H. T. HARVEY & ASSOCIATES
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TASMAN CORRIDOR WETLAND MITIGATION SITE
YEAR-1 MONITORING

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EXECUTIVE SUMMARY

PERMIT NUMBERS

- United States Army Corps of Engineers (USACE) File #18881S
- Regional Water Quality Control Board (RWQCB) File #2188.07(JRW); Site No. 02-43-C0116
- California Department of Fish and Game (CDFG) Notification No. 0101-97

BACKGROUND

The Tasman Wetland Mitigation Site provides mitigation for impacts to U. S. Army Corps of Engineers (USACE) and California Department of Fish and Game (CDFG) jurisdictional areas resulting from the extension of the light rail line across several creeks and drainages, including Calabazas Creek, Stevens Creek, Sunnyvale East channel, and Sunnyvale West channel and from the construction of a levee and other features at the mitigation site. The impacts to USACE jurisdiction included 0.55 acres due to LRT construction and 0.18 acres due to the new levee for a total USACE impact of 0.73 acres, as described in the *Tasman Corridor Mitigation and Monitoring Plan* (H. T. Harvey & Associates 1997) (MMP). Impacts to RWQCB jurisdiction and associated mitigation were the same as that described for the USACE. The impacts to CDFG jurisdiction included 0.55 acres of wetland impacts plus 1.25 acres of ruderal and bare bank areas due to LRT construction. An additional 0.02 acres of CDFG jurisdiction was impacted at the mitigation site where sack concrete slope protection was placed around the culvert inlets on the inboard levee of the Guadalupe River for a total CDG impact of 1.82 acres.

To mitigate for these impacts, the USACE permit calls for the restoration of 3.2 acres of tidal wetlands at the Tasman Wetland Mitigation Site. This 3.2 acres includes 2.27 acres of ruderal uplands and 0.93 acres of non-tidal aquatic habitat (a brackish water pond). The restoration effort involves the conversion of 2.27 acres of uplands to tidal wetlands and the conversion of 1.0 acres of non-tidal aquatic habitat to tidal wetland habitat. This represents a 4.3:1 mitigation ratio (3.2 acres of wetland restoration: 0.73 acres of wetland impact). The RWQCB required the creation of a minimum of 1.46 acres of new jurisdictional wetlands to compensate for 0.73 acres of impacts to U.S. Army Corps of Engineers jurisdiction. However, due the time lapse between the LRT construction and mitigation implementation, the RWQCB subsequently required 2.22 acres of mitigation, essentially changing the mitigation ratio from 2:1 to 3:1. VTA's 3.2-acre site will accommodate this required acreage. The project resulted in 1.82 acres of impacts to CDFG jurisdictional areas and the MMP also called for the restoration of 4.8 acres of new CDFG jurisdictional area (wetlands and uplands on levee slopes). The total area of the mitigation site is 4.8 acres and meets the CDFG jurisdictional mitigation requirements (H. T. Harvey & Associates 1999).

The goal of the wetland mitigation is to restore a fully-tidal brackish marsh similar in structure and function to the adjoining habitat along the Guadalupe River. The wetland mitigation site will be monitored annually for a period of six years or until attainment of the success criteria described in the MMP. An as-built plan was previously prepared after the site was constructed (H. T. Harvey & Associates 1999). Monitoring results will be compared annually to determine whether the site is meeting its performance criteria. A wetland delineation will be conducted in Year-3. This Year-1 report summarizes the results of our biological monitoring as prescribed in the MMP.

RESULTS

The Year-1 monitoring results of the hydrologic monitoring (including sedimentation monitoring), wetland vegetation monitoring, and wildlife surveys are summarized in Table 1 and in the individual sections below.

Table 1. Tasman Wetland Mitigation Site Monitoring Requirements and Year-1 Results

MONITORING PARAMETER	METHOD	FINAL SUCCESS CRITERION	YEAR-1 SUCCESS CRITERIA	YEAR-1 CRITERIA MET/NOT MET	REMEDIAL ACTION REQUIRED
USACE Jurisdictional Area	Wetland delineation	1.46 acres	n/a	n/a	None
Hydrology	Topographic Surveys,	Target elevation of 4.5 ft NGVD by Year-6	None	n/a	None
	Water level datasondes	Slightly muted tides compared to Guadalupe marsh plain; tidal elevations within 0.5 ft of Guadalupe marsh plain	Slightly muted tides compared to Guadalupe marsh plain; tidal elevations within 0.5 ft of Guadalupe marsh plain	Tides are muted; not yet meeting tidal elevations criterion; elevations vary more than 0.5 ft	None
Sedimentation	Topographic surveys and feldspar plots	The majority of the site's marsh plain will aggrade to the target elevation of 4.5 ft NGVD by Year 6 by natural sedimentation processes.	Sedimentation within 15% of predicted sedimentation rate of ~0.25 ft/yr	Yes	None
Wetland Vegetation Monitoring	Quadrat Sampling	85% cover native wetland plant species	5% Cover native wetland plant species	Not met in Year-1 [0%]	None
Avian Surveys	Avian Surveys	Annual review by wildlife biologists to determine adequacy of wildlife use	Annual review by wildlife biologists to determine adequacy of wildlife use	Yes	None

Hydrologic and Sedimentation Monitoring

Hydrological monitoring shows that the wetland mitigation site is experiencing tidal flushing twice daily on a cycle similar to the adjacent Guadalupe River site. However, tidal flows are muted in comparison, and it is unknown if the muted tidal amplitude measured within the site compared to the river will decrease over time.

Topographic surveys show an increase in marsh elevation varying across the site from approximately 0.3 ft to 1.5 ft. Sedimentation results using feldspar marker horizons verify increases in elevation, with higher sediment accretion rates at plots throughout the mitigation site as compared to the adjacent Guadalupe River marsh. The highest feldspar marker horizon accretion rates were found in the northwest section of the mitigation site.

Feldspar horizon marker sedimentation plots show that more sedimentation is occurring in the wetland mitigation site than along the Guadalupe River. The performance criterion for sedimentation states that sedimentation should be within 15% of the sedimentation predicted in the MMP (approximately 3 in per year). Topographic surveys and feldspar horizon marker sedimentation plots both indicate that the site appears to be on a trajectory toward meeting predicted sedimentation goals.

Wetland Vegetation Monitoring

Wetland vegetation monitoring showed minimal recruitment and establishment of native plants in Year-1. The percent cover of native wetland plants will develop more fully as the site establishes and sedimentation continues to occur. Much of the previously existing weedy vegetation and many of the freshwater plants died once the site became inundated with saline water. However, some perennial pepperweed (*Lepidium latifolium*) remains at the site, for which ongoing control and eradication will be necessary.

Avian Surveys

Avian surveys were conducted on a restoration plot established in the mitigation site and on an adjacent reference plot in the Guadalupe River area adjacent the mitigation site during the breeding season and the winter season to compare species richness and abundance between the two sites. The 2009 baseline breeding season surveys took place just prior to the introduction of tidal action to the restoration site, while the 2009/10 winter season surveys took place in the first winter after tidal action had been introduced to the restoration site.

During the breeding season, species richness was similar between the sites. The restoration site offered more diverse habitat types including open water, ruderal, and coastal scrub habitats. The abundance of wetland-associated focal breeding bird species was higher in the reference site than the restoration site, indicating that, prior to restoration activities, the reference site offered higher quality habitat for breeding wetland birds. This was not unexpected, since tidal action had not yet been introduced to the restoration site when the breeding-season surveys were conducted. An analysis of community dissimilarity indicated moderate to high differences in the bird communities of each site. This result was likely driven by the high abundances of wetland birds in the reference area, contrasting with the high richness and abundances of ruderal and waterbird species in the restoration area.

During the winter season, when both the restoration and reference sites were subject to tidal action, species richness was higher at the reference site during the high tide and higher at the restored site during the low tide. The greater richness in the reference site during high tides may have been because the reference plot currently offers a more topographically and floristically varied, higher quality wetland habitat, providing foraging opportunities for a variety of foraging guilds. In contrast, during high tides the restoration site fills almost entirely with water, limiting the amount of habitat available for shorebirds, while the ruderal habitat bordering the basin of the restored site offers relatively little vegetation structure for perching birds to forage and shelter in. On the other hand, the greater species richness in the restored site during low tide could be explained by the fact that the restored site drains much more slowly than the Guadalupe River channel, offering a deep-water refuge for waterbirds, while the mudflats that become exposed are increasingly available to shorebirds and waders. Conversely, waterbirds can be expected to move out of the reference site during low tide and into areas that still hold some standing water, while shorebirds tend to follow the line of the receding tide to maximize their foraging opportunities. The abundance of wetland-associated focal passerine (songbird) species was higher in the reference site than the restoration site, indicating that, prior to restoration activities, the reference site offered higher quality habitat for these birds. In contrast, the abundance of herons and egrets was similar between the two sites, suggesting that the restoration site is offering improved habitat value for these species. An analysis of community dissimilarity indicated high differences in the bird communities of each site. This was likely driven by the high abundances of wetland passerines in the reference area, contrasting with the high richness and abundances of waterbird species in the restoration area.

MANAGEMENT RECOMMENDATIONS

Colonization by perennial pepperweed has the potential to threaten the functions and values of the establishing wetland habitat, unless it is controlled. A low percent cover of perennial pepperweed has persisted at the site in higher elevation areas and along the toe of the levee slope. In order to ensure that perennial pepperweed is adequately controlled, a certified pest control advisor should be contacted to provide specific recommendations on treating perennial pepperweed at the site.

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REPORT COMPILATION & FIELD PERSONNEL

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Steve Carpenter, B.S.	Wildlife Ecologist

INTRODUCTION

BACKGROUND

The Tasman Corridor Project, sponsored by the Santa Clara Valley Transportation Authority (VTA), involved the construction of a light rail public transit line extending from east San Jose to the City of Mountain View in Santa Clara County, California. Impacts to U. S. Army Corps of Engineers (USACE) and California Department of Fish and Game (CDFG) jurisdictional areas resulted from extension of the light rail line across several creeks and drainages, including Calabazas Creek, Stevens Creek, Sunnyvale East channel and Sunnyvale West channel. The light rail project also crossed Coyote Creek, but was permitted separately as part of another project.

A Mitigation and Monitoring Plan (MMP) was prepared to compensate for impacts to USACE and CDFG jurisdictional areas (H. T. Harvey & Associates 1997). The MMP included a design for the restoration of approximately 3.2 acres of tidal brackish marsh and 1.6 acres of grass and ruderal ground cover along the constructed levee slopes adjacent to the west side of the Guadalupe River in northern San Jose. The goal of the wetland mitigation is to restore a fully-tidal brackish marsh similar in structure and function to the adjacent marsh along the Guadalupe River.

The mitigation site is located immediately north of State Route 237 and just west of the Guadalupe River (Figure 1). The site was previously part of the Guadalupe River before being filled with concrete rubble/soil and separated from the river by construction of a levee. The restoration design called for excavation and removal of the concrete rubble and soil to 3.0 ft NGVD, installation of four 48-in culverts through the existing levee at an invert elevation of 0.0 ft NGVD, construction of inlet channels, and construction of a setback levee approximately 1,300 ft long connecting to the Guadalupe River levee (H. T. Harvey & Associates 1997) (Figure 2).

Wetland mitigation site excavation, culvert installation, inlet channel excavation, and rear levee construction were completed at the wetland mitigation site by October 1998 (H. T. Harvey & Associates 1998). However, the tide gates were initially not opened pending completion of a maintenance agreement for the new levee between the Santa Clara Valley Water District (SCVWD) and a determination that the new levee met Federal Emergency Management Agency (FEMA) and USACE requirements for flood protection. The SCVWD/VTA levee maintenance Memorandum of Understanding was completed by 18 December 2001. In 2009 an evaluation of the geotechnical stability of the new levee was performed and an assessment was conducted to determine if it met FEMA and USACE flood protection requirements (Appendix A).

The Guadalupe levees upstream (south) of State Route 237 are certified by FEMA as protective levees against the 100-year flood. However, the levee evaluation determined that the existing levees downstream of State Route 237, including the new mitigation site levee, cannot be certified by FEMA, since these levees were not designed to provide 100-year protection from coastal flooding. However, these levees do protect against the 100-year riverine flood. Since

FEMA's regulations cannot be met for either the existing SCVWD levees or VTA mitigation site levee, USACE criteria was used as the basis of determining adequacy of the VTA levee.

The levee evaluation determined that the VTA levee meets the USACE geotechnical requirements (Appendix A). It is anticipated that with the tide gates open, the VTA levee will take the place of the Guadalupe levee for providing flood protection to the neighboring properties. The studies performed indicate that the protection provided will equal or exceed the protection provided by the Guadalupe levee. The culvert screw gates were opened and tidal action was introduced to the site on 28 May 2009 immediately following the results of the levee evaluation.

ECOLOGICAL MONITORING

The wetland mitigation site will be monitored annually for a period of six years or until attainment of the success criteria described in the MMP. Monitoring results will be compared annually to determine whether the site is meeting its performance criteria. An as-built plan was previously prepared after the site was constructed (H. T. Harvey & Associates 1999).

Ecological monitoring requirements at the Tasman Wetland Mitigation Site include hydrologic monitoring, vegetation monitoring, and avian surveys. Hydrologic monitoring comprises a visual assessment of the slough channels, topographic surveys, sedimentation monitoring using feldspar plots, and water level monitoring. Vegetation monitoring includes a quantitative assessment of average percent cover of native wetland species, natural recruitment, and an assessment of the presence of invasive species. Avian monitoring comprises winter and breeding season surveys to quantify species richness, focal species densities, and community similarity of bird species utilizing the site. The following report describes the Year-1 monitoring and includes monitoring of the site before opening the tide gates in 2009 and after the opening of the gates in 2009 and early 2010.

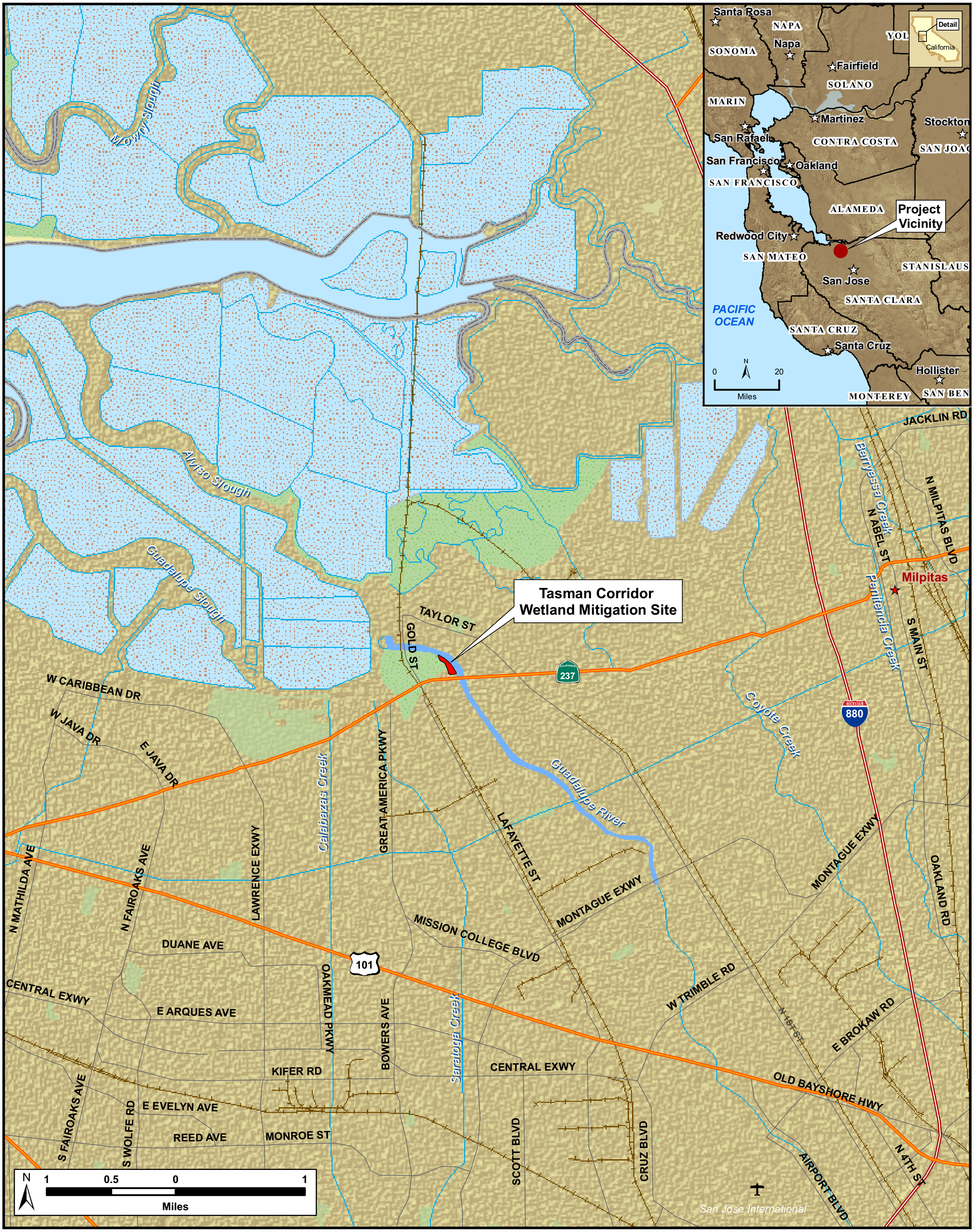

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Figure 1: Vicinity Map

Tasman Corridor Wetland Mitigation Project - Year-1 Monitoring Report (2506-10)

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Figure 2: Site Map

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METHODS

HYDROLOGIC MONITORING

Hydrologic monitoring included a general hydrologic inspection, topographic surveys, water level monitoring, and sedimentation monitoring using feldspar plots. Individual methods for each of these items are described below.

General Hydrology Inspection

Low tide inspections of the constructed inlet channels were made during visits to the site to collect topographic surveys, water level data, and wetland vegetation data. The channels were inspected for areas of sedimentation, slumping, or areas of significant erosion.

Feldspar Marker Horizon Plot Monitoring

H. T. Harvey & Associates installed sedimentation monitoring plots at the mitigation site in 2009 (Figure 3). Feldspar marker horizon plots (0.25 meter^2) were installed to measure short-term vertical accretion (Ball 2005; Cahoon and Turner 1989). On 2 September 2009, seven feldspar plots were established within the wetland mitigation site and three sites were established in the marsh adjacent to the Guadalupe River to determine whether differences in sedimentation patterns exist between these two marshes. Measurements were taken at these sites at one month, three months, and six months.

Topographic Surveys

Topographic surveys were conducted along longitudinal transects to track changes in elevation after opening of the tide gates. H. T. Harvey & Associates ecologists C. Jensen, B.A., S. Carpenter, B.S., and C. Little, M.S. conducted baseline topographic surveys at six fixed cross-sections (approximately along north-south transects) across the site on 19 and 24 March 2009 before the tide gates were opened. In addition, three additional cross-sections were established and surveyed along each of the inlet channels by M. Busnardo, M.S., J. Bourgeois, M.S., and D. Ball, M.S on 2 September 2009 (Figure 3). All of the cross-sections were surveyed with a laser level and rod and tied into local benchmarks provided by RJA & Associates and translated into NGVD29.

A second set of topographic surveys were conducted by C. Jensen, S. Carpenter and D. Ball, and Rachel Burnes, M.S. on 3, 4 and 8 February 2010. This second set of surveys were compared with the initial topographic surveys performed in March and September 2009 to determine whether any change in sedimentation or scour occurred in the site or along the constructed inlet channels during the first year after the tide gates were opened.

The start and end points of cross-sections were physically marked using t-posts or PVC poles, and measurements were taken to establish cross-section stations along the east edge of the levee. All cross-sections start on the east bank. A measuring tape was pulled taut across the length of the cross section. Elevation points were taken along the tape at the beginning and end, grade breaks, the toe of the slope, at the edge of channel, and at the channel centerline. The inlets and outlets of the culverts were also surveyed. The results of the Year-1 surveys will be used to compare surveys in future monitoring years to identify locations and rates of sedimentation, scour, and slough development.

Water Level Monitoring

Water level data was collected to determine the differences in hydrologic function between the wetland mitigation site and the Guadalupe River. Water level monitoring was conducted utilizing two continuous water level recording YSI datasondes (Model 6920) over an 8-day period from 28 October to 4 November 2009. The datasonde locations are shown on Figure 3; the surveyed elevations of the ground surface for each datasonde are included in Table 2.

Table 2. Surveyed Elevation of the Ground Surface for the Datasondes

STATION	ELEVATION (FT) (NGVD 29)
Station 1 Guadalupe River	-1.01
Station 2 Tasman Wetland Mitigation Site	-3.80

Datasondes. Two YSI datasondes (Model #6920) were installed; one at the wetland mitigation site and one on the Guadalupe River by wetland ecologists from H. T. Harvey & Associates on 28 October 2009 on a low tide of –1.08 ft NGVD29. The datasondes were removed on 4 November 2009 and the data downloaded (EcoWatch for Windows). This window in time was chosen to coincide with a spring tide series. On 4 November 2009, the top of each datasonde was surveyed to NGVD 29 by H. T. Harvey & Associates ecologists D. Ball and B. Cleary. The datasondes utilize a differential strain gauge transducer to measure pressure with one side of the transducer exposed to water. The datasondes *in situ* data logging capacity allows for the downloading of data via a cable, which is located inside a protective enclosure, above the highest water level. The data logger was programmed to record one measurement every ten minutes.

One datasonde was installed at the interior of the Tasman wetland mitigation site (Station 1), and one datasonde was installed near the adjacent Guadalupe River marsh under the State Route 237 bridge (Station 2) (Figure 3). These station locations were selected to compare water level fluctuations within the mitigation site to the Guadalupe River and to discern the level of muting of the tidal amplitude by the site's culverts/inlet channels, if any. The datasondes were mounted inside 10-ft sections of 4-in diameter perforated PVC pipe (stilling wells). The datasondes were hung from the interior of perforated PVC stilling wells using heavy-gauge wire attached to the top of the stilling well. The stilling wells were attached to two 10-ft long steel pipes pounded into the mudflat as stabilizers and mounted so that the datasondes were well above the level of any freshly deposited unconsolidated sediment. The distance from the pressure transducer to the top of the stilling well was measured for each datasonde. The steel pipes were permanently installed to allow for annual deployment from the same location.

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Figure 3: Sedimentation and Water Level Monitoring Locations

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All elevation data were converted to NGVD 29 for comparison to survey elevation data for the mitigation site and the Guadalupe River site.

WETLAND VEGETATION MONITORING

H. T. Harvey & Associates wetland ecologist D. Ball, M.S. and plant ecologist B. Cleary, M. S. conducted vegetation monitoring on 17 September 2009. Percent cover of native wetland species was quantitatively evaluated in the mitigation wetland.

Percent Vegetative Cover of Native Wetland Species

The average percent cover by plant species was estimated within the wetland using the quadrat method (Bonham 1989). Six permanent transects were randomly established within the wetland mitigation area. Sampling was conducted using a one-meter square quadrat at random locations along these six permanent transects using a stratified-random design. Vegetation transect locations are shown in Figure 4.

Approximately 0.50% of the mitigation site surface (n = 70) was sampled. Percent vegetative cover of each species observed was visually estimated within each quadrat to the nearest one percent. Total vegetative percent cover, percent cover of each species, and percent cover of bare ground and litter were collected. Slough channels were not included in the percent cover calculations as these areas were originally designed to support little or no vegetation. Following data collection, the relationship between cumulative average percent cover and quadrat number was evaluated to determine if the sample size was adequate (Kershaw 1973). Plant species encountered within the quadrats were identified to species using *The Jepson Manual* (Hickman 1993).

Natural Recruitment

Qualitative surveys were conducted throughout the wetland mitigation site, along the channels, and along the upland/wetland interface to detect naturally recruiting plant species. The plant species and approximate locations of natural recruitment were noted.

Presence of Invasive Plant Species

Surveys were conducted throughout the wetland mitigation site to determine the presence of invasive plant species.

PHOTO-DOCUMENTATION

Photographs were taken from 12 fixed photo-documentation points on 17 September 2009 using a digital camera. Figure 4 shows locations of the photo-documentation points.

AVIAN MONITORING

The purpose of the avian monitoring portion of the MMP is to compare species richness and abundance between the restoration site and an adjacent reference site (Guadalupe River flood plain), and to track the degree to which restoration efforts are functioning to make the restoration area more similar to the tidally-influenced wetlands adjacent to the site. Birds were selected as the focal species group for monitoring because large numbers of species can be detected using simple visual and auditory surveys (as opposed to trapping or more intensive survey efforts that might be necessary to sample other taxa). Also, a number of wetland-associated bird species are known to occur in the vicinity of the study area, and thus, comparison of the use of the restoration site versus an appropriate reference site by wetland-associated bird species would indicate the degree to which the restoration site provides suitable habitat conditions for wildlife. The initial breeding-season bird survey took place just prior to the introduction of tidal action to the restoration site, and provides a pre-restoration baseline of breeding birds at the restoration and reference sites. The initial winter bird survey took place in the first winter after tidal action had been introduced to the restoration site.

Imagery Source: 2006 Santa Clara Aerial, USGS

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Figure 4: Photo Points and Vegetation Transects Map

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H. T. Harvey & Associates wildlife biologist N. Thorngate, M.S. conducted a series of breeding bird surveys on 13, 14, 18, and 20 May 2009 (prior to the opening of the tide gates), and a series of winter bird surveys on 11 and 28 January, and 1 and 2 February 2010 (after the opening of the tide gates). Two survey plots were established: one consisted of the 4.8-acre restoration area, and the second, which served as a reference plot, consisted of a 4.8-acre area of the Guadalupe River channel adjacent to the restoration site. Each survey consisted of a 30-minute area search of each survey plot. Area searches are a standard bird survey method for small habitat patches, where an observer proficient in identifying birds by sight, song, and call searches each plot over a time-constrained period, recording each bird detected within the boundaries of a specified area (Ralph et al 1993, Dieni and Jones 2002). Each area was covered thoroughly, such that the total number of birds observed closely approximates the actual number of birds in the specified area, reducing detection bias compared with stationary counts or line transects. Surveys were conducted twice during high tides and twice during low tides in each season. Each survey was conducted within four hours of local sunrise.

Species richness and abundance were selected as quantitative measures of avian community composition, as these are widely used measures in community ecology. We compared species richness and abundance between the two sites using three metrics – species richness, focal species abundance, and community similarity.

Species Richness

Species richness was calculated for each plot in the restoration area and reference area by season and tide (high or low) by summing the number of species seen during all surveys in each season.

Focal Species Density

The abundance of a subset of species chosen for their affinity to wetland habitats was compared between the plots in the restoration area and reference area using the average number of individuals of each species observed at each site for each season.

Community Similarity

The Bray-Curtis Index of Dissimilarity was used to compare the bird community composition between the restoration area and the reference plot in each season during both high and low tides. Even when plots share similar species richness, the species assemblages in each plot may be very different. Similarity and distance coefficients can be used to categorize the extent to which the species assemblages in sample areas are similar or different. The Bray-Curtis measure is a robust measure of dissimilarity that has been used to evaluate a wide range of ecological data (Faith et al. 1987). We used this measure to calculate the degree of dissimilarity between the restoration area and the reference plot. The Bray-Curtis measure is calculated as:

$$I_{BC} = 1 - (\Sigma | x_i - y_i |)/[\Sigma(x_i + y_i)]$$

where x_i and y_i represent the abundances of the *i*th species in sample x and sample y. The values generated by the Bray-Curtis coefficient range from “0” to “1”, with values closer to “0” indicating areas that have a high degree of similarity (many species in common) and values closer to “1” representing areas that have low similarity (few species in common).

RESULTS AND DISCUSSION

HYDROLOGIC MONITORING

General Hydrology Inspection

The mitigation site and the inlet channels were inspected for erosion and/or slumping. No significant erosion or slumping was observed at the mitigation site. However, the inlet channels experienced some slumping and channel reshaping after the opening of the tide gates and after vegetation removal by the SCVWD during the summer of 2009. Scour also indicates that substantial draining of the mitigation site occurs during tidal cycles and results in some reworking of the inlet channels to a more natural shape. The reshaping of the inlet channels is reflected in the topographic survey data in Appendix C; Figures C7 – C12.

Feldspar Marker Horizon Plot Sampling. The sampling results for the feldspar sedimentation plots show that they have accumulated more sediment over the six-month sampling period (0.32 in) than the plots located along the Guadalupe River (0.13 in) (Table 3). The highest amount of sedimentation in the wetland mitigation site occurred in feldspar plots 1, 2, 3, and 7, which are located at the northwest end of the site. The higher sedimentation in these plots also corresponds to increased elevation in these areas as noted during the topographical surveys (Appendix C; Figures C1 – C6). The difference in the results between the feldspar marker horizon plot results and the topographic survey results in the wetland mitigation site can be attributed to the fact that there are fewer feldspar marker horizon plots (7) and that each of these plots is measuring a discrete location, whereas the topographic surveys are measuring elevation along a transect.

Table 3. Feldspar Sedimentation Results for the Tasman Wetland Mitigation Site (mm)

LOCATION	SEDIMENTATION AT 1 MONTH (mm)	SEDIMENTATION AT 3 MONTHS (mm)	SEDIMENTATION AT 6 MONTHS (mm)	AVERAGE SEDIMENTATION BY SITE (mm)
Tasman Sites				
1	0.7	14.0	17.0	10.6
2	0.0	10.0	17.3	9.1
3	0.8	12.8	7.0	6.9
4	0.7	17.0	2.8	6.8
5[1]	no reading	no reading	no reading	n/a
6	0.5	4.0	4.0	2.8
7	0.5	16.5	21.3	12.8
Average of All Tasman Feldspar Plots (Total average is based on the average of the all readings at 6 months).				**11.6 (0.43 in)/6 months**
Guadalupe River Sites				
8	0.1	7.3	6.5	4.6
9	0.0	2.0	2.0	1.3
10	0.1	9.3	Could not locate site in the vegetation	4.7[2]
Average of Guadalupe River Sites				**3.4 (0.13 in)/6 months**

[1] Sedimentation plot 5 was under water and it was not possible to collect data.
[2] This average is based on only two samples.

Topographic Surveys. The results of the topographic surveys for the wetland mitigation site and the inlet channels are shown in Appendix C (Figures C1 – C12). In general, the topographic surveys show that the wetland mitigation site is demonstrating varying degrees of elevation gain throughout the marsh plain, with elevation gains ranging between 0.3 – 1.5 ft. However, cross sections 1 and 2 are showing some scouring of the deeper ponded areas. Some elevation loss is depicted in cross-sections 1 and 2, but these results may stem from the fact that these areas are ponded too deep to easily perform the topographic surveys. As a result, the surveys for these two cross-sections were performed from a small boat and there may be some error in the results related to this sampling method (Appendix C; Figures C1 – C6).

The inlet channel surveys show that the slough channels have deepened and widened since the tide gates were opened in 2009 (Appendix C; Figures C7 – C12). This is particularly evident in the downstream inlet channel where all three cross-sections show substantial scouring and slumping of the channel (Appendix C; Figures C10 – C12).

The performance criterion for sedimentation at the site states that sedimentation should be within 15% of the sedimentation predicted in Figure D-9 of Appendix D of the MMP (H. T. Harvey & Associates 1997), which indicates a sedimentation rate of approximately 3 inches (76.2 mm) per year, with a predicted aggradation of 1.0 ft during the first three years. Given that the tide gates were not opened until June 2009, the site appears to be well on a trajectory toward meeting the predicted sedimentation goals (~0.25 ft/yr) with 0.3 – 1.5 ft of elevation gain occurring in most of the site.

Water Level Monitoring Using Datasondes

Water levels were calculated in NGVD 29 based on the surveyed elevations. Figure 5 shows the results of water levels for both Station 1 (Tasman wetland mitigation site) and Station 2 (Guadalupe River site) in NGVD29.

The mitigation and monitoring performance criterion state that the tides within the mitigation site should be only slightly muted compared to the tides in the Guadalupe River marsh plain; the reduced tidal elevations should not deviate from those within the Guadalupe River by more than 0.5 ft (H. T. Harvey & Associates 1997). The site is not yet meeting this criterion for tidal function. However, the scouring of the slough channels and predicted sedimentation for the site will likely help the site eventually reach the target criteria over the longer term. In addition, the criterion states that the tidal elevations should not vary by more than 0.5 ft. In 2009, the tidal elevations did vary by more than this amount.

The water level results indicated that the wetland mitigation site (Station 1) is experiencing tidal flushing (with two high and two low tides occurring each day) during spring tides, similar to the Guadalupe River site (Station 2) (Figure 5). However, the tidal amplitude within the mitigation site is muted in comparison to the Guadalupe River site, particularly during the lower high tide event each day. The truncated lower tide water levels at the Guadalupe site indicate that the Guadalupe site is fully drained to the bottom of the channels during water level monitoring, while the wetland mitigation site did not fully drain the site during water level monitoring. The extent of tidal flooding during a combined high tide and storm event was also photographed during a high tide and is shown in Appendix B (Photos B-7 and B-8).

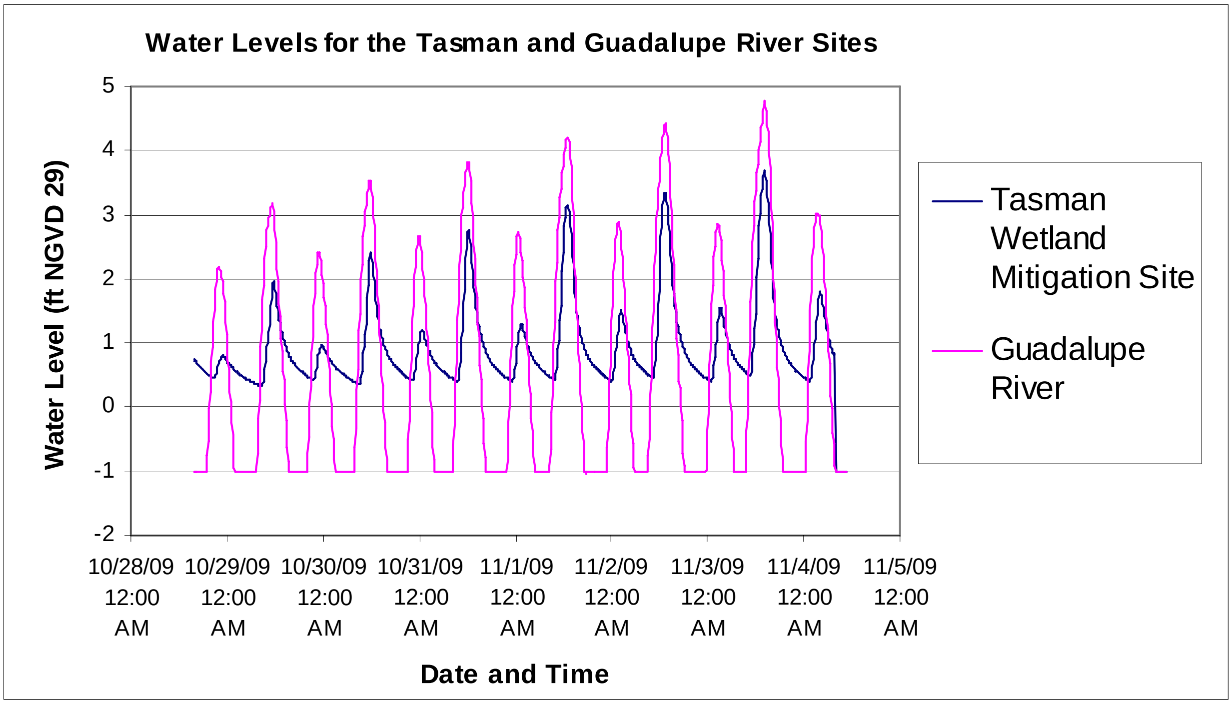


Figure 5. Continuous Water Level Measurements (NGVD 29) - Comparison of Tasman Wetland Mitigation Site to the Guadalupe River Site.

WETLAND VEGETATION

Average Cumulative Percent Cover

The cumulative average percent cover of wetland indicator species was zero for this monitoring year. There was no spatial variability of native wetland plants at the site as none were found in the sampling plots during monitoring. We anticipate that spatial variability and plant diversity will increase as the site develops. The number of samples (70) was chosen to adequately capture the variability throughout the site as it becomes vegetated over time and to establish a baseline of permanent quadrats to be monitored in future years.

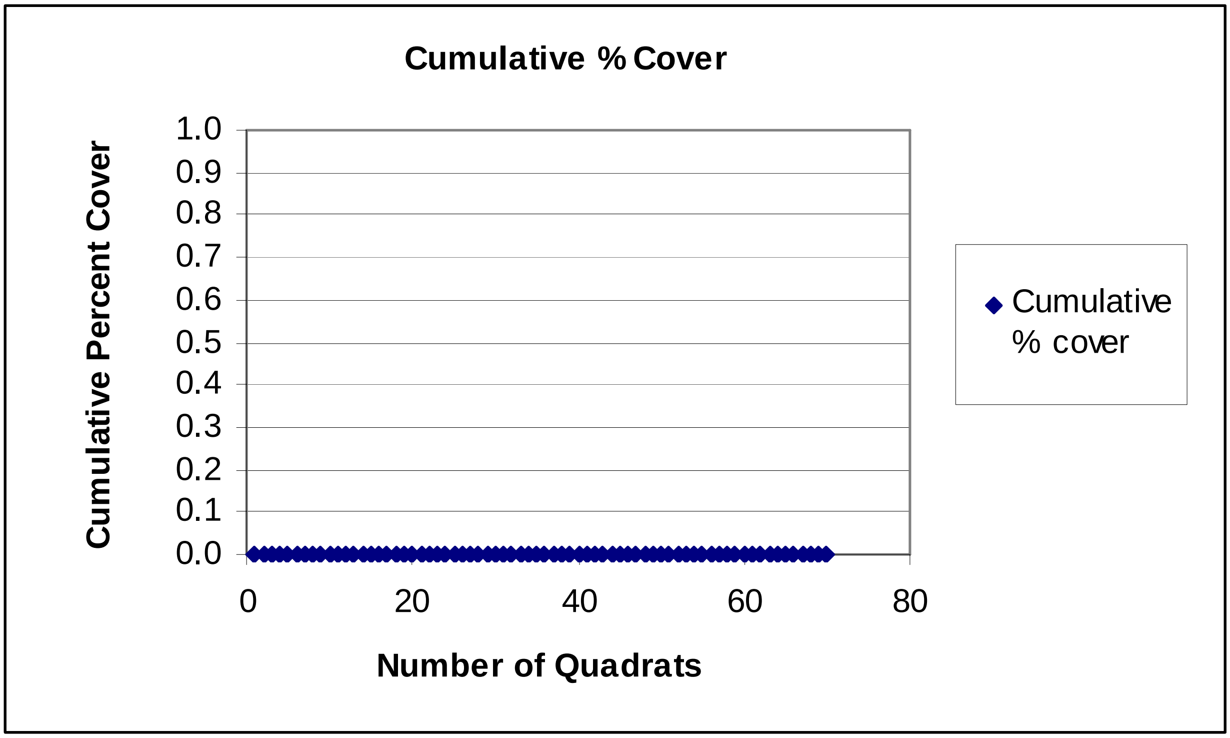


Figure 6. Average Cumulative Percent Cover of Wetland Vegetation[1]

[1] The measure of cumulative percent cover is for wetland vegetation. This is an analysis to determine that an adequate number of samples have been collected to represent the spatial variability at the site.

Percent Cover of Native Wetland Vegetation

During Year-1 wetland vegetation monitoring, no native wetland plants fell within the sampling quadrats (Table 4). Total percent cover of vegetation in Year-1 was 11.2% and totally comprised non-native vegetation. Curly dock (*Rumex* sp.) and perennial pepperweed (*Lepidium latifolium*) are non-native wetland indicator species (Reed 1998).

According to the final success criteria, the wetland mitigation site is required to achieve 85% cover by obligate, facultative wetland, and facultative wetland species by Year 6, dominated by native species. The percent cover requirement for Year-1 is 5% (H. T. Harvey & Associates 1997). The wetland mitigation site did not achieve the Year-1 percent cover criterion as no native wetland vegetation fell within the monitoring quadrats (0% cover).

The lack of cover of native wetland plants in Year-1 can likely be attributed to the site's low elevation in the tidal frame and the season when the tide gates were opened. The site is approximately 1.5 ft lower in the tidal frame than the approximate tidal elevation of the reference marsh on the Guadalupe River flood plain. Additionally, the opening of the tide gates occurred after the spring seeding establishment period for many annual plants and may have precluded the establishment of native wetland plants during the first year. The native wetland plant cover should increase in future years as the marsh plain accumulates sediment and vegetation establishes.

Table 4. Average Percent Cover by Species Vegetation Occurring in the Tasman Wetland

COMMON NAME	SCIENTIFIC NAME	WETLAND INDICATOR STATUS[2]	NATIVE OR NON-NATIVE	2009 AVERAGE PERCENT COVER OF ALL VEGETATION (YEAR-1)
Ruderal[1]	n/a	n/a	n/a	1.4%
Smilo grass	*Piptotherum milaeceum*	n/a	non-native	0.1%
Dock	*Rumex crispus*	FACW-	non-native	0.7%
Perennial pepperweed	*Lepidium latifolium*	FACW	non-native	8.9%
Dead/thatch	n/a	n/a	n/a	20.0%
Mud	n/a	n/a	n/a	48.8%
Water	n/a	n/a	n/a	20.1%
Total Average Percent Cover of all Vegetation (does not include dead vegetation, mud, or water)				**11.1%**
Total Average Percent Cover Wetland Species (as described by the Wetland Indicator Status)				**9.6**

[1] Ruderal includes a mix of weedy plants with each species in amounts too small to quantify

[2] Wetland indicator status taken from Reed (1988). FAC = facultative, FACW = facultative wetland, FACU = facultative upland, UPL = upland, NI = no indicator status given in Reed (1988), NOL = not on list

Natural Recruitment

No natural recruitment of native wetland plant species was noted during Year-1 vegetation monitoring.

Invasive Species

Approximately 9% of the vegetative cover measured during wetland vegetation monitoring can be attributed to perennial pepperweed, which is an invasive species (Table 4). This species colonized the site prior to opening the tide gates in 2009. Much of the infestation was killed by inundation following the tide gate opening in June 2009. However, some of these plants have persisted on the marsh plain in higher elevation areas. Dense stands of perennial pepperweed are also present along the toe of the levee slopes around the entire site. These areas at the toe of slope serve as seed sources for future colonization of the marsh plain.

The MMP (H. T. Harvey & Associates 1997) includes a final success criterion of 85% cover of wetland vegetation, dominated by native plant species. It also states that,

> "an evaluation of the establishment of exotic pest plants such as giant reed and perennial pepperweed. Recommendation for eradication of undesirable vegetation will be included in the annual monitoring report if the pest plants threaten the habitat values of the site."

Perennial pepperweed should be controlled on the marsh plain and levee slopes at the site via a combination of weed whacking and herbicide treatment. The most opportune time for control of perennial pepperweed is early in 2010 and 2011, prior to the colonization of the site by desirable native wetland plant species. A certified pest control advisor should be contacted to provide specific recommendations on treating perennial pepperweed at the site.

Site Maintenance

The site maintenance is generally good, with the exception of perennial pepperweed control as stated above. Non-native weeds and any exotic pest plants should continue to be controlled to ensure the successful establishment of the desired native wetland plant community.

PHOTO-DOCUMENTATION

Selected photos taken during monitoring are included in Appendix B.

AVIAN MONITORING

Breeding Season

Species Richness. During high tide periods, 14 species were observed on the restored plot and 13 species were observed on the reference plot. During low tide periods, 17 species were observed on the restored plot and 16 species were observed on the reference plot (Figure 7). Tables 5 and 6 list the species observed, as well as the plots upon which they were observed, during the high and low tide cycles. Overall the species richness was very similar between plots (Figure 7).

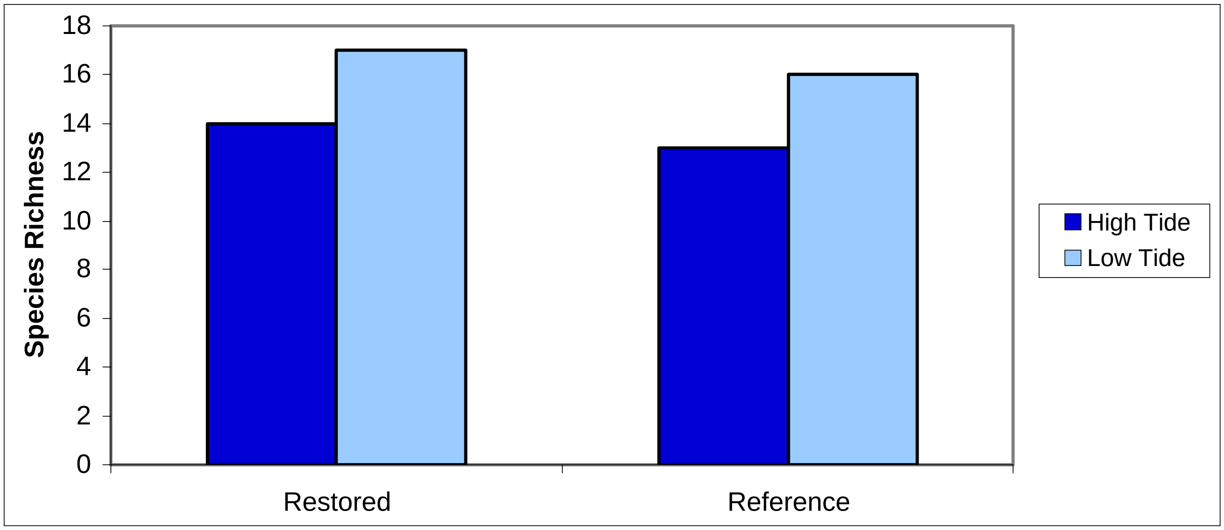


Figure 7. Species richness during the breeding season at the Tasman Mitigation Site and the adjacent reference site.

Table 5. Bird Species Observed during the Breeding Season at the Tasman Mitigation Site and the Adjacent Reference Site during High Tide *

SPECIES	REFERENCE PLOT	RESTORATION PLOT
Anna's Hummingbird		x
Barn Swallow	x	x
Black Phoebe		x
Black-necked Stilt		x
Bushtit		x
California Towhee	x	x
Canada Goose	x	
Caspian Tern		x
Cliff Swallow	x	x
Common Yellowthroat	x	x
Gadwall		x
Great Egret	x	
Green Heron	x	
House Finch	x	x
Lesser Goldfinch	x	
Mallard		x
Marsh Wren	x	
Red-winged Blackbird	x	x
Snowy Egret	x	
Song Sparrow	x	x

* Wetland-associated species are in italics.

Table 6. Bird Species Observed during the Breeding Season at the Tasman Mitigation Site and the Adjacent Reference Site during Low Tide *

SPECIES	REFERENCE PLOT	RESTORATION PLOT
American Goldfinch	x	
Anna's Hummingbird	x	x
Barn Swallow	x	x
Black-necked Stilt	x	x
Bushtit		x
California Towhee		x
Canada Goose		x
Cliff Swallow	x	x
Common Yellowthroat	x	x
European Starling		x
Gadwall		x
Green Heron	x	
House Finch	x	x
Lesser Goldfinch	x	x
Mallard	x	x
Marsh Wren	x	
Mourning Dove	x	x
Rock Pigeon	x	x
Red-winged Blackbird	x	x
Savannah Sparrow	x	
Song Sparrow	x	x

* Wetland-associated species are in italics.

Focal Species Density. Seven wetland-associated species were selected from all species observed on the plots during the breeding season to assess the value of each habitat for wetland bird communities. These species included the green heron (*Butorides virescens*), great egret (*Ardea alba*), snowy egret (*Egretta thula*), marsh wren (*Cistothorus palustris*), common yellowthroat (*Geothlypis trichas*), song sparrow (*Melospiza melodia*), and red-winged blackbird (*Agelaius phoeniceus*). When present, all species were found in substantially higher numbers in the reference plot than in the restoration plot during both high and low tides, indicating that the reference plot offered higher value for wetland bird communities. Such a result was not unexpected given that these breeding-season surveys were conducted prior to the introduction of tidal action within the restoration site. Tables 7 and 8 show the densities of each focal species in each plot during the high and low tides, respectively.

Table 7. Densities of Wetland-Associated Bird Species during the Breeding Season at the Tasman Mitigation Site and the Adjacent Reference Site during High Tide.

	MEAN NUMBER OF BIRDS/SURVEY	
SPECIES	**REFERENCE PLOT**	**RESTORATION PLOT**
Common Yellowthroat	8.5	0.5
Great Egret	0.5	0.0
Green Heron	0.5	0.0
Marsh Wren	7.0	0.0
Red-winged Blackbird	27.0	7.0
Snowy Egret	0.5	0.0
Song Sparrow	10.0	5.0

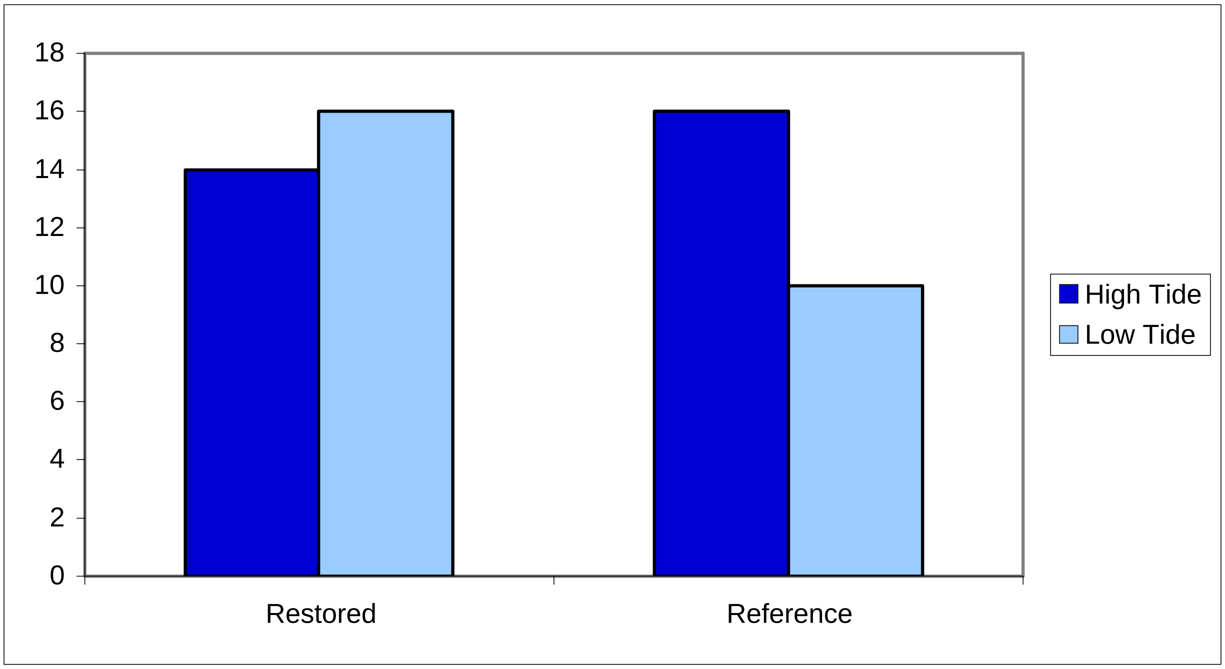
Table 8. Densities of Wetland-Associated Bird Species during the Breeding Season at the Tasman Mitigation Site and the Adjacent Reference Site during Low Tide.

	MEAN NUMBER OF BIRDS/SURVEY	
SPECIES	**REFERENCE PLOT**	**RESTORATION PLOT**
Common Yellowthroat	7.5	0.5
Great Egret	0.0	0.0
Green Heron	0.5	0.0
Marsh Wren	5.5	0.0
Red-winged Blackbird	18.5	7.5
Snowy Egret	0.0	0.0
Song Sparrow	9.5	5.0

Community Similarity. The Bray-Curtis values indicating the degree of community dissimilarity between the restoration site and the reference site during the breeding season, prior to introduction of tidal action to the restoration site, were 0.439 for the low tide cycle and 0.625 for the high tide cycle. These values indicate moderate community similarity during the low tide and low community similarity during the high tide. As discussed above in the analysis of focal species density, the reference site supported greater richness and density of tidal marsh-associated species. In contrast, the restoration site supported a higher richness and density of species associated with ruderal and scrub habitats, such as bushtits (*Psaltriparus minimus*) and house finches (*Carpodacus mexicanus*). The restoration site also tended to harbor more open-water species such as gadwalls (*Anas strepera*), Canada geese (*Branta canadensis*), and mallards (*Anas platyrhynchos*); because tidal action had not yet been introduced to the restoration site when these breeding-season surveys were conducted, the restoration site was dominated by non-tidal open water conditions.

Winter Season

Species Richness. During high tide periods, 14 species were observed on the restored plot and 16 species were observed on the reference plot during the winter season, after tidal action had been introduced to the restored site. During low tide periods, 16 species were observed on the restored plot and 10 species were observed on the reference plot. Tables 9 and 10 list the species observed, as well as the plots upon which they were observed, during the high and low tide cycles. Species richness was higher at the reference site during the high tide and higher at the restored site during the low tide (Figure 8). The greater richness in the reference site during high tides may have been because the reference plot currently offers a more topographically and floristically varied, higher quality wetland habitat, such that a variety of shorebirds and wading birds, and even some waterbirds such as mallards (*Anas platyrhynchos*), can be found foraging on the vegetated flats bordering the channel, while the dense and diverse wetland vegetation on the flats and bordering the stream offers multiple strata of habitat for foraging songbirds. In contrast, during high tides the restoration site fills almost entirely with water, limiting the amount of habitat available for shorebirds, while the ruderal habitat bordering the basin of the restored site offers relatively little vegetation structure for perching birds to forage and shelter in. On the other hand, the greater species richness in the restored site during low tide could be explained by the fact that the restored site drains much more slowly than the Guadalupe River channel, offering a deep-water refuge for waterbirds such as mallards, double-crested cormorants (*Phalacrocorax auritus*), and gadwalls, while the mudflats that become exposed are increasingly available to shorebirds and waders. Conversely, waterbirds can be expected to move out of the reference site during low tide and into areas that still hold some standing water, while shorebirds tend to follow the line of the receding tide to maximize their foraging opportunities.



	High Tide	Low Tide
Restored	14	16
Reference	16	10

Figure 8. Species richness during the winter season at the Tasman Wetland Mitigation Site and the adjacent reference site.

Table 9. Bird Species Observed during the Winter Season at the Tasman Mitigation Site and the Adjacent Reference Site during High Tide *

SPECIES	REFERENCE SITE	RESTORATION SITE
American Coot	x	x
Anna's Hummingbird		x
Audubon's Warbler	x	x
Black Phoebe	x	x
Black-necked Stilt	x	
Canada Goose	x	x
Common Yellowthroat	x	x
Double-crested Cormorant		x
Gadwall		x
Great Egret	x	x
House Finch	x	x
Mallard	x	
Marsh Wren	x	
Pied-billed Grebe		x
Red-winged Blackbird	x	
Snowy Egret	x	x
Song Sparrow	x	x
Spotted Sandpiper	x	
White-crowned Sparrow	x	x
Wilson's Snipe	x	

* Wetland-associated species are in italics.

Table 10. Bird Species Observed during the Winter Season at the Tasman Mitigation Site and the Adjacent Reference Site during Low Tide *

SPECIES	REFERENCE SITE	RESTORATION SITE
American Coot		x
Audubon's Warbler	x	x
Black Phoebe	x	x
Canada Goose		x
Common Yellowthroat	x	x
Double-crested Cormorant		x
Gadwall		x
Great Blue Heron		x
Great Egret	x	x
House Finch	x	
Mallard	x	x
Marsh Wren	x	
Pied-billed Grebe		x
Ruddy Duck		x
Snowy Egret	x	x
Song Sparrow	x	x
Spotted Sandpiper		x
White-crowned Sparrow	x	x

* Wetland-associated species are in italics.

Focal Species Density. Seven wetland-associated species were selected from all species observed on the plots during the winter season to assess the value of each habitat for wetland bird communities. These species included the great blue heron (*Ardea herodias*), great egret, snowy egret, marsh wren, common yellowthroat, song sparrow, and red-winged blackbird. In most cases, focal passerines tended to occur in higher numbers in the reference plot than in the restoration plot during both high and low tides, indicating that the reference plot currently offers somewhat higher value for wetland-associated passerines. Herons and egrets tended to be more equally distributed across the sites, suggesting that the restoration site may be offering improved habitat value for these species. Tables 11 and 12 show the densities of each focal species in each plot during the high and low tides, respectively.

Table 11. Densities of Wetland-Associated Bird Species during the Winter Season at the Tasman Mitigation Site and the Adjacent Reference Site during High Tide

	MEAN NUMBER OF BIRDS/SURVEY	
SPECIES	**REFERENCE PLOT**	**RESTORATION PLOT**
Common Yellowthroat	1.0	1.5
Great Blue Heron	0.0	0.0
Great Egret	0.5	0.5
Marsh Wren	4.5	0.0
Red-winged Blackbird	0.5	0.0
Snowy Egret	0.5	0.5
Song Sparrow	5.0	4.0

Table 12. Densities of Wetland-Associated Bird Species during the Winter Season at the Tasman Mitigation Site and the Adjacent Reference Site during Low Tide

	MEAN NUMBER OF BIRDS/SURVEY	
SPECIES	**REFERENCE PLOT**	**RESTORATION PLOT**
Common Yellowthroat	1.5	0.5
Great Blue Heron	0.0	1.0
Great Egret	0.5	1.0
Marsh Wren	5.0	0.0
Red-winged Blackbird	0.0	0.0
Snowy Egret	0.5	0.5
Song Sparrow	5.5	4.0

Community Similarity. The Bray-Curtis values indicating the degree of community dissimilarity between the restoration site and the reference site were 0.803 for the low tide cycle and 0.669 for the high tide cycle. These values indicate low community similarity during both high and low tides. As discussed above in the analysis of focal species density, the reference site supported greater richness and density of wetland-associated passerines. In contrast, the restoration site supported much higher richness and abundance of waterbirds including gadwalls, Canada geese, mallards, American coots (*Fulica americana*), and double-crested cormorants. As habitat on the restoration site develops, we expect the bird community there to more closely approximate the community of the reference site, yielding lower Bray-Curtis values and indicating that the restoration efforts have succeeded in creating a functional tidal marsh habitat.

MANAGEMENT RECOMMENDATIONS

The Project's MMP requires that recommendations for removal of undesirable vegetation be included in the annual monitoring report if the pest plants threaten the habitat values of the site (H. T. Harvey & Associates 1997). Perennial pepperweed has the potential to threaten the functions and values of the establishing wetland habitat. A low percent cover of perennial pepperweed has persisted at the site in higher elevation areas and along the toe of the levee slope after the opening of the levee. Perennial pepperweed control efforts will begin in Spring 2010. In order to ensure that perennial pepperweed is adequately controlled, a certified pest control advisor will be contacted to provide specific recommendations on treating perennial pepperweed at the site.

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Personal communications

Wines, B. 2009 pers. comm. Personal communication between Brian Wines (RWQCB) and Ann Calnan (VTA) in 2009.

APPENDIX A
VTA Levee Evaluation Report

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March 30, 2009
1437-4A/161552

Mr. Jim Schaaf
SCHAAF & WHEELER
100 North Winchester Boulevard, Suite 200
Santa Clara, California 95050

RE: VTA LEVEE EVALUATION
HIGHWAY 237 AND GUADALUPE RIVER
SAN JOSE, CALIFORNIA

Dear Mr. Schaaf:

In this letter, we present the results of our geotechnical evaluation for the Valley Transportation Authority (VTA) Levee located on the west side of the Guadalupe River and north of Highway 237 in San Jose, California. The approximate location of the project is shown on the Vicinity Map, Figure 1. As you know, we recently performed a preliminary geotechnical review for the project and presented the results in our November 10, 2008 memorandum. A copy of the November 10, 2008 memorandum is enclosed at the end of the report. As discussed in our memorandum, we recommended that additional study be performed to:

a) Evaluate the liquefaction potential at the site in accordance with current engineering standards,
b) Determine the depth of sheet piles supporting the cut along the southern portion of the levee to evaluate levee stability, and
c) Obtain soil samples for laboratory testing to verify our assumptions and judgments.

The results of our additional study are presented in the following sections.

EXPLORATION PROGRAM

To determine the depth of the sheet piles on the southern portion of the levee, we drilled one 4-inch-diameter boring to a depth of 30 feet using portable "Minuteman" equipment. The boring was drilled approximately 12 inches from the existing sheet pile wall. Induction logging was performed using a Mount Sopris Instruments MGX II Logger and EM39 Electromagnetic Induction Probe. The EM39 probe uses transversely mounted coils to measure the electrical conductivity of the material surrounding the bore hole. The tool is designed to read inductance or conductivity within the range of soil and rock. As shown on the induction log, the sheet pile extends to a depth of approximately 20 feet below the adjacent ground surface (15 feet below the bottom of the adjacent wetland).

To characterize the subsurface conditions at the site, we drilled two exploratory borings using truck-mounted hollow-stem auger drilling equipment to a depth of approximately 32½ feet. Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and selective testing. Penetration resistance blow counts were obtained by dropping a 140-pound hammer 30 inches. Modified California 3.0-inch outside diameter (O.D) and Standard Penetration Test (SPT) 2-inch O.D. samples were obtained by driving the samplers 18 inches and recording the number of hammer blows for each 6 inches of penetration. Unless otherwise indicated, the blows per foot recorded on the boring logs represent the accumulated number of blows required to drive the samplers the last two 6-inch increments. When using the SPT sampler, the last two 6-inch increments is the uncorrected SPT measured blow count. Relatively undisturbed samples were also obtained with 2.875-inch inside diameter Shelby Tube sampler, which were hydraulically pushed. The various samplers are denoted at the appropriate depth on the boring logs and symbolized as shown on Figure A-1.

In addition, we advanced four cone penetration tests (CPTs) to a maximum depth of approximately 50 feet. CPT data was obtained at 0.16 feet (5 centimeter) intervals and consisted of cone tip resistance, sleeve friction, dynamic pore pressure and other parameters. All borings and CPTs were permitted and backfilled with cement grout in accordance with Santa Clara Valley Water District guidelines. The approximate locations of the induction test, borings and CPTs are shown on the Site Plan, Figure 2. Logs of our borings, CPTs and induction test are included at the end of this report.

SUBSURFACE

Below approximately 1 to 2 feet of aggregate base, our explorations encountered relatively well compacted very stiff to hard clayey and silty fills to depths of approximately 21½ to 22 feet. Below the fill, predominantly medium stiff to stiff clays, with occasional lenses of sands, 2-foot thick or less, were encountered to a maximum depth explored of 50 feet. Our explorations indicate that the levee foundation soil generally behaves as clay and is relatively stronger than previously assumed in our analysis.

Free ground water was encountered in both borings at a depth of approximately 26½ feet (elevations -4½ to -5½ feet) during drilling. The ground water depth was measured at the time of drilling and may not reflect a stabilized level. Standing water was measured at approximately elevation 1 foot in the wetland area. Previous explorations by others encountered ground water a depths between elevations -6 to -8 feet. All borings and CPTs were backfilled immediately after drilling. Fluctuations in the level of the ground water may occur due to variations in rainfall, underground drainage patterns, and other factors not evident at the time measurements were made.

LABORATORY PROGRAM

The laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical and mechanical properties of the soils underlying the site and to aid in verifying soil classification. Natural water content was measured on 13 soil samples recovered from the borings; in place dry density tests were performed on 11 samples to measure the unit weight of the subsurface soils; two washed sieve analyses were performed to evaluate the percent soil fraction passing No. 200 sieve; and triaxial consolidated undrained shear strength tests were performed on five relatively undisturbed samples of the fill and subsurface clayey soil samples to evaluate the undrained shear strength of these materials. Results of these tests are presented on the boring logs and are attached in this report.

COMPARISON OF LABORATORY TEST RESULTS TO PREVIOUS ASSUMPTIONS

As discussed in our November 10, 2008 memorandum, we assumed soil parameters for the analyses based on our experience with similar soil conditions and based on limited information in reports prepared by others. We recommended that laboratory testing be performed to support our assumptions. Table 1 below presents the assumed soil parameters and laboratory test results. The laboratory test results are also presented graphically on Figures B-1 to B-4.

Table 1. Comparison of Laboratory Test Results to Previous Assumed Soil Parameters

Soil Layer	Assumed Soil Parameters			Laboratory Test Results		
	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Undrained Shear Strength (psf)	Effective Cohesion (psf)	Effective Friction Angle (degrees)	Undrained Shear Strength (psf)
Embankment Fill	10	34	--	200	31	--
Foundation: Clay Alluvium	--	--	1,000	--	--	1,400 to 1,700

In summary, the laboratory test results indicated that the strength of the fill and foundation soil is greater than the soil strength assumed in our November 10, 2008 memorandum for the stress conditions of the project. In addition, our explorations indicated the Bay Mud below the levee fill is not as weak as previously assumed; therefore, it is our opinion that the factors of safety for Cross Section B, Case I (End-of-Construction) and Case IV (Earthquake), will be greater than 1.3 and 1.0, respectively. These factors of safety are recommended in the US Army Corps of Engineers, Design and Construction of Levees, Engineer Manual 1110-2-1913 (2000).

LIQUEFACTION

The site is located within an area zoned by the California Geological Survey as having potential for seismically-induced liquefaction hazards (CGS, 2003). During cyclic ground shaking such as earthquakes, cyclically induced stresses may cause increased pore water pressures within the soil matrix and result in liquefaction. Liquefied soil may lose shear strength and lead to large shear deformations and flow failure (Youd et al., 2001). Liquefied soil can also settle as pore pressures dissipate following an earthquake. Limited field data is available on this subject; however, settlement on the order of 2 to 3 percent of the thickness of the liquefied zone has been measured in some cases.

Soils most susceptible to liquefaction are loose to medium dense, saturated non-cohesive soils with poor drainage, such as sands and silts with interbedded or capping layers of relatively low permeability soil.

Analysis and Results

Based on our explorations and the depth to historically high ground water map prepared by the CGS, we judge a design ground water level at elevation 6 feet is reasonable. As discussed in the “Subsurface” section, several sand layers were encountered below the recommended design ground water depth. These layers were evaluated to assess liquefaction potential and the effects it may have on the levee.

Our liquefaction analyses followed the methods presented by the 1998 NCEER Workshops (Youd et al., 2001) in accordance with guidelines set forth in the CGS Special Publication 117A (2008). The NCEER methods for standard penetration test (SPT) and CPT analyses update simplified procedures presented by Seed and Idriss (1971).

In broad terms, these methods are used to calculate a factor of safety against liquefaction triggering by comparing the resistance of the soil to cyclic shaking to the seismic demand that can be caused during seismic events.

The resistance to cyclic shaking is quantified by the Cyclic Resistance Ratio (CRR), which is a function of soil density, layer depth, ground water depth, earthquake magnitude, and soil behavior. CRR calculations are based on SPT blow counts and CPT tip resistance. To account for effective overburden stresses and soil behavior, we corrected the field measured SPT blow counts for the overburden, stress reduction versus depth, fine-grained soil content, hammer energy ratio, boring diameter, rod length and sampling method (SPT sampler without liners). Our CPT tip pressures were corrected for the overburden and fines content. The CPT method utilizes the soil behavior type index (I_C) and the exponential factor "n" applied to the Normalized Cone Resistance "Q" to evaluate how plastic the soil behaves.

The Cyclic Stress Ratio (CSR) is used to quantify the stresses that are anticipated to develop during cyclic shaking. The formula for CSR is shown below:

$$CSR = 0.65\,(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

Where a_{max} is the peak horizontal acceleration at the ground surface generated by an earthquake, g is the acceleration of gravity, σ_{vo} and σ'_{vo} are total and effective overburden stresses, respectively, and r_d is a stress reduction coefficient. We used a probabilistic peak horizontal acceleration of 0.55g, which corresponds to a 10 percent chance of exceedance in 50 years for our liquefaction analysis.
Soils that have greater than 35 percent of plastic fines, an I_C greater than 2.6, corrected SPT blow counts greater than 30 blows per foot, or corrected CPT tip resistances greater than 160 tons per square foot (tsf) are considered either too plastic or too dense to liquefy. Such soil layers have been screened out during our analyses and are not presented below.

The factor of safety (FS) against liquefaction can be expressed as the ratio of the CRR to CSR. If the FS for a soil layer is less than 1.0, the soil layer is considered liquefiable during a moderate to large seismic event. A summary of our analysis is presented in the table below.

Table 2. Results of Liquefaction Analysis

CPT Number	Depth to Top of Sand Layer (feet)	Layer Thickness (feet)	I_C	*$q_{C1N\text{-}cs}$	Factor of Safety	Estimated Total Settlement (inches)
CPT-3	39.7	1.1	2.1	71	0.2	0.5
CPT-3	48.4	0.7	2.0	86	0.3	0.2
					Total =	**0.7**
CPT-4	41	0.5	2.1	76	0.2	0.2
CPT-4	42.8	0.5	2.1	63	0.2	0.2
CPT-4	49.5	0.3	2.2	69	0.2	0.1
					Total =	**0.5**

* CPT tip pressure corrected for overburden and fines content

Our analysis indicates that several sand layers below the design ground water depth may theoretically liquefy resulting in about ½- to ¾-inch of total settlement. Volumetric change and settlement were estimated using the Ishihara and Yoshimine (1990) method. Since most of the potentially liquefiable sand layers are relatively deep, we judge the potential for liquefaction-induced settlement causing the levee to loose freeboard, lateral spread, and cracks to be low. As discussed in the Southern California Earthquake Center report (SCEC, 1999), differential movement for level ground deep soil sites will be on the order of half the total estimated settlement.

SHEET PILE ANALYSIS

Based on our calculations, a minimum embedment depth of 8½ feet below the bottom of the wetland area is required to retain the cut slope on the southern portion of the levee. As previously discussed, the induction test indicated that the sheet piles extended approximately 15 feet below the bottom of the wet land area. In addition, we performed a maximum bending stress analysis for the sheet piles under seismic conditions. Our calculations indicate that the maximum bending stress on the sheet piles is approximately 5,000 psi, which is less than the allowable bending stress of 29,000 psi for the sheet piles. Based on this information, it is our opinion that the sheet piles provide adequate slope support for the cut slope on the southern portion of the levee.

CONCLUSIONS

In summary, the laboratory test results indicated that the strength of the fill and foundation soil is greater than the soil strength previously assumed in our November 10, 2008 memorandum for the stress conditions of the project. There are some relatively thin, about 1-foot or less, potentially liquefiable sand layers at a depth of 40 feet or deeper, which may result in total settlement of about ½- to ¾-inch; however, we judge the potential for liquefaction causing the levee to loose freeboard, be subject to lateral spreading and cracks to be low since the potential liquefiable sand layers are relatively deep. In addition, our calculations indicated that the sheet piles provide adequate support to maintain static and seismic factors of safety above the recommended minimums of 1.3 and 1.0, respectively, for the cut slope along the southern portion of the levee. Based on this information, it is our opinion that the levee stability and seepage meet geotechnical guidelines as presented in the Army Corps of Engineers, Design and Construction of Levees, Engineer Manual 1110-2-1913 (2000) and FEMA certification requirements. We recommend that the operation and maintenance plans include provisions regarding vegetation and rodent control. In general, the levee should be kept free of large vegetation such as trees and bushes, and rodents should be prevented from burrowing into the levee.

LIMITATIONS

This letter has been prepared for the sole use of Schaaf & Wheeler for application to the VTA Levee in San Jose, California. The opinions presented in this letter have been formulated in accordance with accepted geotechnical engineering practices that exist in the Bay Area at the time this letter was written. No warranty, expressed or implied, is made or should be inferred.

The opinions, conclusions and recommendations contained in this letter are based upon the information obtained from our investigation, which includes data from widely separated discrete locations, visual observations from our site reconnaissance, and review of other geotechnical data provided to us, along with local experience and engineering judgment. The recommendations presented in this letter are based on the assumption that soil and geologic conditions at or between explorations do not deviate substantially from those encountered or extrapolated from the information collected during our investigation. We are not responsible for the data presented by others.

The opinions presented in this letter are valid as of the present date for the property evaluated. Changes in the condition of the property will likely occur with the passage of time due to natural processes and/or the works of man. In addition, changes in applicable standards of practice can occur as a result of legislation and/or the broadening of knowledge. Furthermore, geotechnical issues may arise that were not apparent at the time of our investigation. Accordingly, the opinions presented in this letter may be invalidated, wholly or partially, by changes outside of our control. Therefore, this letter is subject to review and should not be relied upon after a period of three years, nor should it be used, or is it applicable, for any other properties.

If you have any questions concerning this letter, please call and we will be glad to discuss them with you.

Sincerely,

TRC Engineers

Minh Q. Le, P.E.
Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
MINH Q. LE
No. 69024
06/30/10
CIVIL
STATE OF CALIFORNIA

Copies: Addressee (2 and via email)

Attachments:
- Figure 1, Vicinity Map
- Figure 2, Site Plan
- Figure A-1, Keys to Exploratory Borings
- Boring and CPT Logs
- Figure A-2, Induction Log
- Figures B-1 to B-4, Triaxial Tests
- November 10, 2008 Memorandum

REFERENCES

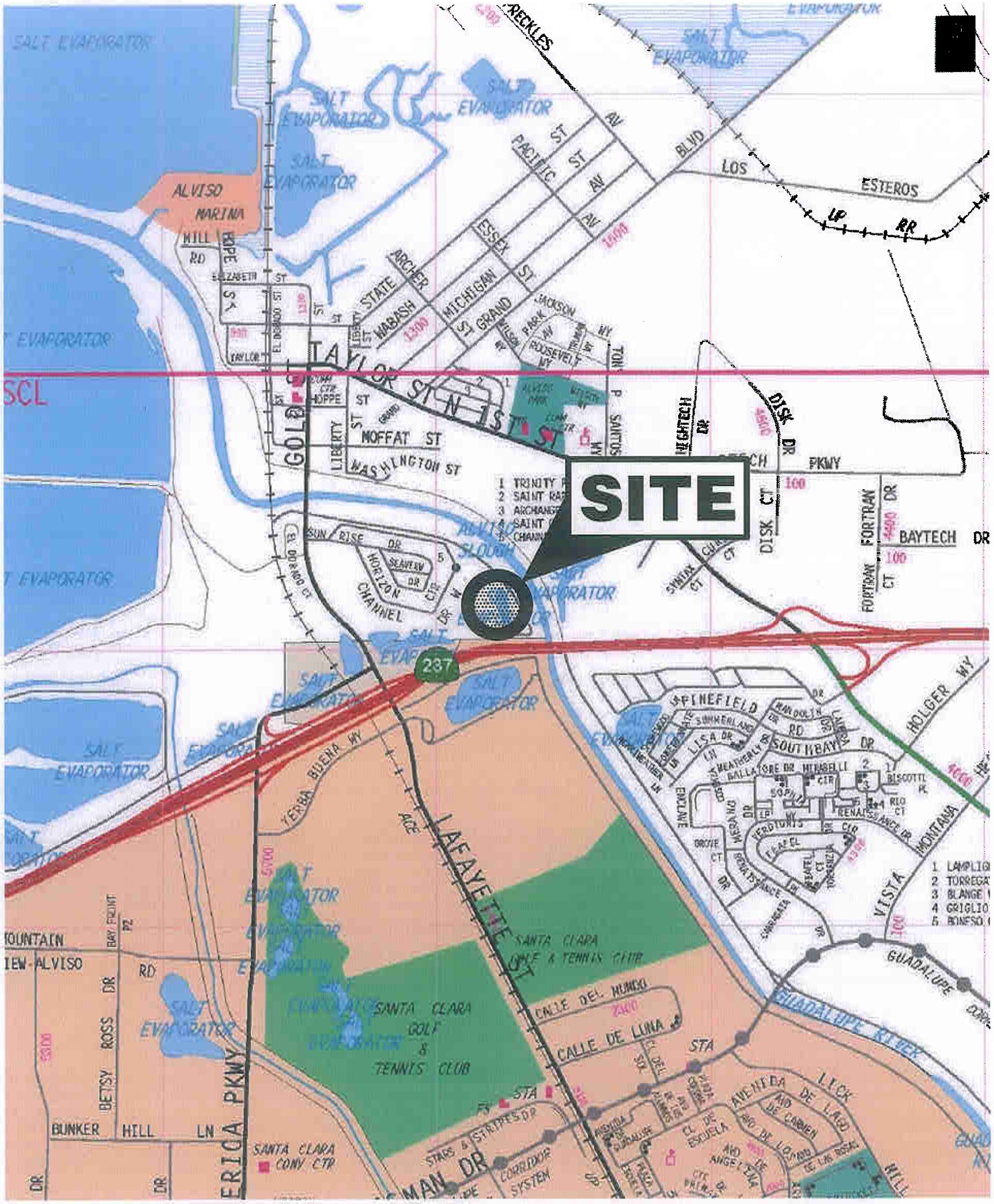
California Geological Survey, 2003, *State of California Seismic Hazard Zones, San Jose West 7.5-Minute Quadrangle, Santa Clara County, California*: Seismic Hazard Zone Report 081.

California Geological Survey, 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117A.

Seed, H.B. and Idriss, I.M., 1971, *A Simplified Procedure for Evaluation soil Liquefaction Potential:* JSMFC, ASCE, Vol. 97, No. SM 9, pp. 1249 – 1274.

Southern California Earthquake Center, 1999, *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California*, March.

Youd et al., 2001, *Liquefaction Resistance of Soils*: Summary Report from the 1996 NCEER and 1998 NCEER/NSF *Workshops on Evaluation of Liquefaction Resistance of Soils*, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vo. 127, No. 10, October.



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2/09*EB

VICINITY MAP

VTA LEVEE EVALUATION
San Jose, California

FIGURE 1
1437-4/161552

LEGEND

- 🔵 - Approximate location of exploratory boring
- ⊕ - Approximate location of cone penetration test
- ● - Approximate location of cone induction test

Base by Ruggeri-Jensen-Azar, dated October 2008.

SITE PLAN
VTA LEVEE EVALUATION
San Jose, California

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FIGURE 2
1437-4/161552

2/09 EB

PRIMARY DIVISIONS			SOIL TYPE		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (Less than 5% Fines)	GW	●	Well graded gravels, gravel–sand mixtures, little or no fines
			GP	○	Poorly graded gravels or gravel–sand mixtures, little or no fines
		GRAVEL WITH FINES	GM	○	Silty gravels, gravel–sand–silt mixtures, plastic fines
			GC	▨	Clayey gravels, gravel–sand–clay mixtures, plastic fines
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (Less than 5% Fines)	SW	░	Well graded sands, gravelly sands, little or no fines
			SP	░	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES	SM	░	Silty sands, sand–silt–mixtures, non–plastic fines
			SC	▨	Clayey sands, sand–clay mixtures, plastic fines
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS LIQUID LIMIT IS LESS THAN 50 %		ML	▥	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	▨	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	▤	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50 %		MH	▥	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	▨	Inorganic clays of high plasticity, fat clays
			OH	▦	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			PT	⁂	Peat and other highly organic soils

DEFINITION OF TERMS

	U.S. STANDARD SIEVE SIZE			CLEAR SQUARE SIEVE OPENINGS			
	200	40	10	4	3/4"	3"	12"

SILTS AND CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

0.08	0.4	2	5	19	76mm

GRAIN SIZES

- ☒ TERZAGHI SPLIT SPOON STANDARD PENETRATION
- ⧗ MODIFIED CALIFORNIA
- ▯ ROCK CORE
- ■ PITCHER TUBE
- ○ NO RECOVERY

SAMPLERS

SAND AND GRAVEL	BLOWS/FOOT*
VERY LOOSE	0–4
LOOSE	4–10
MEDIUM DENSE	10–30
DENSE	30–50
VERY DENSE	OVER 50

RELATIVE DENSITY

SILTS AND CLAYS	STRENGTH+	BLOWS/FOOT*
VERY SOFT	0–1/4	0–2
SOFT	1/4–1/2	2–4
MEDIUM STIFF	1/2–1	4–8
STIFF	1–2	8–16
VERY STIFF	2–4	16–32
HARD	OVER 4	OVER 32

CONSISTENCY

*Number of blows of 140 pound hammer falling 30 inches to drive a 2–inch O.D. (1–3/8 inch I.D.) split spoon (ASTM D–1586).
+Unconfined compressive strength in tons/sq.ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D–1586), pocket penetrometer, torvane, or visual observation.

KEY TO EXPLORATORY BORING LOGS

Unified Soil Classification System (ASTM D–2487)

TRC

FIGURE A–1

EXPLORATORY BORING: EB-1

Sheet 1 of 1

DRILL RIG: MOBILE B-53
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 2-27-09 FINISH DATE: 2-27-09

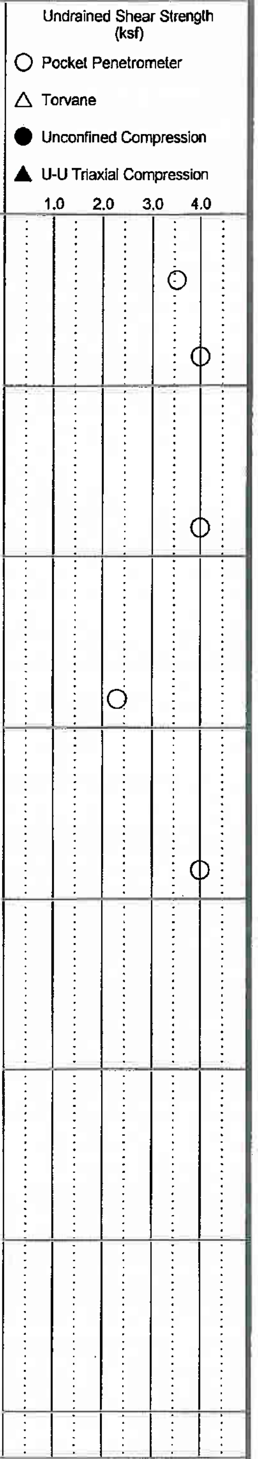
PROJECT NO: 161552
PROJECT: VTA LEVEE EVALUATION
LOCATION: ALVISO, CA
COMPLETION DEPTH: 32.5 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

SURFACE ELEVATION: 22.0 FT. (+/-)

ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	SAMPLER	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	PERCENT PASSING NO. 200 SIEVE
22.0	0	**CLAYEY SAND (SC) [FILL]** medium dense, moist, brown, fine to coarse sand, trace fine subangular to subrounded gravel	SC, FILL					
20.0				16	☒	17		
		LEAN CLAY WITH SAND (CL) [FILL] very stiff to hard, moist, brown, fine sand, low to moderate plasticity		40	⧗	18	112	
	5		CL, FILL					
14.0		**LEAN CLAY (CL) [FILL]** hard, moist, brown, trace fine sand, trace fine subangular to subrounded gravel, low plasticity		35	⧗	16	112	
	10		CL, FILL					
10.5		**LEAN CLAY WITH SAND (CL) [FILL]** very stiff, moist, brown, fine sand, low plasticity						
				22	⧗	19	107	86
	15							
			CL, FILL					
		some brick debris		34	⧗	16	107	71
	20							
0.0		**LEAN CLAY WITH SAND (CL)** medium stiff to stiff, moist, dark grayish brown, fine sand, moderate plasticity						
	25							
⊻			CL		■			
	30	brown			■	23	105	
-10.5		Bottom of Boring at 32½ feet						
	35							



GROUND WATER OBSERVATIONS:
⊻ : FREE GROUND WATER MEASURED DURING DRILLING AT 26.5 FEET

TRC

EXPLORATORY BORING: EB-2

Sheet 1 of 1

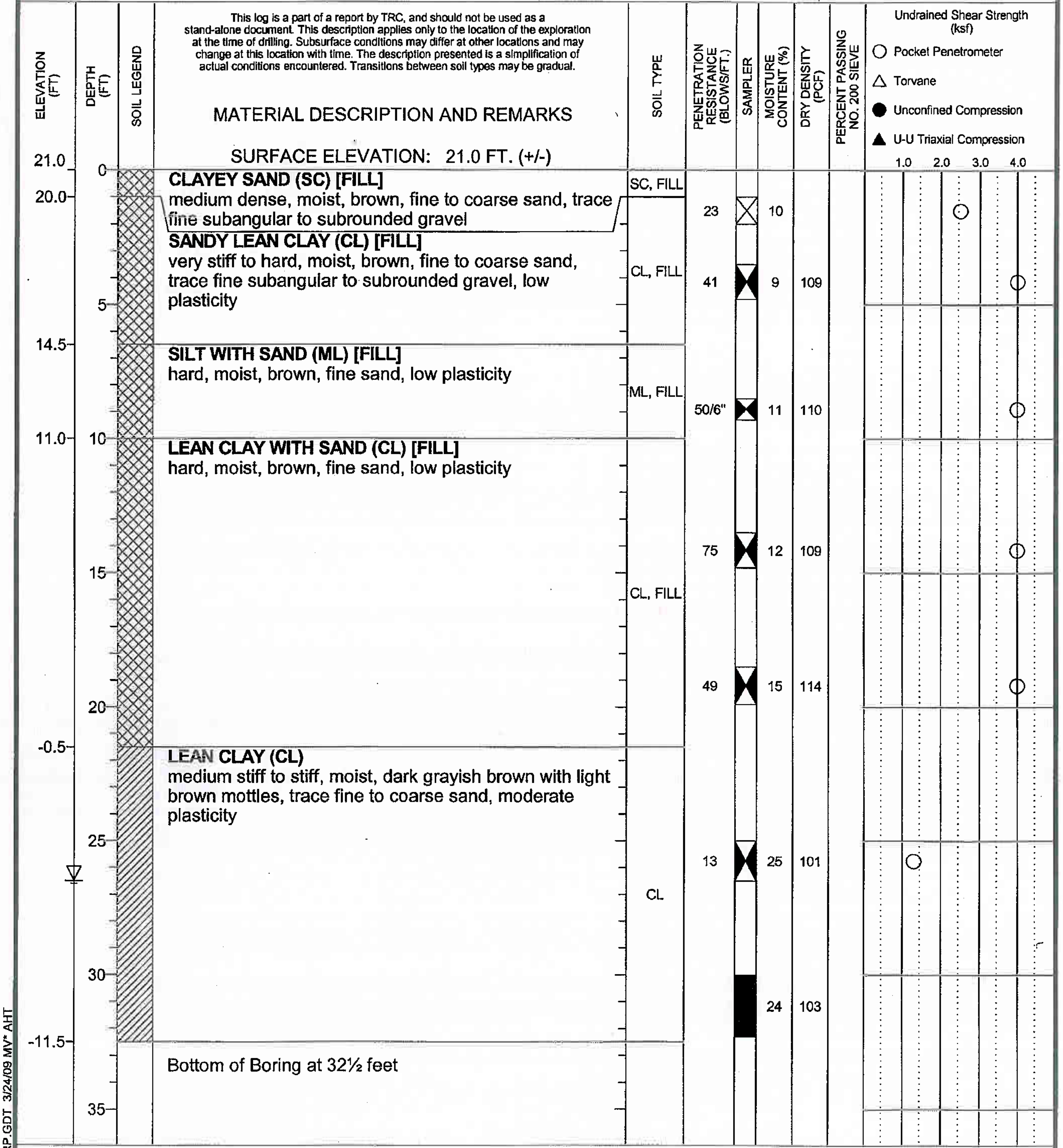
DRILL RIG: MOBIL B-53
BORING TYPE: 8-INCH HOLLOW STEM AUGER
LOGGED BY: AC
START DATE: 2-27-09 FINISH DATE: 2-27-09

PROJECT NO: 161552
PROJECT: VTA LEVEE EVALUATION
LOCATION: ALVISO, CA
COMPLETION DEPTH: 32.5 FT.

This log is a part of a report by TRC, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

MATERIAL DESCRIPTION AND REMARKS

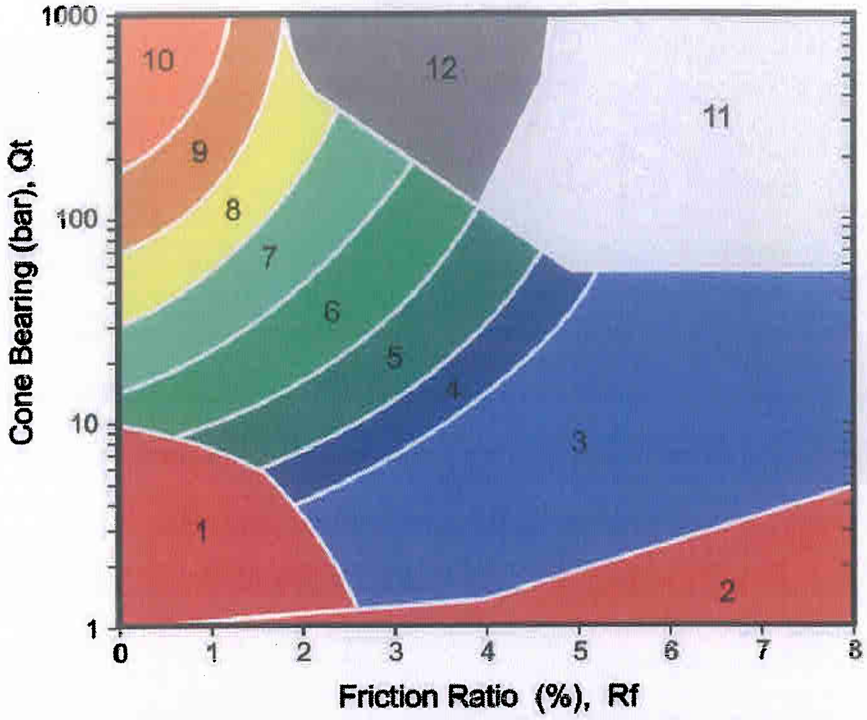
SURFACE ELEVATION: 21.0 FT. (+/-)



ELEVATION (FT)	DEPTH (FT)	MATERIAL DESCRIPTION AND REMARKS	SOIL TYPE	PENETRATION RESISTANCE (BLOWS/FT.)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)
21.0	0	**CLAYEY SAND (SC) [FILL]** medium dense, moist, brown, fine to coarse sand, trace fine subangular to subrounded gravel	SC, FILL	23	10	
20.0		**SANDY LEAN CLAY (CL) [FILL]** very stiff to hard, moist, brown, fine to coarse sand, trace fine subangular to subrounded gravel, low plasticity	CL, FILL	41	9	109
14.5		**SILT WITH SAND (ML) [FILL]** hard, moist, brown, fine sand, low plasticity	ML, FILL	50/6"	11	110
11.0	10	**LEAN CLAY WITH SAND (CL) [FILL]** hard, moist, brown, fine sand, low plasticity	CL, FILL	75	12	109
				49	15	114
-0.5		**LEAN CLAY (CL)** medium stiff to stiff, moist, dark grayish brown with light brown mottles, trace fine to coarse sand, moderate plasticity	CL	13	25	101
					24	103
-11.5		Bottom of Boring at 32½ feet				

GROUND WATER OBSERVATIONS:
∇: FREE GROUND WATER MEASURED DURING DRILLING AT 26.5 FEET

TRC



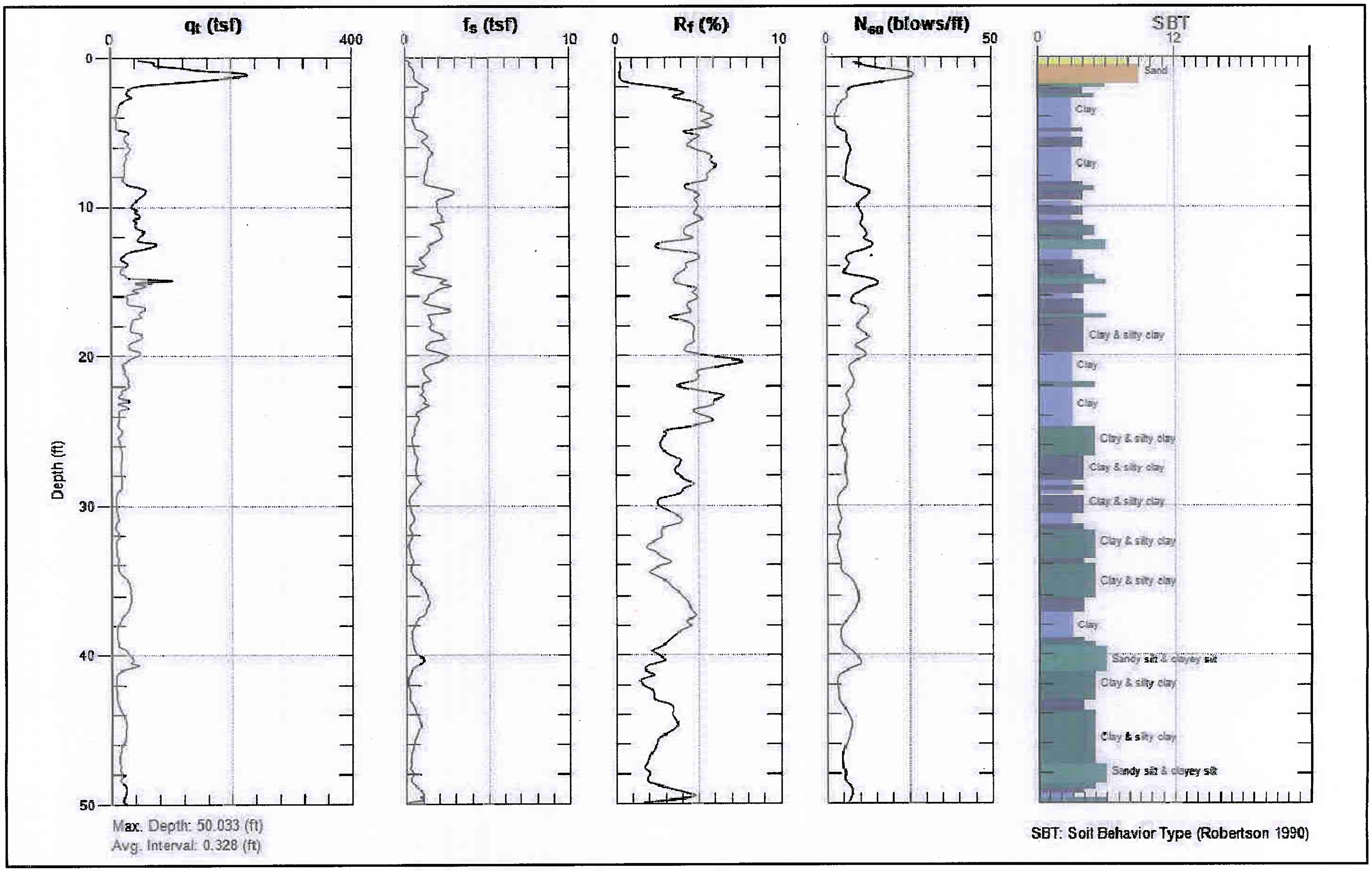
Zone		Q_t / N	Soil Behaviour Type
1	🟥	2	sensitive fine grained
2	🟥	1	organic material
3	🟦	1	clay
4	🟦	1.5	silty clay to clay
5	🟩	2	clayey silt to silty clay
6	🟩	2.5	sandy silt to clayey silt
7	🟩	3	silty sand to sandy silt
8	🟨	4	sand to silty sand
9	🟧	5	sand
10	🟧	6	gravelly sand to sand
11	⬜	1	very stiff fine grained *
12	⬛	2	sand to clayey sand *

* overconsolidated or cemented

Robertson (1990)

KEY TO CONE PENETROMETER TEST

TRC



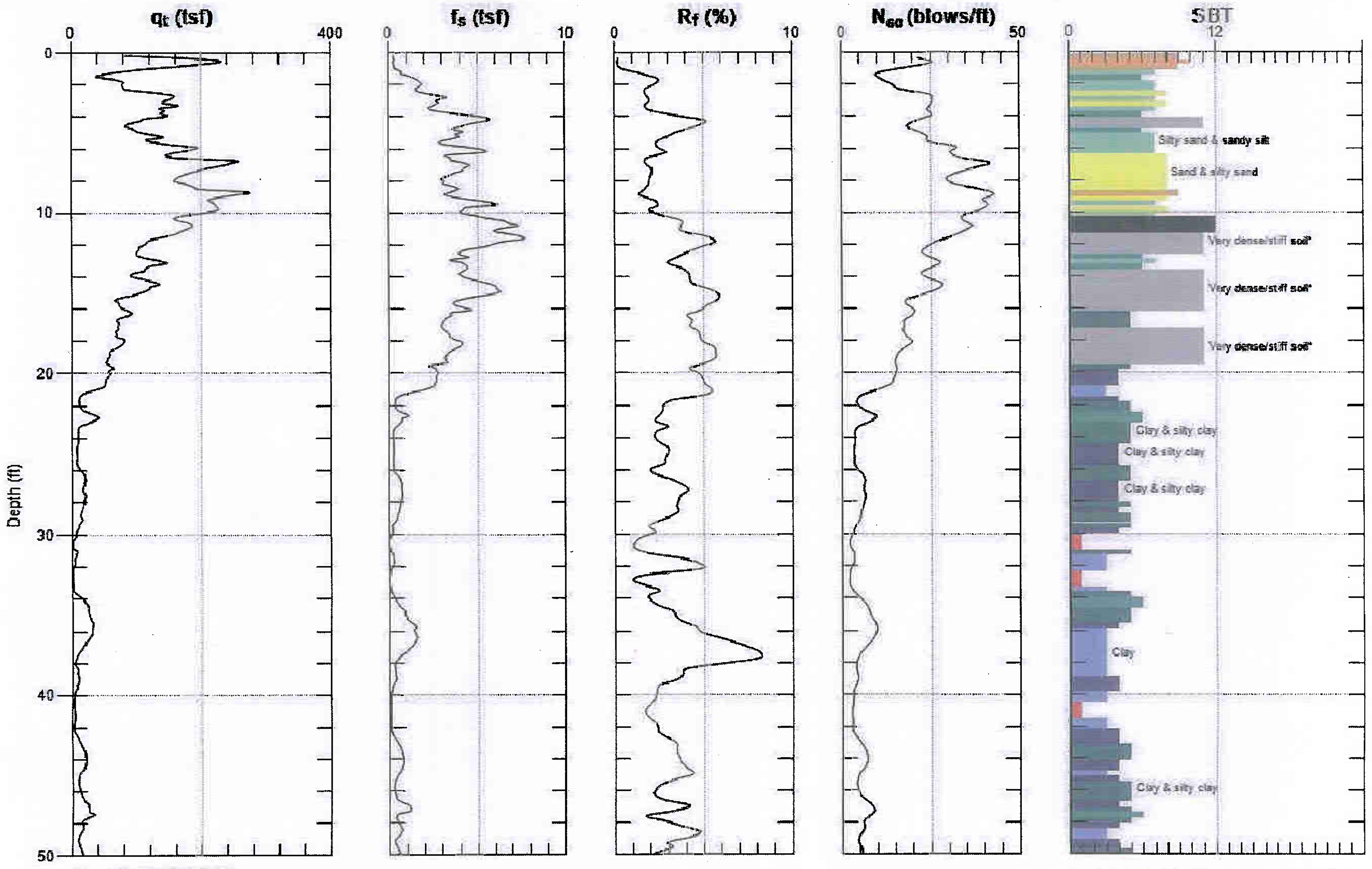
3/09*EB

CONE PENETRATION TEST - CPT-1

VTA LEVEE EVALUATION
San Jose, California

TRC

CPT-1
1437-4/161552



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

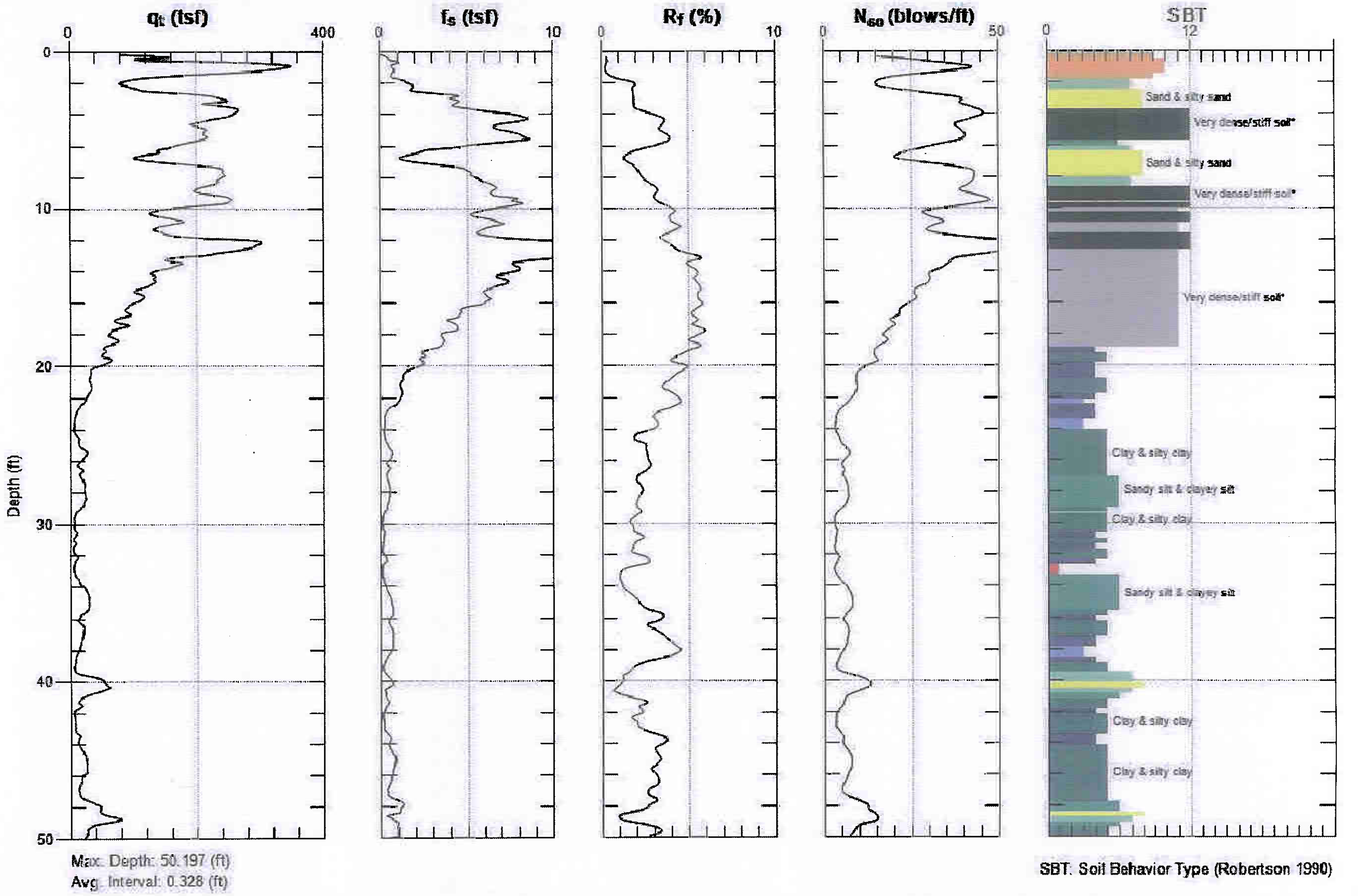
SBT: Soil Behavior Type (Robertson 1990)

3/09*EB

CONE PENETRATION TEST - CPT-2

VTA LEVEE EVALUATION
San Jose, California

CPT-2
1437-4/161552



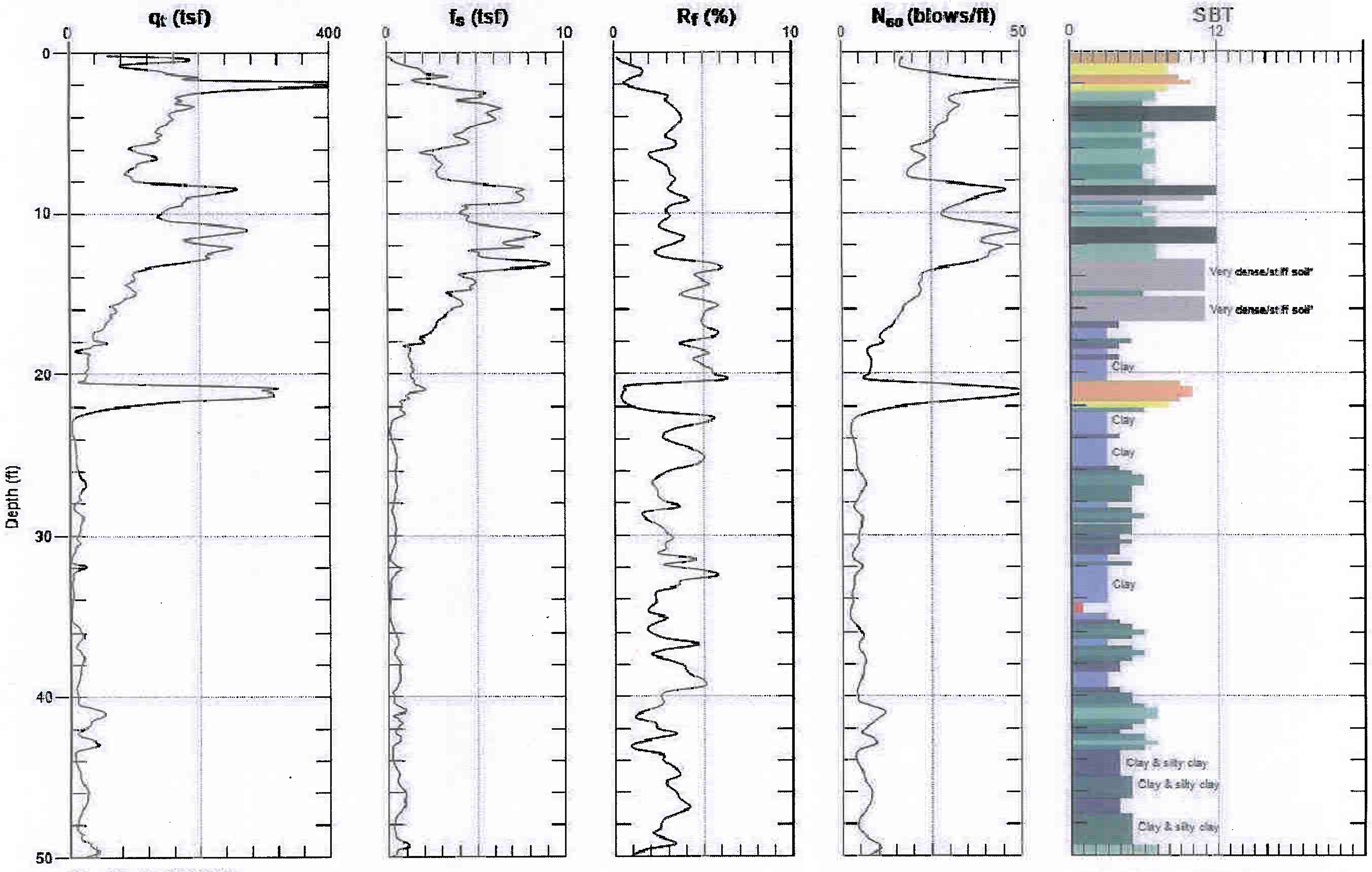
3/09*EB

CONE PENETRATION TEST - CPT-3

VTA LEVEE EVALUATION
San Jose, California

TRC

CPT-3
1437-4/161552



Max. Depth: 50.197 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

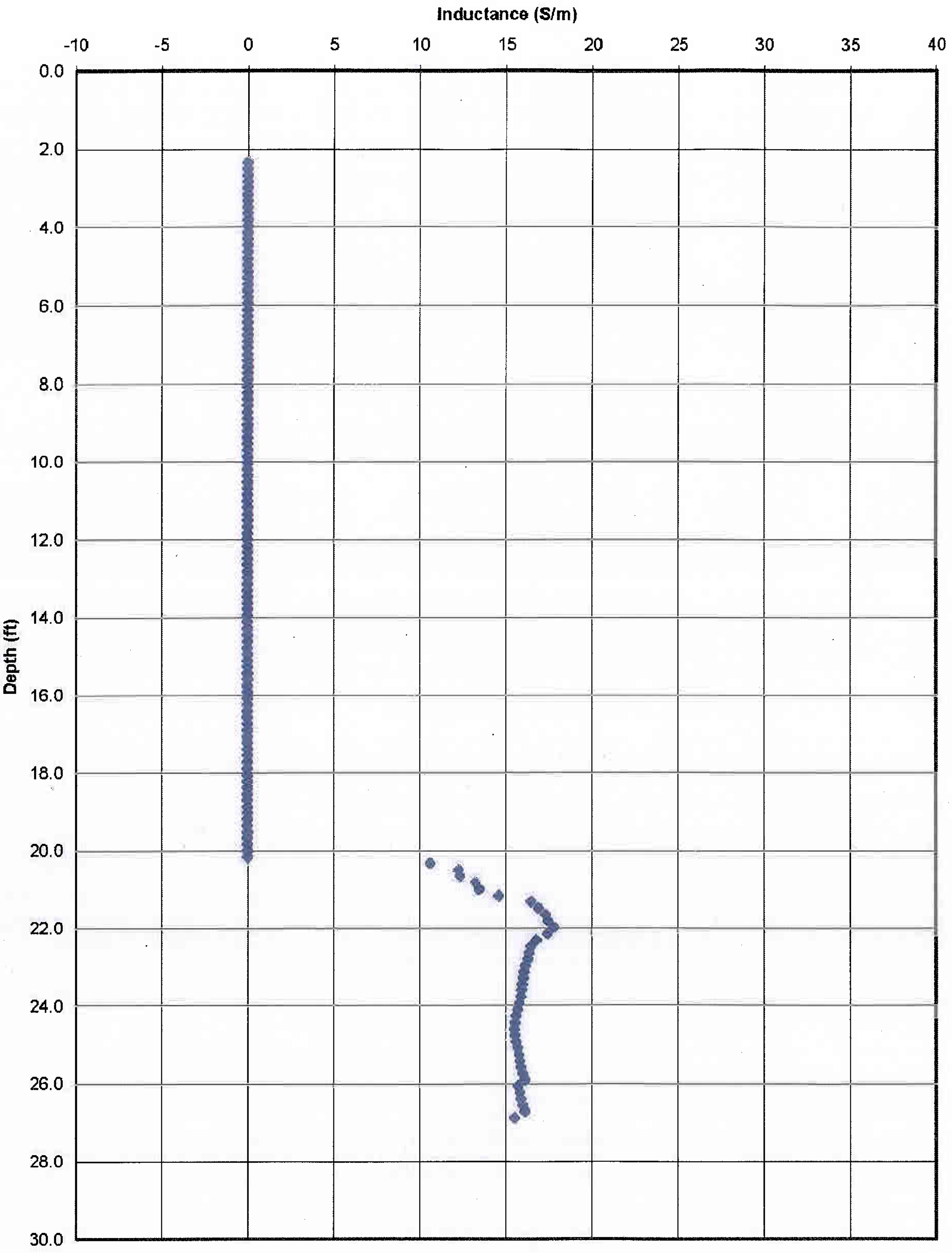
3/09*EB

CONE PENETRATION TEST - CPT-4

VTA LEVEE EVALUATION
San Jose, California

TRC

CPT-4
1437-4/161552



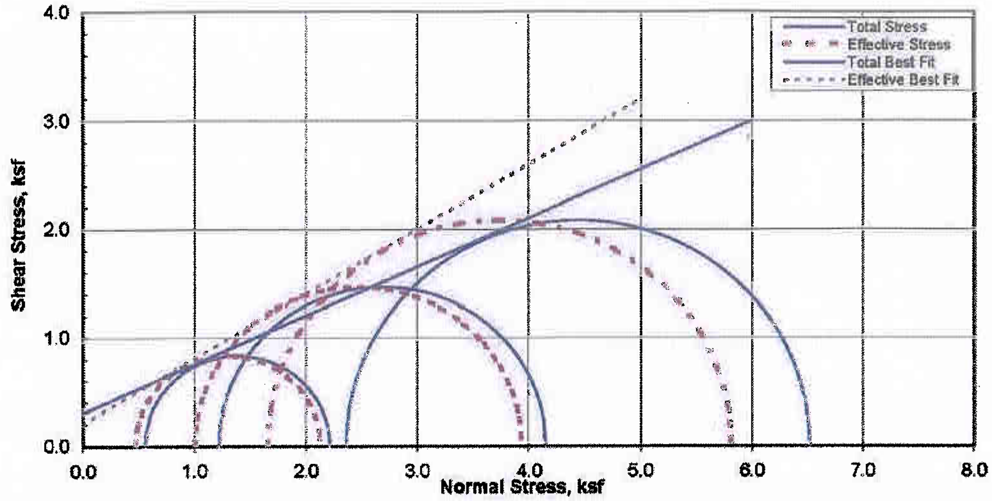
INDUCTION LOG

VTA LEVEE EVALUATION
San Jose, California

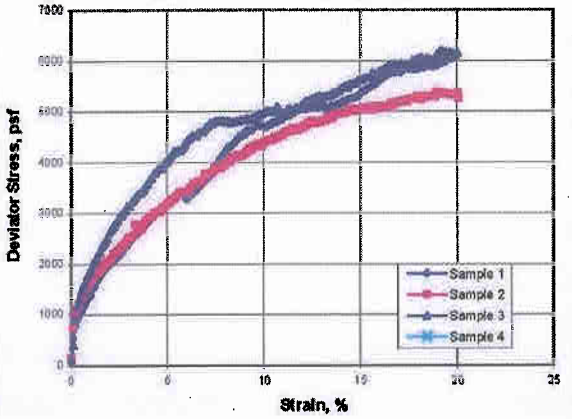
FIGURE A-2
1437-4/161552

Triaxial Consolidated Undrained

(ASTM D4767)



Stress-Strain Response



Job No.: 028-2183 **Date:** 3/10/2009

Client: TRC BY:MD/DC

Project: VTA Levee Review - 161552

Sample 1) EB-1;3A @ 9' Brown CLAY grading to Drk Br CLAY w/Sa

Sample 2) EB-1;4A @ 14' Brown CLAY (Silty)

Sample 3) EB-2;5A @ 19' Dark Gray CLAY (Silty)

Sample 4)

REMARKS: Strengths picked at the peak stress ratio.

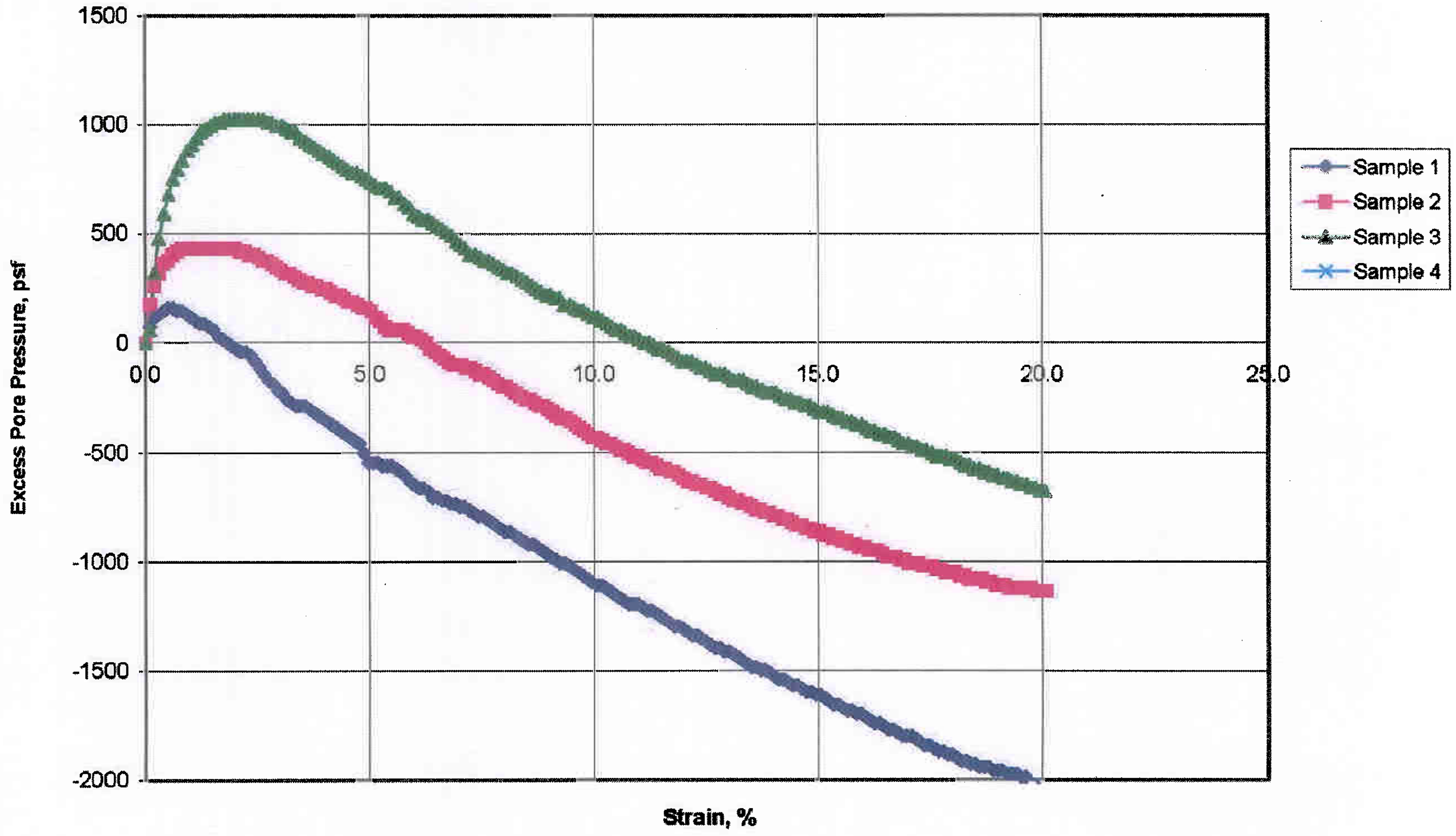
Sample:	1	2	3	4
MC, %	15.8	21.8	19.7	
Dry Dens, pcf	114.6	106.1	109.4	
Sat. %	90.6	97.2	98.5	
Void Ratio	0.470	0.617	0.540	
Diameter In	2.42	2.41	2.41	
Height, In	5.00	5.00	5.00	
	Final			
MC, %	17.8	22.9	20.0	
Dry Dens, pcf	113.8	105.3	109.4	
Sat. %	100.0	100.0	100.0	
Void Ratio	0.481	0.630	0.540	
Diameter, In	2.43	2.43	2.42	
Height, In	4.98	4.97	4.95	
Cell, psi	42.7	46.8	55.2	
BP, psi	38.8	38.3	38.8	
	Effective Stresses At:			
Strain, %	1.4	4.3	5.4	
Deviator ksf	1.656	2.929	4.161	
Excess PP	0.086	0.216	0.706	
Sigma 1	2.131	3.937	5.817	
Sigma 3	0.475	1.008	1.656	
P, ksf	1.303	2.472	3.736	
Q, ksf	0.828	1.464	2.080	
Stress Ratio	4.484	3.905	3.513	
Rate in/min	0.001	0.001	0.001	
Total C	0.3	ksf		
Total Phi	24.2	Degrees		
Eff. C	0.2	ksf		
Eff. Phi	31.0	Degrees		

TRIAXIAL CONSOLIDATED UNDRAINED

VTA LEVEE EVALUATION
San Jose, California

TRC

FIGURE B-1
1437-4/161552

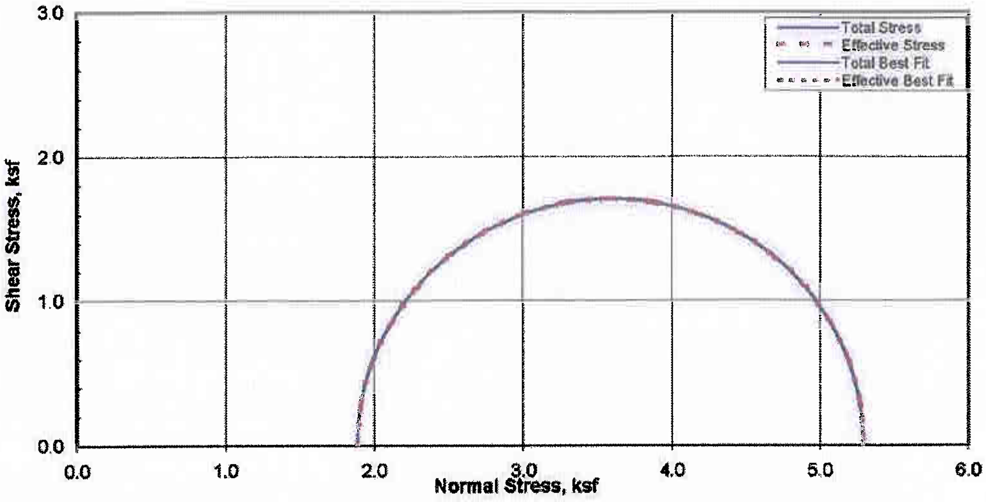


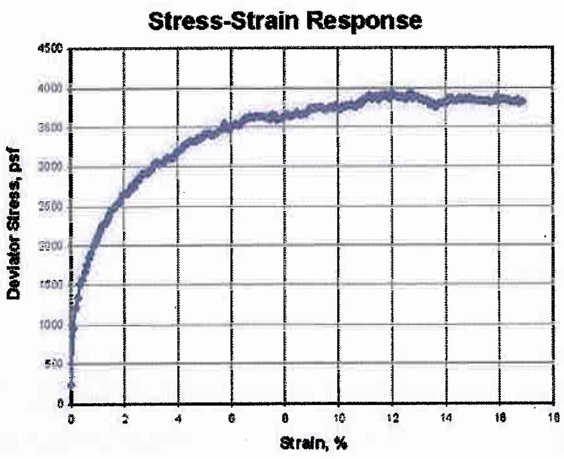
PORE PRESSURE RESPONSE
VTA LEVEE EVALUATION
San Jose, California

FIGURE B-2
1437-4/161552

Triaxial Consolidated Undrained

(ASTM D4767)





Sample:	1	2	3	4
MC, %	22.9			
Dry Dens, pcf	103.3			
Sat. %	98.1			
Void Ratio	0.632			
Diameter In	2.86			
Height, In	6.00			
	Final			
MC, %	21.5			
Dry Dens, pcf	106.6			
Sat. %	100.0			
Void Ratio	0.580			
Diameter, In	2.83			
Height, In	5.92			
Cell, psi	52.4			
BP, psi	39.3			
	Effective Stresses At:			
Strain, %	5.0			
Deviator ksf	3.410			
Excess PP	0.000			
Sigma 1	5.296			
Sigma 3	1.886			
P, ksf	3.591			
Q, ksf	1.705			
Stress Ratio	2.807			
Rate in/min	0.001			
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	N/A	ksf		
Eff. Phi	N/A	Degrees		

Job No.: 028-2183 Date: 3/10/2009

Client: TRC BY: MD/DC

Project: VTA Levee Review - 161552

Sample 1) EB-1;7A @ 30.0' Brown Sa CLAY near Cl SAND w/Gr

Sample 2)

Sample 3)

Sample 4)

REMARKS: Strengths picked at 5% strain.

TRIAXIAL CONSOLIDATED UNDRAINED

VTA LEVEE EVALUATION

San Jose, California

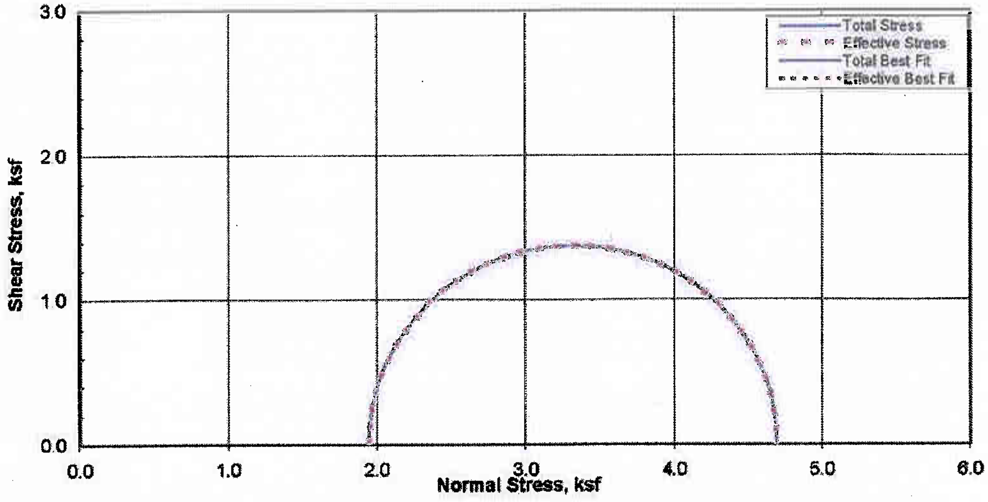
TRC

FIGURE B-3

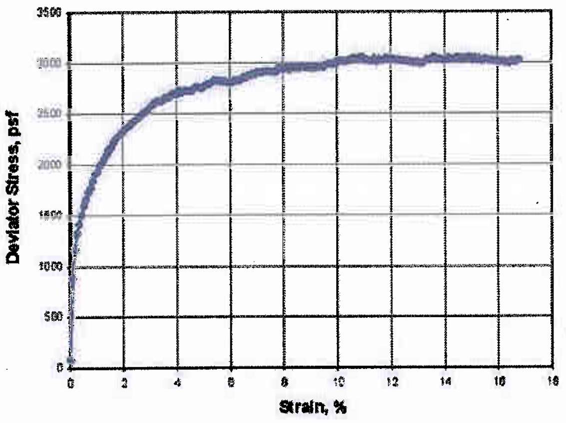
1437-4/161552

Triaxial Consolidated Undrained

(ASTM D4767)



Stress-Strain Response



Sample:	1	2	3	4
MC, %	23.5			
Dry Dens, pcf	103.2			
Sat. %	97.3			
Void Ratio	0.663			
Diameter in	2.86			
Height, in	6.00			
		Final		
MC, %	21.5			
Dry Dens, pcf	107.8			
Sat. %	100.0			
Void Ratio	0.591			
Diameter, in	2.81			
Height, in	5.93			
Cell, psi	52.4			
BP, psi	38.9			
		Effective Stresses At:		
Strain, %	5.0			
Deviator ksf	2.752			
Excess PP	0.000			
Sigma 1	4.696			
Sigma 3	1.944			
P, ksf	3.320			
Q, ksf	1.376			
Stress Ratio	2.415			
Rate in/min	0.001			
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	N/A	ksf		
Eff. Phi	N/A	Degrees		

Job No.: 028-2183 **Date:** 3/10/2009
Client: TRC BY:MD/DC
Project: VTA Levee Review - 161552
Sample 1) EB-2;7A @ 30.0' Brown Sandy CLAY w/ Gravel
Sample 2)
Sample 3)
Sample 4)
REMARKS: Strengths picked at 5% strain.

TRIAXIAL CONSOLIDATED UNDRAINED

VTA LEVEE EVALUATION
San Jose, California

TRC

FIGURE B-4
1437-4/161552

TRC

TRC Engineers
405 Clyde Avenue
Mountain View, CA 94043

Main 650.967.2365
Fax 650.967.2785

Memorandum

To:	Dr. Jim Schaaf, P.E. Schaaf and Wheeler	**From:**	Brian Hubel, P.E., G.E.
		Date:	November 10, 2008
CC:		**Project Name:**	VTA Levee Review
CC:		**Project No.:**	1437-4/161552
CC			
Subject:	**Preliminary Geotechnical Review**		

BACKGROUND

We understand that in 1997 and 1998 the Santa Clara Valley Transportation Authority (VTA) constructed an approximately 2.3 acre wetland area adjacent to the west side of the Guadalupe River in northern San Jose. The project is located adjacent to and north of the Highway 237/Guadalupe River crossing. The wetland area was established by construction of a setback levee approximately 1,300 feet long that connects to the Guadalupe River Levee. The setback levee is approximately 18 feet high, has a top-width of approximately 18 feet, and has creek side and dry side slopes of approximately 2:1 (horizontal: vertical). Water is intended to be regulated into the wetland mitigation area through two 48-inch culverts penetrating the Guadalupe River Levee.

We understand that the wetlands area has not been operated since completion of construction in 1999 because the levee has not yet been certified for FEMA-level flood protection. Our scope of work is to review previously collected subsurface geotechnical information, previous analyses and reports, and survey information , to provide a geotechnical engineering opinion regarding the 'certifiability' of the of the levee. For our review we were provided with the following information

- A geotechnical report titled, "Geotechnical Investigation, Proposed Levee and Improvements Guadalupe River Site, San Jose, California," prepared by Geo/Resource Consultants, Inc. in January 1997.
- A geotechnical construction observation summary report titled, "Construction Observation Tasman West Wetlands, San Jose, California," dated February 1999, prepared by Geo/Resource Consultants, Inc.
- A supplemental geotechnical analysis report titled, "Supplemental Stability Analysis, Tasman West Wetlands, San Jose, California," dated February 1999, prepared by Geo/Resource Consultants, Inc.
- An as-built report titled, "Tasman Corridor Project (Phase I) As-Built Plan for Wetland Mitigation Site, U.S. Army Corps of Engineers File # 18881S92," dated February 3, 1999, prepared by H.T. Harvey & Associates Ecological Consultants.
- Copies of FEMA certification forms Sections 7, 9 and 11 transmitted to the Santa Clara Valley Water District on September 26, 2000 from H.T. Harvey & Associates Ecological Consultants.
- A limited topographic survey of the levee crest and selected cross sections prepared by Ruggeri-Jensen-Azar & Associates, transmitted via email on October 24, 2008.

- As-built plan sheets 5 and 6 titled, "Lower Guadalupe River Flood Protection Project, Plan and Profile, Station 7+400 to Station 7+900," prepared by CH2MHill, dated June 6, 2002.

LEVEE DESIGN REQUIREMENTS FOR FEMA CERTIFICATION

Below, we have briefly summarized the FEMA design requirements for a levee to be certified to provide flood protection. In general, flood protection levees can be certified by local jurisdictions or may be certified by a federal agency such as the Army Corps of Engineers (COE).

- **Freeboard** – Freeboard refers to the vertical distance between the top of the channel embankment and the water surface elevation for the design basis flood. FEMA generally requires a minimum freeboard of 3 feet. An additional 1 foot above the minimum is required on the creekside of the channel embankment within 100 feet on either side of structures, such as bridges. FEMA requirements are described in 44 CFR part 65 of the Federal Code of Regulations. In the past, the COE had similar freeboard requirements. More recently, the COE has changed to a risk-based evaluation of water surface levels coupled with a deterministic channel embankment design for the geotechnical conditions.

  We understand that flood level and freeboard requirements are being evaluated by Schaaf and Wheeler for this project. We also understand that the levee hast a crest elevation of approximately Elevations 20½ to 22 feet and that the base flood elevation is approximately 18 feet. We understand that the as-built plans and current survey are referenced to the NAVD datum.

- **Embankment Erosion Protection** – FEMA requires that engineering analyses be provided to demonstrate that no appreciable erosion of the channel embankment would be expected during the design basis flood as a result of currents or waves, and that anticipated erosion will not result in failure of the channel embankment or foundation directly or indirectly through reduction of the seepage path and subsequent instability. Erosion protection considerations are briefly discussed below.

- **Embankment and Foundation Stability** – FEMA requires engineering analyses that evaluate channel embankment stability. These analyses are generally performed to conform to COE requirements for End-of-Construction, Sudden Drawdown, Steady Seepage at Base Flood Level, and the Earthquake case (COE EM 1110-2-1913, 2000). The end of construction and earthquake cases would be applicable to both creekside and dryside slopes. The steady seepage case is applicable only to the dryside of the levee, and the sudden drawdown case is only applicable to the creekside of the levee.

- **Settlement** – Engineering analyses are required that assess the potential and magnitude of future losses of freeboard as a result of channel embankment settlement and demonstrate that the minimum required freeboard will be maintained.

- **Interior Drainage** – An evaluation of the joint probability of interior and exterior flooding occurring and the adequacy of the facility to evacuate interior floodwater is required. This evaluation should be provided by the project Civil Engineer.

In addition to meeting the design criteria, both Operation Plans and Criteria and a Maintenance Plans and Criteria must be established for the facility. Specific requirements for these plans are not discussed here. Our scope of work is intended to asses the 'certifiability' of the levee in regard to the geotechnical aspects of Embankment and Foundation Stability, and Settlement. In general, our analyses follow US Army Corps of Engineers design guidance for levees.

CONDITIONS AND ANALYSES

Below, we discuss the basis of our assumptions, our analyses, and our preliminary conclusions regarding certification of the levee. Some of the soil parameters used vary from analyses previously performed by others, and lead to different analytical results. Some of these differences are due to differences in engineering judgment and other differences are a result of changes in engineering practice over the past 10 years, in particular related to the evaluation of seismically induced liquefaction and lateral spreading.

Surface Conditions

Based on current survey information and on the 2002 as-built plans it appears that the crest of the levee ranges from about Elevation 20½ to 22 feet (NAVD) and that both the creek side and dry sides of the levee were constructed with slopes of approximately 2:1 (H:V). The toe of the creek side slope has an elevation of about 3 to 6 feet. At the south end of the project there is an approximately 4-foot high vertical condition supported by sheet piles of unknown length. The top of the exposed sheet pile is at Elevation 3 feet and the lower ground is about Elevation -1 feet. Ponded water is observed adjacent to the sheet pile, indicating that the stabilized ground water level is likely between Elevation -1 and 1 feet. Landside levee toes are higher, with elevations of about 13 to 14 feet. During a site visit, some rodent holes were observed in the creek side levee face.

Subsurface Conditions

Geo/Resources' 1997 geotechnical investigation included six (6) exploratory borings. Four of these were located within the proposed levee alignment and two were performed on the existing Guadalupe River Levee, where the proposed culverts were proposed to cross the levee. The borings performed along the proposed levee alignment are identified as B-1, B-2, B-5, and B-6. In addition, two test-pits identified as TP-A and TP-B were performed along the south edge of the wetland area near Highway 237. We understand that Geo/Resource Consultants used the Nation Geodetic Vertical Datum (NGVD) for reference.

Boring Log B-1 indicates that below the surface elevation of approximately -2½ feet, the soils consist of:

- medium stiff clay (CH) to a depth of approximately 9 feet over
- medium stiff sandy clay (CL) to a depth of approximately 12 feet over
- interbedded layers (each of approximately ½ foot) of loose to medium dense clayey sand and soft to stiff sandy clay to the terminal depth of the boring at 26½ feet.

Boring Log B-2 indicates that the boring was performed at a surface elevation of approximately -1 foot , that the soil conditions consisted of:

- 2 feet of clay and rubble fill over
- soft to medium stiff sandy clay to a depth of approximately 5 feet over
- stiff clay (CH) to a depth of approximately 10 feet over
- medium stiff sandy clay (CL) to a depth of approximately 20 feet, over
- dense clayey sand and well graded sand to the terminal depth of the boring of approximately 25 feet.

Boring B-5 was performed from a surface elevation of approximately 10½ feet and encountered approximately

- 14 feet of fill consisting of 8 feet of very stiff clay (CL) and 7 feet of dense gravel fill (GW) over

- about 10 feet of soft Bay Mud silt (MH) over
- interbedded layers of stiff clay (CH) and loose clayey sand (SC) to the terminal depth of the boring at about 35 feet.

Boring B-6 was performed from a surface elevation of approximately -3 feet and encountered approximately:

- 4 feet of loose clayey sand (SC) over
- medium stiff clay (CH) to a depth of approximately 10 feet over
- medium dense clayey sand (SC) to a depth of approximately 15 feet over
- medium stiff to stiff clays and silts (CH, ML, MH) to a depth of 25 feet., the terminal depth of the boring.

These borings were performed with hollow-stem auger drilling equipment. Soils were primarily sampled with Modified California split-spoon sampling. One Shelby tube sample of clay soil was taken in boring B-2 at a depth of approximately 10 feet, and one Shelby tube sample was taken in the clayey/sandy soil at a depth of approximately 24 feet in boring B-5. At the time of drilling, ground water was encountered at elevations of approximately -6, -6, -6.5 and -8 feet in borings B-1, B-2, B-5 and B-6, respectively. Logs for test pits TP-A and TP-B reported similar conditions as boring B-6.

The 1999 Construction Observation Report prepared by Geo/Resource Consultants indicates that two import soils were used for the levee fill. "Cerone" fill was used for the lower portion of the levee and consisted of sandy clay with reported plasticity indices (PIs) of 9 to 18. "San Fernando" fill was used for the upper portion of the levee and consisted of clay with PIs between 10 and 15. Approximately 1- foot of aggregate base was placed at the top of the levee for vehicular paving. It was reported that the levee fill was placed on native soil and was generally compacted to a minimum of 95 percent relative compaction as determined by ASTM D1557.

In a 1999 Supplemental Stability Analysis by Geo/Resources Consultants, it was reported that shear strength testing was performed on the fill and foundation soils although the results were not reported in the text or included in the attachments. Laboratory permeability testing performed on the fill soil indicated hydraulic conductivities of approximately $2x10^{-6}$ cm/sec to $3 x10^{-7}$ cm/sec.

Embankment Erosion Protection

Because of the nature of the levee as set back behind the main Guadalupe River levee, we do not anticipate that stream flows will have high velocity adjacent to the levee, and that the grasses and vegetation observed on the levee will provide adequate erosion protection. Table 1, below, summarizes recommendations for erosion protection for various channel material.

Additionally, we recommend that the operation and maintenance plans include provisions regarding both vegetation and rodent control. In general, the levee should be kept free of large vegetation such as trees and bushes, and rodents should be prevented from burrowing into the levee.

Table 1. Suggested Maximum Permissible Mean Channel Velocities

Channel Material	Mean Channel Velocity (feet/sec)
Fine sand	2
Coarse sand	4
Fine gravel	6
Earth Sandy silt	2
Silty clay	3½
Clay	6
Grass-lined Earth	
Bermuda grass	
Sandy silt	6
Silty clay	8
Kentucky Blue grass	
Sandy silt	5
Silty clay	7
Poor Rock (usually sedimentary)	
Soft sandstone	10
Soft shale	3½
Good Rock (usually igneous or hard metamorphic)	20

Settlement

We anticipate that most of the major settlement for the levee is complete since the project was completed nearly 10 years ago. If new fills are required, additional settlement may occur, especially near the south end where compressible Bay Mud was identified in the borings and test pits. If raising the levee is required, additional engineering analysis will be required.

Seismic Hazards

Based on our review of maps published by the California Geologic Survey, the project site is located within an area zoned by the State as having potential for seismically induced liquefaction hazards (CGS, 2003). The CGS Seismic Hazard report indicates that a peak horizontal ground acceleration of 0.55g has a 10 percent chance of exceedance in 50 years.

Engineering practice in regard to the evaluation of seismically induced liquefaction hazards has progressed significantly in the past 10 years. In general, it is recommended that liquefaction hazard assessment be performed for projects in identified hazard zones, and that the analysis should be based on subsurface exploration to a depth of at least 50 feet below the ground surface, be based Standard Penetration Testing (SPT) with rotary wash drilling equipment or Cone Penetrometer Testing (CPT) equipment. The subsurface information for the project does not include exploration to 50 feet, or SPT, or CPT testing. Additionally, some of the conclusions drawn regarding the potential for liquefaction of clayey sands are inconsistent with current research and practice. Although it is not clear that a liquefaction hazard exists, we recommend that that additional subsurface exploration and laboratory testing be performed to evaluate the liquefaction potential in accordance with current engineering practice.

Seepage and Stability Analyses

We have performed stability analyses in accordance with US Army Corps of Engineers guidelines. Table 2, below, summarizes the recommended minimum factors of safety and our calculated factors of safety. Strength parameters used are based on boring logs and lab testing available in the engineering reports from Geo/Resource, and our engineering judgment. We evaluated two cross sections. Cross Section A is a typical levee section (such as would be encountered near Station 7+800, Sheet 5 dated 6/6/02). Cross Section B is a typical section located on the south end of the levee where the sheet-pile wall is located. Cross Section A was evaluated for both Bay Mud and general alluvium foundation conditions, as the foundation conditions appear to vary for the project. Because Bay Mud was more prominently encountered near the south end of the levee, only Bay Mud foundation conditions were evaluated for Cross Section B. Loading cases checked are described as Case I: End-of-Construction, Case II: Sudden Drawdown, Case III: Steady Seepage from Base Flood and Case IV: Earthquake. Seismic stability was performed using a horizontal pseudo-static coefficient of 0.2g.

The shear strength parameters used in our analysis are presented in Table 2. The soil parameters used are different than parameters used in the Geo/Resource 1997 geotechnical investigation, and are also different than the strength parameters used in Geo/Resource 1999 supplemental slope stability report. Seepage properties used are based on the provided hydraulic conductivity lab test results, and modified based on our judgment and experience with similar soil conditions. Shear strength soil parameters used in our analysis are also based on our experience with similar soil and project conditions. In our opinion, the laboratory data provided does not support the shear strength properties used in the previous slope stability analyses (in particular with regard to the supplemental 1999 report). We recommend additional laboratory testing be performed to verify our assumptions and judgments regarding the soil properties used in our analysis.

Table 2. Soil Properties for Seepage and Stability Analyses

Soil Layer	Vertical Hydraulic Conductivity (cm/sec)	Horizontal to Vertical Hydraulic Conductivity Ratio	Total Unit Weight (pcf)	Effective Cohesion (psf)[1]	Effective Friction Angle[1] (degrees)	Undrained Shear Strength (psf)[2]
Embankment Fill	$1x10^{-5}$	5	130	10	34	1,000
Foundation: Bay Mud	$1x10^{-6}$	5	100	0	28	0.3 x (effective stress)
Foundation: Sand Alluvium	$1x10^{-4}$	5	125	0	32	-
Foundation: Clay Alluvium	$1x10^{-5}$	5	125	0	32	1,000

---

[1] Effective strength parameters are used for Stability Cases I, II, and III. For new levee construction undrained strengths are typically used for Stability Case I, however, because the levee being evaluated is existing effective strength parameters are used. Because of the levee height the shear strengths from the "S" portion of the strength envelope (drained strengths) controlled the evaluation during the multi-step process for Case II.

[2] Undrained shear strengths are used for clay soils for Stability Case IV.

Table 3. Preliminary Slope Stability Analyses Summary

Cross Section	Side of Levee	Loading Case	Recommended Factor of Safety	Calculated Factor of Safety[3,4]
Cross Section A: alluvium foundation conditions	Creekside	Case I[5]	1.3	1.5
	Dryside	Case I	1.3	1.7
	Creekside	Case II	1.0	1.1
	Dryside	Case III	1.4	1.7
	Creekside	Case IV	1.0	1.3
	Dryside	Case IV	1.0	2.3
	Dryside & Creekside	Post-EQ Liquefied	1.0	**?**
Cross Section A: Bay Mud foundation conditions	Creekside	Case I	1.3	1.5
	Dryside	Case I	1.3	1.6
	Creekside	Case II	1.0	1.1
	Dryside	Case III	1.4	1.6
	Creekside	Case IV	1.0	1.1
	Dryside	Case IV	1.0	1.7
	Dryside & Creekside	Post-EQ Liquefied	1.0	**?**
Cross Section B: Bay Mud foundation conditions	Creekside	Case I	1.3	**1.2**
	Dryside	Case I	1.3	2.3
	Creekside	Case II	1.0	1.1
	Dryside	Case III	1.4	2.3
	Creekside	Case IV	1.0	**0.8**
	Dryside	Case IV	1.0	1.4
	Dryside & Creekside	Post-EQ Liquefied	1.0	**?**

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on the information provided, it appears that overall the levee is likely to meet FEMA certification requirements for most of the project from seepage and stability viewpoints although not enough information is available to provide a conclusive opinion at this point. Specific items recommended for additional study are discussed below:

- Liquefaction potential at the site has not been adequately addresses with respect to current engineering standards. Field exploration, laboratory testing, and engineering analysis are

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[3] Factors of safety reported in **Bold** type face are considered below satisfactory levels. Additional soil data, or other information may result in higher or lower factors of safety.

[4] Although Factors of Safety are calculated to be greater than 1 for Case II, some shallow sloughing may occur and should be maintained. Significant instability, affecting the levee integrity is not anticipated.

[5] For new levees, at the end-of construction low factors of safety are often encountered before the soil pore pressures generated during construction have dissipated. Because the levee was constructed many years ago, we evaluated the static slope stability using drained pore pressure strength parameters.

recommended to evaluate the liquefaction hazard at the site. If liquefaction hazards are present at the site, the levee could loose freeboard, laterally spread, crack, or experience other damage. It is usually judged that earthquake hazards and flood hazards do not need to be considered at the same time due the low probability of seismic and flood events occurring at the same time. However, we recommend that liquefaction hazard be evaluated to assess if potential liquefaction distress poses immediate flooding hazard from more common water levels, and that the operation and maintenance plan for the levee includes provisions for post-earthquake inspections and repairs.

- Sheet piles are supporting a cut along the southern portion of the levee. The cut destabilizes the creek side of the levee at this location to unsatisfactory levels (ignoring sheet pile structural capacity). It is not clear if the sheet piles provide adequate slope support. Additional information is needed regarding the sheet piles at this location to verify adequate slope stability.
- Shear strength parameters used in our analyses are based on relatively limited laboratory test data, and are primarily based on our experience with similar projects. Although our assumptions are judged to be reasonable, we recommend that additional field and laboratory testing be performed to verify our assumptions and judgments.

We would be pleased to meet with you to discuss these findings and the recommended next steps toward certification, if desired.

Closure

This document was prepared for the sole use of Schaaf and Wheeler for application to the VTA Wetland Mitigation Levee located on the west side of Guadalupe River, just north of Highway 237, in accordance with generally accepted geotechnical engineering practices at this time and location. No warranty is expressed or implied.

We hope this provides the information you need at this time. If you have any questions, please call and we will be glad to discuss them with you.

Sincerely,

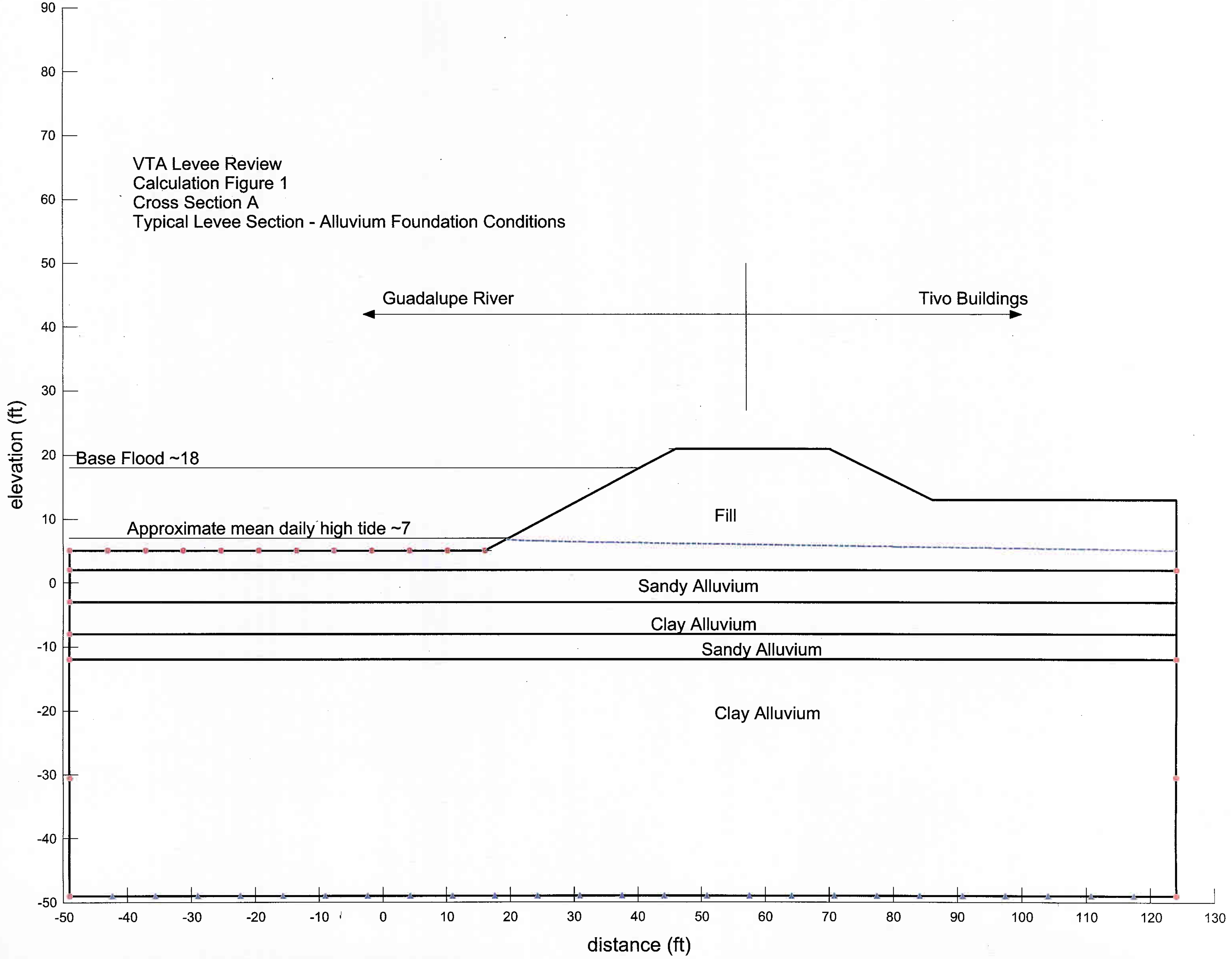
TRC Engineers

Brian Hubel, P.E., G.E.
Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
BRIAN A. HUBEL
No. GE2769
Exp. 6/30/09
GEOTECHNICAL
STATE OF CALIFORNIA

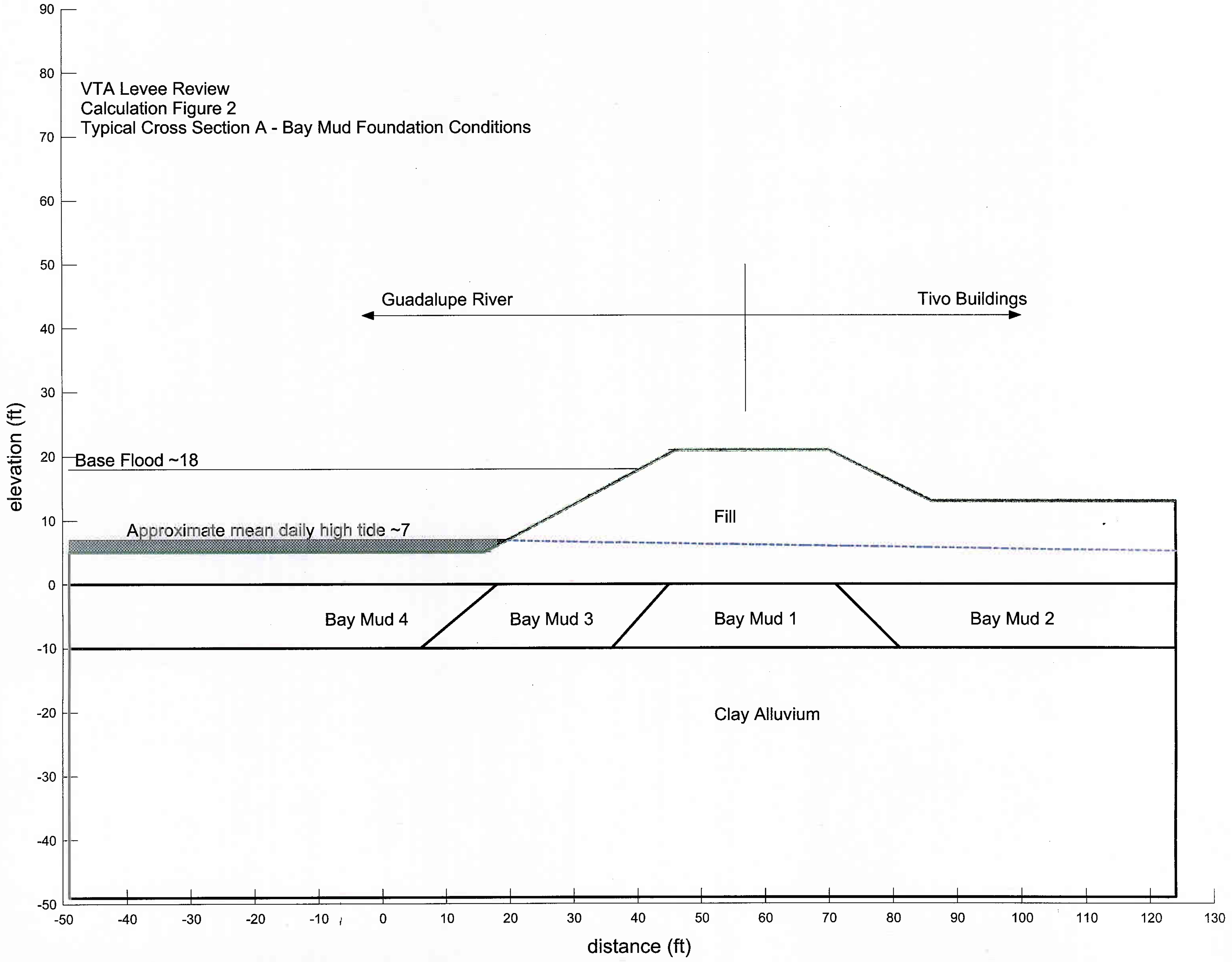
BAH:mql

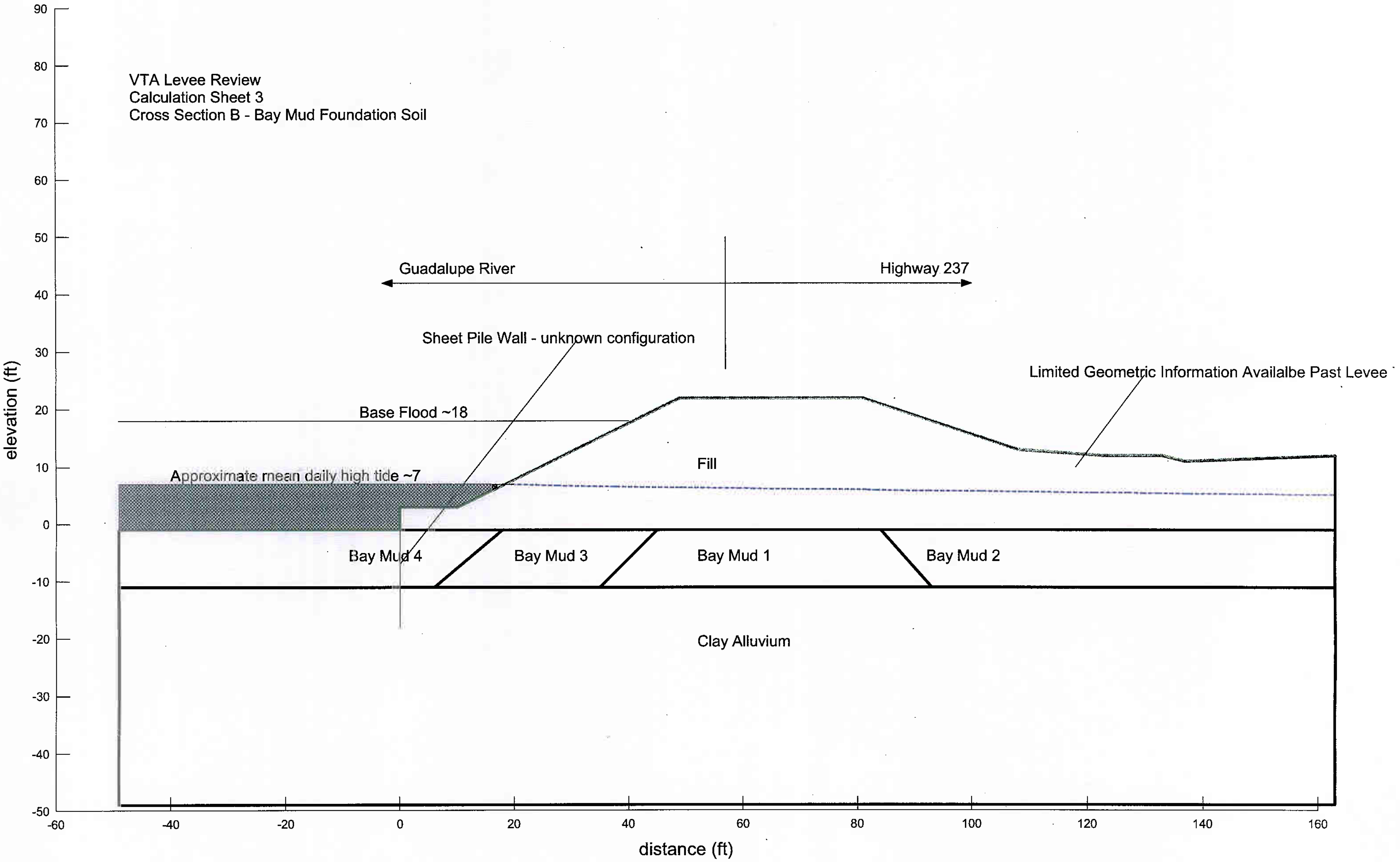
Attachments: Calculation Figure 1 Typical Stability Cross Section A for Alluvium Foundation Conditions
Calculation Figure 2 Typical Stability Cross Section A for Bay Mud Foundation Conditions
Calculation Figure 3 Typical Stability Cross Section B for Bay Mud Foundation Conditions



VTA Levee Review
Calculation Figure 1
Cross Section A
Typical Levee Section - Alluvium Foundation Conditions

VTA Levee Review
Calculation Figure 2
Typical Cross Section A - Bay Mud Foundation Conditions





VTA Levee Review
Calculation Sheet 3
Cross Section B - Bay Mud Foundation Soil

APPENDIX. B
Photo-Documentation

Photo B-1. Water-level datasonde installed during a low tide on the Guadalupe River.

Photo B-2. Water-level datasonde installed during a low tide in the Tasman wetland mitigation site.

Photo B-3. Topographic survey (19 March 2009)

Photo B-4. Vegetation monitoring (17 September 2009)

Photo B-5. Feldspar sedimentation plot; white feldspar is buried beneath a layer of new sediment.

Photo B-6. Feldspar sample cut from sedimentation plot showing a layer of sediment above the white feldspar marker horizon.

Photo B-7. Tasman Wetland Mitigation Site during a combined high tide and heavy rainfall event (Photo taken 20 January 2010).

Photo B-8. Tasman Wetland Mitigation Site (left) and Guadalupe River (right) during a high tide and heavy rainfall event (Photo taken 20 January 2010).

Photo B-9. Photopoint 1 (28 May 2009, before the tide gates were opened)

Photo B-10. Photopoint 1 (17 September 2009)

Photo B-11. Photopoint 2 (28 May 2009, before the tide gates were opened)

Photo B-12. Photopoint 2 (27 September 2009)

Photo B-13. Photopoint 6 (28 May 2009, before the tide gates were opened)

Photo B-14. Photopoint 6 (17 September 2009)

Photo B-15. Photopoint 8 (28 May 2009, before the tide gates were opened)

Photo B-16. Photopoint 8 (17 September 2009)

Photo B-17. Photopoint 12 (28 May 2009, before the tide gates were opened)

Photo B-18. Photopoint 12 (17 September 2009)

Photo B-19. Photopoint 4 showing outboard pilot channel before the tide gates were opened (28 May 2009)

Photo B-20. Photopoint 4 showing outboard pilot channel on 8 February 2010

APPENDIX C
Topographic Cross-Section Survey Results

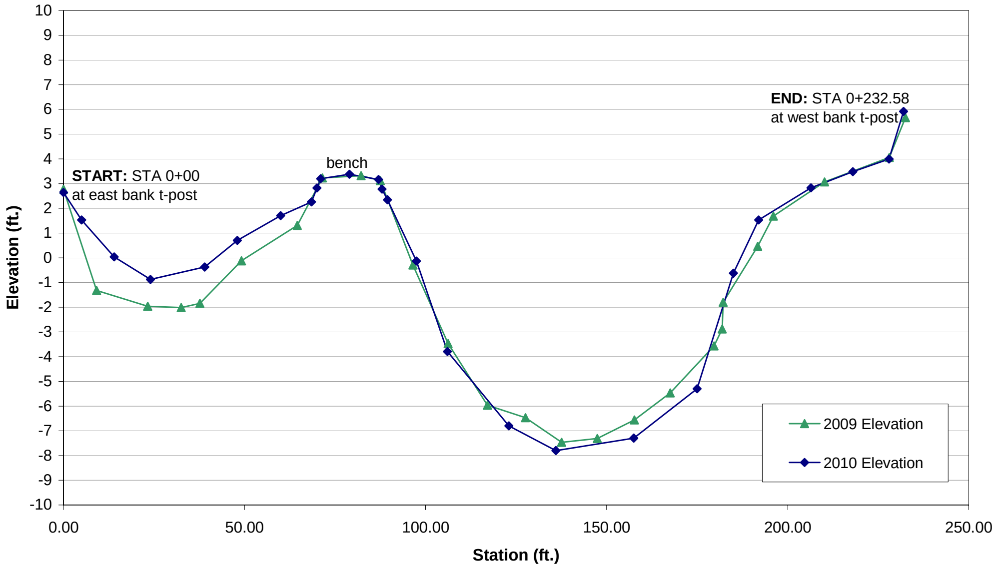


Figure C-1. Wetland Mitigation Levee; Cross-Section 1

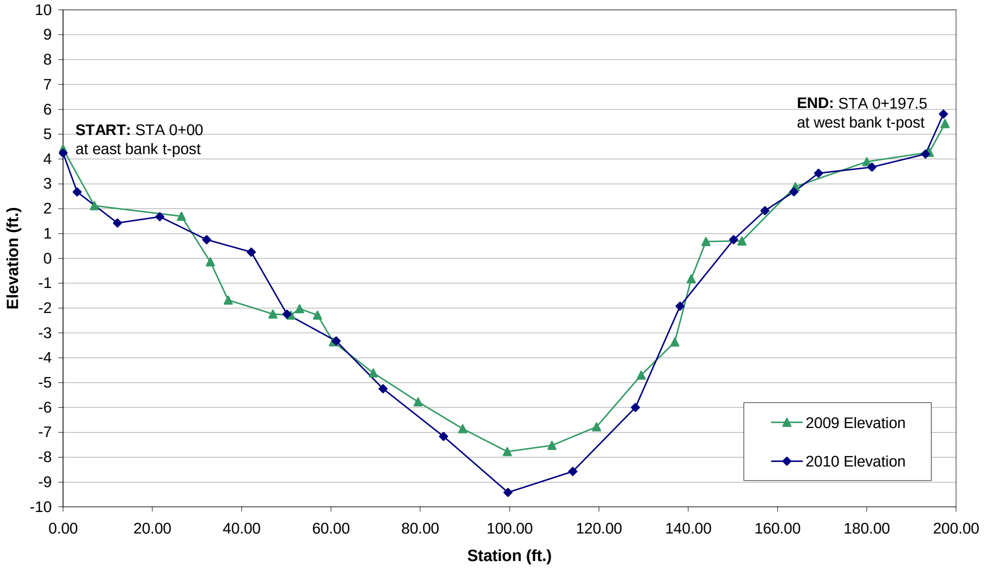


Figure C-2. Wetland Mitigation Levee; Cross-Section 2

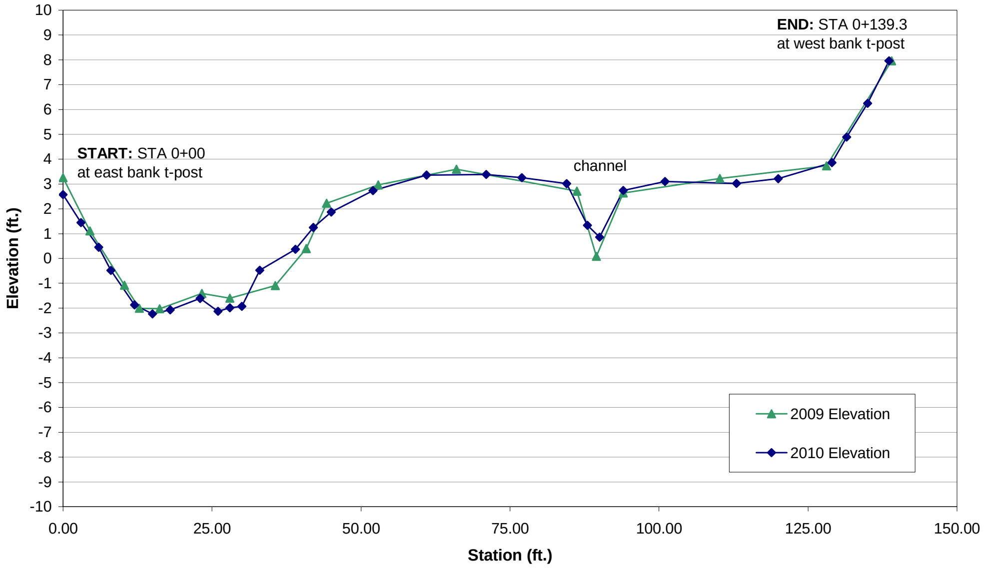


Figure C-3. Wetland Mitigation Levee; Cross-Section 3

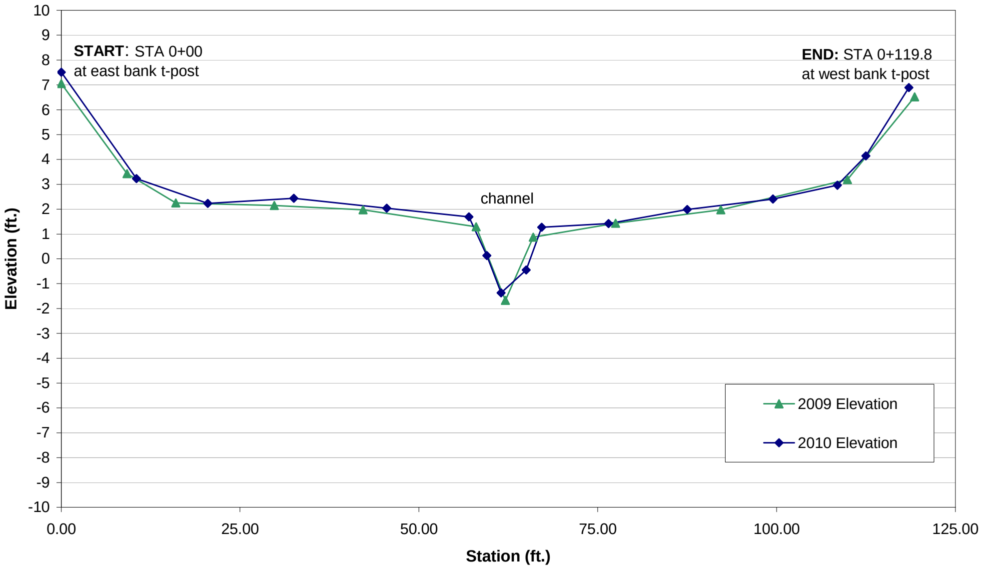


Figure C-4. Wetland Mitigation Levee; Cross-Section 4

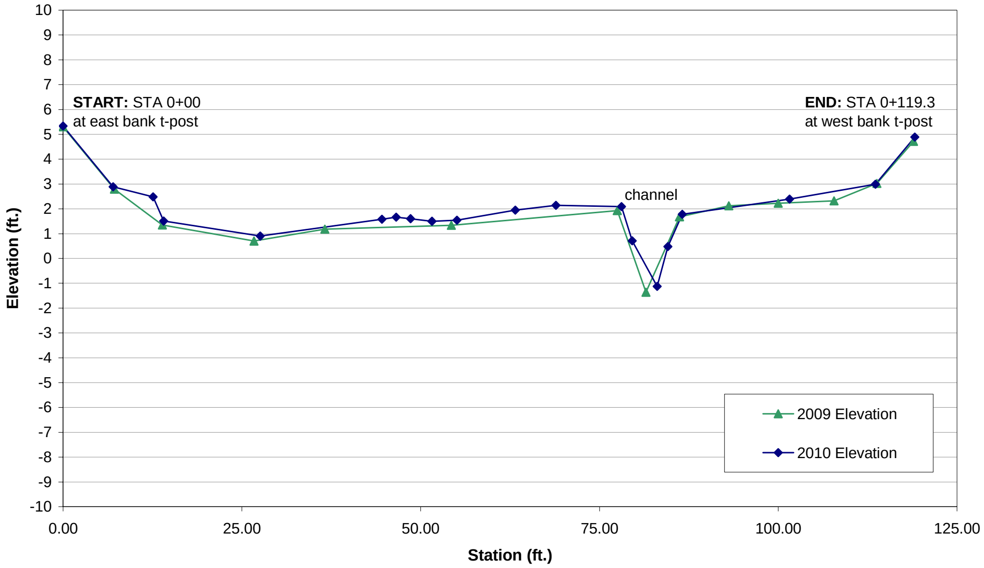


Figure C-5. Wetland Mitigation Levee; Cross-Section 5

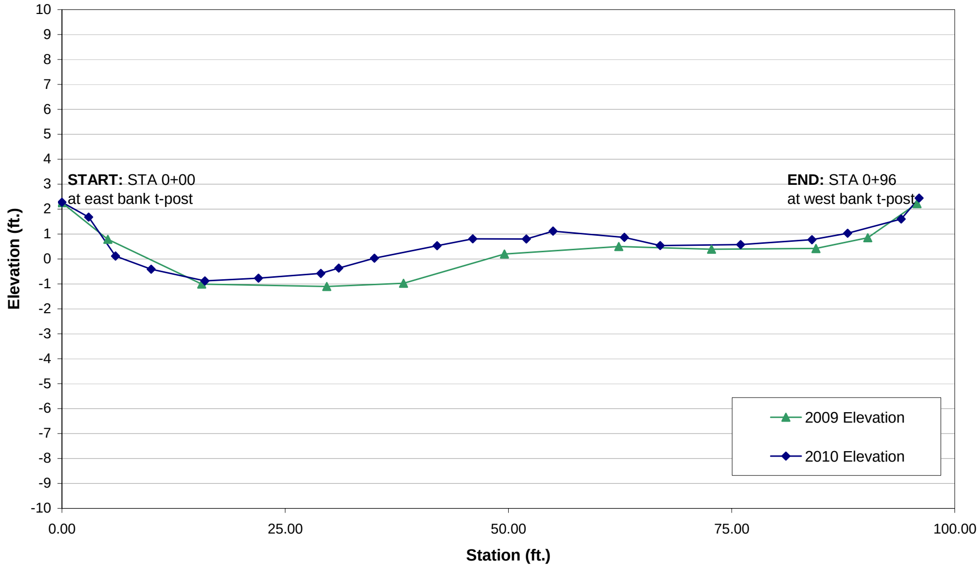


Figure C-6. Wetland Mitigation Levee; Cross-Section 6

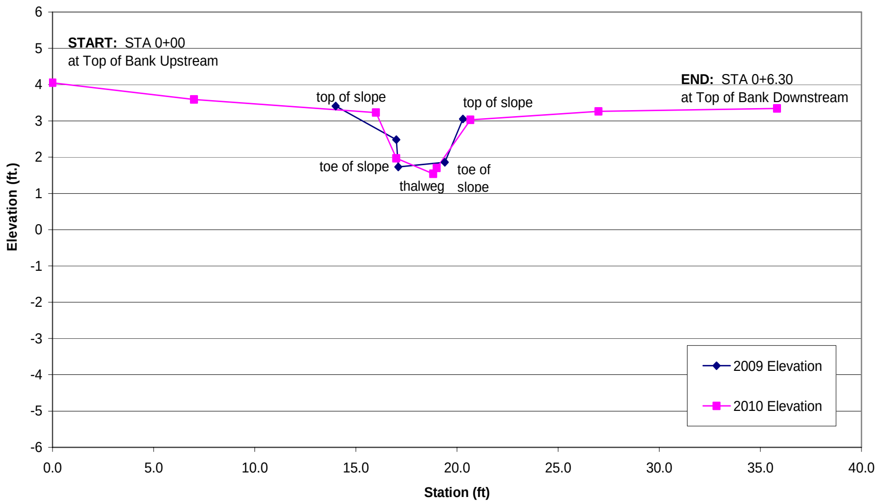


Figure C-7. Upstream Pilot Channel; Cross-Section 7

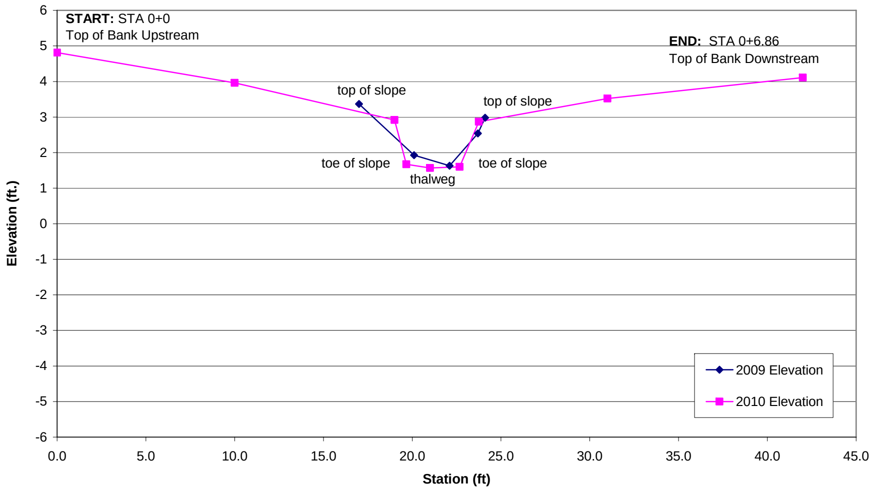


Figure C-8. Upstream Pilot Channel; Cross-Section 8

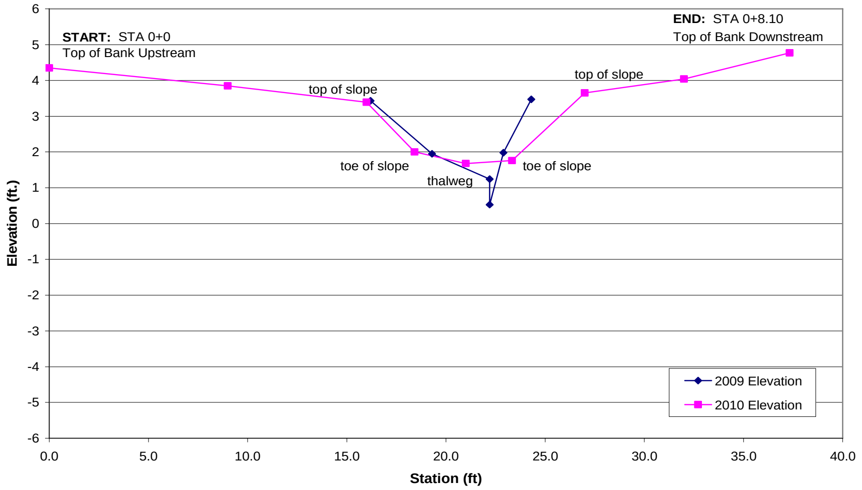


Figure C-9. Downstream Pilot Channel; Cross-Section 9

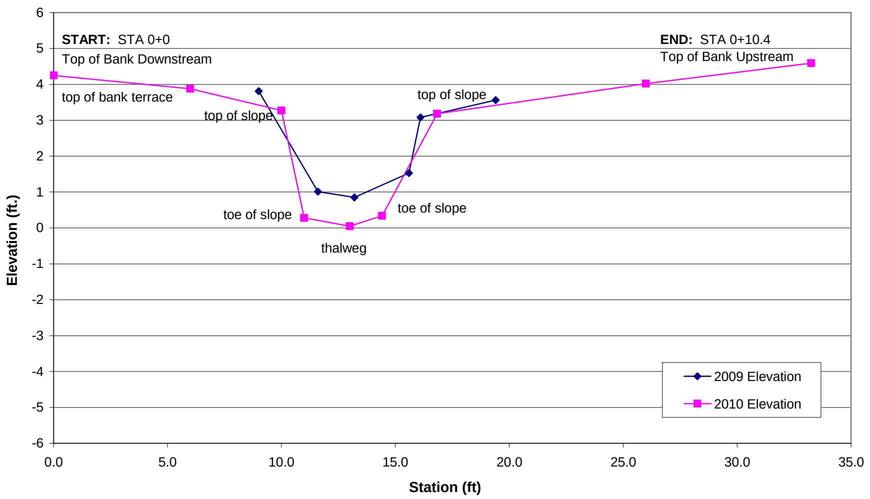


Figure C-10. Downstream Pilot Channel; Cross-Section 10

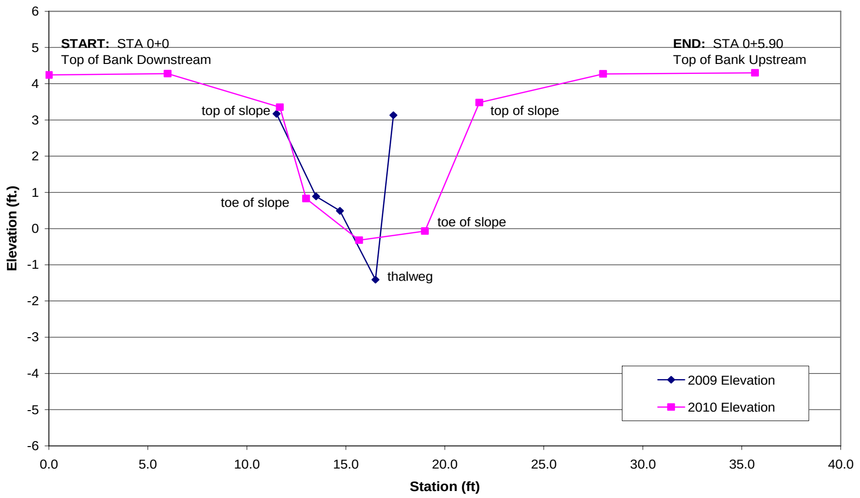


Figure C-11. Downstream Pilot Channel; Cross-Section 11

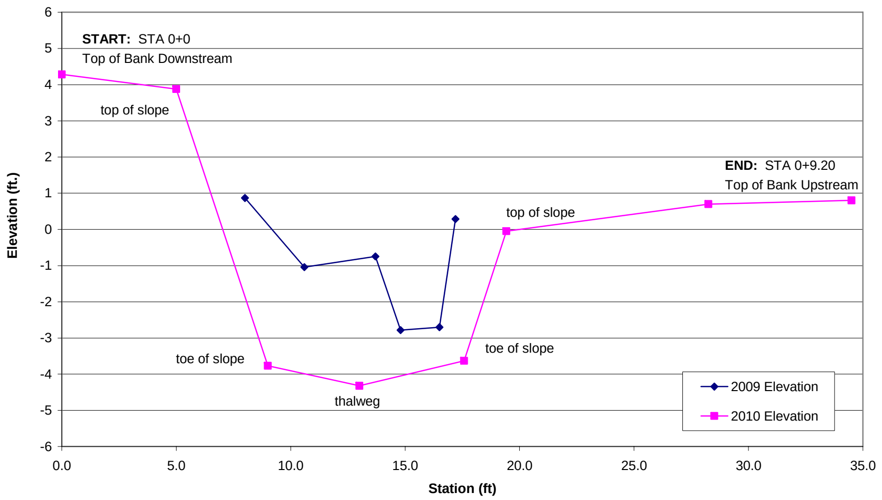


Figure C-12. Downstream Pilot Channel; Cross-Section 12