Slough Railroad Br.	941 4525 Palo Alto Yacht Harbor	941 4537 Palo Alto CM No 8	941 4548 Guadalupe Slough	941 4549 Upper Guadalupe Slough	941 4551 Gold Street Bridge	941 4561 Coyote Creek (Artesian Sl)	941 4575 Coyote Creek (Alviso Sl)	941 4589 Coyote Creek Tributary 2
Period Of Measurement	12/76 - 6/77	11/76 - 2/77	6/84 - 12/84	6/76 - 3/77	12/74 - 3/76	12/76 - 1/77	5/75 - 11/75	11/76 - 3/77	3/75 - 3/76 4/84 - 3/85	6/77 - 1/78
Duration of Measurements	6 mos	4 mos (highs only)	7 mos	10 mos (highs only)	16 mos (highs only)	2 mos	5 mos	5 mos	13 mos + 12 mos	4 mos (highs only)
100-YR	11.5	12	11.5	11.5	11.9	12.3	12.4	12.4	12.5	12.3
HOWL	10.2	-	--	-	10.3	11	11	10.7	10.8	-
MHHW	8.5	-	7.6	-	8.6	9.3	9.3	8.5	9	-
MHW	7.9	-	7	-	8	8.7	8.7	7.9	8.4	-
MTL	4.6	-	3.9	-	4.6	5	5	4.4	4.8	-
MLW	1.2	-	0.8	-	1.1	1.3	1.2	0.8	1.2	-
MLLW	0	-	0	-	0	0	0	0	0	-
LOWL	-	-	-	-	-0.7	-1.7	-1.2	-1	-1.8	-

Blank values indicate that specific tidal plane not computed

Elevations shown in Table 3.3.1 are based on MLLW datum which is not constant over time. It changes for every tidal epoch, approximately 19 years in length, as determined by NOAA. The reasons for the changes are many, but include natural climatic changes, global warming, isostatic changes in the earth's crust, local subsidence due to self-weight consolidation of soft sediments and or groundwater withdrawal, and tectonic causes, to name a few.

3.3.2 Physical Setting

The existing topography in the vicinity of the urban levee was determined by reviewing available documents, site visits, and field survey. The results are described below starting from the western limit at Pond A1, proceeding to the eastern limit at New Chicago Marsh (south of Pond A16). Ponds A22 and A23, which are physically isolated from the other ponds were also surveyed. Typical elevations based on the survey points, referenced to NGVD, are shown on figures. Pond elevations, for reference, are provided in Appendix C.

Pond A1

Figure 3.3.2a shows the pond, the surrounding area, the existing Bay Trail, and the urban levee reference line. Pond A1 is bounded to the north by the Bay, to the west by Charleston Slough and to the east by Mountain View Slough; further to the west is the large Palo Alto Floodbasin, which collects flow from Matadero, Barron, and Adobe Creeks prior to discharging into the Bay.

The far west portion of the pond's southern border abuts the Coast Casey Forebay, which serves as a flood control basin for Mountain View. This portion of the pond levee is relatively low, with elevations ranging from 8' to 9'. As part of the Bay Trail, the levee is paved with asphalt concrete and seems to be in good condition. The pond-side slope is relatively flat, vegetated, and appears to be in good condition.

Proceeding east, the trail rises to elevations of 15' to 17' as it borders the high ground of the Shoreline at Mountain View, which is built on a former landfill that is actively monitored. A wide, lower bench near the water line continues along the pond. Figure 3.3.2b shows a cross-section through this portion of the levee. The slope from the bench to the waterline is mild and vegetated with grasses.

Pond A2W

Figure 3.3.2c shows the pond and adjacent area, the existing Bay Trail, and the urban levee reference line. Pond A2W is bounded to the north by the Bay, to the west by Mountain View Slough, and to the east by Whisman Slough (Stevens Creek). To the south is the eastern half of the Shoreline at Mountain View. This southern border can be divided into three distinct parts depending upon the adjacent, landside features as described below.

The western portion of the pond's south border abuts the Mountain View Tidal Marsh, which is tidally connected to Mountain View Slough via several breaches through a low levee. The levee in this portion is surfaced with aggregate and is not part of the Bay Trail. The elevation of the levee crest gradually slopes up from west to east until it connects with the Bay Trail. A wide lower bench follows the levee crest near the waterline, with typical elevations ranging from 1' to 4'.

The main central portion of the levee is adjacent to high ground (former landfill) and is part of the paved Bay Trail. A wide lower bench continues, parallel to the trail, from the western portion. Figure 3.3.2d shows a cross-section through this portion of the levee. Note that the lower bench elevations range from 2' to 3', and that the levee crest (Bay Trail) is at an approximate elevation of 15'. The short slope from the lower bench to the waterline is highly eroded and nearly vertical.

The eastern portion borders the Stevens Creek Tidal Marsh. Passing through the marsh are two PG&E lines. The lower bench continues at elevations similar to the other portions, and also becomes an unpaved, unofficial part of the Bay Trail. The paved road continues south between the marsh and the landfill but is not a part of the Bay Trail.

Pond A2E/B2

Figure 3.3.2e shows the surrounding area and the urban levee reference line. Pond A2E is bounded to the north by Pond B1, to the west by Whisman Slough, and to the east by Pond B2. To the north of Pond B1 is the Bay. Pond B2 abuts a very short stretch of the urban levee, with the Bay to the north and Pond A3W to the east. Figure 3.3.2f shows a cross-section through this portion of the levee.

Generally, the area south of these ponds consists of the NASA/Ames Research Center in the west and Moffett Field in the east. The far western area is the Stevens Creek Shoreline Nature Study Area, operated by the Mid-Peninsula Regional Open Space District. The western portion of NASA's property contains two large ponded areas. One of them (east of the Nature Study Area) is a large stormwater retention pond. The other pond juts north into Pond A2E, to the north of Moffett Field, and is separated from the retention pond by a low, varying width berm that terminates just north of Moffett Field.

Pond A3W

Figure 3.3.2g shows the area, the existing Bay Trail, and the urban levee reference line. Pond A3W is bounded to the north by the Bay, to the west by Pond B2, and to the east by three different water bodies: 1) Guadalupe Slough; 2) Sunnyvale WPCP oxidation pond; and 3) a small, unnamed salt pond (not part of the purchased salt ponds). Figure 3.3.2h shows a cross-section through this portion of the levee.

South of the pond is Moffett Drain (or Northern Channel), a small ditch draining parts of Moffett Field. Along this ditch are posted numerous signs warning of the potential hazardous materials it may contain. South of the ditch is the Moffett Field Golf Course.

Oxidation Ponds

Figure 3.3.2i shows the area, the existing Bay Trail, and the urban levee reference line. The Oxidation Ponds, which are part of the Sunnyvale WPCP's treatment process, are bounded to the north by Guadalupe Slough, to the west by Pond A3W and the unnamed salt pond, and to the east by Moffett Channel. The unnamed salt pond borders the entire southern limit of the oxidation pond, and is presently not a part of the restoration project; it is owned by Cargill.

Pond A4

Figure 3.3.2j shows the area, the existing Bay Trail, and the urban levee reference line. Pond A4 is bounded to the northeast by Guadalupe Slough and to the northwest by Moffett Channel. South of the pond is the Sunnyvale WPCP, the SMaRT recycling center, and the Twin Creeks Sports Complex. Sunnyvale West Channel links to Moffett Channel and Sunnyvale East Channel borders the southeast corner of Pond A4. Also, there are small ditches separating Pond A4 from the Sunnyvale Water Pollution Control Plant and the SMaRT Center. Sunnyvale East Channel separates Pond A4 from the park. Figure 3.3.2k shows a cross-section through this portion of the levee.

The pond is owned by SCVWD; plans are underway for restoration. Although not a part of the South Bay Salt Pond Restoration Project, the restoration of Pond A4 will have an effect, and will be affected by, the larger project.

The levee south of the pond ranges from approximately 6.5' to 10.5' NGVD. The levee also functions as the Bay Trail along the entire length of the pond, crossing a steel bridge over Sunnyvale East Channel.

Pond A8

Figure 3.3.2l shows the area, the existing Bay Trail, and the urban levee reference line. Pond A8 is bounded to the north and east by Alviso Slough, and to the west by Ponds A7 and A5. The far southwest and south is bordered by Guadalupe Slough. South of the slough is the eastern portion of the Sunnyvale Baylands Park, a bird preserve, and Harvey Marsh.

The elevation of the levee protecting the park from high water in the slough ranges from 10' to 11' NGVD. The levee also is part of the Bay Trail along the park, turning south to follow Calabazas Creek. Several interior levees cross east-west along the southern portion of the pond, some of which support a least tern nesting area.

New Chicago Marsh, Pond A12, Pond A13, Pond A16

Figures 3.3.2m and 3.3.2n shows the area, the existing Bay Trail, and the urban levee reference line. New Chicago Marsh is bounded to the north by Pond A16, to the east by Artesian Slough, to the northwest by Pond A13, to the west by Pond A12, and to the south by the town of Alviso. Along the western edge of the marsh is the SPRR railroad line. The southwest corner of the marsh abuts the Alviso Marina. The plans are in various stages of development, pending funding and status of implementation of the Lower Guadalupe FCP. Figure 3.3.2o shows cross-sections fronting the marsh through this portion of the levee.

The marsh has subsided significantly over time, and would be completely inundated if subjected to tidal water. The levee protecting the town of Alviso varies in elevation from approximately 0' to almost 10.0' NGVD, indicating that flood protection is largely provided by the salt pond levees and the Bay Trail levee.

Pond A22

Figure 3.3.2p shows the area and the urban levee reference line. The pond is bounded to the northeast by Cushing Parkway and to the northwest by Refuge lands, to the west by the SPRR railroad, to the east by Mud Slough and to the south by Pond A23. Figure 3.3.2q shows a cross-section through the levee between the pond and Refuge lands. Since urban development is to the north, only this portion of the pond levee was surveyed.

The pond was dry during the site visit, and borrow ditches were observed 15 to 20 feet from the pondside toe of levee.

Pond A23

Since no urban communities abut against the pond levees, this pond was not surveyed. It should be noted however, that the SPRR railroad runs along the west side of the pond and may prove to be a constraint to full tidal restoration.

3.4 West Bay Complex

3.4.1 Existing Flooding

The current level of flooding can be reasonably ascertained by examination of the FIRMs in the project area. Generally speaking, the salt ponds are currently in the SFHA throughout the project. The 100-year floodplain is shown on Figure 3.4.1a. Areas immediately landward of the salt ponds are typically in SFHA or in Zone X.

Available tidal benchmark data for tide stations near the West Bay Complex are presented in Table 3.4.1.

Table 3.4.1 : Tidal Benchmark Data For West Bay Complex
(elevations in feet, MLLW)

Tidal Plane	941 4501 Redwood Creek	941 4507 Westpoint Slough	941 4509 Dumbarton Bridge	941 4525 Palo Alto Yacht Harbor	941 4537 Palo Alto CM No. 8
100-YR	11.00	11.2	11.6	11.5	11.5
HOWL	9.63	--	10.2	--	--
MHHW	7.96	8.0	8.5	7.6	--
MHW	7.35	7.4	7.9	7.0	--
MTL	4.27	4.3	4.5	3.9	--
NGVD	3.91	--	4.1	--	--
MLW	1.19	1.2	1.2	0.8	--
MLLW	0.00	0.0	0.0	0.0	--
LOWL	-2.10	--	-2.2	-2.1	--

3.4.2 Physical Setting

The results of the field survey are shown in Figures 3.4.2a to 3.4.2e. Note that the aerial photographs are approximately 12 years old and do not show the Sun Microsystems campus between Pond 3 and Highway 84.

Pond S5

Figure 3.4.2a shows the pond, the surrounding area, and the reference line. The pond is bounded by a drainage ditch (Bayfront Canal) and Highway 84 to the south, other ponds to the north and east, and County property to the west where a pump structure exists. Figure 3.4.2b shows a cross-section through this portion of the levee. The pond was dry during the site visit and crystallized salt was observed on the surface.

Pond 3

Figure 3.4.2c shows the pond, the surrounding area, and the reference line. The pond is bounded by Highway 84 to the south, Ravenswood Slough to the northeast, other ponds (ponds 4 and S5) to the northwest and west. Figure 3.4.2d shows a cross-section through the portion of levee fronting Highway 84. The pond was dry in many areas during the site visit, and a significant amount of crystallization of salt was observed on the surface.

Pond SF2

Figure 3.4.2e shows the pond, the surrounding area, and the reference line. Urban development adjacent to the pond include Highway 84 to the north and west and the SPRR railroad to the south.

3.5 BAUMBERG COMPLEX

3.5.1 Existing Flooding

The primary watershed in the Baumberg complex is the Alameda Creek watershed, with a total area of about 695 square miles. The watershed discharges into San Francisco Bay via the Alameda Creek Flood Control Channel.

Previous reports (URS 2002) state that the Alameda Creek Flood Control channel was designed to protect surrounding lands from the standard project flood, which is the flood with a return period of approximately 500 years. Due to siltation, the channel's capacity has lessened, but still provides significant protection from the 100-year event. During this event, the southern levees are above the water surface but the northern levees are overtopped in the downstream reaches (below Ardenwood Blvd.), resulting in moderate flooding of undeveloped areas. Figure 3.5.1 shows the 100-year floodplain, as simulated using hydraulic analysis described in the report.

Available tidal benchmark data for tide stations near the Baumberg Complex are presented in Table 3.5.1.

Table 3.5.1 : Tidal Benchmark Data For Baumberg Complex
(elevations in feet, MLLW)

Tidal Plane	941 4458 San Mateo Bridge (West End)	941 4637 San Mateo Bridge (East End)	941 4621 Coyote Hills Slough	941 4632 Alameda Creek
Period Of Measurement	1/81 – 1/88	1/77 – 3/77	12/76 – 3/77	12/76 – 3/77
Duration of Measurements	7 yrs	3 mos	4 mos	3 mos
100-year Estimated Tide (USACE)	10.7	10.7	--	9.1
Highest Observed Water Level [1]	10.7	9.2	8.3	7.6
Mean Higher High Water (MHHW)	7.7	7.7	6.7	6.1
Mean High Water (MHW)	7.1	7.1	6.1	5.5
Mean Tide Level (MTL)	4.1	4.1	3.3	2.9
Mean Sea Level	4.1	-	-	-
National Geodetic Vertical Datum, 1929 (NGVD) [2]	3.6	3.7	-	-
Mean Low Water (MLW)	1.2	1.2	0.5	0.3
Mean Lower Low Water (MLLW)	0.0	0.0	0.0	0.0
Lowest Observed Water Level	-2.9	-1.8	-0.3	-0.3

Blank values indicate that specific tidal plane not computed

[1] Extreme levels during the period of measurement

[2] Elevation of NGVD is approximate, based on data from NOS and USACE (1984)

3.5.2 Physical Setting

The existing topography in the area of the proposed urban levee is relatively flat, with three significant waterways passing through – the Alameda Creek Flood Control Channel (Coyote Hills Slough), the old Alameda Creek, and Mount Eden Creek. Figure 3.5.2a shows a layout of the study area investigated by URS for the ACFCWCD, including a station line which can be used to determine cross section locations. Figure 3.5.2b shows cross sections through the Flood Control Channel levees. The proposed urban levee essentially follows the reference line previously determined by the Alameda Creek Levee Reconfiguration Project (ACLRP) with minor alterations.

Since a detailed flood control investigation is already being conducted by the ACFCWCD, including field and LiDAR surveys, the visual reconnaissance did not extend into this area. The URS report also includes a detailed photo log of conditions in the area.

4 PROPOSED LEVEE IMPROVEMENTS

4.1 DESIGN ASSUMPTIONS

As previously stated, the reference line of the proposed urban levee was developed by reviewing previous alignments and adjusting them as needed to fit field conditions; thus the reference line follows existing levees and/or trails along the salt pond. It should also be noted that the urban levee ends at intersections with waterways and does not continue upstream along riverbanks; as such, the urban levee functions as a coastal levee as opposed to a riverine levee.

The geometry (height, width, slope) of the proposed urban levee depends on WSEL, extent of wind/wave action, geotechnical stability, construction method, and need for public access along the crest among others. For purposes of this report, several assumptions were made:

- Exposure to wind and waves will depend on the restoration alternative selected for each pond, which is not known at this stage. Therefore two levee sections are assumed for this analysis, one with fetch breaks (limiting wind waves to < 1 foot) and one without fetch breaks. The crest elevations were estimated to be +12 feet (with fetch break) and +14.5 (no fetch break). These elevations are consistent with estimates developed as part of the SFO Airport hydrodynamic studies (ADEC 2002) for tidal marsh restoration at the salt ponds. The SFO studies provide details on the methodology used to estimate wave run-up and wave overtopping.
- The assumed construction method is excavating material from the pond interior using draglines or similar equipment, and placing on the existing levees at the required slopes. Since the material will have a high moisture content, the construction is expected to be slow, and will keep pace with material conditioning and compaction. The large quantity of required material makes importing borrow fill cost-prohibitive, but may be necessary in areas where adequate volume and quality of borrow material is not available.
- A levee crest width of 20-feet and slopes of 3H:1V and 8H:1V were assumed in the analysis. The wide crest will allow placement of additional material if excessive consolidation and/or settlement occurs. The relatively flat side slopes will provide a wide footprint for stability. The cases analyzed were as described in the following.

Crest Elevation	**Side slope**
(ft, NGVD)	(Horizontal : Vertical)
+12	3 : 1
+12	8 : 1
+14.5	8 : 1
+15.5	3 : 1

The Baumberg complex, specifically the Alameda Creek Flood Control Channel, has been studied in more detail by ACFCWCD. A recent report[3] provides a thorough analysis of flood protection requirements for this major waterway which will be utilized in this section.

[3] URS, Alameda Creek Levee Reconfiguration Project, August 2002.

4.2 ANTICIPATED POST-PROJECT HYDRODYNAMICS

Changes in the hydraulic parameters of significance – water levels, velocity, and circulation – will result in changes in conveyance capacity of a slough and a corresponding change in the level of existing flood protection.

Changes in tidal water levels in any of the creeks will be highly dependent on changes in tidal prism in the creek itself. A substantial increase in tidal prism in a fully tidal channel will usually result in a short-term (on the order of months) lowering of high water level, a change in the time of high water level, and increase in channel velocity. The lowering of high water, or “tidal muting”, is explained by the fact that the tidal period (time between successive high or low tides) does not change, and the increase in inundated area and water volume due to pond breaching is compensated by a reduction in tidal range over the entire inundated area.

The data reviewed indicates that at the present time, tidal flooding is not a significant issue on the sloughs flowing adjacent to the salt ponds. However, flooding concerns arise at times of high stormwater flows combined with extreme high tides. If the elevation of high tide near the mouths of the sloughs/creeks do not change, then there will be no apparent change in the level of flood protection that the levees presently offer. However, if there are changes in the level of high tide upstream of the mouth, then the level of flood protection will change.

It is quite likely that breaches through the existing Bayfront pond levees may not result in any increase in high tide elevation upstream of the creek mouth. Instead, there may actually be benefits to flood conveyance since the restored ponds would allow routing of storm flows over existing slough levees (by constructing sills or overflow weirs).

Unfortunately, not all ponds have Bayfront levees, and restoration of some of the interior ponds may involve breaching through the slough levees. Breaches through the existing slough levees may result in changes in levels and time of high water, depending on the size of the existing slough. Sloughs with large cross sectional areas may not result in differences in high tide elevation between the mouth of the sloughs and upstream near the breach. However, some of the smaller sloughs where the high tide is muted due to shallow depths may see an increase in high tide elevation, resulting in flooding concerns farther upstream (due to higher backwater).

4.3 GEOTECHNICAL ASSESSMENT

4.3.1 Assumed Soil Properties for Settlement and Slope Stability Analyses

For 20 representative sections along the levees, preliminary estimates of settlement and the immediately-after-construction factor of safety for slope stability were made. For both analyses, HTE used "typical" Bay Mud properties.

To estimate the magnitude of settlement, a compression ration (C_{EC}) of 0.30 was used for the highly compressible Bay Mud. For the screening-level settlement analysis, an average saturated unit weight of 94 pounds per cubic foot was used for the Bay Mud and 120 pounds per cubic foot was used for the moist unit weight of new and existing fills. The range of average unit weights reflect in part that the Bay Mud is less consolidated (less dense) and has a higher moisture content than the fills. In areas where the center of the new levee will be over or nearly over the original marsh plain, the upper four feet of Bay Mud was assumed to be slightly over-consolidated to a depth of four feet due to desiccation and/or previously lowered groundwater levels. The Bay Mud was assumed to be normally consolidated below four feet in these areas. Where the center of the new levee will overlie existing levee fill, the underlying Bay Mud was assumed to be normally consolidated.

To assess the strength of the Bay Mud foundation materials, HTE reviewed available data. From this review, coupled with past experience on Bay Mud sites, HTE concluded that the undrained shear strength of the Bay Mud beneath the ponds could be characterized as between 200 pounds per square feet (psf) and 400 psf. HTE judges this range to represent the condition of normally consolidated clay and that of slightly over consolidated clay. Both conditions are reasonably to be expected along the length of the salt pond perimeter levee. HTE chose to assess the feasibility-level slope stability of future levee embankments using both of these average shear strength values.

4.3.2 Assumed Levee Profiles For Settlement and Slope Stability Analyses

Four levee design profiles were applied to 20 typical sections for estimating settlement and the immediately-after-construction factor of safety. The factors that differentiated the design profiles were the inclination of the waterside slope and the design crest elevation (DCE). The crest width of at least 20 feet was assumed. The following crest elevations and waterside slopes were used for the four design profiles that were analyzed:

- Design Crest Elevation = +12 ft with a 3:1 waterside slope
- Design Crest Elevation = +12 ft with an 8:1 waterside slope
- Design Crest Elevation = +14.5 ft with an 8:1 waterside slope
- Design Crest Elevation = +15.5 ft with a 3:1 waterside slope

The first two design profiles were used for the condition of having an intermediate fetch break that limits wave run-up, and the last two profiles assumed that there was an unrestricted fetch into the Bay. The flatter slope was used for reduced wave run-up.

4.3.3 Settlement and Slope Stability Analysis

To assess the ultimate thickness of fill needed to maintain the design crest elevations for the levees, HTE modeled consolidation settlement by placing fill in stages, allowing time between the stages for the underlying Bay Mud to consolidate. At or near the end of consolidation under a fill load, additional fill was placed and further consolidation allowed to occur. This process was repeated until the crest elevation was achieved without the crest settling below the design elevation. Only primary consolidation of Bay Mud was considered for this screening-level assessment of settlement. Additional settlement may occur due to secondary compression and shear deformation in the Bay Mud, compaction of the fills and regional subsidence. Settlement estimates for various crest elevations and thicknesses of Bay Mud deposits are presented on Figures 4.3.3a through 4.3.3m for the various cross-sections.

For soft clay sites such as Bay Mud, the controlling stability condition is that which occurs during-construction or immediately-after-construction of the levee embankment. If the levee fill were to be placed over a long period of time (years), the underlying clay would have time to drain and gain strength. For the long-term (drained) condition, slope stability is seldom a concern. For final slope configurations of 3:1 or flatter, HTE concludes that long-term stability for the urban levees will not be a concern. The critical case occurs when the load is first placed. At that time, no drainage has occurred and, as a direct result, no strength gain has occurred. Most of the new load is taken up by increases in pore pressure within the soil as the new loads are placed. Strength gain occurs only as the excess pore water pressure drains, allowing the soil particles to press together increasing the strength. For this reason, the controlling condition for clay sites occurs either as the loads are placed or immediately after they have been placed.

For this feasibility study, HTE assessed the factor of safety of the levee embankments assuming that they were built instantaneously and that they were initially constructed to between 1 to 1.5 feet above their design crest elevations. This additional fill thickness allows for a combination of overbuild to account for settlement as well as a simplified way to model for a somewhat greater mass of fill that will occur due to immediate compression and shear deformation in the foundation materials.

The purpose of the stability analysis presented herein is to provide an initial screening assessment to demonstrate the impact that existing levee and pond elevations have on the typical levee heights and slopes. The primary focus was the stability assessment on the bay side of the levee where the greatest quantities of fill will be placed. In some areas, the existing levees are quite low and new fill thicknesses will create substantial loading on the upslope side of the levee. The stability in the upslope direction was not addressed during the stability analyses for this feasibility study. Further stability assessments will be needed during levee siting and development of the levee configurations. The geotechnical engineers will need to assess the factors of safety during various stages of filling for areas where the immediately-after-construction factor of safety is less than the design value selected by the owners (commonly a factor of safety of 1.3). Specific guidance on stability is provided by the USACE (USACE 1970, USACE 2000).

As part of the stability screening procedure, stability charts were prepared for use in assessing the stability of the levees for various heights of fill embankment and various thicknesses of Bay Mud deposits. These charts were developed both for 3:1 and 8:1 slopes. Charts were prepared for two Bay Mud shear strengths, 200 psf and 400 psf. These stability analysis charts are presented on Figures 4.3.3n and 4.3.3o. In preparing these charts, an assumed average unit weight of 115 pcf applied to both the new fill as well as the underlying Bay Mud. These charts were derived using the stability design chart titled “Stability Analysis for Slopes and Cohesive Soils Undrained Conditions” published in the Naval Facilities Design Manual DM7, page 7.1-319. Using these stability analysis charts, the short term / immediately-after-construction stability for the new levee embankments was estimated. For those embankments whose design crest elevations were at Elevation14.5 and 15.5 feet, it was assumed that the embankments would be constructed 1.5 feet higher than the design crest elevation to allow for the initial aspects of settlement. Where the embankment at design crest elevation is 12 feet, it was assumed that one additional foot of fill would be placed to elevation 13. The resulting computed factors of safety for these thicknesses of embankment fill are presented for various thicknesses of Bay Mud on Figures 4.3.3a through 4.3.3m

The stability charts were prepared for "thicknesses" of Bay Mud. Previous sections of this report have described the collection of existing data and reported that the elevation of the base of the mud was typically between Elevation –5 and –15 feet NGVD. The elevation within the ponds is typically between - 3 to + 3 feet. Clearly, the thickness of the mud is a function of the elevation of the bottom of the ponds as well as the elevation of the base of the mud. For this feasibility level study, the data available on the base of the mud is not precise beyond the general descriptions that have been used.

4.3.4 Levee Construction Issues

The two major geotechnical factors affecting the construction of new levees are the strength of the Bay Mud and its compressibility. The low strength will limit the height at which the levees can be initially constructed in some areas. For much of the salt marsh perimeter, the levees may need to be constructed in two stages. The time between filling stages will allow the underlying clays to consolidate and gain strength. The levees will continue to settle after they are constructed to their designed crest elevations. The levee crests will need to be designed with sufficient width to accommodate placing additional fill to maintain the levee crest design elevations.

In addition to settlement of the Bay Mud, there is an on-going longer term consolidation occurring in the deeper sediments in the Santa Clara Valley basin. This deep consolidation is the result of groundwater withdrawal. Though the extraction of groundwater is now well managed and the rates of settlement have decreased substantially, some on-going regional settlement is still occurring. Extrapolating the rate of regional consolidation over the last twenty to thirty years suggests that an additional foot of settlement can be expect over a period of 30 years along portions of the urban levee. In addition, sea level rise, and sedimentation within the restored ponds adjacent to the levee will occur. These factors would be addressed by raising the levee periodically.

4.3.5 Geotechnical Discussion and Conclusions

The discussion presented below is based on limited analysis (see Section 3.2.2), which is believed to be sufficient at this stage in the planning process of the project. Ultimate designers will need to consider such items as differential settlement, seepage, liquefaction, deformation, stability, etc.

The most important factor affecting stability of the embankments in this locale is the strength of the foundation soils. The strength of the foundation soils limits the height to which the levee can be built and how fast it can be built. The primary guideline for raising an embankment on Bay Mud is to limit the height of each stage of construction so that an adequate factor of safety against stability failure for that stage of construction is maintained. For thick fills it may be necessary to have several stages of construction, allowing the mud to consolidate and gain strength between the stages. For the South Bay levees, many of the levees will be of sufficient height that they will have to be constructed in two stages, with the second stage bringing the levee to its design crest elevation. One of the methods that can be used to improve stability is to place wide stability berms at the levee toe. These berms offset the tendency of the toe to heave, creating a buttress or weight at the levee toe, increasing its overall stability. Using flat slopes can also improve the stability of slopes. Wick drains may be used to accelerate consolidation. However, the increased risk of underseepage that a drainage blanket would cause must be assessed.

Levees will need to be overbuilt to allow for settlement that will occur after the levees have been constructed. The suggested plan would be to construct them 1 to 1.5 feet above the design crest elevations and to make the crest levees wide enough so that as the levees settle, additional fill can be placed to achieve the final design crest elevation while still having a sufficiently wide final levee crest.

Levees need to control seepage. Seepage can be subdivided into two primary seepage zones: that which occurs through the levee embankment and that which flows through aquifers or other formations beneath the levees. During the site visits, indications of seepage through the levees was not observed. The new urban levees will be much taller and broader and will be well compacted. The risk of adverse seepage through the new levee embankments will be small.

For the deeper seepage, there may be changes with time in the groundwater profile. The project will change the average head of water in the ponds compared to the head when they stored water for salt manufacturing. A closer look will have to be taken to see if the project will be raising or lowering the heads because the ponds elevations vary. With the low permeability of the Bay Mud which underlies the ponds, the changes in groundwater levels below the Bay Mud will likely be small. Further assessment may conclude the affect on groundwater levels may be insignificant relative to other stressors on the hydrogeology.

4.4 DESIGN AND COST ESTIMATE

Based on the design assumptions described in the previous section, a planning-level design and concept level cost estimate was developed for the urban levee. The estimate is presented as a cost for each pond, and includes construction, engineering, surveying, geotechnical studies, environmental studies, construction administration, and contingencies. For simplicity, a single cross-sectional design was developed for each pond to represent the typical features of the pond.

4.4.1 Alviso Complex

The Alviso complex passes through varied topographical features, and thus has greater variation in cross-section than the other complexes. The design cross-section for each pond in the Alviso complex is shown in Figure 4.4.1. Quantities and costs associated with each pond levee are shown on Tables 4.4.1a and 4.4.1b.

4.4.2 West Bay Complex

The typical design cross-section for pond S5 and pond 3 in the West Bay complex is shown in Figure 4.4.2, and Tables 4.4.2a and 4.4.2b show the quantities and costs anticipated for each pond in the West Bay complex.

4.4.3 Baumberg Complex

The Alameda Creek Levee Reconfiguration Project (URS, 2000) provided several options to increase the level of flood protection to its original, design level. The Alameda Creek Flood Control channel (or Coyote Hills Slough) was designed to provide protection from the Standard Project Flood (SPF), which roughly corresponded to a 500-year event. Currently, the channel does not provide this level of protection, and of various options proposed, option D best resembles the Salt Pond Restoration Project features. This option would include multiple levee breaches along the channel, allowing flow into ponds between the channel and Old Alameda Creek to the north. The levee breaches assumed in the URS report would also expose these ponds to tidal influence, with a corresponding increased potential for coastal flooding to urban areas.

It is important to note that the levees in the ACLRP are designed to meet criteria that differs from the Alviso and West Bay complexes. First, the levees must contain fluvial flood waters, as opposed to coastal flood waters. The water surface profile of the channel in the easternmost region of the complex is much higher than at the mouth of the channel, resulting in levee crest elevations that are several feet higher than would be required for coastal flooding only. Secondly, the SPF is a more severe flood event than the 100-year event used in the other complexes.

The continuous urban levee reference line in the Baumberg complex was assumed to follow the alignment proposed in the ACLRP. Additionally, the ACLRP provided an estimated quantity of material needed to construct the urban levee. Thus, in lieu of performing new studies and analyses, and given the advanced nature of the previous investigation, the information provided in the ACLRP will be utilized in this section.

The summary of quantities and cost estimate for the Baumberg complex is shown in Table 4.4.3

4.5 ITEMS FOR FURTHER STUDY

Much of the information presented in this report is based on assumptions which warrant further investigation. Some of these issues are:

- Alignment – Along several ponds there are possible alternative alignments of the urban levee, some of which should be seriously considered. Pond A2E currently shows a levee protecting a large private pond which may not need protection. Pond A3W shows a levee protecting a small ditch (Moffett Drain) with a levee to the landside of the ditch; this alignment may be revised landward by filling in the ditch, if desired. New Chicago Marsh shows a levee along the Marsh border with the town of Alviso; this may result in flooding of the Marsh and a loss of significant marsh habitat. A levee following either the SPRR tracks, or the boundaries of Ponds A12-A13-A16 may be an alternate alignment to protect the marsh from inundation. These are a sampling of the possibilities for further study.
- Coordination with related projects such as Pond A4, Pond A16, Lower Guadalupe River Flood Control Project. These projects will have some effect on the hydraulics of the project vicinity. Whether these related projects improve or degrade the overall system is a matter for additional study.
- Flood insurance considerations – the benefit of providing levees which meet FEMA certification requirements is to remove communities from areas with flood insurance requirements. However, there is no requirement on the part of the levee owner to meet FEMA requirements, rather only to provide the same protection currently provided.
- Loose and medium dense sands are present below the water table at the site. These soils are subject to liquefaction during a large magnitude earthquake in the vicinity. HTE's preliminary judgement is that broad, well-compacted levee embankments with properly maintained freeboard and with moderately flat side slopes can tolerate considerable liquefaction induced deformation without significant risk of overtopping or breaching. A more thorough evaluation of seismic risk and deformation will need to be addressed during subsequent design phases.

Table 4.4.1a - Estimated Quantities for Levee Construction

Alviso Complex

Pond	Begin Station	End Station	Length	Levee Crest Elevation (ft, NGVD)	Slope	Total Volume (CY)
A1	0	600	600	12.0	8:1	19,000
				14.5	3:1	13,000
				14.5	8:1	29,000
A1	600	1300	700	12.0	8:1	18,000
				14.5	3:1	9,000
				14.5	8:1	32,000
A1	1300	4300	3000	12.0	8:1	127,000
				14.5	3:1	91,000
				14.5	8:1	185,000
A2W	0	3750	3750	12.0	8:1	190,000
				14.5	3:1	139,000
				14.5	8:1	264,000
A2W	3750	4800	1050	12.0	8:1	79,000
				14.5	3:1	50,000
				14.5	8:1	105,000
A2E	0	1700	1700	12.0	8:1	67,000
				14.5	3:1	50,000
				14.5	8:1	99,000
A2E	1700	4100	2400	12.0	8:1	152,000
				14.5	3:1	106,000
				14.5	8:1	205,000
A2E	4100	6800	2700	12.0	8:1	164,000
				14.5	3:1	117,000
				14.5	8:1	222,000
A3W	6800	9400	2600	12.0	8:1	146,000
				14.5	3:1	96,000
				14.5	8:1	203,000
A3W	9400	11400	2000	12.0	8:1	117,000
				14.5	3:1	84,000
				14.5	8:1	159,000
A22	0	12000	12000	12.0	8:1	218,000
				14.5	3:1	214,000
				14.5	8:1	369,000

Table 4.4.1b - Concept Level Costs for Levee Construction
Alviso Complex

	LENGTH	Levee Crest Elevation 12' 8:1 Slope				Levee Crest Elevation 14.5' 3:1 Slope				Levee Crest Elevation 14.5' 8:1 Slope			
	(ft)	QTY	UNIT	UNIT COST	COST	QTY	UNIT	UNIT COST	COST	QTY	UNIT	UNIT COST	COST
Alviso Complex													
Pond A1	4300												
Mob / Demob		1	LS	$120,000	$120,000	1	LS	$130,000	$130,000	1	LS	$180,000	$180,000
Earthwork		164,000	CY	$10	$1,640,000	113,000	CY	$15	$1,695,000	246,000	CY	$10	$2,460,000
Clear & Grub		5.0	AC	$5,000	$25,000	5.0	AC	$5,000	$25,000	5.0	AC	$5,000	$25,000
Engineering (15%)		1	LS	$250,000	$250,000	1	LS	$258,000	$258,000	1	LS	$373,000	$373,000
Contract Administration (8%)		1	LS	$133,000	$133,000	1	LS	$138,000	$138,000	1	LS	$199,000	$199,000
Environmental / Permits (5%)		1	LS	$83,000	$83,000	1	LS	$86,000	$86,000	1	LS	$124,000	$124,000
Contingency (20%)		1	LS	$333,000	$333,000	1	LS	$344,000	$344,000	1	LS	$497,000	$497,000
Total Pond A1					$2,584,000				$2,676,000				$3,858,000
Pond A2W	4800												
Mob / Demob		1	LS	$200,000	$200,000	1	LS	$210,000	$210,000	1	LS	$280,000	$280,000
Earthwork		268,000	CY	$10	$2,680,000	188,000	CY	$15	$2,820,000	369,000	CY	$10	$3,690,000
Clear & Grub		6.0	AC	$5,000	$30,000	6.0	AC	$5,000	$30,000	6.0	AC	$5,000	$30,000
Engineering (15%)		1	LS	$407,000	$407,000	1	LS	$428,000	$428,000	1	LS	$558,000	$558,000
Contract Administration (8%)		1	LS	$217,000	$217,000	1	LS	$228,000	$228,000	1	LS	$298,000	$298,000
Environmental / Permits (5%)		1	LS	$136,000	$136,000	1	LS	$143,000	$143,000	1	LS	$186,000	$186,000
Contingency (20%)		1	LS	$542,000	$542,000	1	LS	$570,000	$570,000	1	LS	$744,000	$744,000
Total Pond A2W					$4,212,000				$4,429,000				$5,786,000
Pond A2E	6800												
Mob / Demob		1	LS	$290,000	$290,000	1	LS	$300,000	$300,000	1	LS	$390,000	$390,000
Earthwork		384,000	CY	$10	$3,840,000	272,000	CY	$15	$4,080,000	525,000	CY	$10	$5,250,000
Clear & Grub		8.0	AC	$5,000	$40,000	8.0	AC	$5,000	$40,000	8.0	AC	$5,000	$40,000
Engineering (15%)		1	LS	$582,000	$582,000	1	LS	$618,000	$618,000	1	LS	$794,000	$794,000
Contract Administration (8%)		1	LS	$310,000	$310,000	1	LS	$330,000	$330,000	1	LS	$423,000	$423,000
Environmental / Permits (5%)		1	LS	$194,000	$194,000	1	LS	$206,000	$206,000	1	LS	$265,000	$265,000
Contingency (20%)		1	LS	$776,000	$776,000	1	LS	$824,000	$824,000	1	LS	$1,058,000	$1,058,000
Total Pond A2E					$6,032,000				$6,398,000				$8,220,000
Pond A3W	4600												
Mob / Demob		1	LS	$200,000	$200,000	1	LS	$200,000	$200,000	1	LS	$270,000	$270,000
Earthwork		263,000	CY	$10	$2,630,000	180,000	CY	$15	$2,700,000	361,000	CY	$10	$3,610,000
Clear & Grub		5.0	AC	$5,000	$25,000	5.0	AC	$5,000	$25,000	5.0	AC	$5,000	$25,000
Engineering (15%)		1	LS	$398,000	$398,000	1	LS	$409,000	$409,000	1	LS	$545,000	$545,000
Contract Administration (8%)		1	LS	$212,000	$212,000	1	LS	$218,000	$218,000	1	LS	$291,000	$291,000
Environmental / Permits (5%)		1	LS	$133,000	$133,000	1	LS	$136,000	$136,000	1	LS	$182,000	$182,000
Contingency (20%)		1	LS	$531,000	$531,000	1	LS	$545,000	$545,000	1	LS	$727,000	$727,000
Total Pond A3W					$4,129,000				$4,233,000				$5,650,000
Pond A22	12000												
Mob / Demob		1	LS	$170,000	$170,000	1	LS	$240,000	$240,000	1	LS	$280,000	$280,000
Earthwork		218,000	CY	$10	$2,180,000	214,000	CY	$15	$3,210,000	369,000	CY	$10	$3,690,000
Clear & Grub		14.0	AC	$5,000	$70,000	14.0	AC	$5,000	$70,000	14.0	AC	$5,000	$70,000
Engineering (15%)		1	LS	$338,000	$338,000	1	LS	$492,000	$492,000	1	LS	$564,000	$564,000
Contract Administration (8%)		1	LS	$180,000	$180,000	1	LS	$262,000	$262,000	1	LS	$301,000	$301,000
Environmental / Permits (5%)		1	LS	$113,000	$113,000	1	LS	$164,000	$164,000	1	LS	$188,000	$188,000
Contingency (20%)		1	LS	$450,000	$450,000	1	LS	$656,000	$656,000	1	LS	$752,000	$752,000
Total Pond A3W					$3,501,000				$5,094,000				$5,845,000

Table 4.4.2a - Estimated Quantities for Levee Construction

West Bay Complex

Pond	Begin Station	End Station	Length	Levee Crest Elevation (ft, NGVD)	Slope	Total Volume (CY)
WB-S5	0	3000	3000	12.0	8:1	55,000
				14.5	3:1	54,000
				14.5	8:1	92,000
WB-3	3000	7000	4000	12.0	8:1	57,000
				14.5	3:1	52,000
				14.5	8:1	111,000
WB-3	7000	11800	4800	12.0	8:1	81,000
				14.5	3:1	70,000
				14.5	8:1	148,000

Table 4.4.2b - Concept Level Costs for Levee Construction

West Bay Complex

	LENGTH	Levee Crest Elevation 12' 8:1 Slope				Levee Crest Elevation 14.5' 3:1 Slope				Levee Crest Elevation 14.5' 8:1 Slope			
	(ft)	QTY	UNIT	UNIT COST	COST	QTY	UNIT	UNIT COST	COST	QTY	UNIT	UNIT COST	COST
West Bay Complex													
Pond S5	3000												
Mob / Demob		1	LS	$40,000	$40,000	1	LS	$60,000	$60,000	1	LS	$70,000	$70,000
Earthwork		55,000	CY	$10	$550,000	54,000	CY	$15	$810,000	92,000	CY	$10	$920,000
Clear & Grub		3.0	AC	$5,000	$15,000	3.0	AC	$5,000	$15,000	3.0	AC	$5,000	$15,000
Engineering (15%)		1	LS	$85,000	$85,000	1	LS	$124,000	$124,000	1	LS	$140,000	$140,000
Contract Administration (8%)		1	LS	$45,000	$45,000	1	LS	$66,000	$66,000	1	LS	$75,000	$75,000
Environmental / Permits (5%)		1	LS	$28,000	$28,000	1	LS	$41,000	$41,000	1	LS	$47,000	$47,000
Contingency (20%)		1	LS	$113,000	$113,000	1	LS	$165,000	$165,000	1	LS	$187,000	$187,000
Total Pond S5					$876,000				$1,281,000				$1,454,000
Pond 3	8800												
Mob / Demob		1	LS	$110,000	$110,000	1	LS	$140,000	$140,000	1	LS	$200,000	$200,000
Earthwork		138,000	CY	$10	$1,380,000	121,000	CY	$15	$1,815,000	259,000	CY	$10	$2,590,000
Clear & Grub		10.0	AC	$5,000	$50,000	10.0	AC	$5,000	$50,000	10.0	AC	$5,000	$50,000
Engineering (15%)		1	LS	$215,000	$215,000	1	LS	$280,000	$280,000	1	LS	$396,000	$396,000
Contract Administration (8%)		1	LS	$114,000	$114,000	1	LS	$149,000	$149,000	1	LS	$211,000	$211,000
Environmental / Permits (5%)		1	LS	$72,000	$72,000	1	LS	$93,000	$93,000	1	LS	$132,000	$132,000
Contingency (20%)		1	LS	$286,000	$286,000	1	LS	$373,000	$373,000	1	LS	$528,000	$528,000
Total Pond 3					$2,227,000				$2,900,000				$4,107,000

Table 4.4.3a - Estimated Quantities for Levee Construction

Baumberg Complex

Begin Station	End Station	Length	TOTAL OVERBUILT AREA, SF	Slope	Total Volume (CY)
0	38140	38140	varies	2:1	194,000

Table 4.4.3b - Concept Level Costs for Levee Construction

Baumberg Complex

	LENGTH (ft)	QTY	UNIT	UNIT COST	COST
All	38140				
Mob / Demob		1	LS	$190,000	$190,000
Earthwork		194,000	CY	$12	$2,328,000
Clear & Grub		44.0	AC	$5,000	$220,000
Engineering (15%)		1	LS	$382,000	$382,000
Contract Administration (8%)		1	LS	$204,000	$204,000
Environmental / Permits (5%)		1	LS	$127,000	$127,000
Contingency (20%)		1	LS	$510,000	$510,000
Total Pond 3					$3,961,000

5 REFERENCES

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Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Reference Line

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 1.1a

GRAPHIC SCALE

0 2000

1" = 2000'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Reference Line

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 1.1b

GRAPHIC SCALE
0 2000
1" = 2000'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond
Restoration Project

LEVEE EVALUATION

N

Legend

- Existing Bay Trail
- Existing Salt Pond Levee
- Existing Publicly-Maintained Levee
- Reference Line

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 1.1c

GRAPHIC SCALE
0 2000
1" = 2000'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Reference Line

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 1-1d

GRAPHIC SCALE

0 1500

1" = 1500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Reference Line

Aerial Photo Source: USGS Bay Area Regional Database
Date: July 1993

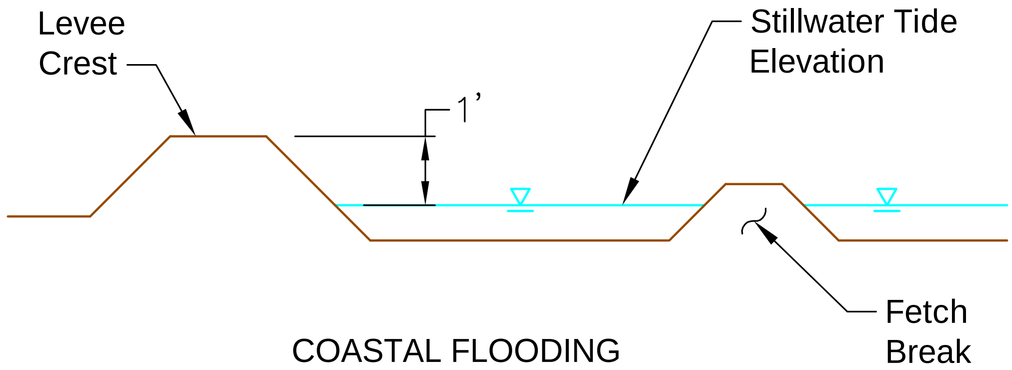
Figure 1.1e

GRAPHIC SCALE

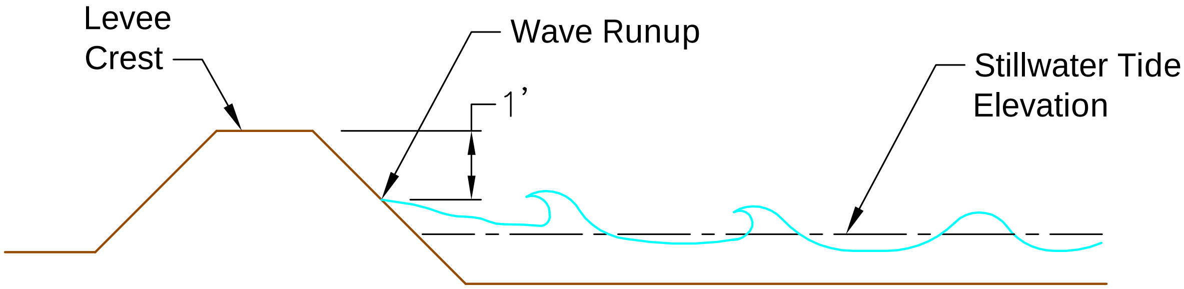
0 2500

1" = 2500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



COASTAL FLOODING
NO WIND/WAVE ACTION



COASTAL FLOODING
WIND/WAVE ACTION

US ARMY CORPS OF ENGINEERS
FREEBOARD REQUIREMENTS

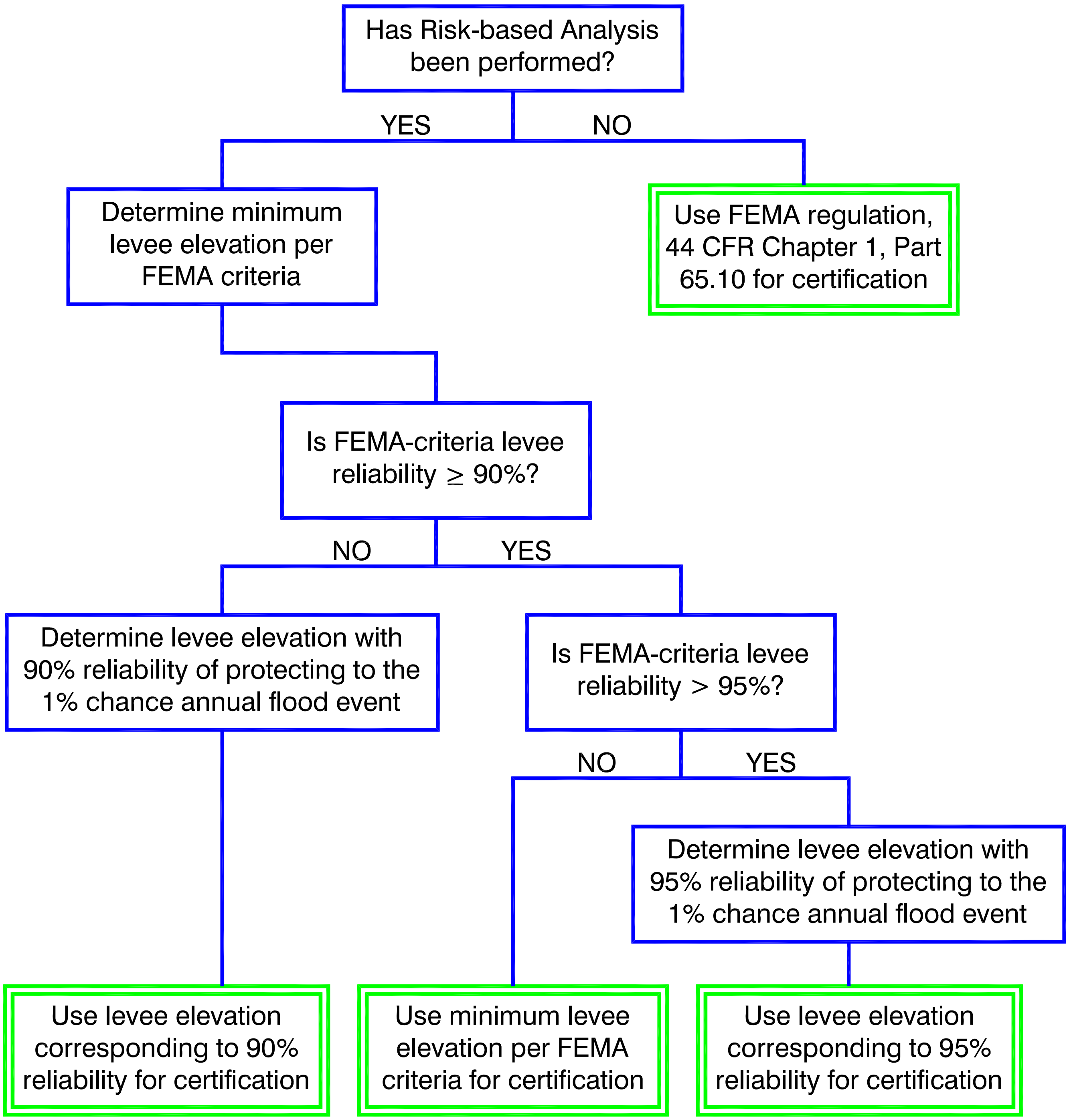
Salt Pond Restoration Project

LEVEE EVALUATION

Figure 2.2.2

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

LEVEE CERTIFICATION DECISION TREE



FEMA Criteria = 1% chance median annual flood event plus three feet of freeboard
Reliability = % chance non-exceedance given the 1% chance annual event occurs

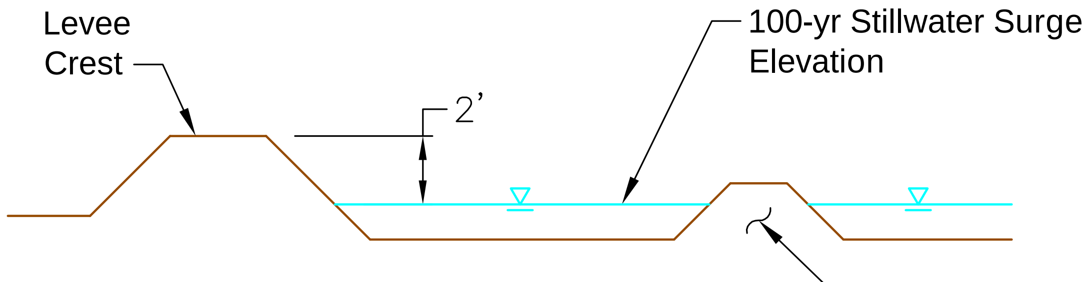
Source: "Risk Analysis and Uncertainty in Flood Damage Reduction Studies"

Salt Pond Restoration Project

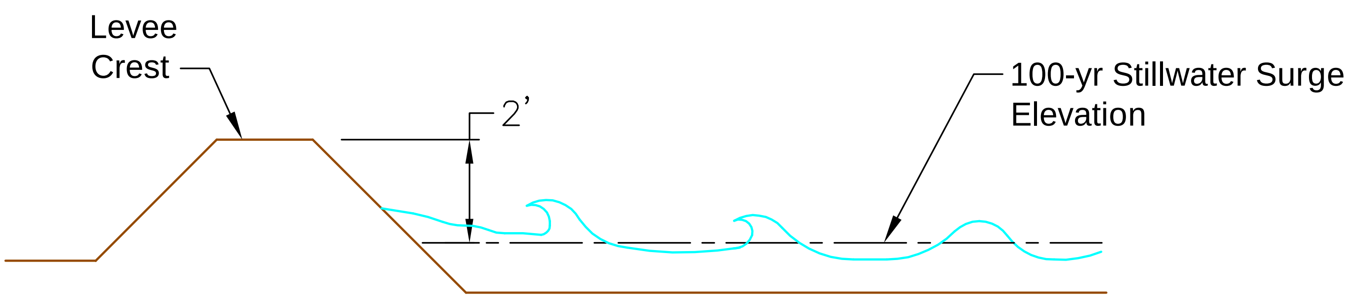
LEVEE EVALUATION

Figure 2.2.4

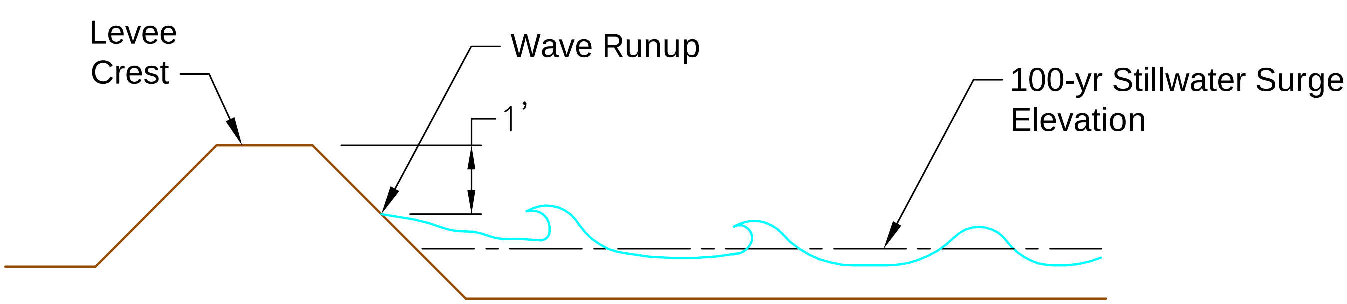
MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



COASTAL FLOODING
NO WIND/WAVE - MINIMUM CONDITIONS



COASTAL FLOODING
WIND/WAVE ACTION - MINIMUM CONDITIONS



COASTAL FLOODING
WIND/WAVE ACTION

<u>FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA)</u>
<u>FREEBOARD REQUIREMENTS</u>

Salt Pond Restoration Project

LEVEE EVALUATION

Figure 2.3.2

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Soil Boring Location

Cone Penetration Test (CPT) Location

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.2.2a

GRAPHIC SCALE
0 2000
1" = 2000'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Soil Boring Location

Cone Penetration Test (CPT) Location

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.2.2b

GRAPHIC SCALE

0 2000

1" = 2000'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond
Restoration Project

LEVEE EVALUATION

N

Legend

Soil Boring Location

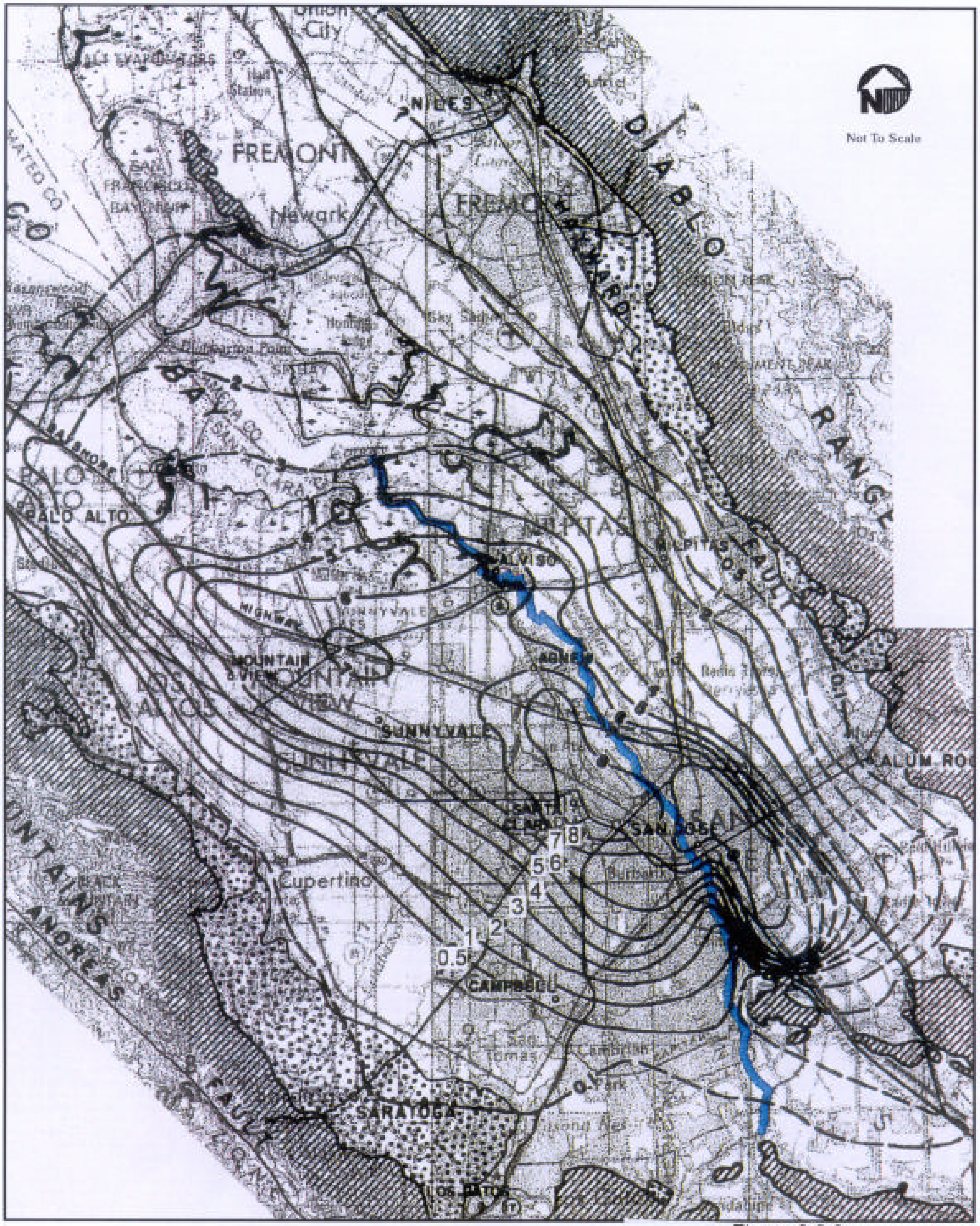
Cone Penetration Test (CPT) Location

Aerial Photo Source: USGS
Bay Area Regional Database
Date: Oct 1991

Figure 3.2.2c

GRAPHIC SCALE
0 1500
1" = 1500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



Map showing equal lines of land subsidence (in feet) in Santa Clara Valley from 1934 to 1967 (from Poland and Ireland, 1968) overlaid onto topographic mapping from the USGS.

Guadalupe River is highlighted in blue.

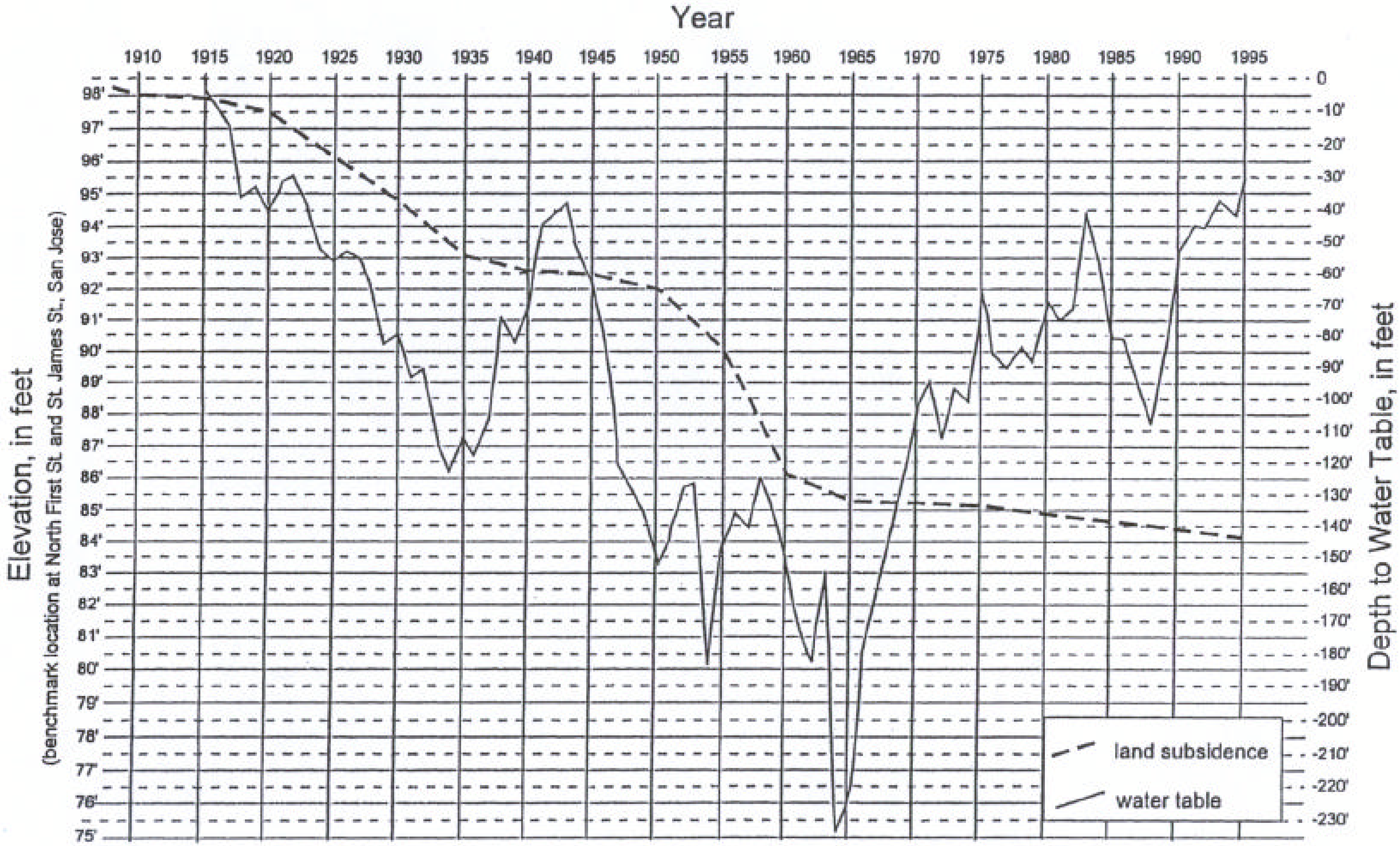
Figure 3.2.3a
(Source: Lower Guadalupe River Planning Study, SCVWD, 2002)

Map Showing Equal Lines of Land Subsidence in Santa Clara Valley (1934-1967)
Lower Guadalupe River Project

Source: NHC, 2000.

CH2MHILL

W042002001SJC 153735.06.01 LandSubsidence_SCV.fh9 6/2/02(EC)



Source: Atwater, B.F., Hedel, C.W., and Heeley, E.J., "Late Quaternary depositional history, Holocene sea-level changes, and vertical crustal movement, southern San Francisco Bay, CA." Geological Survey Professional Paper 1014, 15p.

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Figure 3.2.3b
(Source: Lower Guadalupe River Planning Study, SCVWD, 2002)

Land Subsidence and Depth to Water Table in San Jose (1906-1995)

Lower Guadalupe River Project

CH2MHILL

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

FEMA Special Flood Hazard Area

Tide Gage

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.3.1a

GRAPHIC SCALE
0 2000
1" = 2000'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

FEMA Special Flood Hazard Area

Tide Gage

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

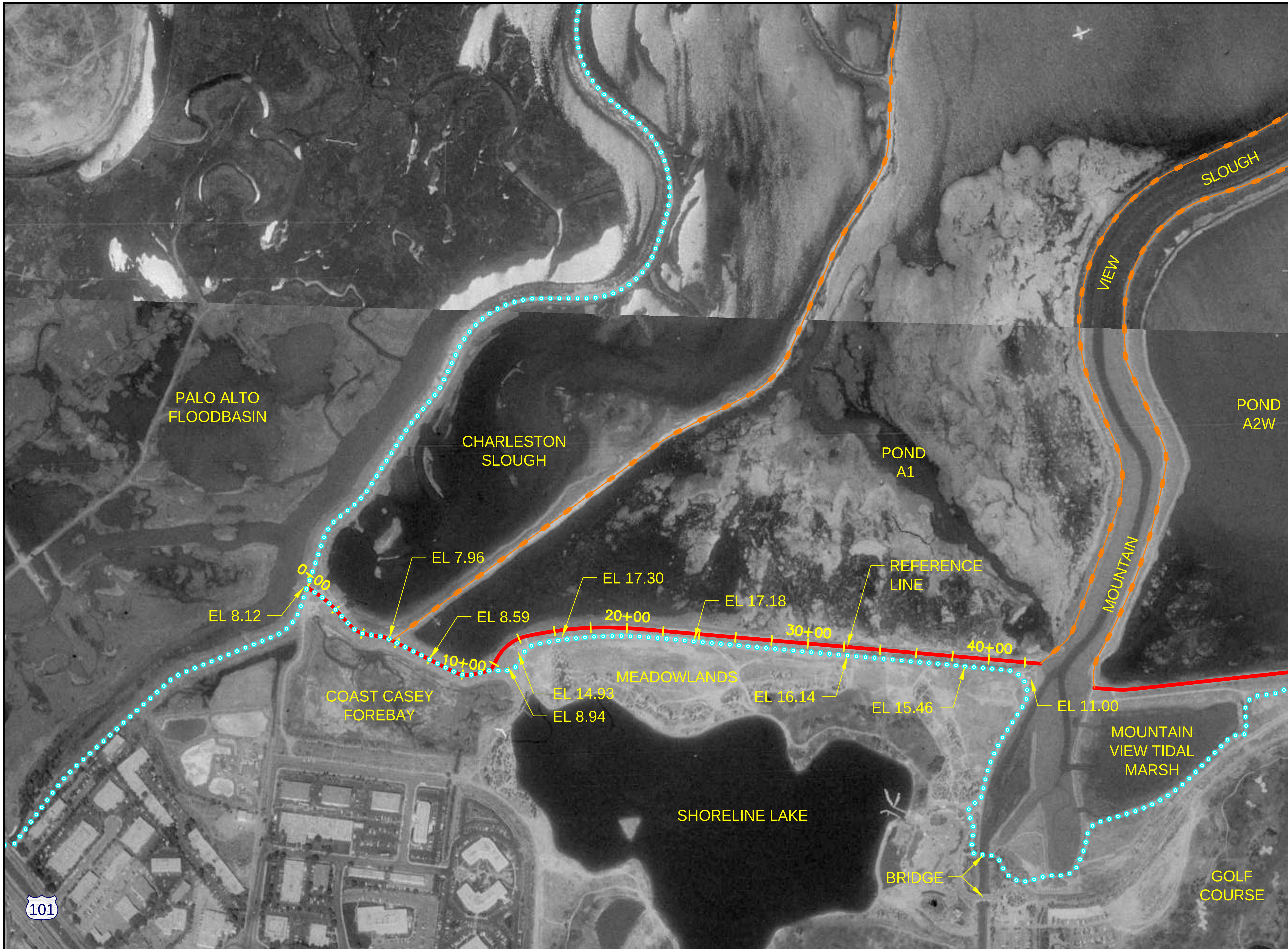
Figure 3.3.1b

GRAPHIC SCALE

0 2000

1" = 2000'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

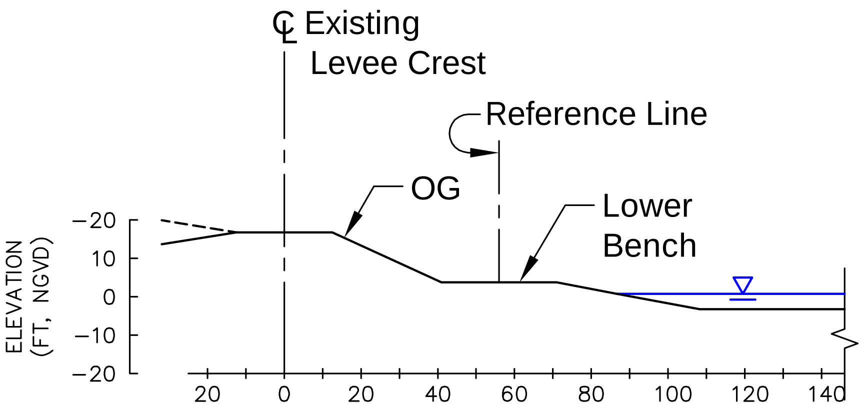
Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.3.2a

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



TYPICAL SECTION
LOOKING WEST

STATION 24+00±
LOOKING WEST

POND A1

Salt Pond Restoration Project

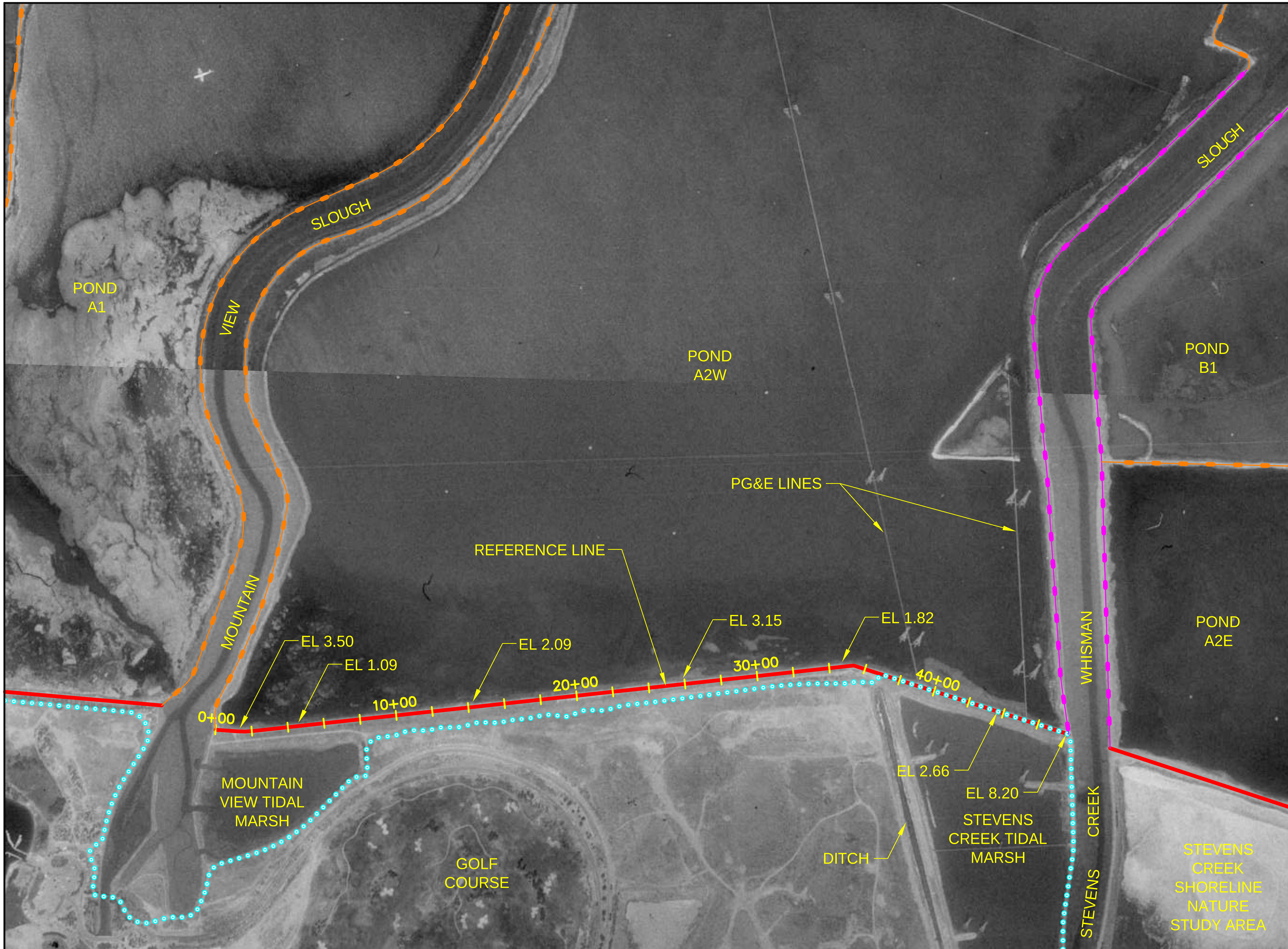
LEVEE EVALUATION

Figure 3.3.2b

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond
Restoration Project

LEVEE EVALUATION



N

Legend

Existing Bay Trail

Existing Salt Pond Levee

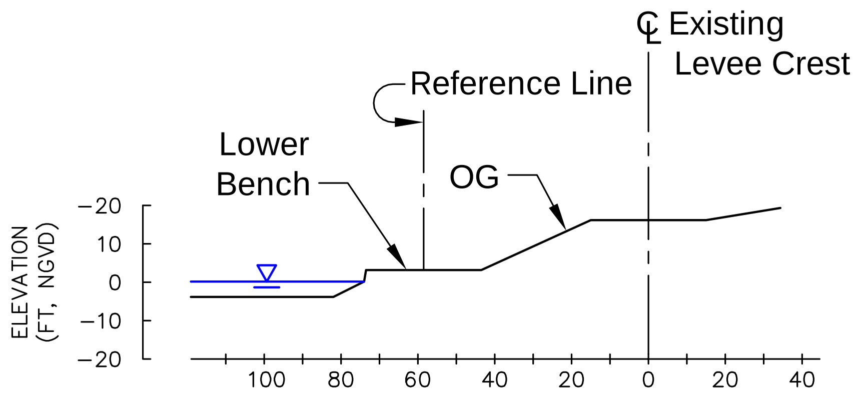
Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.3.2c

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



TYPICAL SECTION
LOOKING EAST

STATION 20+00±
LOOKING EAST

POND A2W

Salt Pond Restoration Project

LEVEE EVALUATION

Figure 3.3.2d

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

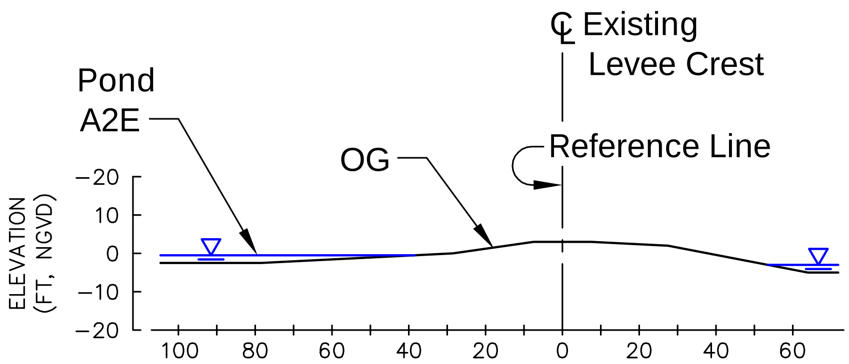
Figure 3.3.2e

GRAPHIC SCALE

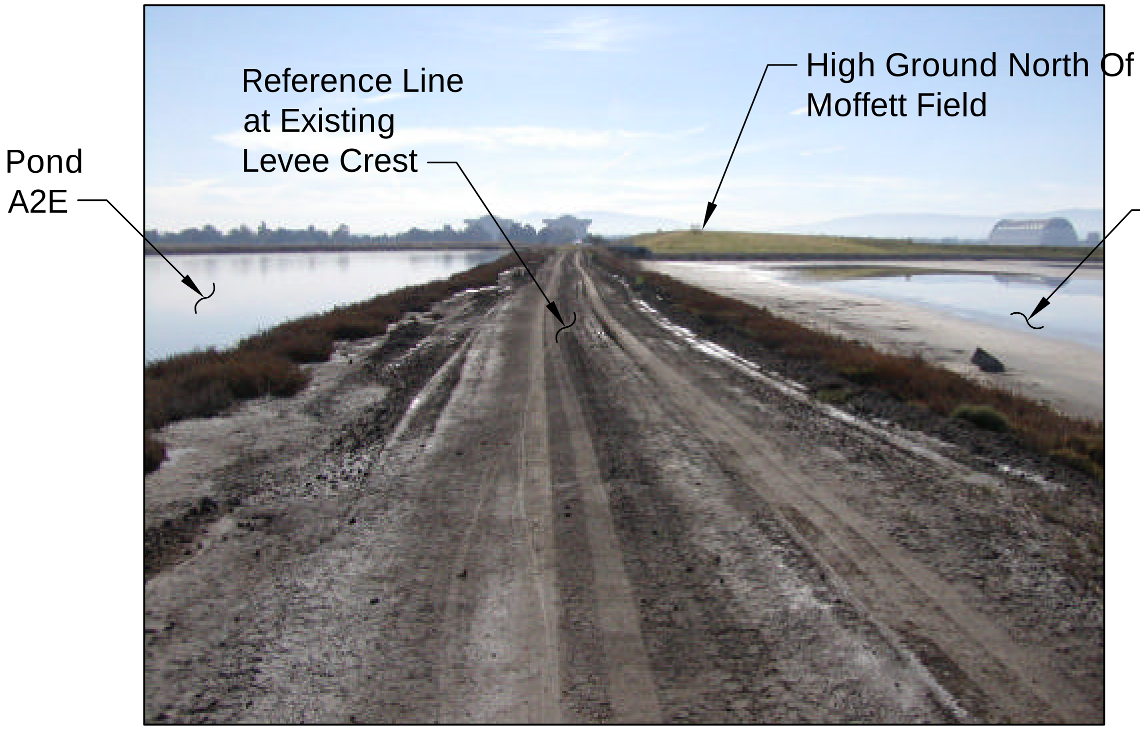
0 500

1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



TYPICAL SECTION
LOOKING SOUTHEAST



STATION 45+00±
LOOKING SOUTHEAST

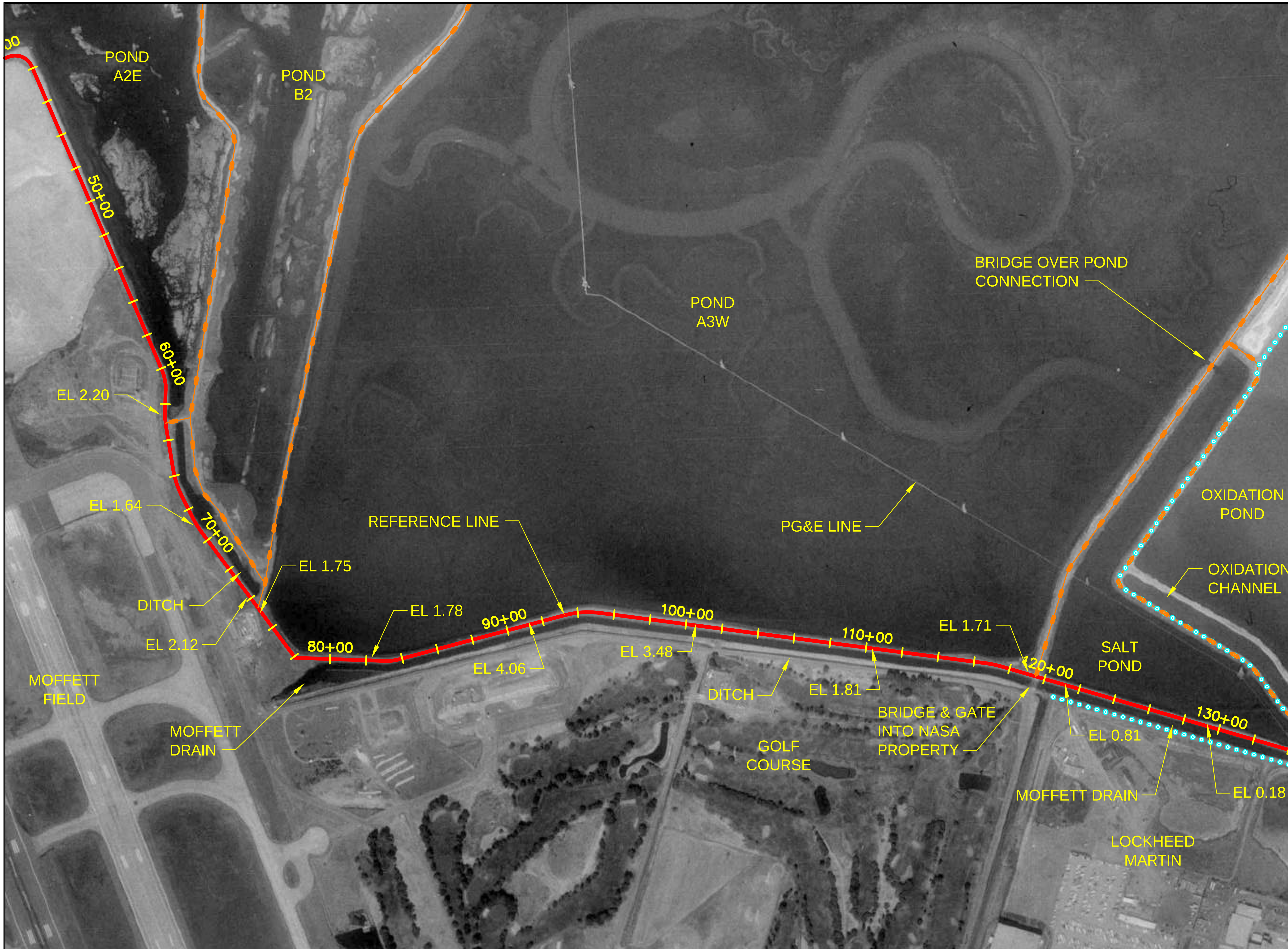
POND A2E

Salt Pond Restoration Project

LEVEE EVALUATION

Figure 3.3.2f

MOFFATT & NICHOL
ENGINEERS
WALNUT CREEK, CALIFORNIA



Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

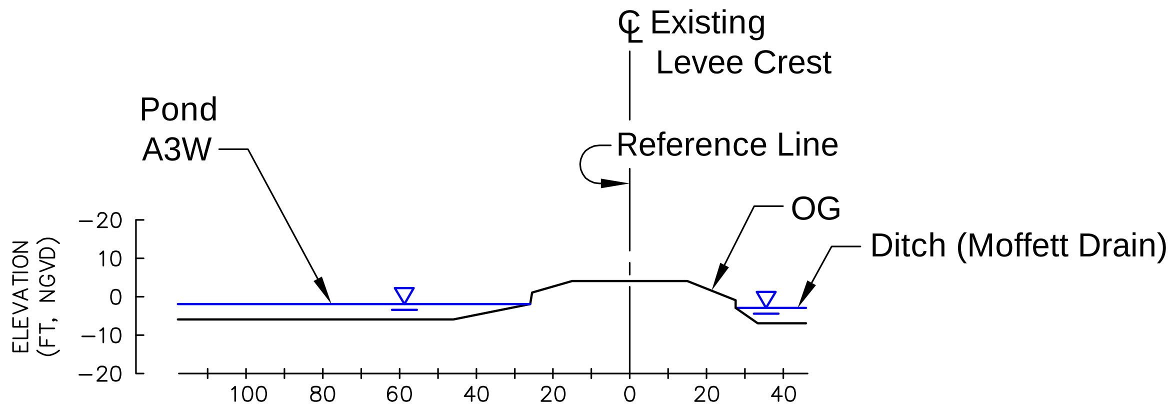
Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.3.2g

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



TYPICAL SECTION
LOOKING EAST

STATION 85+00±
LOOKING EAST

POND A3W

Salt Pond Restoration Project

LEVEE EVALUATION

Figure 3.3.2h

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.3.2i

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

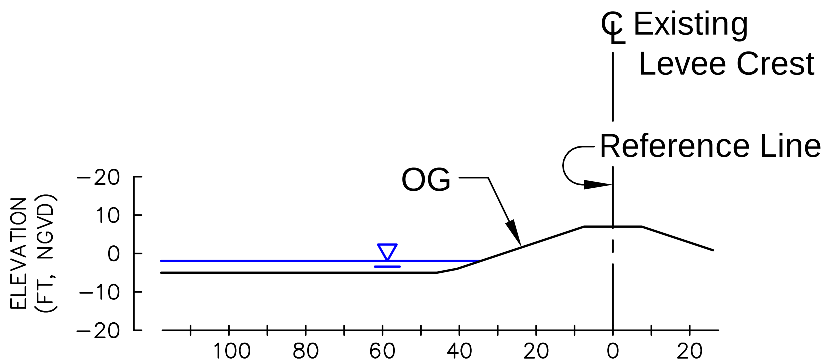
Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.3.2j

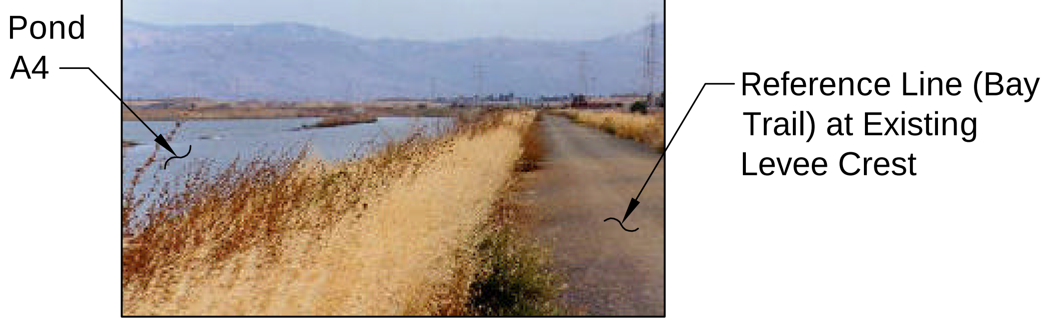
GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

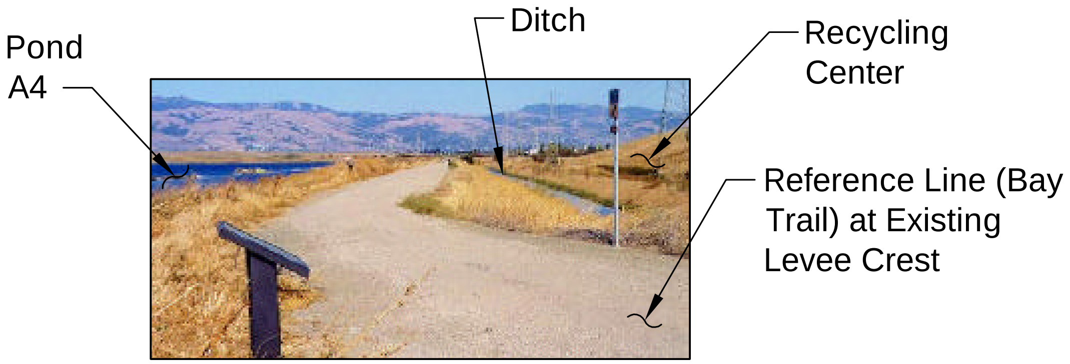


Note: Section is schematic only.

TYPICAL SECTION
LOOKING EAST



STATION 30+00±
LOOKING EAST



STATION 14+00±
LOOKING EAST

POND A4

Photos: Bay Trail Guided Photo Tour
www.abag.ca.gov/bayarea/baytrail/vtour/map3/access/Syvlblds/syvlblds1.htm

Salt Pond Restoration Project LEVEE EVALUATION	Figure 3.3.2k MOFFATT & NICHOL ENGINEERS WALNUT CREEK, CALIFORNIA

Salt Pond
Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.3.2l

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond
Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.3.2m

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database Date: Oct 1991

Figure 3.3.2n

GRAPHIC SCALE

0 500

1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Town Of
Alviso

Existing Levee

New Chicago
Marsh Pond

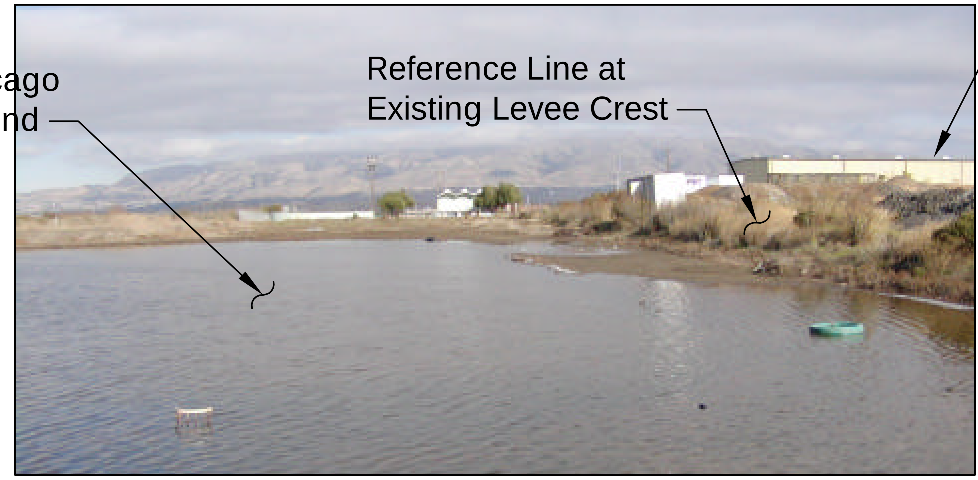
Note: Reference Line location does not appear in photo.

STATION 56+00±
LOOKING NORTH

New Chicago
Marsh Pond

Reference Line at
Existing Levee Crest

Town Of
Alviso



STATION 47+00±
LOOKING EAST

NEW CHICAGO MARSH

Salt Pond Restoration Project

LEVEE EVALUATION

Figure 3.3.2o

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond
Restoration Project

LEVEE EVALUATION

N

Legend

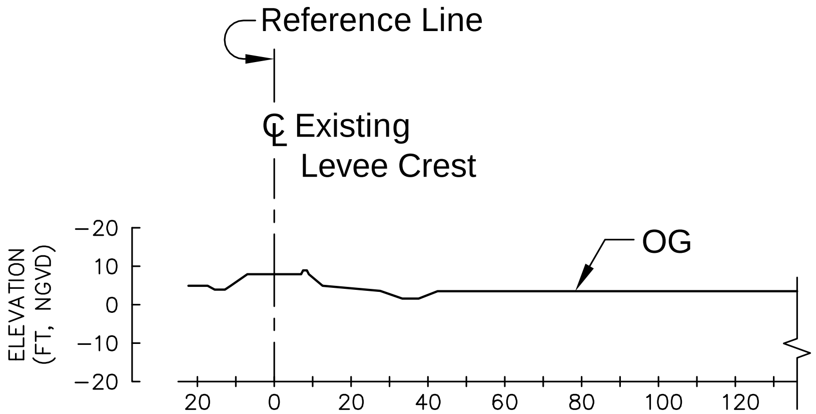
- Existing Bay Trail
- Existing Salt Pond Levee
- Existing Publicly-Maintained Levee

Aerial Photo Source: USGS
Bay Area Regional Database
Date: Oct 1991

Figure 3.3.2p

GRAPHIC SCALE
0 600
1" = 600'

MOFFATT & NICHOL
ENGINEERS
WALNUT CREEK, CALIFORNIA



STATION 30+00±
LOOKING NORTH

STATION 30+00±
LOOKING NORTH

POND A22

Salt Pond Restoration Project

LEVEE EVALUATION

Figure 3.3.2q

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

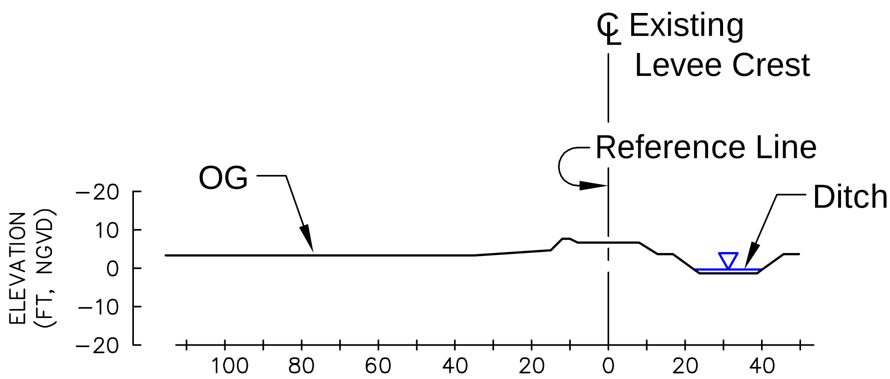
Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.4.2a

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



Note: The portion of pond shown in the section was dry at the time of investigation.

TYPICAL SECTION
LOOKING EAST

STATION 14+00±
LOOKING EAST

WEST BAY POND S5

Salt Pond Restoration Project

LEVEE EVALUATION

Figure 3.4.2b

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

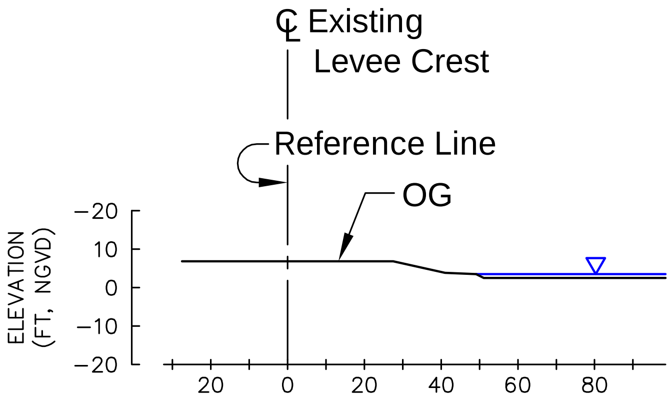
Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.4.2c

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



TYPICAL SECTION
LOOKING WEST

STATION 48+00±
LOOKING WEST

WEST BAY POND 3

Salt Pond Restoration Project

LEVEE EVALUATION

Figure 3.4.2d

MOFFATT & NICHOL
ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond
Restoration Project

LEVEE EVALUATION

N

Legend

Existing Bay Trail

Existing Salt Pond Levee

Existing Publicly-Maintained Levee

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.4.2e

GRAPHIC SCALE
0 500
1" = 500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

Salt Pond
Restoration Project

LEVEE EVALUATION

N

Legend

FEMA Special Flood Hazard Area

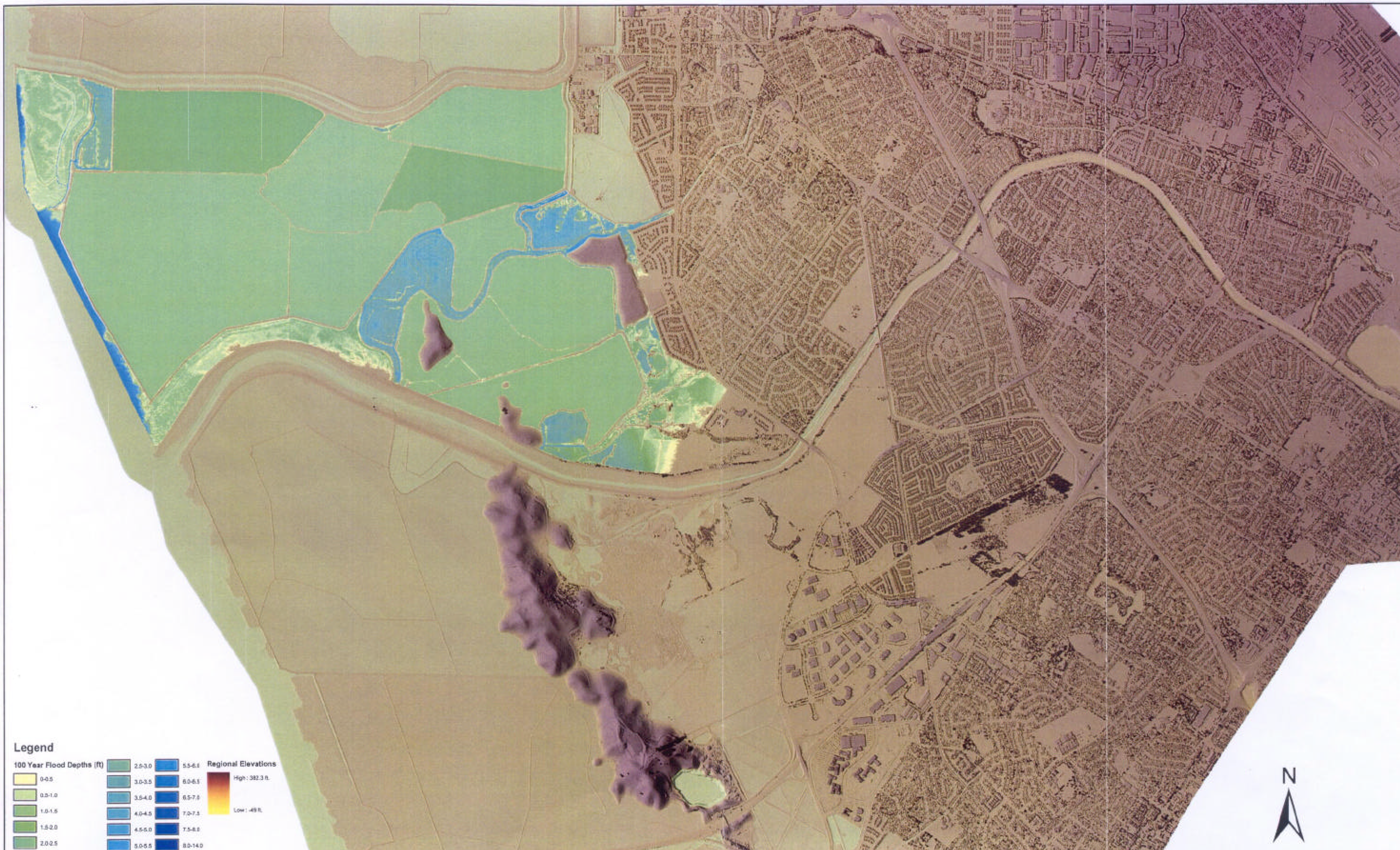
Tide Gage

Aerial Photo Source: USGS Bay Area Regional Database
Date: Oct 1991

Figure 3.4.1

GRAPHIC SCALE
0 1500
1" = 1500'

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



Elevation data derived from LIDAR topographic survey, 2001
Datum: NAD 1983
Projection: State Plane, California Zone III
Units: Meters
Vertical Datum: NGVD 1929

0 1,250 2,500 5,000 7,500 10,000 Feet

URS

Figure 3.5.1: 100 – Year Floodplain
(Source: Alameda Creek Floodplain Study, URS, 2003, Prepared for ACFCWCD)

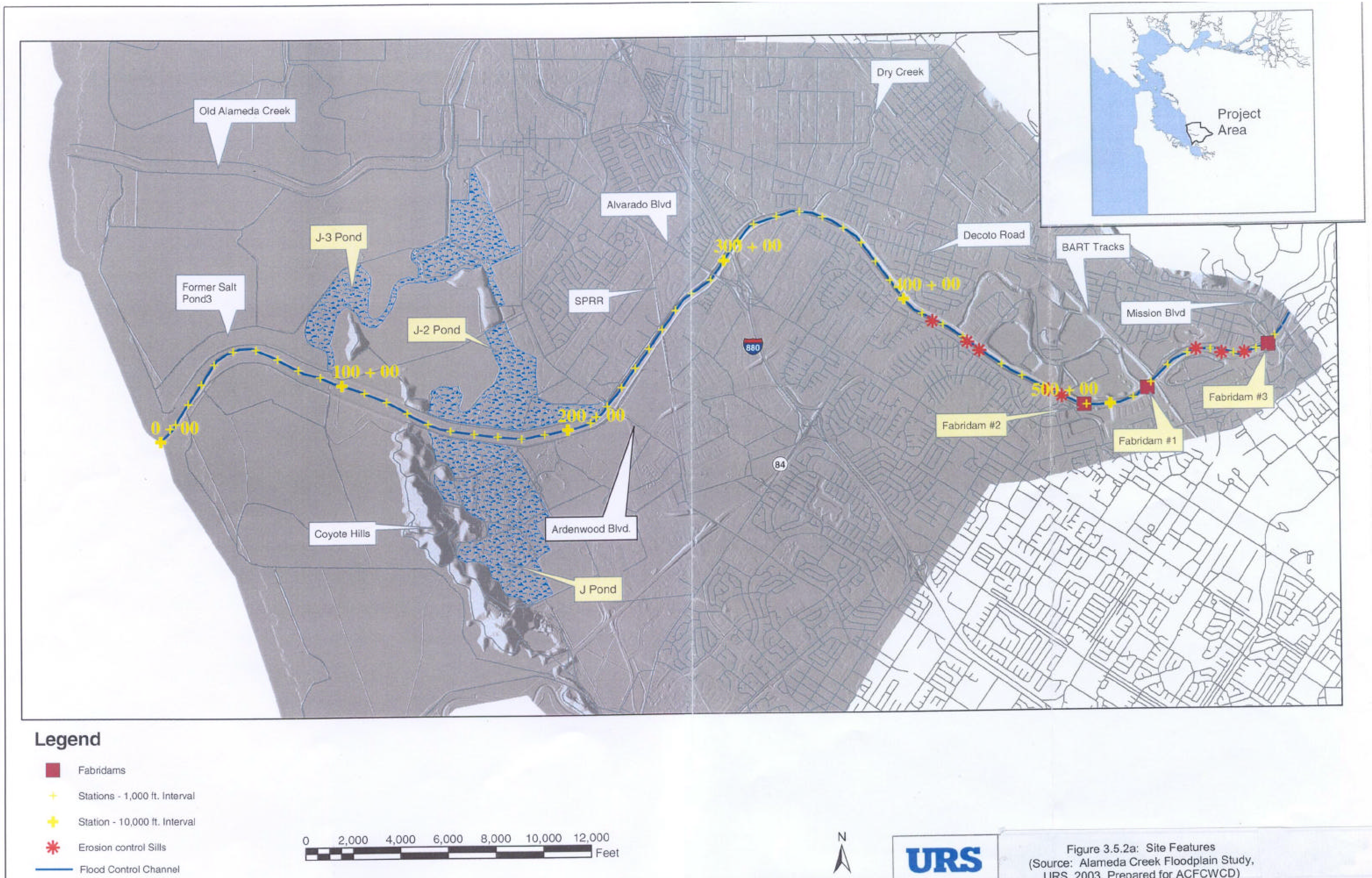


Figure 3.5.2a: Site Features
(Source: Alameda Creek Floodplain Study, URS, 2003, Prepared for ACFCWCD)

Figure 8 Comparison of LIDAR Survey and County In-Channel Survey at Selected Cross Sections

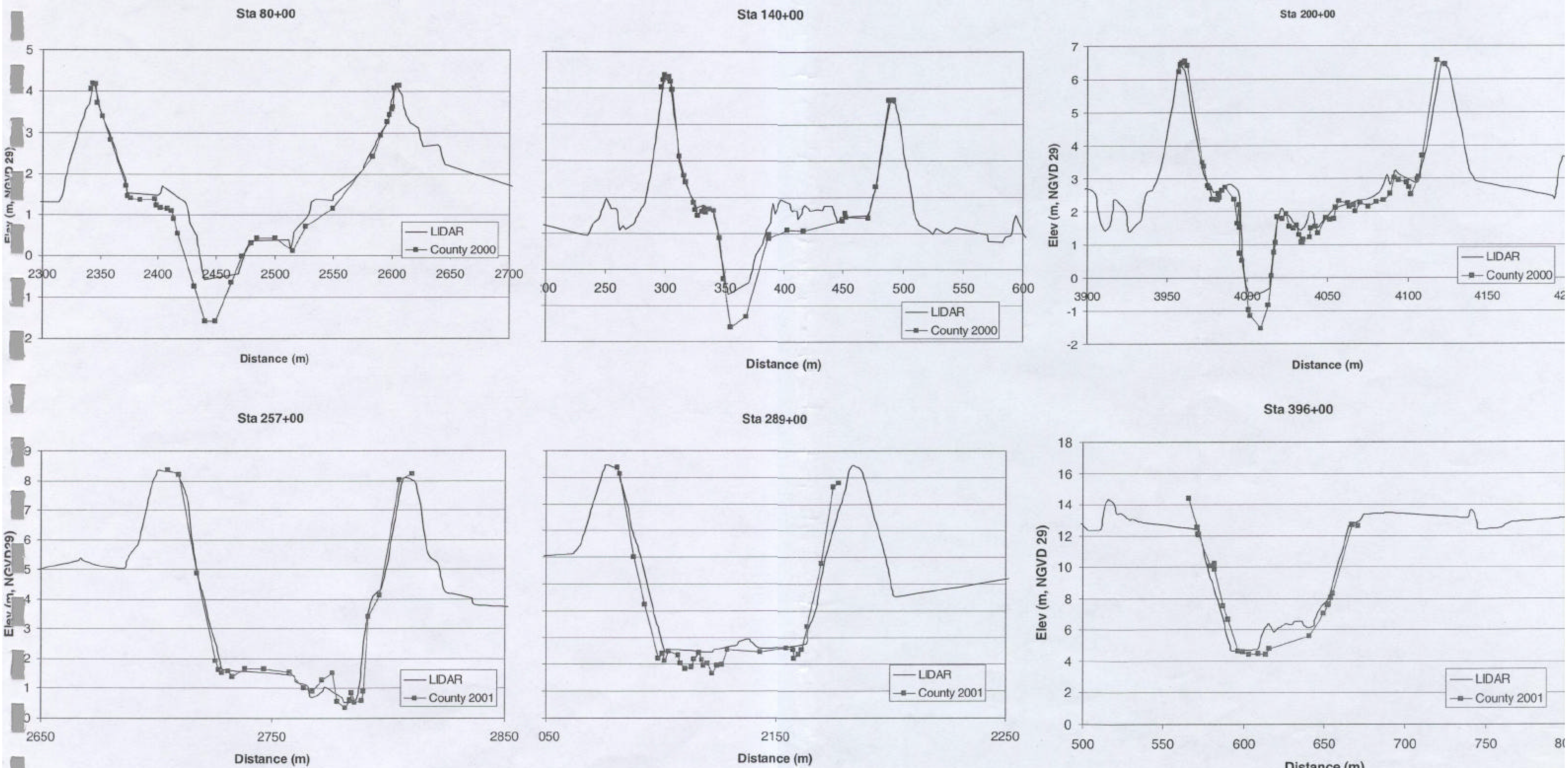
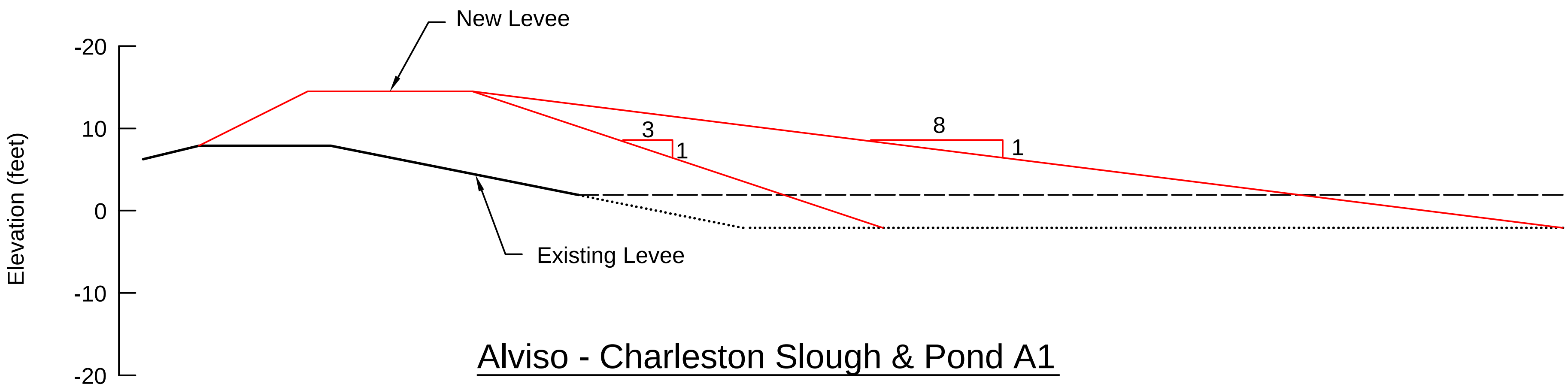


Figure 3.5.2.b: Levee Cross Sections,
See Figure 3.5.2a for Stationing

(Source: Alameda Creek Floodplain Study,
URS, 2003, Prepared for ACFCWCD)



Alviso - Charleston Slough & Pond A1

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.8	1.6	1.4			
14.5	8 : 1	1.7	1.3	1.1	3.3	2.7	2.3			
12.0	3 : 1	1.2	1.0	< 1.0	2.4	2.0	1.9			
12.0	8 : 1	1.9	1.5	1.3	3.9	3.0	2.7			

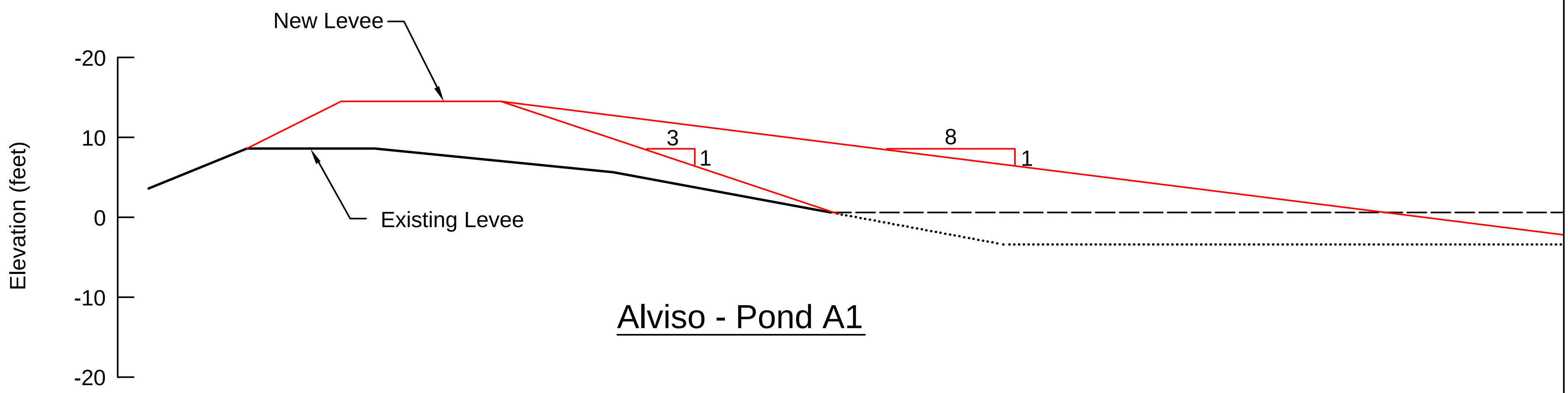
SCALE

0 20 feet

1 inch = 20 feet

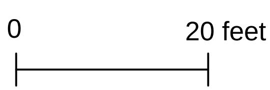
Station 0+00 - 6+00 (Way Point 76) South Bay Salt Ponds Alviso, California	Cross-Section

Hultgren - Tillis Engineers	Project No. 561.01	Figure 4.3.3a



Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.7	1.4	1.4	1	1	1
14.5	8 : 1	1.5	1.2	1.0	3.0	2.5	2.1	< 1	1	1
12.0	3 : 1	1.0	< 1.0	< 1.0	2.1	1.8	1.7	< 1	1	1
12.0	8 : 1	1.8	1.4	1.2	3.6	2.8	2.5	< 1	1	1

SCALE



1 inch = 20 feet

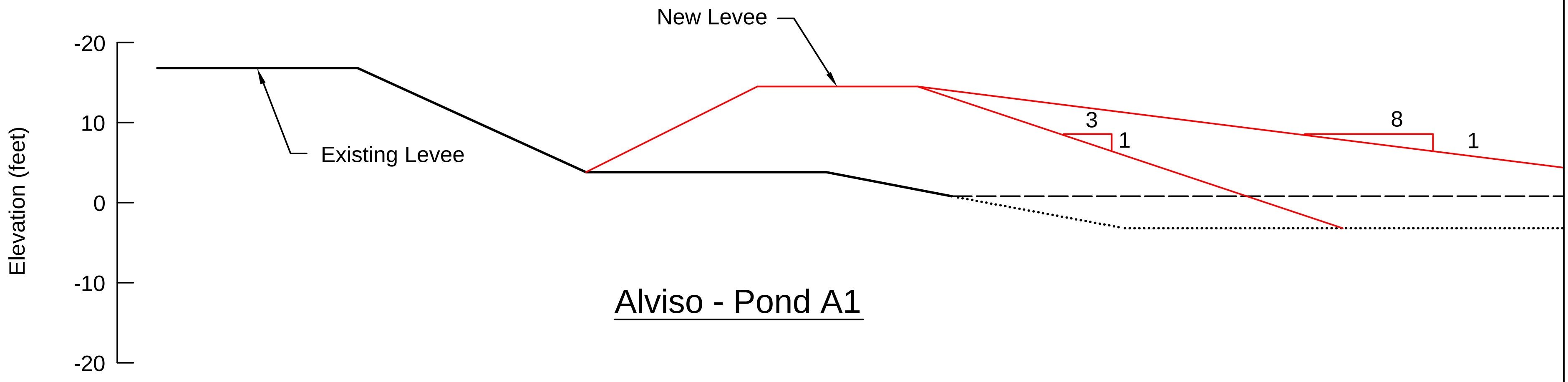
Station 6+00 - 10+00 (Way Point 77)
South Bay Salt Ponds
Alviso, California

Cross-Section

Hultgren - Tillis Engineers

Project No. 561.01

Figure 4.3.3b



Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.8	1.5	1.4	1	3	4
14.5	8 : 1	1.5	1.2	1.1	3.1	2.5	2.2	1	2	3
12.0	3 : 1	1.1	< 1.0	< 1.0	2.2	1.9	1.7	1	2	3
12.0	8 : 1	1.8	1.4	1.3	2.6	2.9	2.6	1	2	3

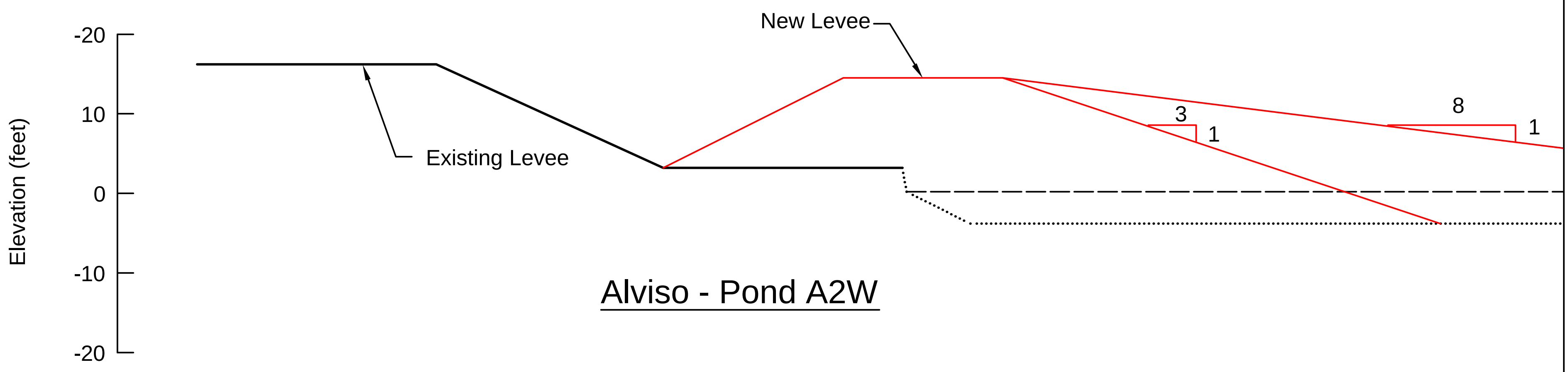
SCALE

0 20 feet

1 inch = 20 feet

Station 15+00 - 43+00 (Way Point 78) South Bay Salt Ponds Alviso, California	Cross-Section

Hultgren - Tillis Engineers	Project No. 561.01	Figure 4.3.3c



Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability: Strength = 200 (psf), Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)	Strength = 400 (psf), Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)	Settlement (feet): Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.7	1.4	1.4	2	3	5
14.5	8 : 1	1.5	1.2	1.0	3.0	2.4	2.0	2	3	4
12.0	3 : 1	1.0	< 1.0	< 1.0	2.0	1.8	1.7	2	3	4
12.0	8 : 1	1.7	1.3	1.2	3.4	2.7	2.5	2	3	4

SCALE

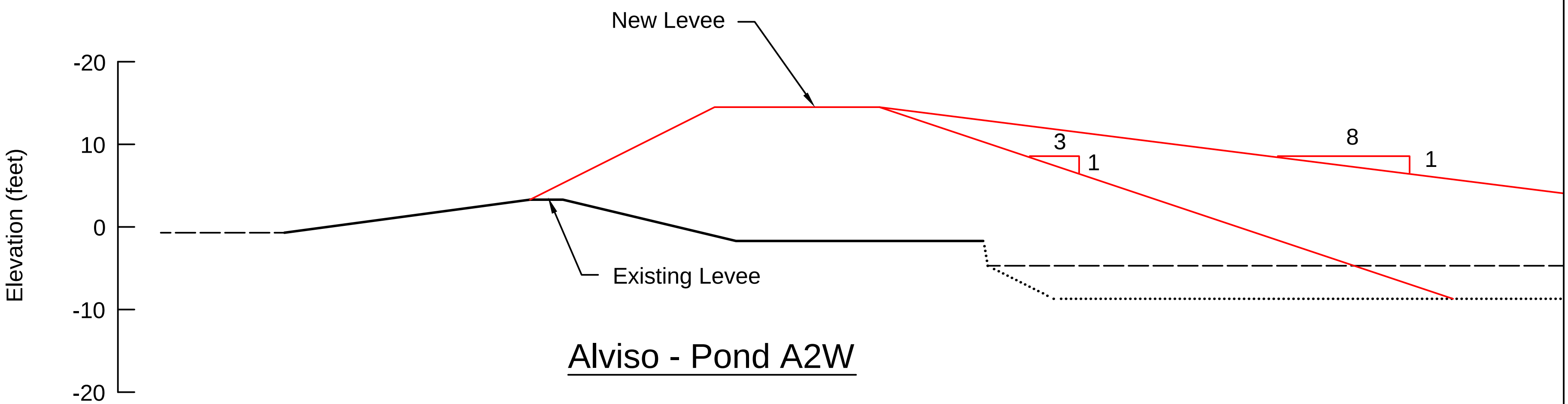
0 20 feet

1 inch = 20 feet

Station 0+00 - 37+00 (Way Point 80)
South Bay Salt Ponds
Alviso, California

Cross-Section

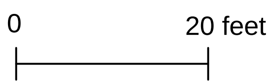
Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3d



Alviso - Pond A2W

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.4	1.1	1.0	2	3	5
14.5	8 : 1	1.2	1.0	< 1.0	2.5	2.0	1.5	2	3	5
12.0	3 : 1	< 1.0	< 1.0	< 1.0	1.6	1.4	1.3	2	3	4
12.0	8 : 1	1.4	1.1	< 1.0	2.8	2.2	1.8	2	3	4

SCALE

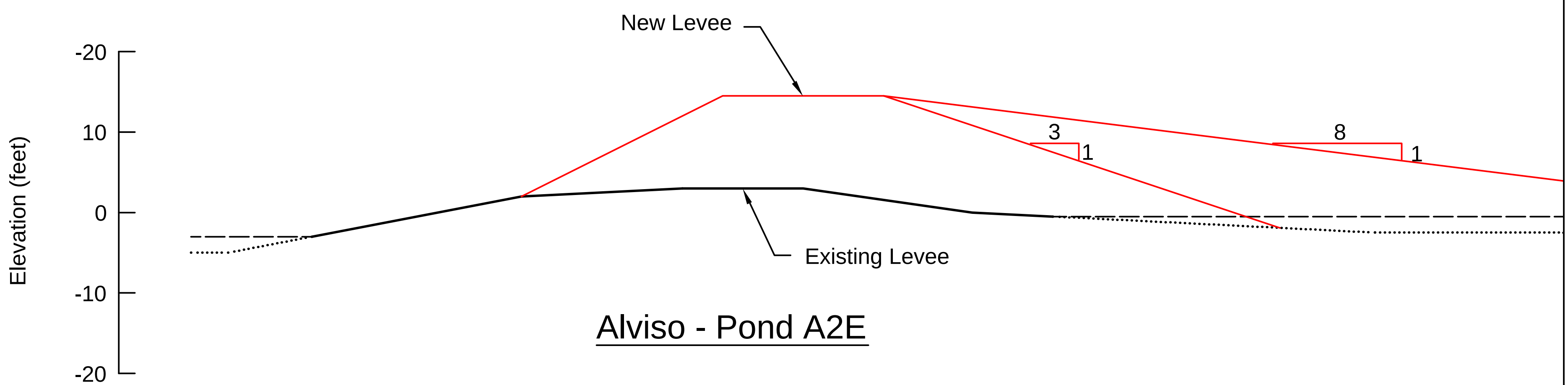


1 inch = 20 feet

Station 38+00 - 48+00 (Way Point 81)
South Bay Salt Ponds
Alviso, California

Cross-Section

Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3e



Alviso - Pond A2E

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.6	1.4	1.3	1	2	3
14.5	8 : 1	1.4	1.2	<1.0	2.9	2.4	2.0	1	2	3
12.0	3 : 1	< 1.0	< 1.0	< 1.0	1.9	1.7	1.6	1	2	2
12.0	8 : 1	1.7	1.3	1.1	3.3	2.7	2.3	1	2	2

SCALE

0 20 feet

1 inch = 20 feet

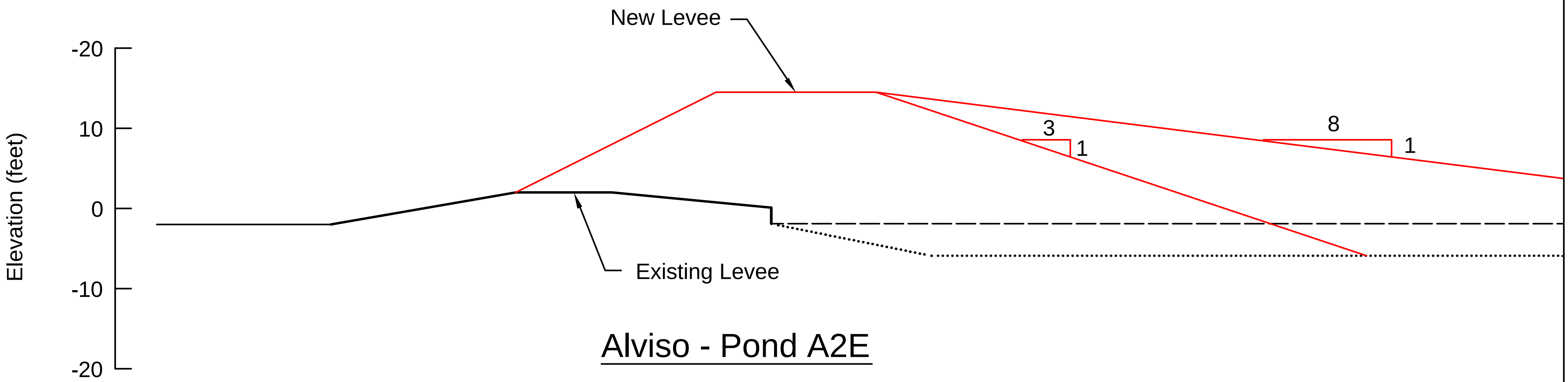
Station 0+00 - 17+00 (Way Point 82)
South Bay Salt Ponds
Alviso, California

Cross-Section

Hultgren - Tillis Engineers

Project No. 561.01

Figure 4.3.3f



Alviso - Pond A2E

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.5	1.3	1.2	2	3	5
14.5	8 : 1	1.4	1.1	<1.0	2.8	2.2	1.8	2	3	5
12.0	3 : 1	< 1.0	< 1.0	< 1.0	1.8	1.6	1.4	2	3	4
12.0	8 : 1	1.5	1.2	1.1	3.1	2.5	2.2	2	3	4

SCALE

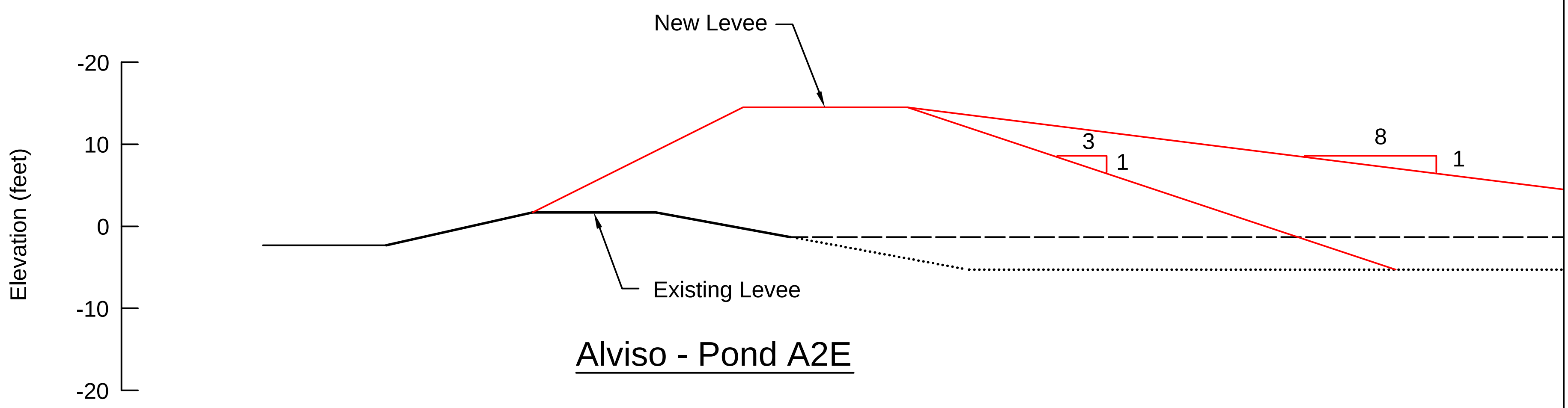
0 20 feet

1 inch = 20 feet

Station 17+00 - 41+00 (Way Point 83)
South Bay Salt Ponds
Alviso, California

Cross-Section

Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3g



Alviso - Pond A2E

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.5	1.3	1.2	2	3	5
14.5	8 : 1	1.4	1.1	<1.0	2.8	2.2	1.8	2	3	5
12.0	3 : 1	< 1.0	< 1.0	< 1.0	1.8	1.5	1.5	2	3	4
12.0	8 : 1	1.6	1.2	1.1	3.2	2.5	2.2	2	3	4

SCALE

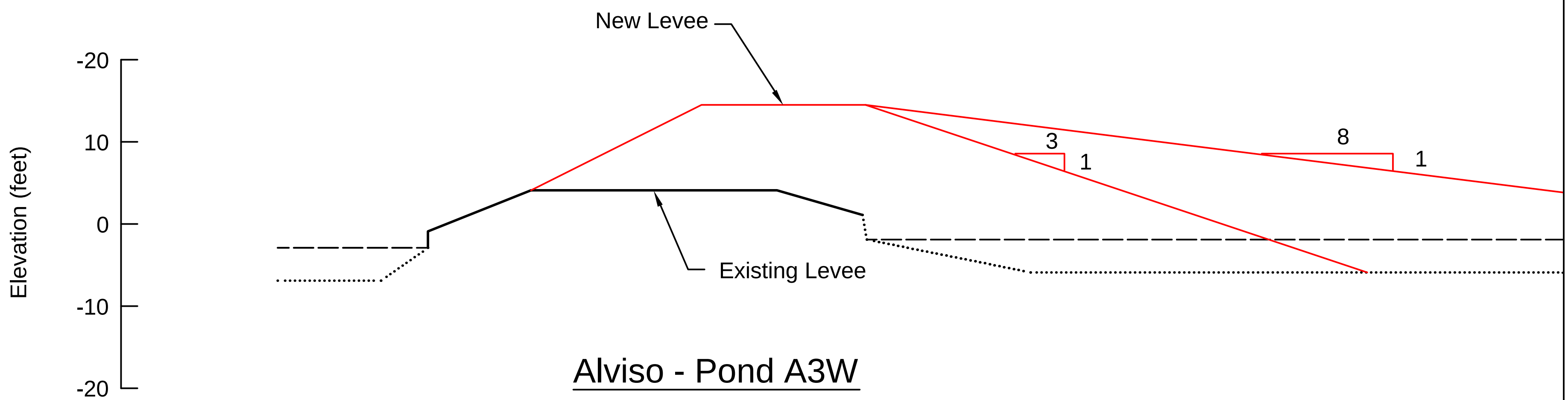
0 20 feet

1 inch = 20 feet

Station 41+00 - 62+00 (Way Point 84)
South Bay Salt Ponds
Alviso, California

Cross-Section

Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3h



Alviso - Pond A3W

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability: Strength = 200 (psf), Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)	Strength = 400 (psf), Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)	Settlement (feet), Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.5	1.3	1.2	2	3	5
14.5	8 : 1	1.4	1.1	<1.0	2.8	2.2	1.8	2	3	5
12.0	3 : 1	< 1.0	< 1.0	< 1.0	1.8	1.6	1.4	2	3	4
12.0	8 : 1	1.5	1.2	1.1	3.1	2.5	2.2	2	3	4

SCALE

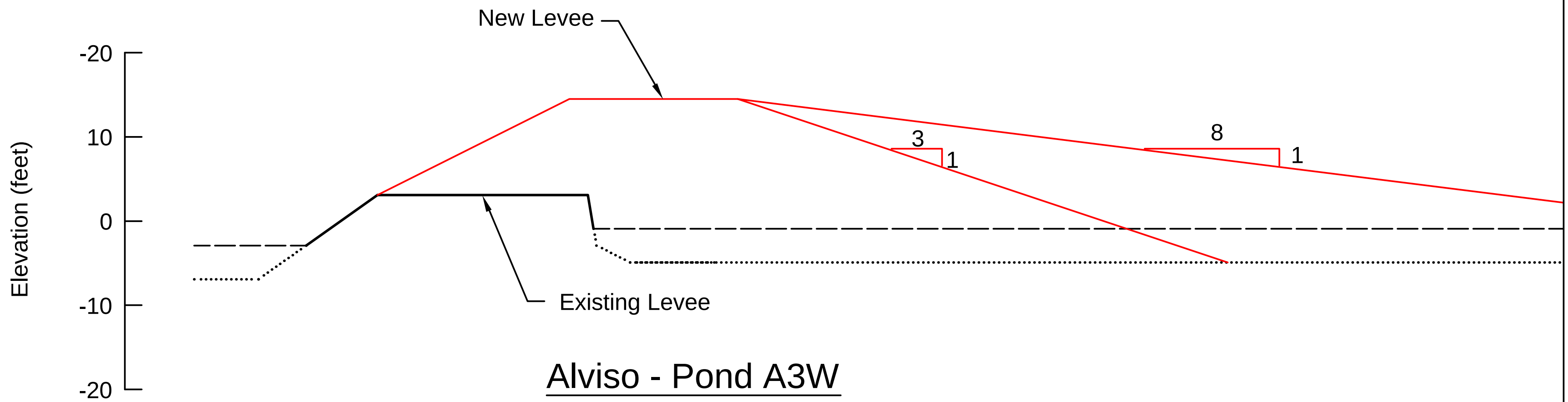
0 20 feet

1 inch = 20 feet

Station 70+00 - 94+00 (Way Point 86)
South Bay Salt Ponds
Alviso, California

Cross-Section

Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3i



Alviso - Pond A3W

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.6	1.3	1.2	2	3	5
14.5	8 : 1	1.4	1.2	<1.0	2.8	2.3	1.9	2	3	5
12.0	3 : 1	< 1.0	< 1.0	< 1.0	1.9	1.7	1.6	2	3	4
12.0	8 : 1	1.7	1.3	1.1	3.3	2.7	2.3	2	3	4

SCALE

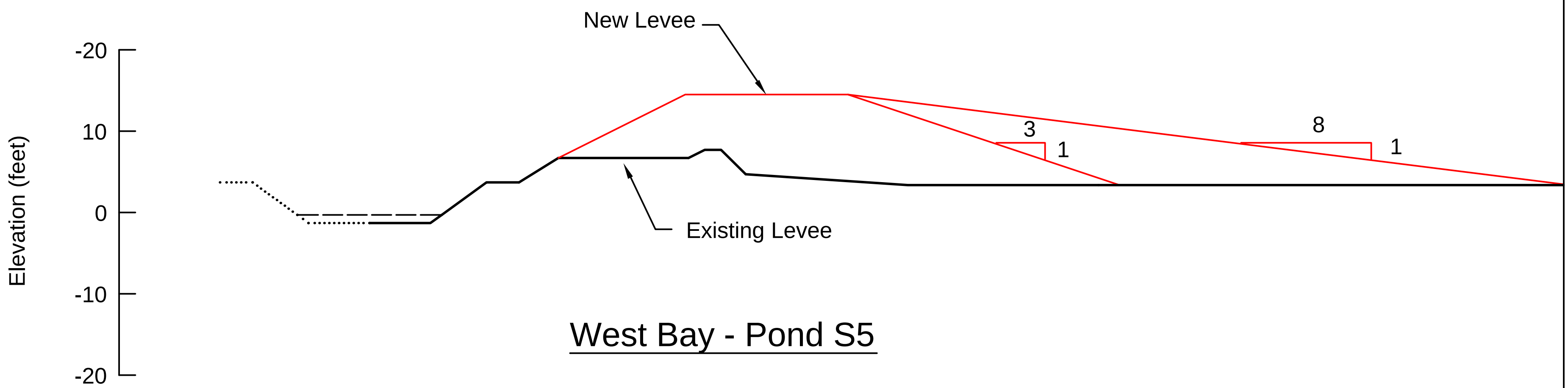
0 20 feet

1 inch = 20 feet

Station 94+00 - 110+00 (Way Point 87)
South Bay Salt Ponds
Alviso, California

Cross-Section

Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3i



West Bay - Pond S5

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.9	1.7	1.6	2	3	4
14.5	8 : 1	1.8	1.4	1.2	3.6	2.8	2.5	2	3	4
12.0	3 : 1	1.3	1.2	1.1	2.7	2.4	2.3	1	3	4
12.0	8 : 1	2.2	1.7	1.4	> 4.0	3.3	2.9	1	3	4

SCALE

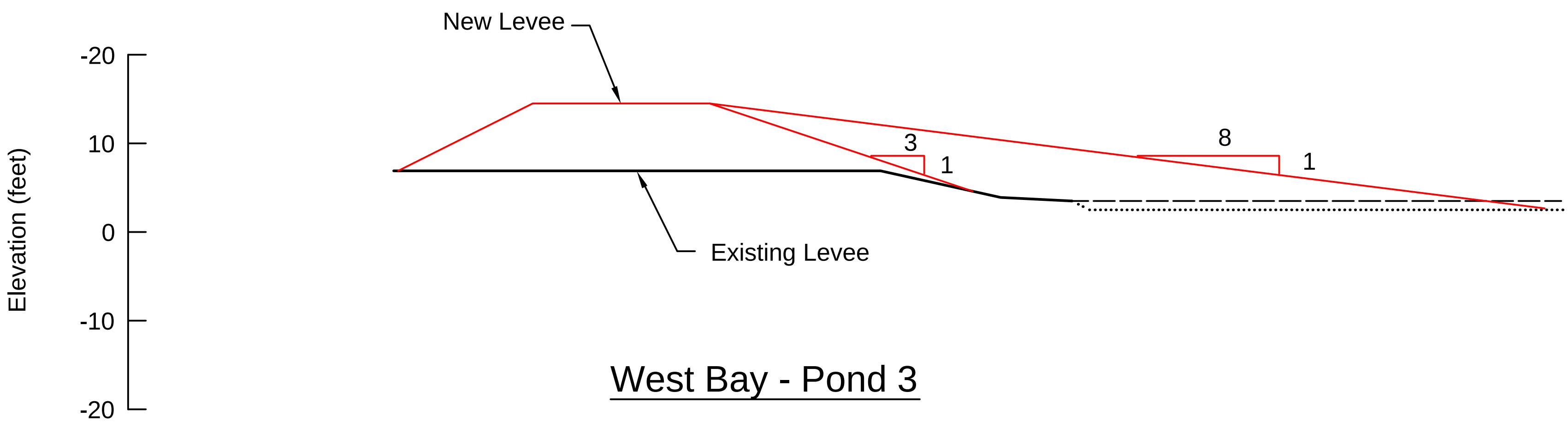
0 20 feet

1 inch = 20 feet

Station 4+00 - 30+00 (Way Point 89)
South Bay Salt Ponds
West Bay, California

Cross-Section

Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3k



West Bay - Pond 3

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability: Strength = 200 (psf), Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)	Strength = 400 (psf), Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)	Settlement (feet), Bay Mud Thickness 5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.9	1.7	1.6	1	1	2
14.5	8 : 1	1.8	1.4	1.2	3.6	2.8	2.5	1	1	2
12.0	3 : 1	1.3	1.2	1.1	2.7	2.4	2.3	< 1	1	1
12.0	8 : 1	2.2	1.7	1.4	> 4.0	3.3	2.9	< 1	1	1

SCALE

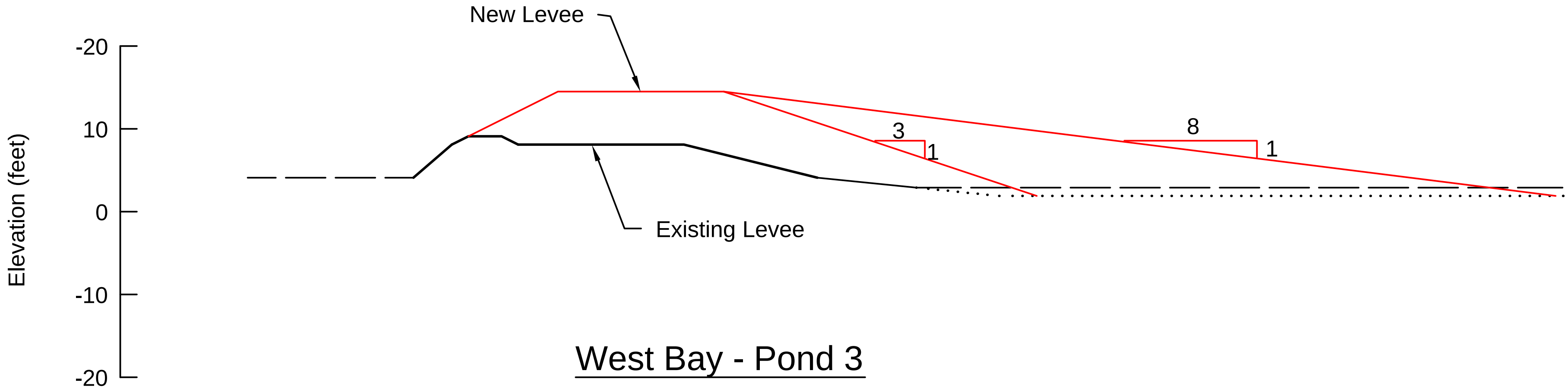
0 20 feet

1 inch = 20 feet

Station 30+00 - 70+00 (Way Point 91)
South Bay Salt Ponds
West Bay, California

Cross-Section

Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3I



West Bay - Pond 3

Crest Elev. (ft)	Slope H : V	Factor of Safety for Slope Stability						Settlement (feet)		
		Strength = 200 (psf)			Strength = 400 (psf)					
		Bay Mud Thickness			Bay Mud Thickness			Bay Mud Thickness		
		5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)	5.0 (ft)	10.0 (ft)	15.0 (ft)
15.5	3 : 1	< 1.0	< 1.0	< 1.0	1.9	1.7	1.6	1	2	3
14.5	8 : 1	1.7	1.3	1.2	3.4	2.7	2.5	1	2	3
12.0	3 : 1	1.2	1.1	1.1	2.5	2.3	2.1	1	1	2
12.0	8 : 1	2.0	1.6	1.4	4.0	3.3	2.8	1	1	2

SCALE

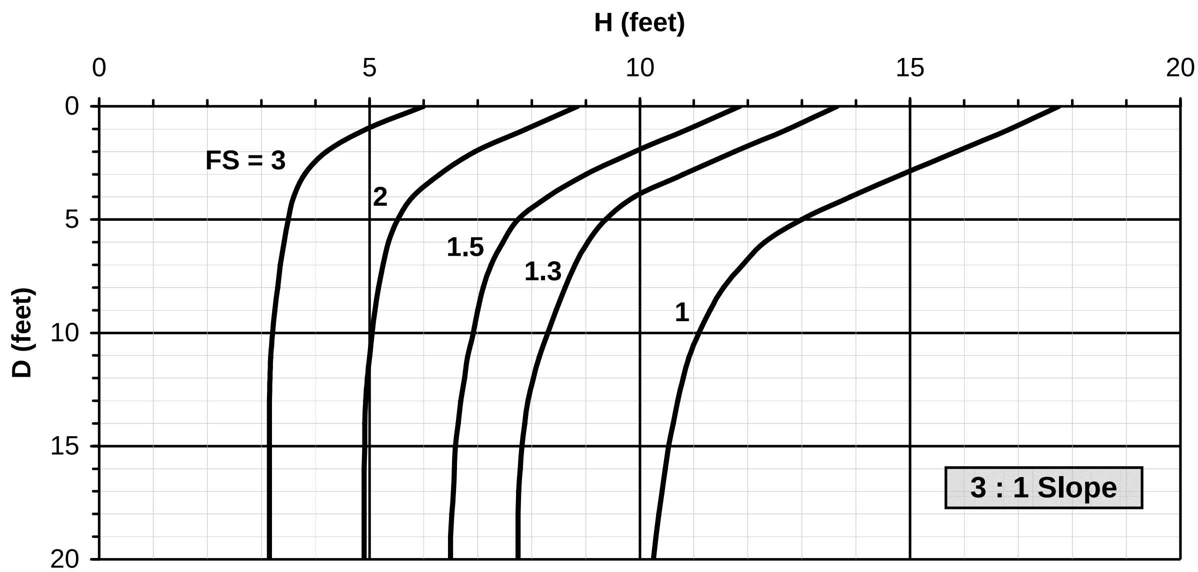
0 20 feet

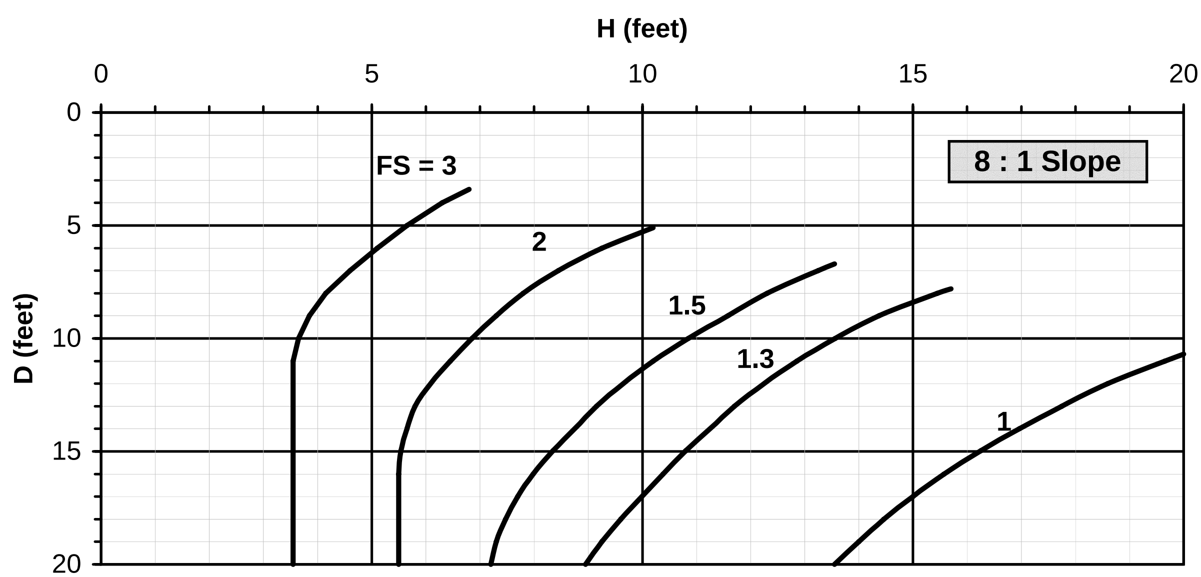
1 inch = 20 feet

Station 110+00 - 118+00 (Way Point 92)
South Bay Salt Ponds
West Bay, California

Cross-Section

Hultgren - Tillis Engineers | Project No. 561.01 | Figure 4.3.3m





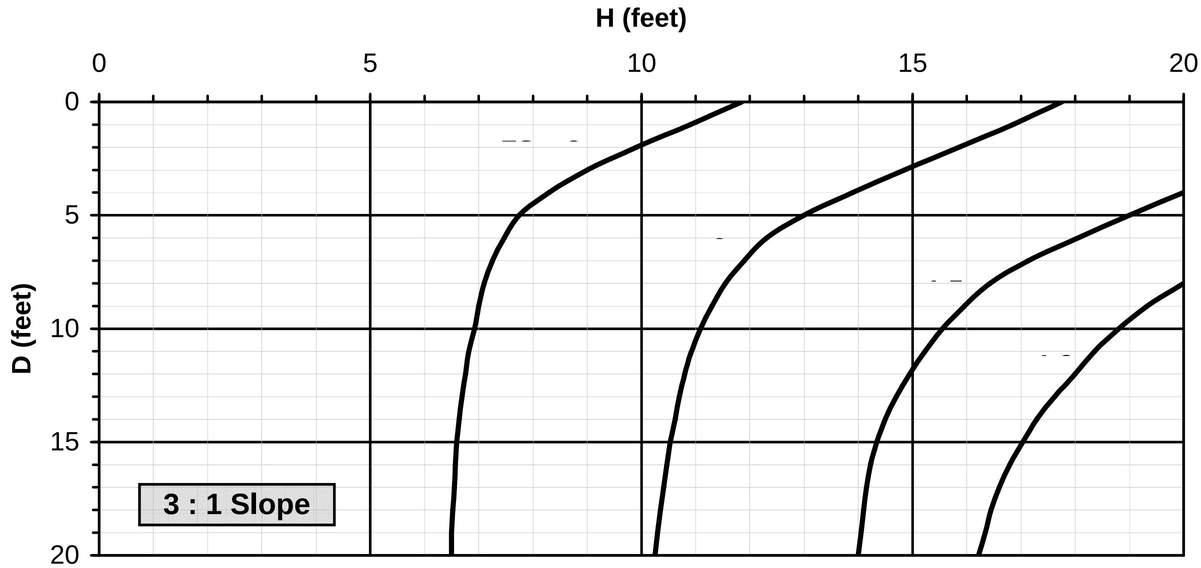
Note:

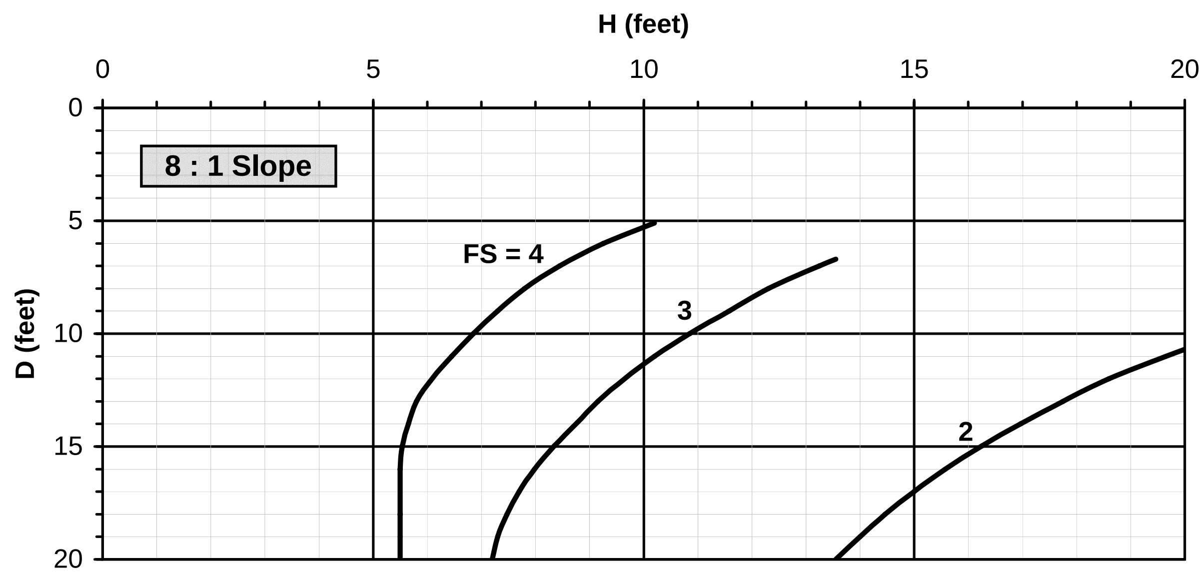
H: Height of the new levee embankment above the toe of the existing levee
D: Thickness of Bay Mud below the toe of the existing levee

South Bay Salt Ponds
Alviso, California

Stability Analysis Charts
Shear Strength, C = 200 psf

Hultgren - Tillis Engineers	Project No. 556.01	Figure 4.3.3n





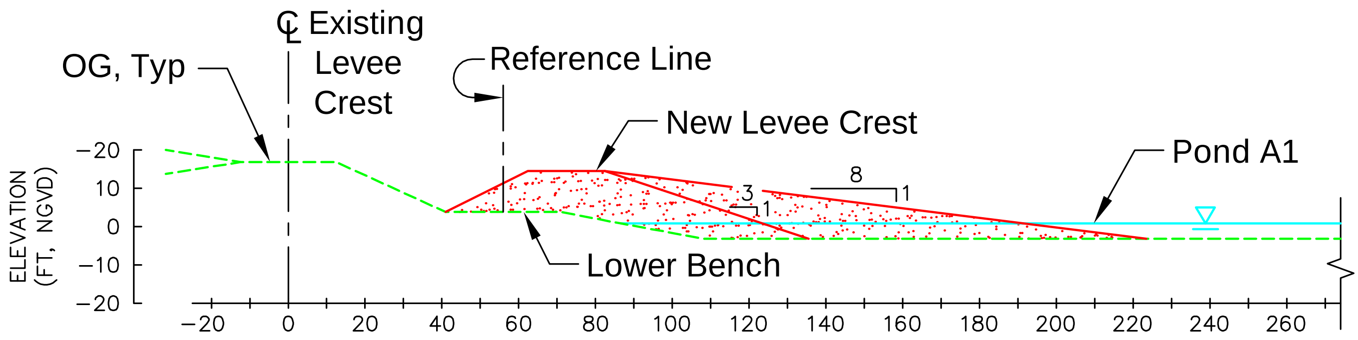
Note:

H: Height of the new levee embankment above the toe of the existing levee

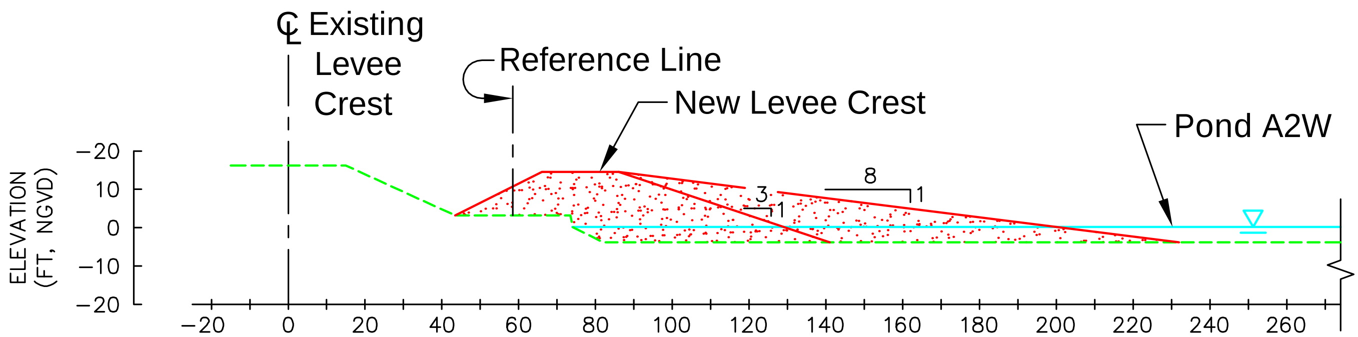
D: Thickness of Bay Mud below the toe of the existing levee

South Bay Salt Ponds *Alviso, California*	**Stability Analysis Charts** **Shear Strength, C = 400 psf**

Hultgren - Tillis Engineers	Project No. 556.01	Figure 4.3.3o

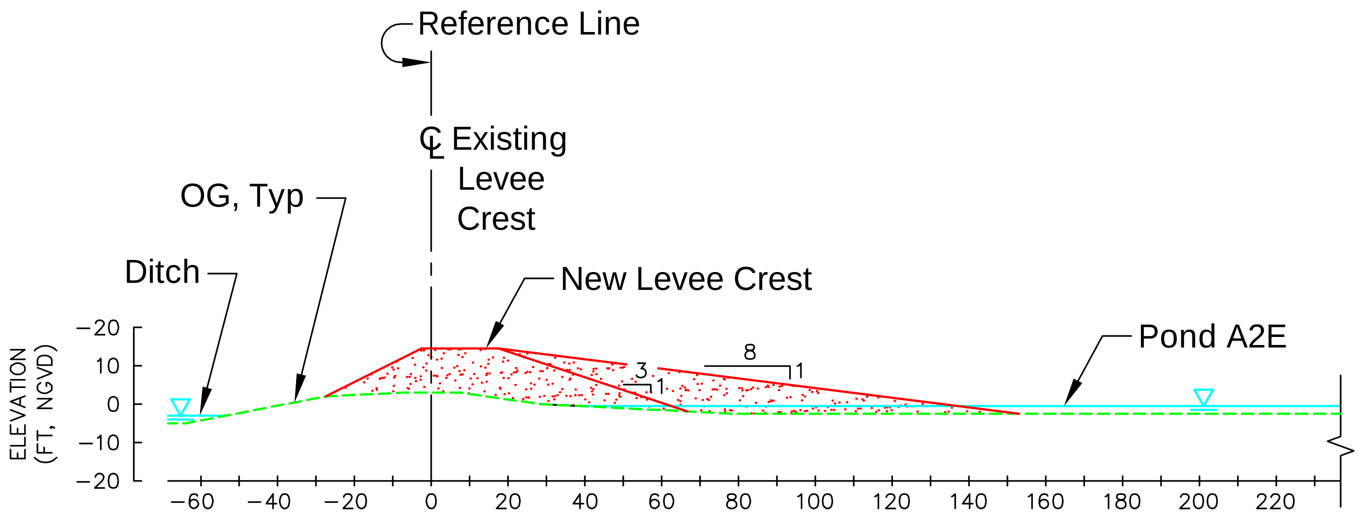


ALVISO COMPLEX - POND A1
PROPOSED TYPICAL SECTION

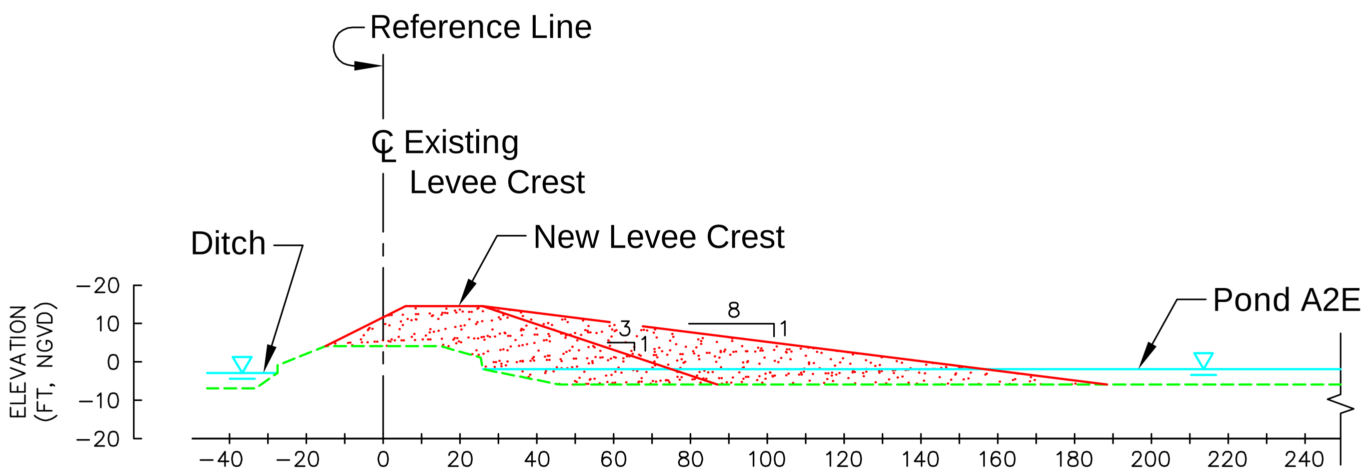


ALVISO COMPLEX - POND A2W
PROPOSED TYPICAL SECTION

Salt Pond Restoration Project LEVEE EVALUATION	Figure 4.4.1 MOFFATT & NICHOL ENGINEERS WALNUT CREEK, CALIFORNIA



ALVISO COMPLEX - POND A2E
PROPOSED TYPICAL SECTION



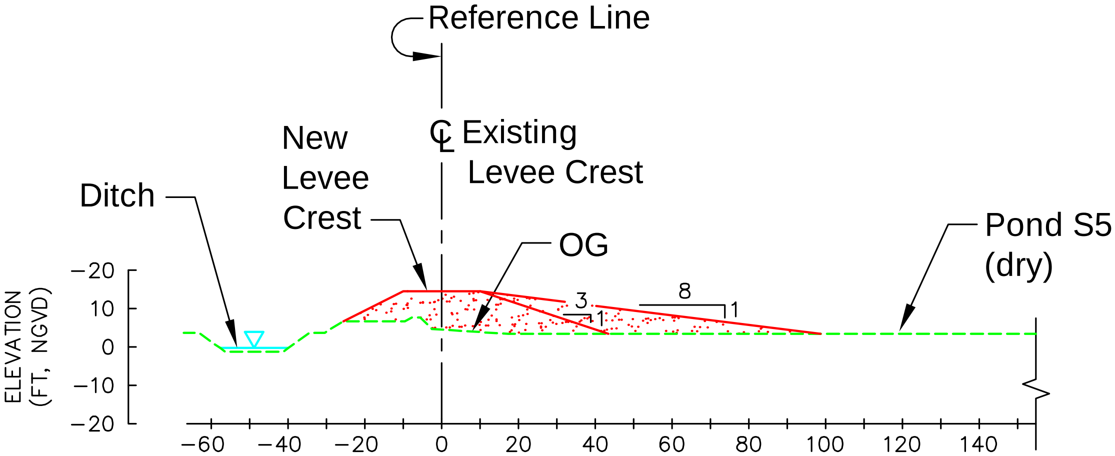
ALVISO COMPLEX - POND A3W
PROPOSED TYPICAL SECTION

Salt Pond Restoration Project

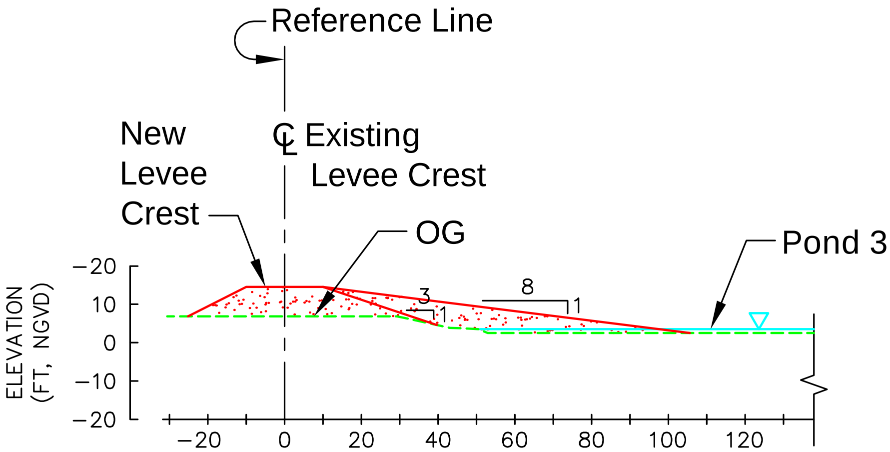
LEVEE EVALUATION

Figure 4.4.2

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



WEST BAY COMPLEX - POND S5
PROPOSED TYPICAL SECTION



WEST BAY COMPLEX - POND 3
PROPOSED TYPICAL SECTION

Salt Pond Restoration Project

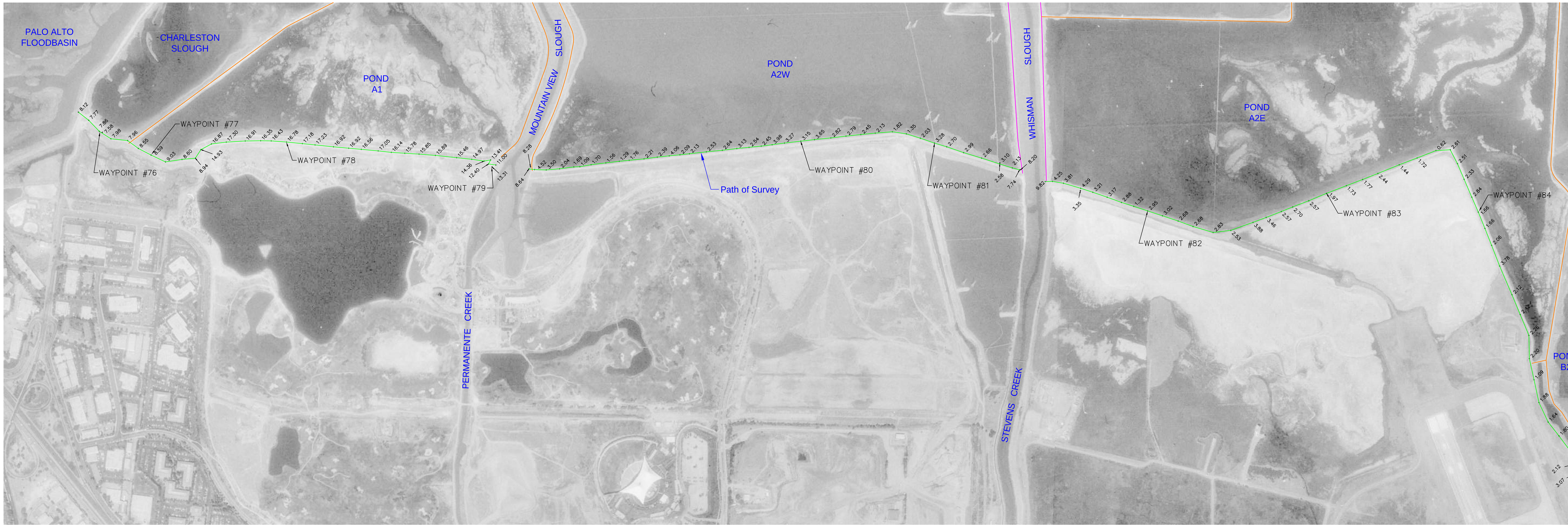
LEVEE EVALUATION

Figure 4.4.3

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

APPENDICES

A. Survey Data



Notes:

1. Elevations shown are in Feet, relative to NGVD 1929.
2. Waypoints shown are locations where cross-sections were measured (visual reconnaissance level).
3. GPS Survey performed by Tucker & Associates, Jan 2004.
4. Additional benchmarks established on site, not shown on Figure.

GRAPHIC SCALE

0 500

1" = 500'

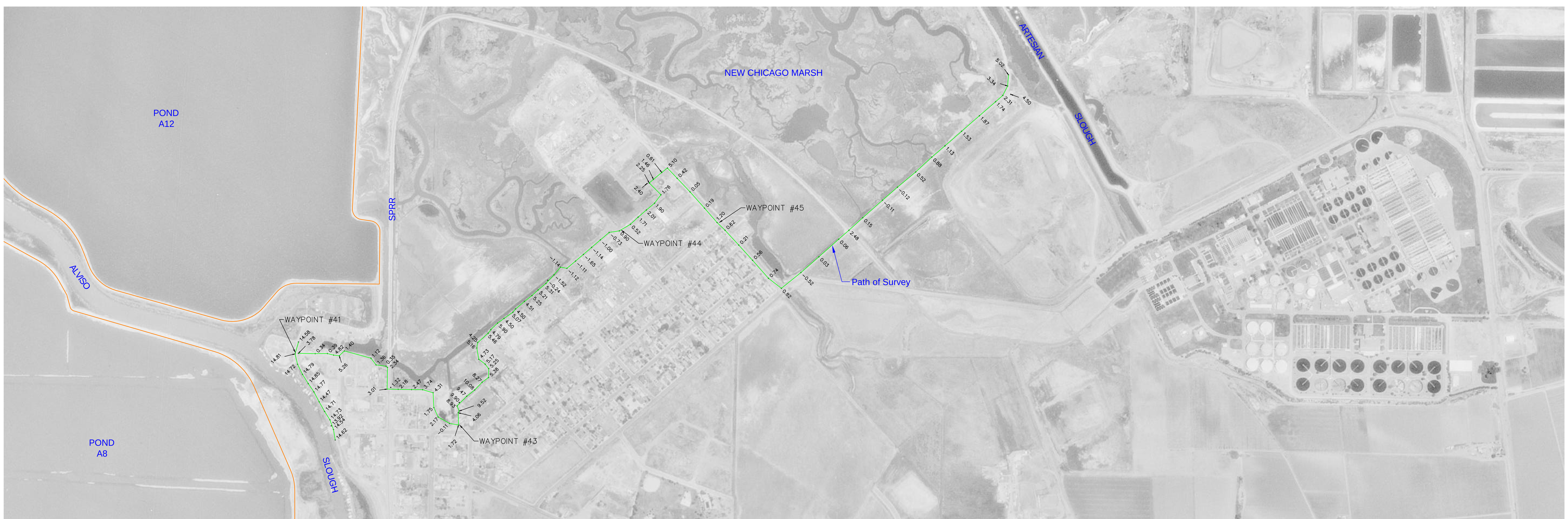
Salt Pond
Restoration Project

LEVEE EVALUATION

N

Alviso Complex (West)

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



Notes:

1. Elevations shown are in Feet, relative to NGVD 1929.
2. Waypoints shown are locations where cross-sections were measured (visual reconnaissance level).
3. GPS Survey performed by Tucker & Associates, Jan 2004.
4. Additional benchmarks established on site, not shown on Figure.

GRAPHIC SCALE

0 500

1" = 500'

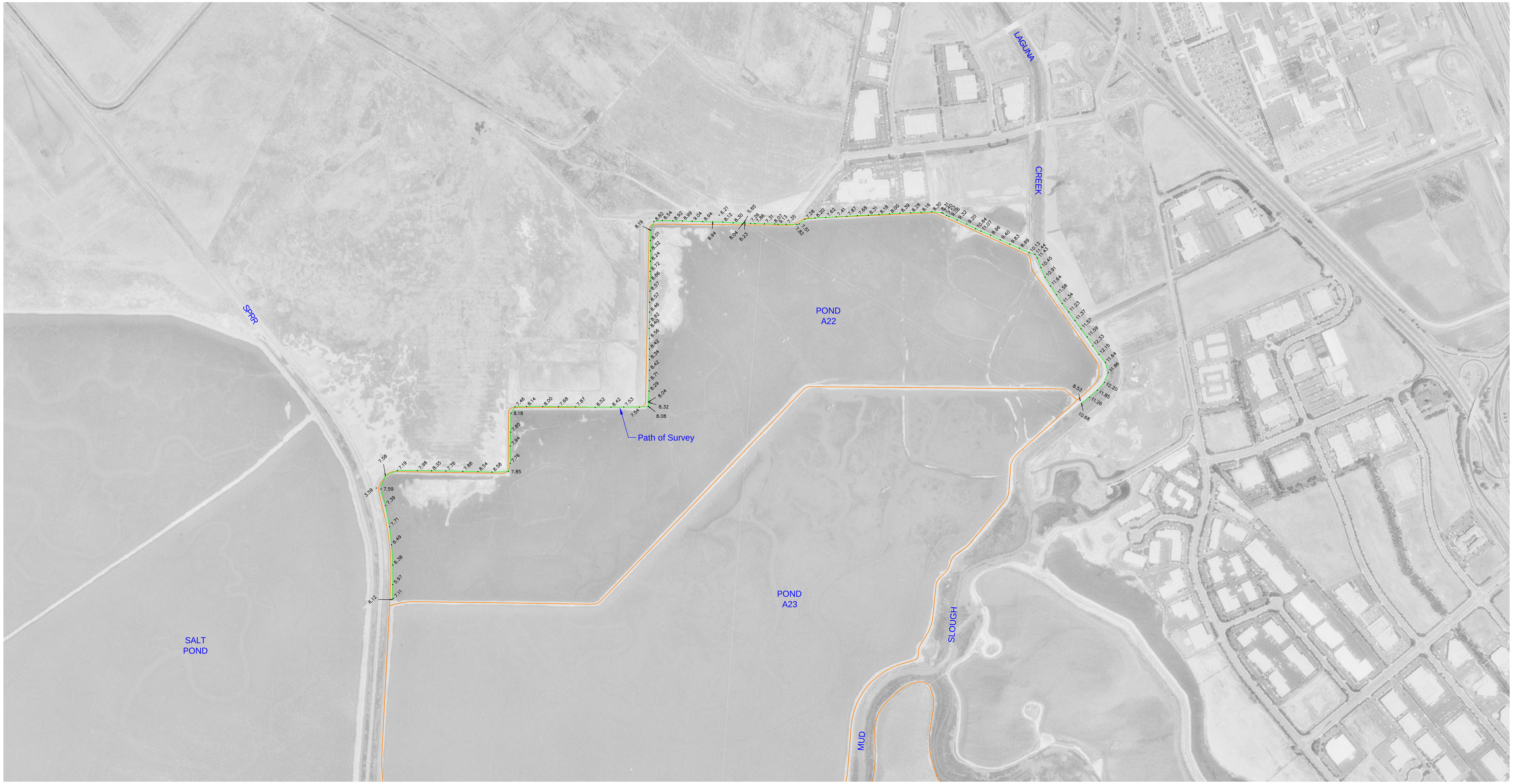
Salt Pond
Restoration Project

LEVEE EVALUATION

N

Alviso Complex (East)

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA



GRAPHIC SCALE

0 500

1" = 500'

Notes:

1. Elevations shown are in Feet, relative to NGVD 1929.
2. Waypoints shown are locations where cross-sections were measured (visual reconnaissance level).
3. GPS Survey performed by Tucker & Associates, Jan 2004.
4. Additional benchmarks established on site, not shown on Figure.

Salt Pond
Restoration Project

LEVEE EVALUATION

N

Alviso Complex
(Northeast)

MOFFATT & NICHOL ENGINEERS
WALNUT CREEK, CALIFORNIA

GRAPHIC SCALE

0 500

1" = 500'

Notes:

1. Elevations shown are in Feet, relative to NGVD 1929.
2. Waypoints shown are locations where cross-sections were measured (visual reconnaissance level).
3. GPS Survey performed by Tucker & Associates, Jan 2004.
4. Additional benchmarks established on site, not shown on Figure.

Salt Pond Restoration Project

LEVEE EVALUATION

N

West Bay Complex

MOFFATT & NICHOL ENGINEERS

WALNUT CREEK, CALIFORNIA

B. Field Data Forms

Alviso Pond Complex

Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"023"
Latitude	37.487784
Longitude	-121.959286
Pond Name	
Station from	
Station to	
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	
Photos Up-Station	
Comments	Gate

Alviso Pond Complex

Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"024"
Latitude	37.487528
Longitude	-121.964382
Pond Name	
Station from	64+00
Station to	78+00
Crest Elev	
Crest Width	12'
Crest Condition	grasses
Pond Side Slope Inclination	19 degrees
Pond Side Crest Height	4'
Pond Side Slope Erosion	minimal
Pond Side Slope Vegetation	grasses
Pond Side Toe Condition	12' wide bench
Pond Side Ditch	20' wide, 2' deep, mud
Land Side Slope Inclination	20 degrees
Land Side Crest Height	5'
Land Side Slope Erosion	minimal
Land Side Slope Vegetation	grasses
Land Side Toe Condition	ditch at toe
Land Side Ditch	8' wide, 3' deep, grass & pickleweed, .5' to 1' vertical face adjacent to levee, remnant fence on far side of ditch
Levee Material	probable Bay Mud
Seepage	none
Photos Back-Station	131, 133
Photos Up-Station	132, 134
Comments	4" diameter active burrows in levee crest

Alviso Pond Complex

Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"030"
Latitude	37.484911
Longitude	-121.964388
Pond Name	
Station from	54+00
Station to	63+00
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	3'
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	
Photos Up-Station	
Comments	fill extends into pond 75' bet Sta 52+00 and 54+00, no ditch on pond side over this range, levee crest width down to 6' in some areas

Alviso Pond Complex

Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"031"
Latitude	37.482444
Longitude	-121.965620
Pond Name	
Station from	
Station to	
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	
Photos Up-Station	
Comments	end of slump area between Gate and this Way Point, approx. 1' below balance of levee further to the north, levee paved w/gravel

Alviso Pond Complex
Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"032"
Latitude	37.482461
Longitude	-121.966869
Pond Name	A22
Station from	32+00
Station to	45+00
Crest Elev	
Crest Width	15'
Crest Condition	3/8"-gravel
Pond Side Slope Inclination	65 degrees
Pond Side Crest Height	3'
Pond Side Slope Erosion	heavy
Pond Side Slope Vegetation	sparse (eroding
Pond Side Toe Condition	5 degrees to ditch
Pond Side Ditch	20' from toe
Land Side Slope Inclination	42 degrees
Land Side Crest Height	3'
Land Side Slope Erosion	moderate w/6' scarp
Land Side Slope Vegetation	grasses w/some pickleweed
Land Side Toe Condition	ditch at toe
Land Side Ditch	6' wide x 1' deep
Levee Material	Sandy Silt
Seepage	no signs
Photos Back-Station	138, 139
Photos Up-Station	140, 141
Comments	

Alviso Pond Complex
Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"033"
Latitude	37.481539
Longitude	-121.969094
Pond Name	A22
Station from	25+00
Station to	31+00
Crest Elev	
Crest Width	16'
Crest Condition	3/8"-gravel
Pond Side Slope Inclination	40 degrees
Pond Side Crest Height	3'
Pond Side Slope Erosion	moderate to heavy
Pond Side Slope Vegetation	sparse/eroded
Pond Side Toe Condition	Silty Sand, 5 degrees to ditch
Pond Side Ditch	15' from toe, 15' wide ditch
Land Side Slope Inclination	34 degrees
Land Side Crest Height	3'
Land Side Slope Erosion	minimal
Land Side Slope Vegetation	grasses/pickleweed
Land Side Toe Condition	ditch at toe
Land Side Ditch	6' wide, 1' deep
Levee Material	Sandy Silt
Seepage	None
Photos Back-Station	142, 143
Photos Up-Station	144, 145
Comments	

Alviso Pond Complex
Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"035"
Latitude	37.480591
Longitude	-121.970668
Pond Name	A22
Station from	13+00
Station to	25+00
Crest Elev	
Crest Width	20'
Crest Condition	3/8"-gravel
Pond Side Slope Inclination	26 degrees
Pond Side Crest Height	3'
Pond Side Slope Erosion	minimal
Pond Side Slope Vegetation	minimal
Pond Side Toe Condition	5 degrees, barren
Pond Side Ditch	20' to ditch, 15' wide
Land Side Slope Inclination	24 degrees
Land Side Crest Height	2'
Land Side Slope Erosion	minimal
Land Side Slope Vegetation	grasses/pickleweed
Land Side Toe Condition	ditch at toe
Land Side Ditch	8' wide, 2' deep
Levee Material	Sandy Silt
Seepage	None
Photos Back-Station	146, 147
Photos Up-Station	148, 149
Comments	access ramp into pond @ WP34 (photo 148)

Alviso Pond Complex
Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"038"
Latitude	37.478225
Longitude	-121.973122
Pond Name	A22
Station from	0+00
Station to	13+00
Crest Elev	
Crest Width	15'
Crest Condition	3/8"-gravel
Pond Side Slope Inclination	34 degrees
Pond Side Crest Height	3'
Pond Side Slope Erosion	heavy (wind wave)
Pond Side Slope Vegetation	none
Pond Side Toe Condition	5 degrees to ditch
Pond Side Ditch	8' to toe, standing water
Land Side Slope Inclination	35 degrees
Land Side Crest Height	3'
Land Side Slope Erosion	minimal
Land Side Slope Vegetation	pickleweed
Land Side Toe Condition	tidal slough at toe
Land Side Ditch	tidal slough, 8-10' wide
Levee Material	Sandy Silt
Seepage	none
Photos Back-Station	155, 156
Photos Up-Station	157, 158
Comments	Splash berm on pond side

Alviso Pond Complex

Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"040"
Latitude	37.429621
Longitude	-121.978502
Pond Name	
Station from	8+00
Station to	
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	Pan [159-167]
Photos Up-Station	
Comments	

Alviso Pond Complex

Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"041"
Latitude	37.429240
Longitude	-121.980614
Pond Name	
Station from	~0+00
Station to	
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	
Photos Up-Station	168 and 169
Comments	Approx. location of intersection with existing Flood Control levee

Alviso Pond Complex

Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"043"
Latitude	37.427529
Longitude	-121.975382
Pond Name	
Station from	North of Catherine St.
Station to	between Gold & State/Liberty
Crest Elev	High fill ground to the east, low to the west
Crest Width	
Crest Condition	storm drain crosses here
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	170 to 174
Photos Up-Station	
Comments	

Alviso Pond Complex

Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"044"
Latitude	37.432484
Longitude	-121.970246
Pond Name	
Station from	State and Pacific (N End)
Station to	
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	175-178
Photos Up-Station	
Comments	

Alviso Pond Complex
Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"045"
Latitude	37.432839
Longitude	-121.966698
Pond Name	
Station from	State at Spreckels
Station to	83+00
Crest Elev	
Crest Width	
Crest Condition	asphalt road
Pond Side Slope Inclination	19 degrees towards marsh
Pond Side Crest Height	
Pond Side Slope Erosion	minimal
Pond Side Slope Vegetation	grasses
Pond Side Toe Condition	
Pond Side Ditch	none (marsh)
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	179-180
Photos Up-Station	
Comments	

Alviso Pond Complex
Site Visit November 20, 2003

Date	11/20/2003
Persons	Ed and Dilip
Way Point No.	"046"
Latitude	37.417048
Longitude	-121.987065
Pond Name	
Station from	Pond #85
Station to	Guadalupe St. at San Tomas Aquino Ct.
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	181-183
Photos Up-Station	
Comments	

Alviso Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"076"
Latitude	37.43584
Longitude	-122.09869
Pond Name	Charleston Slough
Station from	
Station to	
Crest Elev	
Crest Width	16'
Crest Condition	paved
Pond Side Slope Inclination	11deg
Pond Side Crest Height	6'
Pond Side Slope Erosion	none
Pond Side Slope Vegetation	some, pickleweed
Pond Side Toe Condition	pond
Pond Side Ditch	pond
Land Side Slope Inclination	14 deg
Land Side Crest Height	13'
Land Side Slope Erosion	none
Land Side Slope Vegetation	pickleweed
Land Side Toe Condition	pond
Land Side Ditch	pond
Levee Material	clay
Seepage	none
Photos Back-Station	1
Photos Up-Station	2
Comments	

Alviso Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"077"
Latitude	37.43506
Longitude	-122.09683
Pond Name	A1
Station from	6+00
Station to	10+00
Crest Elev	
Crest Width	16'
Crest Condition	paved
Pond Side Slope Inclination	upper 30': 6 deg lower 25': 10 deg
Pond Side Crest Height	8'
Pond Side Slope Erosion	none
Pond Side Slope Vegetation	pickleweed
Pond Side Toe Condition	pond
Pond Side Ditch	pond
Land Side Slope Inclination	22 deg
Land Side Crest Height	16'
Land Side Slope Erosion	none
Land Side Slope Vegetation	marsh
Land Side Toe Condition	
Land Side Ditch	none
Levee Material	clay
Seepage	none
Photos Back-Station	3, 5, 6
Photos Up-Station	4
Comments	photos 7, 8, 9: pond to southwest

Alviso Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"078"
Latitude	37.43551
Longitude	-122.09216
Pond Name	A1
Station from	15+00
Station to	44+00
Crest Elev	
Crest Width	25'
Crest Condition	paved, Bay Trail
Pond Side Slope Inclination	upper: 25 deg; 30' wide bench lower: 11 deg
Pond Side Crest Height	upper: 13', lower: 3'
Pond Side Slope Erosion	none
Pond Side Slope Vegetation	pickleweed on lower slope
Pond Side Toe Condition	pond
Pond Side Ditch	pond
Land Side Slope Inclination	none, landfill
Land Side Crest Height	none, landfill
Land Side Slope Erosion	none, landfill
Land Side Slope Vegetation	none, landfill
Land Side Toe Condition	none, landfill
Land Side Ditch	none
Levee Material	clay
Seepage	none
Photos Back-Station	10
Photos Up-Station	11
Comments	

Alviso Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"079"
Latitude	37.43479
Longitude	-122.08584
Pond Name	Permanente Creek
Station from	
Station to	
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	
Photos Up-Station	
Comments	panorama photos 12 to 15

Alviso Pond Complex

Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"080"
Latitude	37.43557
Longitude	-122.07615
Pond Name	A2W
Station from	0+00
Station to	37+00
Crest Elev	
Crest Width	30'
Crest Condition	paved upper, gravel lower
Pond Side Slope Inclination	landfill slope 25 deg, 30' bench, near vertical scarp at lower slope
Pond Side Crest Height	14'
Pond Side Slope Erosion	lower slope eroding
Pond Side Slope Vegetation	none
Pond Side Toe Condition	pond
Pond Side Ditch	pond
Land Side Slope Inclination	landfill
Land Side Crest Height	landfill
Land Side Slope Erosion	landfill
Land Side Slope Vegetation	landfill
Land Side Toe Condition	landfill
Land Side Ditch	landfill
Levee Material	clay/some debris
Seepage	none
Photos Back-Station	16
Photos Up-Station	17
Comments	

Alviso Pond Complex

Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"081"
Latitude	37.43568
Longitude	-122.07125
Pond Name	
Station from	38+00
Station to	48+00
Crest Elev	
Crest Width	30' at lower bench
Crest Condition	gravel lower
Pond Side Slope Inclination	near vertical at lower, 13 deg at berm
Pond Side Crest Height	bench:2', berm: 5'
Pond Side Slope Erosion	active
Pond Side Slope Vegetation	pickleweed on lower, grasses at berm
Pond Side Toe Condition	pond
Pond Side Ditch	pond
Land Side Slope Inclination	8 deg to marsh
Land Side Crest Height	4'
Land Side Slope Erosion	none
Land Side Slope Vegetation	marsh
Land Side Toe Condition	none/marsh
Land Side Ditch	none
Levee Material	clay
Seepage	none
Photos Back-Station	18
Photos Up-Station	19
Comments	panorama photos 20-23 (from 19 to 18)

Alviso Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"082"
Latitude	37.43403
Longitude	-122.06377
Pond Name	A2E
Station from	0+00
Station to	17+00
Crest Elev	
Crest Width	15' + 20' = 35'
Crest Condition	clay, slick, low, cast-up
Pond Side Slope Inclination	8 deg
Pond Side Crest Height	3'
Pond Side Slope Erosion	none
Pond Side Slope Vegetation	barren
Pond Side Toe Condition	pickleweed, 10' wide, 20:1 slope (cast-up from pond slope)
Pond Side Ditch	none
Land Side Slope Inclination	11 deg
Land Side Crest Height	5'
Land Side Slope Erosion	none
Land Side Slope Vegetation	pickleweed in upper, grass towards ponded water
Land Side Toe Condition	ponded
Land Side Ditch	ponded
Levee Material	clay
Seepage	none
Photos Back-Station	24
Photos Up-Station	25
Comments	pond on both sides, consider moving FCL behind ponded water

Alviso Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"083"
Latitude	37.43460
Longitude	-122.05838
Pond Name	A2E
Station from	17+00
Station to	new station line
Crest Elev	
Crest Width	12'
Crest Condition	slick, clay
Pond Side Slope Inclination	5 deg, 20' wide side slope
Pond Side Crest Height	3'
Pond Side Slope Erosion	active
Pond Side Slope Vegetation	pickleweed near toe only
Pond Side Toe Condition	2' vertical to water
Pond Side Ditch	none visible
Land Side Slope Inclination	10 deg
Land Side Crest Height	4'
Land Side Slope Erosion	none visible
Land Side Slope Vegetation	pickleweed
Land Side Toe Condition	dessicated mud
Land Side Ditch	none visible
Levee Material	clay
Seepage	none
Photos Back-Station	26
Photos Up-Station	27
Comments	

Alviso Pond Complex

Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"084"
Latitude	37.43431
Longitude	-122.05326
Pond Name	A2E
Station from	
Station to	
Crest Elev	
Crest Width	15'
Crest Condition	slick, clay
Pond Side Slope Inclination	10 deg
Pond Side Crest Height	3'
Pond Side Slope Erosion	moderate
Pond Side Slope Vegetation	upper: barren, lower: pickleweed
Pond Side Toe Condition	pond
Pond Side Ditch	none observed
Land Side Slope Inclination	13 deg
Land Side Crest Height	4'
Land Side Slope Erosion	none
Land Side Slope Vegetation	pickleweed/grasses
Land Side Toe Condition	mudflat
Land Side Ditch	none
Levee Material	clay
Seepage	none
Photos Back-Station	28
Photos Up-Station	29
Comments	

Alviso Pond Complex

Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"085"
Latitude	37.42740
Longitude	-122.04434
Pond Name	
Station from	
Station to	
Crest Elev	
Crest Width	
Crest Condition	
Pond Side Slope Inclination	
Pond Side Crest Height	
Pond Side Slope Erosion	
Pond Side Slope Vegetation	
Pond Side Toe Condition	
Pond Side Ditch	
Land Side Slope Inclination	
Land Side Crest Height	
Land Side Slope Erosion	
Land Side Slope Vegetation	
Land Side Toe Condition	
Land Side Ditch	
Levee Material	
Seepage	
Photos Back-Station	
Photos Up-Station	30
Comments	photo of ditches/channels

Alviso Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"086"
Latitude	37.42756
Longitude	-122.04266
Pond Name	A3W
Station from	70+00
Station to	94+00
Crest Elev	
Crest Width	30'
Crest Condition	trail, compacted clay
Pond Side Slope Inclination	16 deg
Pond Side Crest Height	3'
Pond Side Slope Erosion	near vertical
Pond Side Slope Vegetation	grass
Pond Side Toe Condition	pond at toe
Pond Side Ditch	none, pond
Land Side Slope Inclination	22 deg
Land Side Crest Height	5'
Land Side Slope Erosion	none
Land Side Slope Vegetation	grassy
Land Side Toe Condition	2' vertical to water
Land Side Ditch	drainage channel
Levee Material	clay
Seepage	
Photos Back-Station	31
Photos Up-Station	32
Comments	

Alviso Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"087"
Latitude	37.42730
Longitude	-122.04005
Pond Name	
Station from	94+00
Station to	110+00
Crest Elev	
Crest Width	25'
Crest Condition	trail, compacted clay
Pond Side Slope Inclination	vertical
Pond Side Crest Height	4' scarp
Pond Side Slope Erosion	active
Pond Side Slope Vegetation	none
Pond Side Toe Condition	pond
Pond Side Ditch	none, pond
Land Side Slope Inclination	35 deg
Land Side Crest Height	6'
Land Side Slope Erosion	none/some
Land Side Slope Vegetation	grasses
Land Side Toe Condition	drainage channel
Land Side Ditch	drainage channel
Levee Material	clay
Seepage	
Photos Back-Station	33
Photos Up-Station	34, 35
Comments	

Alviso Pond Complex

Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"088"
Latitude	37.43167
Longitude	-122.03053
Pond Name	
Station from	
Station to	
Crest Elev	
Crest Width	10'
Crest Condition	gravel
Pond Side Slope Inclination	27 deg
Pond Side Crest Height	6.5'
Pond Side Slope Erosion	none - significant burrowing
Pond Side Slope Vegetation	grasses
Pond Side Toe Condition	20' to waterline, mudflat
Pond Side Ditch	
Land Side Slope Inclination	29 deg
Land Side Crest Height	4'
Land Side Slope Erosion	riprap (concrete debris)
Land Side Slope Vegetation	none - riprap
Land Side Toe Condition	oxidation pond
Land Side Ditch	oxidation pond
Levee Material	unknown
Seepage	
Photos Back-Station	36
Photos Up-Station	37
Comments	panorama photos 38 to 41 (from 37 to 36)

West Bay Pond Complex

Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"089"
Latitude	37.48540
Longitude	-122.17195
Pond Name	
Station from	
Station to	
Crest Elev	
Crest Width	6' wide berm, 1' high + 16' wide levee
Crest Condition	
Pond Side Slope Inclination	1:15 for 20', then flat
Pond Side Crest Height	3'
Pond Side Slope Erosion	severe, near vertical
Pond Side Slope Vegetation	bare, eroding
Pond Side Toe Condition	salt pan/flat
Pond Side Ditch	none
Land Side Slope Inclination	32 deg to berm, 36 deg to ditch
Land Side Crest Height	3' bench to crest, 4' water to bench
Land Side Slope Erosion	mild
Land Side Slope Vegetation	grass
Land Side Toe Condition	F.C. channel
Land Side Ditch	F.C. channel
Levee Material	clay
Seepage	none
Photos Back-Station	42 pond, 43 channel
Photos Up-Station	44 pond, 45 channel
Comments	

West Bay Pond Complex

Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"090"
Latitude	37.48415
Longitude	-122.16598
Pond Name	
Station from	
Station to	
Crest Elev	
Crest Width	45'
Crest Condition	grass
Pond Side Slope Inclination	23 deg
Pond Side Crest Height	2'
Pond Side Slope Erosion	active
Pond Side Slope Vegetation	bare
Pond Side Toe Condition	mudflat, 20:1, 25' wide
Pond Side Ditch	pond, 1' deep
Land Side Slope Inclination	34 deg
Land Side Crest Height	2'
Land Side Slope Erosion	moderate
Land Side Slope Vegetation	sparse
Land Side Toe Condition	mudflat (dried pond)
Land Side Ditch	none
Levee Material	clay
Seepage	none
Photos Back-Station	46 pond, 48 "landside"
Photos Up-Station	47 pond, 49 "landside", panorama 50-55
Comments	possible alternate alignment to create detention basin on "landside"

West Bay Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"091"
Latitude	37.48266
Longitude	-122.16076
Pond Name	
Station from	
Station to	
Crest Elev	
Crest Width	>50' (~55' to trail)
Crest Condition	
Pond Side Slope Inclination	12 deg
Pond Side Crest Height	3'
Pond Side Slope Erosion	moderate
Pond Side Slope Vegetation	sparse
Pond Side Toe Condition	8' wide mudflat to channel, 20:1 slope
Pond Side Ditch	none - channel 1' deep, salt on channel edges
Land Side Slope Inclination	n/a
Land Side Crest Height	n/a
Land Side Slope Erosion	n/a
Land Side Slope Vegetation	n/a
Land Side Toe Condition	n/a
Land Side Ditch	n/a
Levee Material	clay
Seepage	none
Photos Back-Station	56
Photos Up-Station	57
Comments	12" HDPE pipeline along shore, extends to flood slough.

West Bay Pond Complex
Site Visit December 16, 2003

Date	12/16/2003
Persons	Ed and Dilip
Way Point No.	"092"
Latitude	37.48813
Longitude	-122.14269
Pond Name	
Station from	
Station to	
Crest Elev	
Crest Width	20' + berm 8' wide x 1' high
Crest Condition	trail
Pond Side Slope Inclination	14 deg
Pond Side Crest Height	4'
Pond Side Slope Erosion	active
Pond Side Slope Vegetation	none
Pond Side Toe Condition	12' wide mudflat to channel
Pond Side Ditch	channel
Land Side Slope Inclination	41 deg
Land Side Crest Height	4'
Land Side Slope Erosion	none
Land Side Slope Vegetation	grasses along slope
Land Side Toe Condition	
Land Side Ditch	Ravenswood Slough
Levee Material	clay
Seepage	none
Photos Back-Station	58 pond, 60 marsh (landside)
Photos Up-Station	59 pond, 61 marsh (landside)
Comments	

C. Pond Elevations

ALVISO PONDS - AREA, ELEVATION, TIDAL PRISM

No	Pond Number	Pond Area (ac)	Pond Elev (NGVD)	Distance to MHHW (ft)	Volume To MHHW (AF)
1	B2	28	0.5	4.2	116
2	B2	108	0.5	4.2	452
3	B2	51	0.5	4.2	214
4	B1	158	-1.3	6.0	945
5	A23	180	1.2	3.0	532
6	A23	275	1.2	3.0	813
7	A22	89	3.0	1.2	104
8	A22	184	3.0	1.2	213
9	A21	142	2.3	1.8	256
10	A20	67	1.8	2.3	156
11	A19	276	1.8	2.4	649
13	A17	136	1.1	3.0	406
14	A16	241	0.6	3.6	856
15	A15	252	0.7	3.4	866
16	A14	351	-0.1	4.8	1,671
17	A13	283	-1.1	5.2	1,470
18	A12	314	-2.0	6.1	1,902
19	A11	268	-1.8	6.5	1,738
20	A10	253	-0.8	5.5	1,402
21	A9	372	0.5	4.2	1,562
22	A8-South	175	-0.5	5.2	912
23	A8	444	-3.4	8.1	3,592
24	A7	269	-0.8	5.5	1,474
25	A5	661	-1.9	6.6	4,391
26	A3W	606	-3.2	7.9	4,780
27	A3N	185	-1.5	6.2	1,140
28	A2W	457	-0.9	5.6	2,566
29	A2E	315	-3.0	7.7	2,424
30	A1	285	-1.9	6.6	1,892
Subtotal		**7,425.0**			**39,496**

March 2003 acquisition area only. Ponds A4 and A18 not shown
Source : Siegel & Bachand, 2002

BAUMBERG PONDS - AREA, ELEVATION, TIDAL PRISM

No	Pond Number	Pond Area (ac)	Pond Elev (NGVD)	Distance to MHHW (ft)	Volume To MHHW (AF)
31	1	297	2.2	1.8	541
32	1c	65	3.7	0.3	18
33	2	692	2.1	1.9	1,294
34	2c	32	2.7	1.2	40
35	3c	180	2.9	1.0	187
36	4	202	2.9	1.1	216
37	4c	168	3.2	0.8	138
38	5	172	2.4	1.6	275
39	5c	96	3.0	1.0	99
40	6	183	2.4	1.6	299
41	6a	329	1.1	2.9	957
42	6b	293	1.7	2.3	683
43	6c	85	2.8	1.2	103
44	7	217	2.5	1.5	319
45	8	156	2.8	1.2	192
46	8a	310	4.0	0.0	0
47	8-middle	42	2.8	1.2	52
48	8-north	31	2.8	1.2	39
49	9	386	2.8	1.2	444
50	10	269	2.3	1.6	441
51	11	128	3.0	1.0	124
52	12	117	2.9	1.1	128
53	13	134	3.3	0.7	91
54	14	172	3.7	0.3	46
Subtotal		4,756			6,725

March 2003 acquisition area only.
Source : Siegel & Bachand, 2002

WEST BAY PONDS - AREA, ELEVATION, TIDAL PRISM

ID	Pond Number	Pond Area1 (ac)	Pond Elev2 (NGVD)	Pond Distance to mhhw4 (ft)	Pond Void Space to mhhw5 (AF)
94	1	446	2.0	2.3	1,030
95	2	141	1.9	2.5	346
96	3	296	2.1	2.3	679
97	4	307	2.1	2.2	681
98	5	35	2.5	1.9	67
110	s5	38	2.5	1.6	61
111	sf2	239	2.1	1.9	460
Subtotal		1,503			3,324

March 2003 acquisition area only.
Source : Siegel & Bachand, 2002

D. Geotechnical Scope of Work for Subsequent Assessment

A California Corporation
Specializing in Geotechnical Engineering
Hultgren-Tillis Engineers

April 26, 2004
Project No. 561.01

Moffatt & Nichol Engineers
2001 N. Main St., Suite 360
Walnut Creek, California 94596

Attention: Dr. Dilip Trivedi

Task C - Develop Scope for Subsequent Geotechnical Assessment
Urban Flood Management Requirements - Draft (March 2004)
South Bay Salt Pond Restoration Project

Gentlemen:

INTRODUCTION

This letter presents a work plan and cost estimate for a subsequent geotechnical investigation and embankment design of the planned urban flood control levees for Alviso and West Bay units. The intent of this letter is to layout and explain how the geotechnical engineer may approach the investigation and design of the levees. This letter does not propose to establish requirements for the investigation. Rather, it presents an approach that one firm (Hultgren - Tillis Engineers) considers viable.

Changing the land use of the salt ponds to a tidal marsh will subject the landward-most perimeter levees to tidal and flood stages at the margin of the bay. In the past, these perimeter levees retained ponded water that had water surface elevations that were controlled by salt pond operations. The urban levee project involves raising the existing levees to reach specified elevations to provide tidal flood protection. The enlarged levee embankments will extend into the former salt ponds in most locations. Recompacting the existing levee fills will likely be part of the final design, disrupting potential seepage paths within the former salt pond levees and creating strong, erosion resistant perimeter levees.

The site is located along the southern fringe of San Francisco Bay. Weak clays and silts (Bay Mud) underlie the urban levee alignments and extend to elevations ranging from about –5 to –15 feet relative to NGVD-1929 in the majority of the Alviso and West Bay units. Deeper layers of Bay Mud occur locally at existing creeks that create buried, Bay-Mud-filled channels or fingers extending from the bay towards the land. The Bay Mud layer can contain intermediate sand layers or lenses that may be susceptible to liquefaction.

The Bay Mud typically overlies alluvial deposits, consisting primarily of medium stiff to stiff clays and loose to dense sands. Loose to medium dense sands below the groundwater table may be at risk of liquefying during a large earthquake. Assessing the liquefaction risk, estimating deformation and evaluating the resulting potential change in risk of overtopping/breaching will need to be addressed.

Additional existing geotechnical data may be available for many areas near various sections of the planned urban levees in addition to that which was collected as part of this feasibility level assessment. Collection of the additional data will be part of subsequent preliminary design phases and should be accomplished before initiating new exploration.

Earth embankments will be used to raise and broaden the existing levees. In many areas, the underlying Bay Mud is too weak to allow the levees to be constructed to their final heights without special considerations. The critical stability concern for new, raised and/or widened levees will be stability of the embankment during and immediately after construction. Stability failures can occur in soft Bay Mud foundation materials if levee embankment fills are placed too high over a short period of time. The overloaded ground beneath the levee fill will sink and the adjacent mudflat heave up. This type of failure is common when filling over soft ground to rapidly. It is commonly referred to as a "mudwave".

Special considerations may include placing the fill in stages, wide stability berms, wick drains, and/or removal of the Bay Mud from beneath the new embankment footprint. For much of the urban levee alignment, the levees may need to be constructed in two stages. The time between filling stages will allow the underlying clays to consolidate and gain strength. Wide stability berms placed at the levee toe can be used to improve stability. These berms offset the tendency of the toe to heave, creating a buttress or weight at the levee toe, increasing its overall stability. Using flat slopes can also improve the stability of slopes. Wick drains through the deeper sections of Bay Mud may be used to accelerate consolidation and corresponding strength gain. However, one must assess the increased risk of underseepage that a drainage blanket would cause. In areas where the Bay Mud is thin, excavating the weak clay and replacing with compacted fill may be the preferred scheme.

In some areas, the existing levees are quite low and new fill thickness will create substantial loading on the urban side of the levee. The stability of the urban side of the levee will also need to be addressed.

An assessment of seepage and its control should be a part of the geotechnical investigation. The investigation should consider seepage that occurs through the levee embankment and that which flows through aquifers or other formations beneath the levees.

Where the urban levee will be adjacent to an existing landfill, placement and design of the urban levee will need to be coordinated with the landfill owners and regulators. Issues of seepage, settlement and stability may need to be reviewed by water quality and solid waste regulating agencies.

EXPLORATION APPROACH

Key factors that need to be addressed by geotechnical exploration for the urban levee include:

1. General stratigraphy,
2. Strength and compressibility of the weak clays (Bay Mud),
3. Liquefaction potential,
4. Under seepage.

To be cost effective, most of the exploration will need to be done from the existing levees. Geotechnical data to characterize the general stratigraphy and to assess both liquefaction potential and seepage can be fully developed from borings and cone penetration testing accomplished from the existing levee crests. The strength and compressibility of the Bay Mud can only be partially evaluated from exploration conducted through the existing levees. Undisturbed samples will need to be taken beyond the toe of the existing levee, beneath the planned expanded footprint of the urban levee.

For the purpose of this cost estimate, we are assuming that approximately 65,000 linear feet (~20 km) of urban levee will need to be upgraded and that the total length needs exploration.

Our preliminary recommendation is that exploration points be spaced at about 650 feet (200 meters). We believe that effective exploration along the existing levee alignment can be achieved by conducting cone penetration tests (CPT's) at two thirds of the exploration locations and by drilling rotary wash borings at one third of the locations. Borings and CPT's should be drilled/pushed to about 50 feet below the existing levee crests. The CPT truck rig has a gross vehicle weight of 20 tons. We assume that a 20-ton truck can traffic the existing levee crests during the summer and early autumn. However, the generally unimproved surface will likely not be trafficable for heavy equipment from November to April. We recommend that exploration be scheduled between June and September. Small bridges must be crossed to access some areas. The load capacity of the bridges should be checked before exploration.

To assess the undrained shear strength in the Bay Mud beyond the existing levee footprint, undisturbed samples will need to be collected from beneath the pond areas. This is traditionally done with a small drill rig mounted on a raft or small barge. A raft/barge would be lifted into and out of the ponds by a crane. The crane will pick the drill rig up and set it on the raft/barge.

Our cost estimate uses this approach. An alternate approach would be to attach an Osterberg sampler (a sampler for taking undisturbed samples) to the boom of a crane or long reach excavator. The crane/excavator would push the sampler to a desired depth for the sample and the sample collected. Samples will only need to be collected in the soft clay (Bay Mud) which we estimate in the range of 5 to 15 feet thick along most of the urban levee alignment. The balance of the needed data will be collected from exploration conducted through the existing levees. If exploration from the levee indicates the base of the Bay Mud is quite shallow, the levee designers may conclude that it will be most economical to excavate the Bay Mud from beneath the planned widened section of the levee embankment. If this is the case, gathering undisturbed Bay Mud samples from beneath the ponds for strength and compressibility testing will be less important and sample collection may not be needed in these areas. Overall, we suspect that the cost of sample collection from beneath the ponds can be less than we have allowed in our fee estimate.

LABORATORY TESTING

Strength characterization will consist of unconsolidated-undrained triaxial strength tests (UU tests) in Bay Mud, both beneath the existing levee and beneath the pond where new embankment fill will be placed. This testing is used to assess the current strength under existing loading conditions. Consolidated-drained triaxial strength tests (CU tests) in Bay Mud can be used along with the UU tests results in two-stage analyses to assess the margin of safety when placing new fill over existing slopes underlain by soft to medium stiff clays. Consolidation tests will measure the compressibility and rate of consolidation of the Bay Mud. These values can be used to predict how fast the Bay Mud will consolidate and gain strength. Other testing should include various classification testing, including sieve analyses, Atterberg limits, moisture and density measurements.

ENGINEERING ANALYSES

The geotechnical engineer would assemble and plot data collected from the subsurface exploration and laboratory testing programs together with that data which may be available from other sources. From this data, strength envelopes and compressibility characteristics will be assessed. Preliminary levee profiles would be plotted on cross-sections of the existing topography. Subsurface stratigraphy would be added to the sections. The stability of the proposed embankment sections would be checked for the immediately-after-construction condition. If a low factor of safety is indicated, additional consideration should be given on how best to achieve the design crest elevation. These may include: (a) removing the Bay Mud from beneath the planned embankments where the Bay Mud is thin; (b) placing new levee embankment fill in stages and allowing strength gain before placing a subsequent stage; (c) using wick drains to accelerate consolidation and strength; and (d) placing wide stability berms to buttress the new embankments.

Settlement analyses would be made to predict how much the fills will settle and to provide a basis for estimating how much the levee may need to be raised to maintain the design crest elevation as the levee settles.

The engineer will need to assess the existing levees and make recommendations as to whether sections of the levees should be excavated and recompacted.

The potential for seepage through the embankments would be evaluated. Potential for seepage beneath the embankments will also be considered. If adverse seepage is predicted, the engineer would develop schemed for controlling or preventing the seepage.

The engineer will evaluate the seismic risk for the levees. This will include assessing ground motions, checking liquefaction potential and estimating embankment deformation from a strong seismic event. The engineer will evaluate the potential impact that liquefaction and/or deformation may have on the flood protection reliability of the levees.

The engineers' conclusions and recommendations would be submitted in a written report together with the results of the field exploration and laboratory testing programs.

ASSUMPTIONS USED AS THE BASIS FOR EXPLORATION AND TESTING COST ESTIMATES

Drilling permits will need to be taken out in each county. Fees and inspection rules vary. We have included an allowance of $3,000 for permit and inspection fees.

CPT's are usually priced on a footage basis. Sixty-six CPT's pushed to an average depth of 50 feet indicates 3,300 linear feet of CPT. A cost per foot for CPTs of $9.00 was used, which includes collecting the data, grouting the hole and reducing the data. We assumed 300 linear feet per day for 11 days of per diem for two people (22 man-days). We assumed 18 CPT rig and crew hours for mobilizations/demobilizations, including resupplying the rig on weekends. We have assumed a 15 percent markup by the retaining geotechnical engineering firm on the total CPT costs.

Exploratory borings are usually priced on an hourly basis. Thirty-three borings would be drilled from existing levees to average depths of 50 feet. An average production rate of two borings per 8-hour shift extends to 132 drilling hours. We used an operated drill rig cost of $165 per hour. We assumed 24 drill rig and crew hours for mobilizations/demobilizations, including resupplying the drill rig on weekends. We have assumed a 20 percent markup by the retaining geotechnical engineering firm on the total drilling costs from the levee crests. The 5 percent difference in mark ups between CPT and drilling is to allow for brass liner usage and consumable materials provided by the geotechnical engineering firm.

To collect undisturbed samples beneath the ponds, we assumed that two boring locations could be sampled per 8-hour day. For 33 locations, this would be 17 days. Exploration below each pond would include lifting the raft/barge into the pond, lifting the small drill rig onto the raft/barge, recovering the samples, moving to another location within the pond and sampling there also, and then lifting the equipment from the pond. An hourly rate of $350 is used for the drilling subcontractor, including the costs of raft/barge, small drill rig, crane and three-man crew. A single lump sump price of $5,000 was used for mobilization/demobilization and resupplying the raft/barge-mounted drill rig working in the ponds. We have assumed a 15 percent markup by the retaining geotechnical engineering firm on the total drilling costs within the ponds.

We used Cooper Testing Laboratory's published Schedule of Charges for soil mechanics laboratory testing. We budgeted 132 unconsolidated-undrained triaxial strength tests in Bay Mud, nine consolidated-drained triaxial strength tests (three series of three points each) in Bay Mud, 20 consolidation tests and 10 pin-hole tests. Other testing will include sieve analyses, Atterberg limits, moisture and density measurements. Again, we used a 15 percent markup by the retaining geotechnical engineering firm.

If the entire 65,000 linear feet of levee alignment is authorized for investigation at one time, we estimate that the cost for the investigation and development of geotechnical design criteria would be about $483,000.

SUBDIVIDING THE GEOTECHNICAL INVESTIGATION

Our base estimate of $483,000 assumes that all of the levees are investigated and analyzed as one project. The project could be subdivided into several projects consisting of one or more ponds. We have supplemented our fee estimate with an approximation of what the fee might be if done as several independent projects. In doing so, we assumed that 5 percent of the overall project's field exploration and laboratory testing costs and that 30 percent of the overall project's project management, engineering, report preparation and consultation costs would reoccur as lump sums in each subdivided project. The balance (95 percent of exploration and laboratory and 70 percent of the project management, engineering, report preparation and consultation) would be divided equally into the number of subdivided projects. Using this approach, we estimate the costs of the geotechnical investigation would be as follows:

Number of projects the overall project is subdivided into:	Cost per subdivided project	Combined cost of the subdivided projects
1	$483,000	$483,000
2	$282,000	$564,000
3	$215,000	$645,000
4	$182,000	$728,000
5	$162,000	$810,000
6	$148,000	$888,000
7	$139,000	$973,000
8	$131,000	$1,048,000
9	$126,000	$1,134,000
10	$121,000	$1,210,000

Should you require further information, please call.

Sincerely yours,

Hultgren-Tillis Engineers

Edwin M. Hultgren
Geotechnical Engineer

EMH:la

Attachment: Cost Estimate Breakdown

Filename: 56101 Supplemental Geotechnical Assessment.doc

Cost Estimate for Subsequent Geotechnical Investigation
Urban Leve Flood Management Requirements - Draft (March 2004)
South Bay Salt Pond Restoration Project

	Quantity	Units	Rate	Geotechnical Engineer's Services	Outside/ Subcontracted Services	Mark Up on Outside Services	Extension	Task Totals
Project Management / Work Plan / Site Visits / Utility Clearances / Permits								**$27,330**
Technician/Word Process	20	hr	$50	$1,000				
Staff Engineer	40	hr	$80	$3,200				
Senior Engineer	80	hr	$120	$9,600				
Principal Engineer	60	hr	$160	$9,600				
Permits	10	ea	$300		$3,000	15%	$3,450	
Truck	40	hr	$12	$480				
Exploration								
Cone Penetration Tests								**$50,757**
CPT Mob/Demob	18	hr	$180		$3,240	15%	$3,726	
CPTs	3300	LF	$6.00		$19,800	15%	$22,770	
Grouting	3300	LF	$2.00		$6,600	15%	$7,590	
Interpretation	3300	LF	$1.00		$3,300	15%	$3,795	
Perdiem	22	m-da	$120		$2,640	15%	$3,036	
Excess Travel/Setup time	8	hr	$180		$1,440	15%	$1,656	
Senior Engineer	66	hr	$120	$7,920				
Truck	22	hr	$12	$264				
Deep Borings from Existing Levees								**$70,918**
Driller Mob/Demob	24	hr	$165		$3,960	20%	$4,752	
Drilling	132	hr	$165		$21,780	20%	$26,136	
CleanUp / CuttingsDisposal	17	hr	$165		$2,805	20%	$3,366	
Shelby Tubes	99	ea	$10		$990	20%	$1,188	
Grout Materials	99	bag	$10		$990	20%	$1,188	
Perdiem	51	m-da	$120		$6,120	20%	$7,344	
Truck	132	hr	$12	$1,584				
Staff Engineer (Logging)	166	hr	$80	$13,280				
Senior Engineer (Coord)	34	hr	$120	$4,080				
Principal Engineer (Coord, Edit)	50	hr	$160	$8,000				
Undisturbed Sampling Beneath Ponds								**$90,143**
Driller Mob/Demob	1	LS	$5,000		$5,000	15%	$5,750	
Drilling	132	hr	$350		$46,200	15%	$53,130	
Shelby Tubes	99	ea	$15		$1,485	15%	$1,708	
Grout Materials	33	bag	$10		$330	15%	$380	
Perdiem	68	m-da	$120		$8,160	15%	$9,384	
Truck	136	hr	$12	$1,632				
Staff Engineer (Logging)	132	hr	$80	$10,560				
Senior Engineer (Coord)	34	hr	$120	$4,080				
Principal Engineer (Coord, Edit)	22	hr	$160	$3,520				
Laboratory testing								**$41,187**
UU Strength	132	ea	$100		$13,200	15%	$15,180	
CU Strength	9	ea	$420		$3,780	15%	$4,347	
Consolidation	20	ea	$295		$5,900	15%	$6,785	
Shelby tube trimming	29	ea	$15		$435	15%	$500	
Pin-Hole	10	ea	$300		$3,000	15%	$3,450	
Gradation	40	ea	$100		$4,000	15%	$4,600	
Moisture/Density Shelby	50	ea	$24		$1,200	15%	$1,380	
Moisture/Density	100	ea	$18		$1,800	15%	$2,070	
Atterberg limits	20	ea	$125		$2,500	15%	$2,875	
Engineering Analyses								**$155,200**
Staff Engineer	400	hr	$80	$32,000				
Senior Engineer	600	hr	$120	$72,000				
Principal Engineer	320	hr	$160	$51,200				
Recommendations and Report Preparation								**$24,690**
Staff Engineer	80	hr	$80	$6,400				
Senior Engineer	60	hr	$120	$7,200				
Principal Engineer	40	hr	$160	$6,400				
Word Processor	80	hr	$50	$4,000				
Printing	30	ea	$20		$600	15%	$690	
Consultation/Meetings								**$23,168**
Senior Engineer	80	hr	$120	$9,600				
Principal Engineer	80	hr	$160	$12,800				
Truck	64	hr	$12	$768				
			TOTAL	**In House**	**Subs**	**Mark Up**		
Totals			**$483,394**	**$281,168**	**$174,255**	**$27,971**		

Key to Colors:	**Green**	=	If this project is to be subdivided into two or more projects, ninety five percent of the cost for exploration and laboratory testing can be proportionately adjusted for the length of levee under consideration. Five percent of the total exploration and testing costs would be reapplied to each project if the overall project is subdivided into two or more projects.
	Brown	=	If the project is divided into two or more projects, we recommend assuming that costs other than exploraton and laboratory testing be estimated as about 30 percent of the overall project's non-data costs (costs other than exploration or laboratory testing) be assigned to each subdivided project and that the other 70 percent of non-data costs be proportinaltey assigned to each subdivided project.