Martin Slough Enhancement Project
Eureka, California
Basis of Design Report

Michael Love & Associates
Hydrologic Solutions
PO Box 4477 • Arcata, CA 95518 • (707) 822 -2411

GHD

August 2015

Martin Slough Enhancement Project, Eureka, CA

Basis of Design Report

Prepared for:

Redwood Community Action Agency
904 G Street
Eureka CA 95501

Funded by:

California Department of Fish and Wildlife
Fisheries Restoration Grant Program, Agreement #P1110309

California Department of Water Resources
Urban Streams Restoration Program, Agreement #4600009869

Prepared by:

Michael Love

Michael Love P.E.
California Licensed Engineer, No. C71681
Principal Engineer
Michael Love & Associates, Inc.
mlove@h2odesigns.com | (707) 822-2411

Rachel Shea P.E., M.S.
California Licensed Engineer, C72614
Engineering Geomorphologist
Michael Love & Associates, Inc.

Antonio Llanos

Antonio Llanos P.E.
California Licensed Engineer, No. C65621
Project Engineer
Michael Love & Associates, Inc.
llanos@h2odesigns.com

Steve Allen, P.E.
California Licensed Engineer, C62021
Principal Engineer
GHD, Inc.
steve.allen@ghd.com| (707) 443-8326

August 2015

Contents

1.0 Introduction, Project Goals, and Scope of Report

The Martin Slough and Elk River estuary are part of the larger Humboldt Bay ecosystem that accommodates a variety of waterfowl, wading birds and shorebirds, several species of fish and other aquatic organisms, passerines, and raptors. Not much is known relative to the historic composition of the lower portions of Martin Slough. However, it is apparent from its elevation relative to tidewater and its geomorphic features that the lower portions of Martin Slough consisted of estuarine habitat, likely composed of some salt marsh and slough channels along with other more brackish water habitats.

Although much of the historic estuary has been converted to other land use, some estuarine habitat still exists. That habitat had been degraded by the installation of once common top hinged cast iron tide gates placed on the outlet end of multiple corrugated metal culvert pipes at the confluence of Martin Slough with Swain Slough and other land management practices. These modifications also have had a pronounced effect on flood routing and sedimentation in the lower channel. Existing problems that have been identified in Martin Slough include obstructed fish access, poor fish habitat, poor sediment routing, lack of riparian habitat, and frequent prolonged flooding that has a negative economic impact on current land uses.

The pre-development vegetation of Martin Slough is presumed to have been a mixed Sitka Spruce (Picea sitchensis)/willow (Salix spp.) forest transitioning to tidal salt marsh. Extreme upper limits of the project area could possibly have been forested in portions by coast redwood (Sequoia sempervirens). Transition between forest and tidal salt marsh would likely have been comprised of brackish water and high groundwater tolerant willows, sedges (Carex spp.), bulrush (Scirpus ssp.) and rush (Juncus spp.). Salt marsh vegetation may well have dominated much of the study area prior to the dike construction. The tidal flats could well have been vegetated by pickleweed (Salicornia virginica) and salt grass (Distichlis spicata). In the non-forested transitional areas brackish vegetation may have been soft rush (Juncus effusus), silverweed (Potentilla anserina), small-headed bulrush (Scirpus microcarpus), and tufted hairgrass (Deschampsia cespitosa).

The purpose of the Martin Slough Enhancement Project is to improve aquatic and riparian habitat and reduce flooding throughout the project area. Specific goals of the Project include the following:

1. Improve fish access from Swain Slough,
2. Reduce flood impacts to current land use,
3. Improve sediment transport,
4. Increase the amount of riparian corridor and riparian canopy,
5. Improve water quality (increased circulation, decrease nutrient inputs, decrease sedimentation),
6. Increase the extent of the estuarine ecotone in Martin Slough, providing a gradual transition from brackish water to freshwater habitats, and
7. Enhance and create off-channel/backwater habitats.

1.1 Feasibility Study

In 2001, the Natural Resources Division of Redwood Community Action Agency (RCAA) funded Winzler & Kelly (W&K), now GHD Inc. (GHD), to develop an enhancement plan to improve fish access, enhance aquatic habitat, improve sediment transport, and reduce flooding impacts on land use activities within Martin Slough. Michael Love & Associates (MLA), Graham Matthews & Associates (GMA) and Coastal Analysis, LLC (CAL) also participated in the project. RCAA administered the project and is responsible for the Technical Advisory Committee (TAC) and landowner coordination. The TAC was comprised of agency representatives, land owners, and land managers plus the team of consultants and representatives of RCAA. The TAC had the following entities represented at one or more meetings:

City of Eureka	Lisa Shikany (Planning), Gary Boughton (Engineering), Mike Zoppo (Property Management)
Course Co (golf course lessees)	Don Roller, Ray Davies, Bruce Perisho
Land Owners	Gene Senestraro, Bob Barnum
State Coastal Conservancy	Michael Bowen
U.S. Army Corps of Engineers	David Ammerman (Permitting)
NOAA Fisheries	Keytra Meyers, Margaret Tauzer, Chuck Glasgow
CA Department of Fish & Game	Michelle Gilroy
County of Humboldt	Rob Burnett and Chris Whitworth (Public Works), Alyson Hunter and Tom Hofweber (Community Development)
California Coastal Commission	Jim Baskin
RCAA	Don Allan, Michele Copas
Michael Love & Associates	Michael Love
Winzler & Kelly (GHD)	Steven Allen

W&K , MLA and CAL prepared a planning level report for the project, entitled Martin Slough Enhancement Feasibility Study, Eureka California (W&K et al., 2006). The Enhancement Study characterized current conditions and limiting factors within Martin Slough and developed four alternative enhancement approaches that enhance aquatic and riparian habitat. The TAC selected Alternative 4 as the preferred project.

1.2 Preliminary (30%) Design Development

The design development team of GHD and MLA, under direction from RCAA, proceeded to develop preliminary (30%) designs for the preferred alternative (Alternative #4) from the Feasibility Study. Input was received from the TAC and other stakeholders, including the new owners of Gene Senestraro's property, the North Coast Regional Land Trust (NCRLT). With feedback, additional modeling, and further design, the project elements remained essentially the same as they were presented in the Feasiblity Study. A Basis of Design Report was prepared describing the proposed project in detail (GHD & MLA, 2013)

1.3 Design and Construction of New Tide Gates

In 2014, the design for the new tide gates described in the 2013 Basis of Design Report was completed and the tide gates were constructed. These tide gates are adequately sized to function as needed once the upstream Martin Slough Enhancement Project is implemented.

1.4 Current Design Efforts

The project design team continued design development and refinement of the project. Utility constraints within the project footprint were better defined following preparation of the preliminary design plans. These included difficulties associated with relocating (deepening) a 12 inch gas main that crosses the project in three locations, and location of sanitary sewer lines installed as part of the City of Eureka's Martin Slough Interceptor Project. RCAA, GHD and MLA worked through specific issues as part of the refining the project design, including

- coordinating with PG&E to determine how to address the gas lines on the project site,
- coordinating with the City of Eureka regarding their infrastructure improvements with the Martin Slough Interceptor project,
- developing conceptual options for maintaining a fresh water supply for on-going irrigation needs on the golf course,

- coordinating with the NRLT regarding changes in the alignment of the main channel and location of new tidal wetland habitat (Pond C) to avoid conflicts with the PG&E 12 inch gas main

- coordinating with NRLT regarding location and extents of riparian planting

- discussions with CDFW regarding proposed project changes and creating freshwater pond habitat for coho salmon rearing.

The result of these design efforts is the current refined project, as described in this basis of design report and in the 65% design plans (dated July 2015).

1.5 Project Location and Land Use

The Martin Slough Enhancement Project is located in and adjacent to the southeast portion of the City of Eureka and terminates with its confluence with Swain Slough as shown in Table 1-1. Martin Slough is the lowest tributary to Elk River via Swain Slough. The mouth of Martin Slough is separated from Swain's Slough by a berm and tide gates. The Martin Slough watershed includes both City and County jurisdictions, with the project area owned by the City of Eureka (approximately 120 acres) and a private landowner (approximately 40 acres). The project area is partially within the coastal zone.

The Martin Slough watershed land use includes a mix of residential, agricultural, timberlands, and municipal infrastructure. Humboldt County's Eureka Community Plan includes future residential development of the southeastern portion of the Martin Slough watershed. This currently forested area has been phased out of timber production zone (TPZ) status to allow for residential or mixed-use development. This conversion could modify the watershed hydrology and potentially result in increased storm water runoff. Its actual effect on peak flows within Martin Slough will be dependent on the measures taken by future development to address storm water runoff, currently set for no net increase by the County.

The project area is currently zoned Public Facility and Agriculture Exclusive. Municipal infrastructure directly within the project area includes the City maintained Fairway Drive, PG&E 12 inch, 6 inch, and 4 inch natural gas lines, a sewage interceptor line with pump station, and the Eureka Municipal Golf Course. The Humboldt Community Services District also has existing sewer infrastructure near Fairway Drive.

Martin Slough has a watershed area of approximately 5.4 square miles, and natural channel length of over 10 miles with approximately 7.5 miles of potential salmonid fish habitat supporting coho salmon, steelhead trout, and cutthroat trout. However, the existing tide gates partially block upstream salmonid migration. The lower portion of the watershed flows through low gradient bottomland containing the golf course and pastureland. Many of the stream channels flow from gulches that contain mature second-growth redwood forests. The upper portions of the watershed are either in urban settings, or are recently harvested timber lands slated for future residential areas.

The Martin Slough project area consists of the general flood plain between Swain Slough and the upper (second) Fairway Drive stream crossing in the lower Martin Slough watershed (Table 1-1). Existing problems that have been identified in the Martin Slough study area include limited fish access, poor fish habitat, large sediment loads, poor sediment routing, lack of riparian habitat, and frequent prolonged flooding that has a negative economic impact on current land use.

The project area was not well mapped prior to the original installation of tide gates but similar areas around Humboldt Bay that were accurately mapped indicate that these transition areas between the freshwater portion of the stream and the tidal marshes consisted of a complex of channel networks with diverse habitat types and vegetation that supported a wide variety of native fish and wildlife. With the conversion to agricultural uses, the channel network was filled in to make crop land and later grazing land. Riparian vegetation was removed and the channel was straightened. The diversity of habitats, including backwater nursery areas for salmonids and riparian forest supporting a wide variety of avian species, was eliminated.

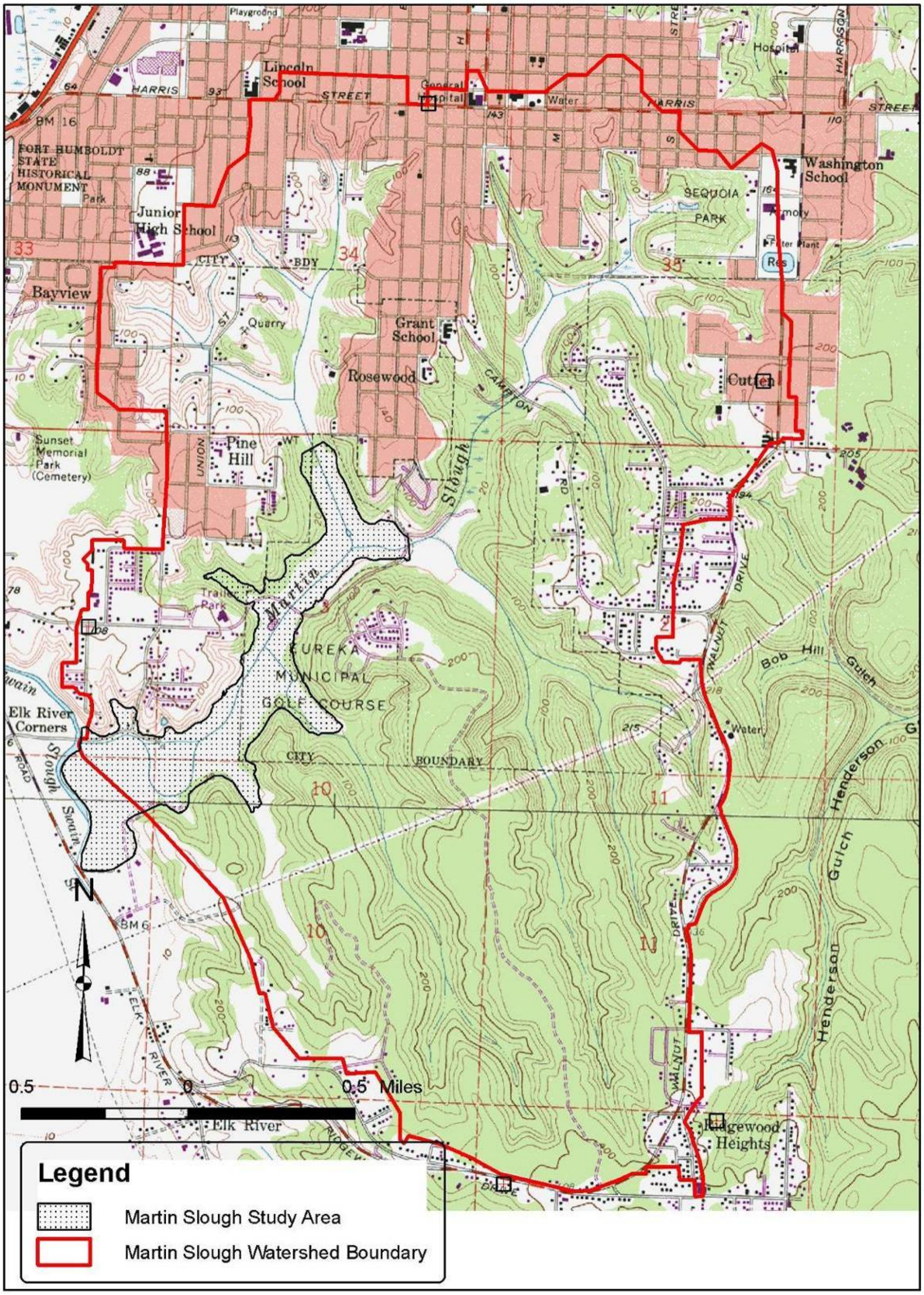


Table 1-1. Martin Slough Project Area and Watershed Boundary.

2.0 Project Description

Preliminary designs for four alternatives for the Martin Slough Enhancement Project were developed and presented in the *Martin Slough Enhancement Feasibility Study* prepared for Redwood Community Action Agency (W-K et al., 2006). Alternative 4 was selected by the TAC as the preferred alternative. Alternative 4 was then developed to the 30% design level and presented in a Basis of Design Report (2013). The project has been further developed to the 65% design level, and includes reintroduction of a limited tidal influence into lower Martin Slough through new tide gates, enlargement of the channel to accommodate the daily tidal flux, and constructing numerous off-channel ponds and wetlands. Together these components would create a self-sustaining muted tidal system while providing increased aquatic habitat and improved routing of floodwaters and sediment through the project site.

The proposed project includes multiple components that are all interrelated (Table 2-1). These components include:

- Replacement of the existing tide gates (completed in 2014)
- Construction of seven tidal wetlands
- Construction of one freshwater pond
- Enlarging the existing slough channel
- Installation of large wood structures for fisheries habitat and grade control
- Repair and raise the existing berm between Martin Slough and Swain Slough
- Generally improve drainage in the areas of play within the golf course
- Replacement of one existing agricultural culverts with a bridge to access the agricultural pastures and barn
- Replacement of two existing agricultural culverts with two new culverts to provide livestock access to the agricultural pastures
- Rerouting a freshwater tributary out of an existing ditch and installing one culvert crossing for utility maintenance access
- Replacement of eight golf course bridges that span the channel on the golf course
- Planting of wetland and riparian vegetation

Hydraulic, hydrologic, and geomorphic analyses were used to develop the interrelated project components through an iterative design process. The following sections describe the project components, with subsequent chapters describing the methods and results used in developing the design.

All elevations presented in this report are in NAVD88.

Description

New tide gates to increase conveyance and restore limited tidal influence, construction and enlargement of tidal and freshwater wetlands to increase floodwater storage and provide enhanced fisheries and waterfowl habitat, and enlarged channel to increase floodwater and tide water conveyance through project area.

A & B (0.75 acres & 2.3 acres) - salt marsh plain 50 ft wide paralleling slough channel and 70 ft wide along abandoned meander.

C (1.7 acres) - Salt marsh with low elevation pond connected to springs.

D & E (0.8 acres & 1.3 acres) - Expanded brackish wetlands, containing deep open water, littoral benches and elevated outlet sill that minimizes salinity intrusion during wet season.

F (1.7 acres) - Backwater slough with island and deep open water and littoral bench on inside of bend.

G (0.5 acres) - Predominantly freshwater alcove pond. Deep open water with emergent vegetation along banks.

North Fork (0.8 acres) - Restored channel with marsh plan and side channel.

South East Tributary (0.3 acres) - Restored channel with small freshwater pond connected to existing tributary.

New channel dimensions - Trapezoidal shape with 1.5:1 (H:V) side slopes and bottom elevation ranges from -1.0 to 2.8 ft. Stable tidal channel geometry based on published relationships of diurnal tidal prism and slough channel dimensions.

Reach	Top Width (ft)	Length (ft)
1	39	1,050
2	41	600
3	34	1,400
4	33	650
5	31	750
6	29	650
7	24	1,140

MARTIN SLOUGH ENHANCEMENT PROJECT

Figure 2-1. Summary of Martin Slough Enhancement Project Components. Reach Number and Pond Name are Circled.

2.1 Tide Gate Replacement

2.1.1 Design Objectives

A new tide gate structure was constructed in 2014 to replace the existing undersized tide gate structure where Martin Slough drains into Swain Slough. The replacement tide gate was designed to meet multiple objectives including:

- Allow a muted tidal prism to enter Martin Slough to provide adequate tidal exchange for sediment and nutrient flushing and enlargement of estuarine habitat.
- Maintain the tidal water below an elevation of 6 feet to protect adjacent pasture grasses and turf from salt-burn.
- Mimic the natural variability of the tidal cycle within the muted tide range to support a variety of salt marsh and open water habitats.
- Reduce the duration that floodwaters inundate overbank areas within the golf course and cattle pasture.
- Maximize the amount of time the tide gates are open to provide for upstream and downstream movement of aquatic organisms.
- Maximize the amount of time water velocities through the gate openings meet passage criteria for adult and juvenile salmon and steelhead.

2.1.2 Tide Gate Description

The new tide gate structure increased the outflow capacity by nearly three times than the previous tide gate and is capable of introducing a muted tide into Martin Slough. The replacement structure consists of three 6-foot by 6-foot gates installed into a new triple bay concrete box culvert (Table 2-2). The three main tide gates open when water levels in Martin Slough are higher than in Swain Slough. The two outer gates are side-hinged and the middle gate is top-hinged. Side-hinged gates provide a larger opening, which produces less hydraulic resistance and provides for better fish passage conditions than top-hinged gates. The proposed configuration, with the middle gate being top-hinged, helps center the outflow velocities to reduce risk of scouring the adjacent berm and Pine Hill Road Bridge supports.

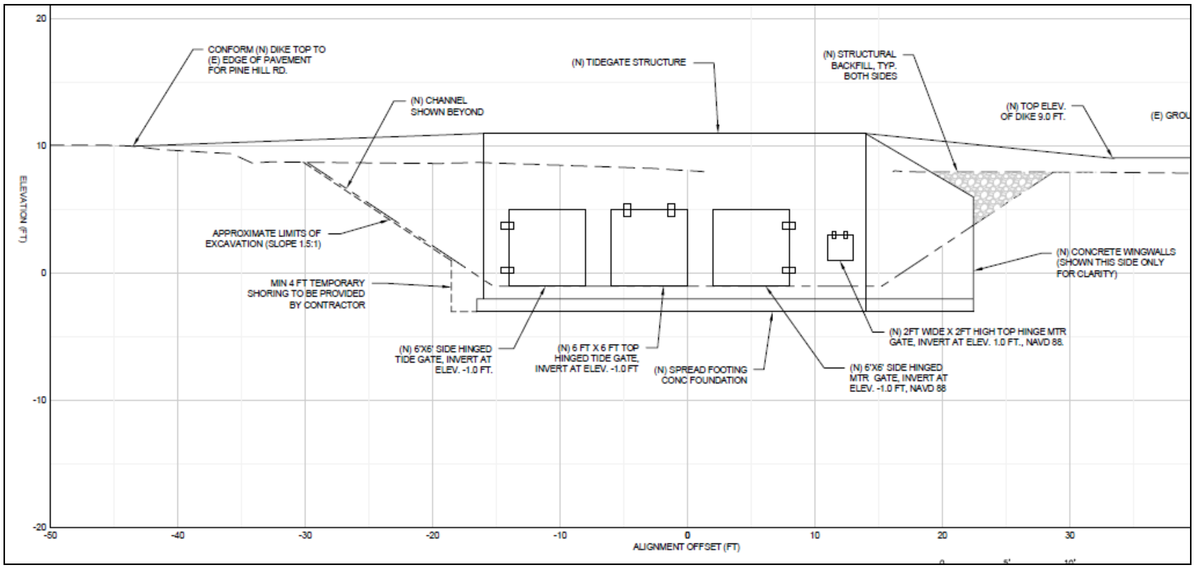


Table 2-2. Cross section on view from Swain Slough of proposed replacement tide gate structure.

The invert elevations of the three main tide gate doors are at an elevation of -1.0 feet, which is slightly deeper than the current elevation of Swain Slough. This elevation is below the crest of a tidal sill at the mouth of Elk River, which prevents the tide in Swain Slough from dropping below elevation 1.5 feet. The southern side-hinged gate is equipped with an adjustable muted tide regulator (MTR), which will be made operational once the overall project is constructed and the Martin Slough channel is enlarged to convey the increased tidal prism.

An auxiliary gate with an adjustable MTR is included as a separate opening in the structure. The auxiliary MTR gate door is top-hinged with a 2-foot high by 2-foot wide gate set at an invert elevation of 1.0 feet. Note that auxiliary gate has an approximate effective opening of 1.5-foot high by 2-foot wide because it is top hinged.

2.1.3 Muted Tide Regulator (MTR) Gates

Within the new tide gate structure, the southern 6-foot by 6-foot side hinged tide gate and the 2-foot high by 2-foot wide auxiliary gate are equipped with MTR systems. The two MTR-controlled gates allow for a limited amount of tidal water to flow into the project area, creating a muted tide within Martin Slough. An MTR system is designed to hold open a tide gate when water levels on the outside of the gate are higher than on the inside, when it typically would be closed. This allows for tidal water to flow through the gate and upstream into Martin Slough. The MTR system for each gate contains an adjustable mechanical lever attached to a float that closes the gate when Martin Slough water levels reach the designated elevation, preventing tidal flooding inside of the levees. The combination of the MTR gates controlling tidal inflow and the main tide gate doors allowing outflow on an ebb tide provide muted tidal conditions inside of Martin Slough.

The project design includes use of the MTR gates to produce the desired muted tidal conditions while preventing tidal flooding on adjacent lands. When Swain Slough water levels are higher than Martin Slough, tidal inflow will begin filling Martin Slough through the two MTR gates. The MTR equipped 6-foot by 6-foot side-hinged gate will then close when Martin Slough water levels reach an elevation of approximately 4.0 feet. The auxiliary door will continue to remain open until water levels in Martin Slough reach an elevation of 5.7 feet, and then will close. These elevations and gate sizes were selected to create the desired muted tide and protect upstream land uses and infrastructure.

2.1.4 Interim Operations of the MTR Gates

Under current conditions, the MTR on the 6-foot by 6-foot gate is disconnected, and will not be operated until the larger Martin Slough Enhancement Project is completed. The MTR on the auxiliary door is currently operational, and is set to sustain a small muted tide similar to conditions created by the recently replaced leaky tide gates. The small muted tide is intended to sustain the salt marsh vegetation that became established in the lower reaches of Martin Slough and to allow passage of aquatic species.

2.1.5 Muted Tide and Design Tidal Prism in Martin Slough

Tidal prism is defined in this project as the total tidal volume exchanged between mean higher high water (MHHW) and mean lower low water (MLLW) on an ebb tide. A muted tidal prism is a tidal prism that has a smaller amplitude than a tide in an unconstrained system. The muted tidal prism in Martin Slough will be controlled by tidal conditions in Swain Slough, tide gate opening geometry, water surface elevation at which the MTR gate closes, available tidal prism storage within Martin Slough, and routing of tidal waters.

The replacement tide gate structure will allow a muted tide to enter Martin Slough that has a mean lower low water (MLLW) equal to that of Swain Slough; approximately elevation 1.5 feet. This elevation is controlled by a persistent tidal sill at the mouth of Elk River. A maximum allowable muted tide elevation of 6 feet within Martin Slough was established to avoid brackish waters in the channel affecting the root-zone of the golf course turf, which will have a minimum elevation of 7 feet after several low areas within the golf course are raised. The muted tide created by the project is designed to have a MHHW of 5.5 feet, which is approximately 1.2 feet lower than MHHW in Humboldt Bay and Swain Slough.

A design tidal prism of approximately 20 acre-feet was identified to be feasible for the project area. This volume was selected to achieve several project objectives. The design tidal prism is similar to the historical tidal prism determined from aerial photograph measurements of channel widths of the abandoned meander bend on the Northcoast Regional Land Trust (NRLT) property. This prism is sufficient to maintain a slough channel that has capacity to route floodwaters efficiently during ebb tides, reducing the duration of overbank flooding. Also, a tidal prism of this size will result in a stable channel that fits under the existing Lower Fairway Drive bridge crossing with sufficient space for the golf cart path that crosses in that location.

2.1.6 Fish Passage through the Replacement Tide Gates

Fish passage through the new tide gates was analyzed previously as part of the 30% design submittal (GHD and MLA, 2013). The analysis assumes the gates are operated as previously described, and do not represent interim operations currently being implemented. The analysis was conducted for adult anadromous salmon and steelhead and for juvenile salmonids using velocity and depth criteria established by CDFG (2002) and NOAA Fisheries (2001), and provided in Table 2-3. Low and high passage design flows were also determined using these agency criterion. The analysis used the project HEC-RAS model to evaluate hydraulics through the gates. The model was executed for each passage design flow. This involved running one year of Swain Slough tides with a constant freshwater inflow set equal to the specified passage design flow. The model results allowed an assessment of passage conditions at each design flow across the full range of tidal conditions.

Upstream passage was defined as fish entering Martin Slough from Swain Slough and downstream passage was defined as fish leaving Martin Slough. Passage was computed for both in-flowing and out-flowing conditions as the percent of time during a 365-day period that one or more of the tide gates is open and provides suitable water depth and velocity for fish passage.

Table 2-3. CDFG and NMFS Fisheries fish passage depth and velocity criteria and range of fish passage flows applied Martin Slough replacement tide gates (from GHD and MLA, 2013).

Lifestage	Range of Passage Flows		Minimum Water Depth	Maximum Water Velocity
	Low	High		
Adult Salmon and Steelhead Trout	3.6 cfs	89 cfs	1.0 ft	6 fps
Juvenile Salmon and Steelhead Trout	1 cfs	27 cfs	0.5 ft	2 fps [1]

[1] Because of the short length of the tide gate structure, a water velocity corresponding to juvenile salmonid burst swim speeds was used for analyzing juvenile passage instead of the 1 fps recommended by CDFG and NOAA Fisheries.

Table 2-4 presents the results of upstream and downstream fish passage analysis for the replacement tide gate. Minimum water depths will always be adequate through the lower two 6-foot by 6-foot tide gates because their inverts are set at an elevation of -1.0 feet, 2.5 feet below the elevation of the Elk River tidal sill, which prevents Swain Slough water levels from dropping below 1.5 feet. On an incoming tide, the 6-foot by 6-foot MTR Gate will remain open for a portion of time, providing a minimum water depth of 1.0 feet. When the MTR gate closes and the auxiliary door remains open, a minimum flow depth of 0.5 feet will occur at lower tides.

Upstream and downstream passage of adult salmon and steelhead is provided through the tide gate structure over 90 percent of the time for the High Passage Design Flow. Passage is limited due to closure of the gates. During Low Passage Design Flows, upstream movement of adult salmon and steelhead is slightly limited by gate closures and an additional small percentage of the time due to water depth limitations through the auxiliary door. Downstream movement is limited by gate closures, and up to 17 percent of the time by excessive velocities and/or insufficient depths through the auxiliary door.

Upstream passage of juvenile salmonids is provided through the tide gate structure over 90 percent of the time for the range of fish passage flows. Downstream movement of juveniles is limited by gate closures and excessive velocities through the auxiliary door and, to a much lesser extent, through the MTR gate.

Table 2-4. Computed upstream and downstream fish passage at the Martin Slough replacement tide gates for adult and juvenile salmonids (from GHD and MLA, 2013).

Fish Species & Lifestage	Stream Flow	Percent of Time Gates Open	Percent of Time Passable	
			Upstream Movement	Downstream Movement
Juvenile Salmon & Steelhead:				
Low Passage Design Flow	1 cfs	98.3%	98.1%	54.7%
High Passage Design Flow	27 cfs	95.5%	94.3%	64.7%
Adult Salmon & Steelhead:				
Low Passage Design Flow	3.6 cfs	95.5%	92.8%	78.9%
High Passage Design Flow	89 cfs	91.7%	91.7%	91.7%

2.2 Martin Slough and Tributary Channels

The proposed muted tide for Martin Slough will introduce tidal influence within the channel throughout the project area, restoring it to a tidal slough channel. Approximately 6,300 feet of the existing Martin Slough channel, 1,000 feet of the North Fork Tributary, and 700 feet of the Southeast Tributary will be enlarged within the project area to increase conveyance for both flood flows and tidal exchange. Additionally, several other small tributaries were enlarged as part of the pond improvements associated with the project.

The channel improvements for the mainstem of Martin Slough and the North Fork Tributary within the project area were designed as tidal channels. The daily exchange of tidal water largely governs the size and shape of these channels. Tidal channels are typically U-shaped and increase in width and depth with increasing tidal prism (volume of water conveyed by the channel during a tidal cycle). Therefore, empirically derived tidal sizing equations were used to size these tidal channels (Williams et al., 2002; Coats et al. 1995, and PWA and Faber, 2004).

2.2.1 Mainstem Channel

The project encompasses the mainstem of Martin Slough from Swain Slough, through the NRLT pasture and Eureka Municipal Golf Course, to approximately 500 feet downstream of the Fairway Drive culvert crossing. The channel within the project area was divided into seven reaches, numbered from downstream to upstream (Table 2-1). Reaches were generally segmented at the confluences of the proposed tidal ponds or tributary channels because they contribute significantly to the tidal prism of the downstream channel reach.

Channel Profile

The mainstem of Martin Slough is designed with a thalweg elevation of -1.0 feet (NAVD88) at the tide gates. This is equal to the invert elevations of the new replacement tide gates and slightly deeper than the receiving Swain Slough. Swain Slough is expected to deepen slightly because of the increased tidal prism from Martin Slough.

Upstream of the tide gate, the channel starts at elevation -1.0 feet and slopes up at 0.02% to 0.06% for 5,080 feet, ending at elevation 0.8 feet just upstream of the confluence with the North Fork Tributary. Upstream of the North Fork Tributary, the mainstem channel will have a series of five log weirs spaced 60 feet apart and designed with 6-inch drops to allow for upstream passage of juvenile salmonids at all tidal stages. The downstream and upstream-most weirs are set at elevation 2.0 and 4.0 feet, respectively, creating an overall slope of 0.8%. The upstream weir is set at approximately the elevation of the existing channel thalweg. The log weirs will both stabilize the tidal to fluvial channel transition reach and prevent brackish water from moving upstream of the project area.

Channel Shape

The Martin Slough tidal channel will be constructed with a trapezoidal shape having side-slopes of 1.5H:1V, which was the recommended maximum side slope in the project geotechnical report (Appendix A) . The shape of the channel is expected to evolve into a more parabolic shape, or “U-shape”, typical of tidal channels. The resulting stable channel geometries will have top widths ranging from 39 feet wide in Reach 1, along the lower portions of the NRLT Property near the tide gates, to 24 feet wide in Reach 7, which matches the width of the existing channel at this location. The constructed channel depths, as measured from the top of bank to bottom of channel, will range between 9 feet and 7.4 feet.

Channel Alignment

The project maintains the current alignment for most of the mainstem slough channel’s length, with some small shifts to add a limited amount of sinuosity and to keep the new channel away from underground utilities (gas and sewer) and adjoining properties. Changes of note in the alignment are listed below:

- Close to the tide gate the channel flows parallel to Pine Hill Road. In this reach, the channel will be moved 10 to 15 feet away from the roadway to reduce the risk of scour and erosion along the roadway embankment.
- The channel will be realigned upstream and downstream of the existing NRLT barn to accommodate a new bridge crossing.
- Three short sections of channel will be meandered to the north on the golf course to create some sinuosity and hydraulic diversity.

2.2.2 North Fork Tributary Channel

The North Fork Tributary flows through the northeastern finger of the golf course before draining into the mainstem of Martin Slough. The lower 1,000 feet of this channel will be enhanced as part of the project. The North Fork channel was designed as a tidally influenced channel that is predominately freshwater year-round.

The reach from mainstem Martin Slough to Pond G follows the current alignment to avoid conflicts with existing sanitary sewer lines. The existing irrigation pond along the North Fork Tributary will be eliminated and replaced with an off-channel alcove, called Pond G (Section 2.3.1). Elimination of the in-channel pond is intended to reduce sedimentation and the need for periodic dredging. Upstream of Pond G, the channel will be graded to increase conveyance, sinuosity, and complexity.

The channel will have a top width of 28 feet at the downstream end and narrows to 15 feet at the upstream end, where it transitions to the existing channel. The channel thalweg will have elevations ranging between 0.8 feet and 1.5 feet, and a slope of 0.06% in the downstream direction. Channel side slopes will be 1.5H:1V. A self-draining bench will be located along to the western bank of the tributary upstream of Pond G. This bench will slope at 2.6% from elevation 6 feet to 5 feet to promote emergent wetland vegetation. A 1 foot deep side channel with a bottom width of 3 feet and 2H:1V side slopes will be located parallel to the mainstem to drain the bench.

2.2.3 Historical Meander on NRLT Property

The meander on the NRLT property is the historical alignment of Martin Slough. The mainstem was relocated to its current alignment in the early part of the 20th century. Currently the meander is connected to the mainstem via two 30-inch diameter reinforced concrete pipe (RCP) culverts; one located at each end of the meander. This allows for local drainage of the meander and results in some muted tidal influence. Reoccupation of the meander would require relocating the two 12 inch gas line crossings to provide sufficient cover over the lines. Gas line relocation was found to be cost prohibitive for the project, and realignment of the mainstem channel into the meander was eliminated as a feasible alternative. Instead, as part of the project, the meander habitat will be enhanced and fish access to the meander will be improved, making it a valuable backwater channel while protecting the two gas line crossings.

Table 2-5. Summary of proposed tidal channel cross section dimensions and contributing tidal prism for each reach of Martin Slough.

Reach	Station (Length)	Channel Slope	Bottom Width	Typical Top of Bank Width	Typical Channel Depth	Contributing Tidal Prism (MHHW - MLLW)[1]
1	0+50 to 11+00 (1,050 feet)	0.018%	17 feet	39 feet	9.0 feet	2.8 AF Channel 0.5 AF Marsh Plain A and B 1.8 AF Historical Meander
2	11+00 to 17+00 (600 feet)	0.017%	15 feet	41 feet	8.8 feet	1.4 AF Channel 1.6 AF Pond C
3	17+00 to 31+00 (1,400 feet)	0.014%	12 feet	34 feet	8.6 feet	3.0 AF Channel
4	31+00 to 37+50 (650 Feet)	0.031%	11 feet	33 feet	7.9 feet	1.3 AF Channel 0.6 AF Pond D
5	37+50 to 45+00 (750 feet)	0.040%	11 feet	31 feet	7.7 feet	1.4 AF Channel 1.6 AF Pond E
6	45+00 to 51+30 (650 feet)	0.062%	5 feet	29 feet	7.8 feet	0.8 AF Channel 1.6 AF Pond F
7	51+50 to 62+90 (1,140 Feet)	0.045% Slough 0.8% Log Weirs	3 feet	24 feet	7.4 feet	0.6 AF Channel 1.0 AF Pond G and NF Trib.
					Total Tidal Prism for Design Conditions	**20.0 AF**

[1] Measured at downstream end of reach

The two existing RCPs connecting the meander to the mainstem will be replaced with 30-foot long 4-foot diameter RCPs. The purpose of these crossings will be to maintain livestock and to preserve the existing invert elevations of the meander bend to protect the existing gas line crossings. The downstream and upstream RCPs will be placed at an invert elevation of 2.5 feet and 3.0 feet, respectively, approximately equal to the invert elevations of the existing RCPs. Two tidal benches will be graded adjacent to the north bank of the meander to form a tidal marsh plain.

The larger RCPs will improve fish access. Additionally, the increased tidal prism from the project and routing of the Southeast Tributary into the meander are anticipated to cause a small increase in flow circulation within the meander, thus improving water quality and creating some brackish to freshwater habitat during the wet season. The 4-foot diameter RCP size was selected to maintain 2 feet of cover over the pipes to accommodate the cattle crossing.

To help ensure the gas line crossings are protected from any channel scour, they will be armored with two layers of articulating concrete mat. The bottom layer will be draped in the direction of the flow and keyed into the channel bed upstream and downstream of the pipeline to create a scour cutoff. The top layer will be draped across the channel and up the banks to provide scour protection to the banks. The top elevations of these mats within the channel will be approximately equal to the existing channel bed at the crossings and will be below the elevation of the new culvert inverts.

2.2.4 Southeast Tributary Channel

The Southeast Tributary enters the project area at the southeast corner of the NRLT property and then flows north in a ditch and pipe along the fence line adjoining the golf course, crossing over the 12 inch gas line and draining into Martin Slough. Because of the location and shallow depth of the 12 inch gas line, restoration of the Southeast Tributary will involve realigning the channel on the NRLT property to avoid crossing the gas line.

The new alignment of the channel will follow the southern fence line on the property, with the tributary draining into the historical meander. The realigned channel will be approximately 700 feet long and will have small amplitude meanders within a thirty foot wide corridor planted with riparian vegetation. The lower two thirds of the channel will experience some tidal influence, while the upper third will be above tidal influence. At the southeast corner of the NRLT property, a small (0.03 acres) freshwater pond will be constructed and is intended to provide overwinter rearing habitat for coho salmon. The pond outlet elevation is 6.7 feet, well above tidal influence, and the thalweg elevation of the channel at the confluence with the historical meander is 3.0 feet, resulting in an average channel slope of 0.58%. The new Southeast Tributary channel was sized based on a 2-year 24 hour precipitation event which yielded a flow of approximately 9 cfs, a channel with a slope of 0.58%, 2-foot bottom width and 1.5H:1V side slopes. The bankfull channel is 1.3 feet deep with an inset floodplain bench with 4H:1V side slopes above bankfull that extends to existing ground.

A culvert crossing will be constructed across the Southeast Tributary, near its confluence with the historical meander. This crossing is to provide maintenance access for PG&E to an existing power pole located on the property line. The culvert crossing will consist of a 4-foot diameter RCP, approximately 20 feet long. The invert of the culvert would be set level at elevation 2.0 feet, thus embedding it 1 foot below the channel bed. A natural bed will form within the culvert.

2.3 Constructed Wetlands and Ponds

As part of this project, seven tidal wetlands and one non-tidal pond will be constructed. The tidal wetlands are denoted as Ponds lettered A through G (Table 2-1). They are intended to:

1. Provide tidal prism storage to sustain a tidal slough channel throughout the project area,
2. Create a diversity of aquatic habitats suitable for marine, estuarine, and freshwater species, and
3. Provide floodwater storage to reduce the frequency and duration of overbank flooding.

The freshwater pond, located on the Southeast Tributary, is intended to provide rearing habitat for juvenile coho.

Three of the seven constructed tidal wetlands (Ponds D, E, and G) involve enlargement of existing ponds on the golf course that currently experience some level of tidal influence through the interim operation of the Auxiliary MTR gate, which mimics the previous leaky tide gates. One new tidal wetland (Pond F) will be constructed on the golf course and three new salt marsh wetlands (Marsh Plains A and B, and Tidal Marsh Complex C) will be created on the NRLT Property.

The locations and configurations of proposed tidal wetlands on the golf course were developed with review and recommendations provided by golf course architect Gary Linn, representing CourseCo Inc. Ponds on the NRLT Property were located to minimize fragmentation of the pasture, avoid the need to relocate the existing 12-inch gas line owned by PG&E, include the freshwater tributary entering from the southeast corner of the property, and incorporate freshwater springs entering in the northeast corner of the property.

The proposed tidal wetlands will be spatially arranged to create a continuum of estuarine environments, as found in naturally functioning tidal estuaries, including both open water and shallow tidal benches. The wetlands, Pond C and Pond D, will capture freshwater from tributaries and springs. Salinity concentrations in all the tidal ponds are expected to fluctuate from summer to winter, being higher in the summer when less freshwater is entering the project area. The non-tidal freshwater pond on the Southeast tributary will be constructed with its outlet elevation above tidal influence, and therefore should not experience any inflow of brackish water.

Table 2-6 summarizes the locations, sizes, and depths of the proposed tidal wetlands and ponds.

Table 2-6. Summary of the tidal and non-tidal wetlands within the Martin Slough project area.

Pond	Area	Maximum Residual Depth
A	0.75 acres	NA
B	2.3 acres	NA
C	1.7 acres	NA
D	0.8 acres	2.25 feet
E	1.3 acres	3.0 feet
F	1.7 acres	3.0 feet
G	0.5 acres	3.8 feet
Southeast Tributary Pond (Non-Tidal)	0.03 acres	4.7 feet

2.3.1 Summary of Wetland and Pond Geometry

The proposed tidal wetlands were configured to create areas suitable for both open-water and aquatic vegetation. They will create both in-channel and off-channel habitats, and were arranged to generate circulation patterns that will maintain suitable water quality. In the downstream portions of the project area, where a predominately-marine environment will persist, Marsh Plains A and B and Tidal Marsh Complex C (Pond C) were configured to support salt marsh vegetation and low-order tidal channels.

Ponds C through F and the North Fork Tributary will have littoral benches that gently slope between elevations 4 feet and 6 feet. The benches will be located adjacent to deeper open waters and intended to support emergent wetland vegetation. At this elevation, the benches will be located within the intertidal zone. During the dry season when saltwater and freshwater stratify, much of these benches are expected to be within the freshwater lens within the upper water column. Inundation depths will generally be up to two feet during high tides, making the benches suitable for supporting wetland vegetation.

Generally, the pond bottoms were set near elevation zero feet, or at an elevation which maintains a residual depth of 2 feet or more. This depth is expected to prevent colonization by emergent wetland vegetation in the open-water portion of the ponds. The pond shorelines adjacent to open-water areas will generally have side-slopes of 3H:1V, which will limit growth of emergent wetland plants while providing a gradual enough slope to allow waterfowl and other wildlife to enter and exit the water.

Large wood structures will be placed on the pond benches and anchored into the soil. They will also be located in the deeper water to provide cover for fish, and provide perches for birds.

2.3.2 Marsh Plains A and B

Marsh Plains A and B will parallel the Martin Slough channel and are approximately 50 and 75 feet wide, respectively. Marsh Plain A is located just upstream of the new tide gate and runs along the south bank of the channel for 750 feet. Marsh Plain B is located along approximately 1,400 feet of the historic meander. The marsh plain surfaces will undulate to encourage zonation of marsh vegetation and formation of first and second order tidal channels by concentrating runoff during ebb tides. Marsh plains begin at elevation 4.8 feet and slope gently at 0.1% to 1.4% to elevation 5.8 feet, from the back of the marsh plain, the ground will slope at 3H:1V to meet existing ground. Marsh Plain B is segmented into two sections at the gas line crossing to allow maintenance access.

Large wood will be placed onto the marsh plains and anchored using soil anchors. The large wood is intended to increase topographic complexity, provide cover for fish, and provide perches for birds.

2.3.3 Tidal Marsh Complex C

Tidal Marsh Complex C (Pond C) was configured to create complex salt and brackish marsh habitats. It will be located in a historical marsh plain and will convey flow from an intermittent spring-fed freshwater tributary of Martin Slough that enters in a ditch from an adjacent property to the north. It will contain a main tidal slough channel and at least two tributary slough channels, an island, and an in-channel pond. The channels and pond will be surrounded by approximately 1.4 acres of salt marsh plain. Several first and second-order tidal channels are expected to form from the higher ground that projects into the marsh plain and from the seeps and springs located along the toe of the adjacent hillslope.

The spring-fed freshwater tributary that is currently ditched will be realigned and transition into the head of Pond C. A series of four log weirs located approximately 100-feet downstream of the property line will be placed in the realigned channel to help stabilize the channel profile. Each step will have a 6-inch drop. The crest elevation for the upstream most-step will be at 6.5 feet to match the upstream ditch flowline. The upper two steps are set above the highest muted tidal elevation of 5.8 feet, thus preventing saline water from backwatering up the existing drainage ditch and onto the adjacent property.

The main Pond C tidal channel was designed based on stable tidal channel geometry for the design tidal prism. The channel will have a top width of 11 feet and depths ranging from 1.8 to 3.4 feet at MHHW. The tidal channel will have a thalweg elevation of 0.9 feet where it joins Martin Slough. A flow-through side channel will be connected to the main tidal channel to form an upland island. Marsh plain will comprise the ground around the island and adjacent to the side channel. The side channel around the island will have a bottom width of 3 feet and near vertical sides.

At the upstream end of the tidal marsh complex will be a brackish tidal pond that is approximately 150 feet long, 6 feet wide, and maintains a residual depth of approximately 1 foot during low tide. The pond will provide a low-velocity refuge for tidewater goby and out-migrating smolts. Large wood structures placed within the slough channels will sustain scour pools and provide hydraulic controls during ebb and flood tides.

The tidal marsh plains in this tidal marsh complex were designed with a range of elevations that will support a diversity of tidal marsh species. The marsh plain begins at elevation 5.0 feet and gently slope back to elevation 6.0 feet while the width and side slope varies with shape of the wetland, but is generally between 1.5% and 2.0%. From the back of the marsh plain the ground slopes at 3H:1 V to match to existing ground.

2.3.4 Pond D

Pond D is an existing in-line pond on the East Tributary that will be enlarged, creating fresh to brackish water habitat. The outlet elevation of Pond D is constrained by a shallow 12-inch diameter gas line that passes under the tributary. A sill armored with two layers of articulated concrete matting with a top elevation of 4.0 feet will be constructed to protect the gas line crossing. Upstream of the gas line crossing is a bridge installed in 2014. This pedestrian bridge will be maintained and, if necessary, it may be moved during construction and then reinstalled after construction. The East Tributary will flow from Pond D, into the mainstem over a series of six log weirs that raise the bed elevation from 2.25 feet in the mainstem to 4.25 feet downstream of the gas line sill. Each log weir will have 6 inches of drop and will allow for upstream passage of adult and juvenile salmonids during all tidal conditions (low to high). The upstream-most weir is designed to create sufficient depth over the gas line crossing at juvenile and adult salmonid migration flows.

The bottom elevation of Pond D will be at 2.0 feet and the outlet sill will maintain a residual pond depth of 2.25 feet. The bottom width of the pond will be up to 17 feet and have side slopes ranging from of 4H:1V to 6H:1V. The perimeter of the pond is surrounded by an approximately 25-foot wide marsh plain that slopes gently from elevation 4 to 5 feet, and then at 3H:1V to existing ground. This will provide a bench that is expected to experience primarily freshwater to slightly brackish conditions and have suitable depth to support emergent aquatic vegetation.

Two complex wood structures are planned for Pond D to provide cover and foraging habitat for fish, and perches for birds.

2.3.5 Pond E

Pond E is an existing pond connected to the Martin Slough mainstem that will be enlarged. It is located just downstream of the existing Lower Fairway Drive bridge crossing. In addition to a low-flow entrance at the downstream end, Pond E will also have a high flow connection at the upstream end of the pond. An earthen sill will define the downstream entrance to the pond with a crest elevation of 3.0 feet and approach thalweg slope of 10H:1V. The pond bottom will be located at elevation 0.0 feet to produce a residual depth of 3 feet. Side slopes in the pond vary but do not exceed 2H:1V and are intended to support emergent aquatic vegetation. The existing ground along the right bank of the mainstem and in the middle of the pond will define the edge of the pond and provide two island features that preserve existing vegetation. Two “saddles” along the edge of the pond adjacent to the mainstem will allow high flow exchange with the pond when water surface elevations exceed 4.25 feet.

At least two complex wood structures are planned for Pond E to provide fish cover and foraging habitat, and to provide perches for birds.

2.3.6 Pond F

Pond F will be a new pond located just upstream of Lower Fairway Drive and connected to Martin Slough via an approximate 200-foot long channel. The pond is located in an existing low-lying area that often experiences ponding and inundation during and following frequently occurring overbank flows. An earthen sill with a crest elevation of 3.0 feet and channel thalweg slope of 10H:1V on its upstream and downstream end will be located near the downstream end of the outlet channel to control water levels in the pond. The pond bottom elevation will be between elevation 0.0 and 0.5 feet, creating a residual depth of between 2.5 and 3.0 feet. A 25-foot wide island feature will be located in the pond and partially bordered by a side channel bench at an elevation of 5 feet to maintain emergent freshwater vegetation. Side slopes of the pond vary from 3H:1V to 7H:1V.

At least three complex wood structures are planned for Pond F to provide cover and foraging habitat for fish, and to perches for birds.

2.3.7 Pond G

Pond G will be a new off-channel alcove on the North Fork Tributary located within the footprint of the existing irrigation pond. The existing pond will be converted to an off-channel alcove to improve sediment transport within the North Fork Tributary and reduce the rate of sedimentation within the pond. The outfall of Pond G will be located along the North Fork Tributary approximately 200 feet upstream of the confluence with the mainstem channel of Martin Slough. An earthen sill with a crest at elevation 3.8 feet will control water levels in the alcove. The upstream face of the sill will have a slope of 2H:1V, while the downstream face will have a more gentle slope of 10H:1V. The pond bottom will be at elevation 0.0 feet, creating a residual depth of 3.8 feet. Side slopes of 4H:1V from the bottom to existing ground form the pond edge along the east side of the pond. Along the west side, the side slopes upward at approximately 3H:1V to elevation 5.0 feet, where it forms a bench and high flow connection to the North Fork Tributary. The bench is set at an elevation that supports emergent freshwater wetland vegetation.

At least one to two complex wood structures are planned for Pond G to provide cover and foraging habitat for fish, and perches for birds.

2.3.8 Southeast Tributary Pond

Where the Southeast Tributary enters the NRLT property, a small freshwater pond will be constructed. This was requested by CDFW and is intended to create overwintering habitat for juvenile coho salmon. The pond will be fed by the tributary, which has a drainage area of 0.5 square miles. The small freshwater pond with a bottom elevation of 2.0 feet and 2H:1V side slopes. The outlet elevation will be 6.7 feet, preventing saline waters from entering the pond and maintaining a residual depth of 4.7 feet. Outflow from the pond will flow into the realigned Southeast Tributary, which will drain into the historical meander.

At least one complex wood structures is planned for the Southeast Tributary Pond to provide cover and foraging habitat for fish, and perches for birds.

2.4 Engineered Log Structures

Engineered Log Structures (ELS) will be utilized at various locations throughout the project to stabilize the channel profile (log weirs), create bed and bank scour (log constrictors and root wad deflectors) and to provide avian and aquatic habitat (log cover structures and root wad habitat structures). These structures are designed to remain stable at all evaluated flow conditions by (1) burying portions of the logs, (2) anchoring to log piles, and (3) using conventional soil anchors. Each of the five ELS serves a distinct purpose as described below. Details and drawings of each type of structure can be found in the 65% Submittal plan set prepared by GHD and MLA.

2.4.1 Log Weir

Log weirs will be placed to span the channel and are used to provide vertical stabilization of the channel profile at oversteepened transitions. These grade control structures provide channel stability to prevent headward erosion in the event of a change in channel elevation. The Log weirs proposed for Martin Slough will consist of two weir logs placed perpendicular to the flow and embedded into the banks, two pile logs, placed vertically at the toes of the channel to provide an anchor, and two training logs placed at either end of the log weir crest and placed at an angle to help direct flow to the center of the channel and to help prevent bank erosion around the ends of the weirs. The top weir log will have a notch cut into it to concentrate low flow into a plunging nappe at the channel center. This is done to increase depth over the weir at low flows for fish passage.

The log weirs have been designed with 6-inch drops to meet fish passage criteria for juvenile salmonids and spaced sufficiently to provide energy dissipation and pool development. Log weirs will be placed in three locations:

1. At the upstream end of Pond C to transition the spring-fed tributary into the pond,
2. From the outlet of Pond D to the confluence of the mainstem of Martin Slough to backwater the gas line crossing and transition to the profile elevation of the mainstem of Martin Slough at the confluence, and
3. At the transition of the mainstem Martin Slough from the project tidal slough channel to the existing mainstem channel.

2.4.2 Log Constrictor

Log constrictors are placed to concentrate or direct flow towards the center of the channel. Constrictors are often placed near tributary channel or pond outfalls to create sweeping velocities and reduce deposition at the confluence. Log constrictors consist of a vertical pile log placed on the downstream end of the structure, and two constrictor logs embedded horizontally into the bank and anchored to the pile log. The two horizontal logs are placed at an angle to each other forming two protrusions into the channel and anchored to the pile log. An additional soil anchor will be located toward the buried end of the upstream constrictor log. The constrictor structure extends a minimum of 3-feet into the channel and is buried into the bank. The pile log will extend 2 to 3 feet above the top constrictor log to facilitate catching woody material and provide an avian perch. The bottom log is skewed in the upstream direction to create limited scour along the toe of the bank and create a small pool with cover from the top log. The top log is skewed facing downstream to guide flow towards the center of the channel.

2.4.3 Root Wad Deflector

Root wad deflectors are utilized in a similar manner as the Log Constrictors and are placed in areas where flow can be directed to the channel center or opposite bank. A single root wad is buried into the bank with the root mass exposed and facing slightly upstream. A minimum of 2 soil anchors are used to stabilize the root wad. For root wads longer than 10 feet, soil anchors shall be located every 5-feet on alternating sides. The flow divergence created by the root mass will scour a pool at and immediately downstream of the Root Wad Structure.

2.4.4 Log Cover Structure

Log cover structures are utilized along pond margins to provide habitat and cover for aquatic organisms and avian perches. They consist of three vertical piles, and three cover logs interlaced to form a triangle and anchored to each pile. Woody vegetation and slash will also be incorporated into these wood features to create complex edge and cover features for fish to occupy.

2.4.5 Root Wad Habitat Structure

The Root wad habitat structures are identical to the Root Wad Deflector, but placed on the intertidal marsh plain. The root mass is placed facing the channel and provides cover and habitat for aquatic organisms during high tides and high flows. They also provide an avian perch. Small scour pools are likely to form around the root mass increasing habitat complexity.

2.5 Raising of Low Areas on Golf Course to Elevation 7.0

Currently, the golf course has numerous low areas on the floodplain that do not drain after storm events because the water ponds, increasing the potential for stranding of coho salmon and tidewater goby as floodwaters recede and leave ponds that become isolated from the creek. As part of design conditions, the low areas within the golf course that pond will be filled to a minimum elevation of 7 feet so they drain towards the channel, reducing the likelihood of fish stranding and improving drainage. Additionally, the new tide gate has a much larger outflow capacity, increasing rate that flood waters drain and reducing the amount of time the golf course greens will be inundated.

2.6 Replacement Bridge Crossing and Sheet Pile Walls

The NRLT property is managed for cattle grazing and uses the barn adjacent to the Martin Slough channel. Access to the barn is via a ranch road crossing of Martin Slough that consists of a 5-foot diameter RCP culvert. This undersized culvert will be replaced with a new 34-foot span bridge to provide access to the barn and adjacent pastures. The roadway width over the bridge will be approximately 45 feet to accommodate loading and unloading of cattle. Sheet pile retaining walls will be installed along the channel to accommodate the bridge given the limited available space between the barn on the south bank and the access road on the north bank. The sheet pile will extend upstream and downstream of the bridge crossing and will flair at both ends to create a gradual hydraulic transition. The width between the sheet piles will be 25 feet, while the bottom width of the enlarged slough channel will be 15 feet. The width between the sheet pile walls was designed to have negligible hydraulic influence on conveyance of both high stream flows and the ebb and flood of the muted tide.

2.7 Revegetation

The 65% Design Plans include the planting areas and species densities for the project area. The goal is to create native, forested riparian, wetland and tidal marsh habitats along the Martin Slough channel and expanded ponds. The excavated reaches of Martin Slough and expanded ponds would be revegetated with low growing brackish and freshwater wetland (sedges and rushes) and riparian forest (Sitka spruce, willow, wax myrtle, and alder). Plant material, to the extent feasible, would be salvaged from the project impact footprint. All enhancement areas disturbed during grading and other construction activities would be treated with erosion control seeding with native grasses, forbs and shrubs. Active planting is currently proposed, however natural recruitment of native plant species would be desirable to augment the active planting activities. Exclusion fencing will be constructed around the perimeter of the riparian forest to protect the plantings in the pasture. Fencing is not needed on the golf course (City) property as no cattle are allowed on the City property.

Areas of the golf course that are outside the riparian and wetland areas, i.e., fairways, affected by construction activities will be revegetated with grasses suitable for golf course fairways. Pasture areas affected by construction activities will be revegetated with pasture grasses. Revegetation of the berm between Martin and Swain Sloughs will occur after placement of soil to reinforce and enhance the berm. Before placing the soil, the existing sod layer will be removed and stored on site. After the berm is shaped and compacted, the sod layer will be placed back on the berm. As this area is actively grazed during summer, the existing vegetation is to be maintained similar to the existing conditions.

Active vegetation maintenance in the enhancement areas would be regularly performed to ensure that the target riparian forest habitat develops along the riparian corridor areas. Options for limiting undesirable vegetation include intermittent controlled flash grazing (cattle, goat or sheep), manual removal, and mechanical removal. Special attention would be given to non-native invasive species such as dense-flowered cordgrass, and maintenance activities will be coordinated with regional eradication programs, including both timing and methods for removal of specific species. If grazing is employed, exclusion fencing would be placed to protect channel banks, newly establishing revegetation plantings, and areas of naturally recruiting desirable native plants. Flash grazing may be carefully employed to control weed cover in active planting areas and natural recruitment areas but will be managed to avoid excessive damage to native plantings and recruits.

3.0 Background Resources

There was a substantial amount of supporting data used to develop and analyze the proposed project. The following sections summarize these data.

3.1 Project Base Map

A digital terrain model (DTM) produced from aerial photogrammetry and provided by the City of Eureka was used as the basis of the project topography. The aerial photogrammetry was flown in 2001 by Cartwright Aerial Surveys, Inc. of Sacramento and provided 2-foot contours of the project area. The topography only extended down to the edge of water at the time of the flight, which excludes much of the topography within the existing channel and golf course ponds. Horizontal control for the survey is North American Datum 1983 (NAD83) California State Plane, Zone 1, in feet and vertical control is North American Vertical Datum of 1988 (NAVD88) in feet.

To support the design (W-K et al., 2006), Spencer Engineering conducted additional survey of the channel and golf course ponds in 2005 to capture topography below the water. A total of 45 channel cross sections were surveyed within the channel that included the top of bank, toe of bank, and channel thalweg. Spot elevations were surveyed in the ponds located adjacent to Hole 4 and Hole 17. To facilitate development of the preliminary design, the channel topography below the water was added to the 2001 DTM.

In July 2013, RCAA conducted additional “fill-in” survey within the low areas on the golf course for use in defining the areas requiring fill to raise the ground to elevation 7.0 feet. Lastly, RCAA conducted an as-built survey of the constructed tide gate in the spring of 2015.

3.2 Geotechnical Investigation

Geotechnical recommendations for design development and construction were needed for this project. While this information was not readily available during the early development of the 30% design, the field work and recommendations have been recently completed by SHN Consulting Engineers & Geologists, Inc. and the results of the geotechnical investigation are presented in Appendix A. The report covers project elements such as new channels, ponds, the tide gate, new bridges, enhancement of the existing berm, and construction considerations such as temporary cut slopes and temporary access roads.

SHN was also asked to provide a written section on the geologic setting of the site for use in environmental compliance such as CEQA and other regulatory permits. That report is included in Appendix E.

3.3 Project Hydrology

Martin Slough has a watershed area of approximately 5.5 square miles and consists of a mix of residential, agricultural, timberlands, and municipal infrastructure in Eureka, California. Humboldt County’s Eureka Community Plan includes future residential development of the southeastern portion of the Martin Slough watershed.

As is characteristic throughout the region, the majority of precipitation falls between November and April, with drier weather persisting for the remaining months. Due to its low elevation and proximity to the Pacific Ocean, the Martin Slough watershed receives almost all of its precipitation in the form of rainfall. On average, the lower lying portions of the watershed receive approximately 40 inches of rainfall annually.

Earlier phases of the project included measuring streamflow and rainfall within the project area, characterizing rainfall-runoff patterns within the Martin Slough watershed, and developing fish passage design flows (W-K et al, 2006). Products from this work were directly used in developing and analyzing the proposed project.

3.3.1 Gaged Streamflows

Wet-Season Streamflows

Graham Matthews and Associates (GMA) established a streamflow gaging station on the mainstem of Martin Slough, immediately downstream of the culvert on the upper Fairway Drive crossing. Stage data was collection at 15-minute intervals using a continuous stage recorder. GMA also installed in the project area a recoding tipping-bucket rain-gage. Flows and precipitation were gaged in Martin Slough from February 12, 2003 through July 22, 2003. Flows (but not precipitation) were again gaged from November 7, 2003 through January 9, 2004 (Table 3-1). Field methods and findings are described in Appendix D of the 2006 Martin Slough Enhancement Feasibility Study (W-K et al, 2006).

To help place the relatively short monitoring record into a long-term hydrologic perspective, monthly total precipitation for February 2003 through January 2004 were evaluated using rainfall data from Woodley Island in Eureka (CDEC, 2011). The monthly totals were compared to average (normal) monthly precipitation for the period of record (1905 through 2011) (Table 3-2). February and March of 2003, and January 2004 had rainfall totals close to normal for those months. April 2003 was the wettest April on record and December 2003 was much wetter than average. Much of the rain during these months was low-intensity and spread-out over time. As a result, local streams and rivers did not experience large flows.

The flow record obtained from the gaging of Martin Slough contains several higher flow events. The associated return period of the largest event recorded likely did not exceed 2-years. This is based partly on a review of the nearby Little River near Trinidad flow records (USGS Station No. 18010102), which has a 55-year period of record and shows flow patterns similar to Martin Slough. From February 2003 to January 2004 the largest peak flow in Little River had a return period slightly greater than 1.5-years.

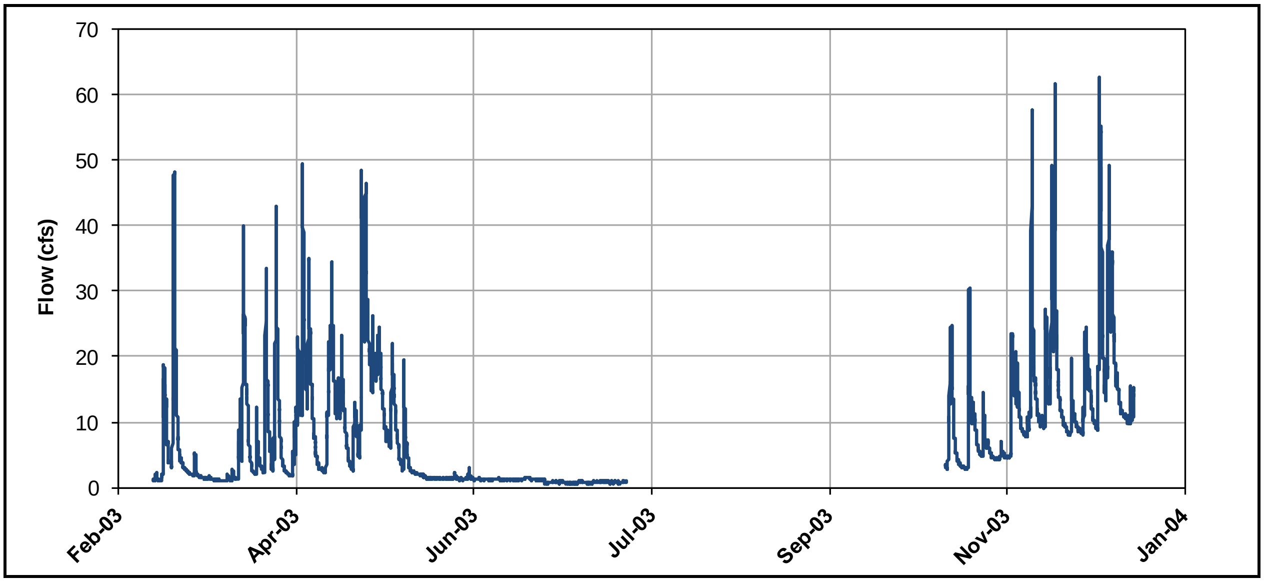


Table 3-1. Gaged flows in Martin Slough at the Upper Fairway Drive crossing (February 2003 through July 2003, and November 2003 through Januyary 2004). Low-flows were not gaged during the summer dry season.

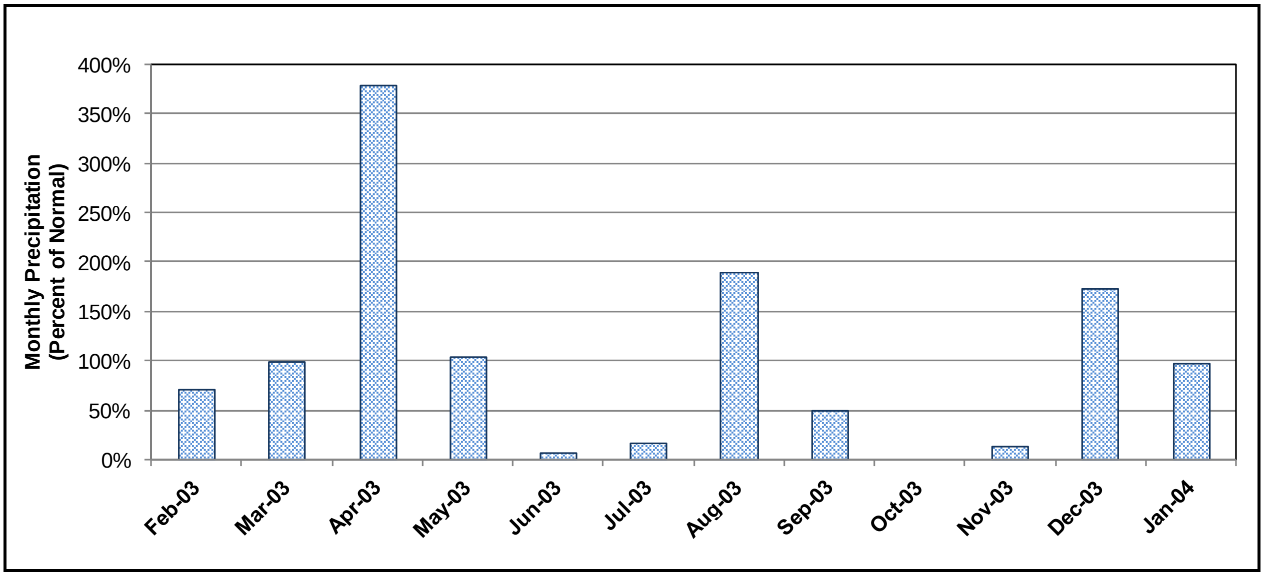


Table 3-2. Recorded monthly precipitation in Eureka (Woodley Island) during 2003 as a percent of normal for that month. Months plotted span the period-of-record for the Martin Slough flow gaging station at the Upper Fairway Drive crossing.

Summer Baseflow

Gaging of flows on the mainstem of Martin Slough ended on July 22, 2003. From this data, average baseflows were 1.2 cfs in June and 0.81 cfs in July of 2003.

Additional baseflow characterization was needed to aid in developing the preliminary project design and support changes to the golf course irrigation water supply system. MLA measured baseflow in Martin Slough between August 4 and October 15, 2008 (MLA, 2010). Baseflow was measured at two locations; the North Fork Tributary upstream of the irrigation pond and the Martin Slough Mainstem immediately upstream of the North Fork Tributary confluence. Flows were measured on a weekly basis. Table 3-3 summarizes the typical summer baseflow measured in the two locations. Flows from an August 21, 2008 rain event are not reported in the flow ranges.

Table 3-3. Summer baseflow measured in Martin Slough and the North Tributary from August 4 through October 15, 2008 (MLA, 2010).

Location	Drainage Area	2008 Summer Baseflow
Mainstem Upstream of North Fork Confluence	2.75 square miles	0.23 cfs to 0.42 cfs (Average 0.31 cfs)
North Fork Upstream of Mainstem Confluence	1.01 square miles	0.11 cfs to 0.25 cfs (Average 0.14 cfs)

Flow statistics from the Little River near Trinidad gaging station (USGS Station No. 11481200, 55-year period-of-record) were used to place the 2008 summer baseflow in Martin Slough in the context of inter-annual flow variability. Average monthly flow in the Little River for August, September and October of 2008 were within the lowest 20 percentile for those months (MLA, 2010). This suggests that the observed

2008 baseflow conditions within Martin Slough were also relatively low when compared to inter-annual flow variability.

3.3.2 Rainfall-Runoff Hydrographs

As part of the feasibility study, synthetic hydrographs for each tributary entering the project area were developed for 24-hour precipitation events with intensities of 2-year, 10-year, and 100-year return periods (W-K et al., 2006). Runoff was computed for Martin Slough in a calibrated HEC-HMS model applying Soil Conservation Service methods (NRCS, 2002). Simulations were conducted by applying land coverages (i.e. dense urban, sparse urban, timber) to the 44 defined sub-basins for existing conditions.

The results of the hydrologic modeling were used as inputs to the hydraulic modeling as part of the project design development. The applied locations of inflows to the project area are shown in Table 3-4. Contributing sub-basins to each inflow location are color-coded in Table 3-5. For inflow at Martin 3, sub-basins that have independent discharge locations into Martin Slough were grouped to simplify analysis and reporting.

The predicted peak flows at each inflow location for the 24-hour precipitation events with 2, 10 and 100-year return periods are provided in Table 3-6. For the 100-year precipitation event, inflow locations were combined to create three inflow hydrographs to the project area rather than six: Martin 1 and 2, Martin 3 and 4, and Martin 5 and 6.

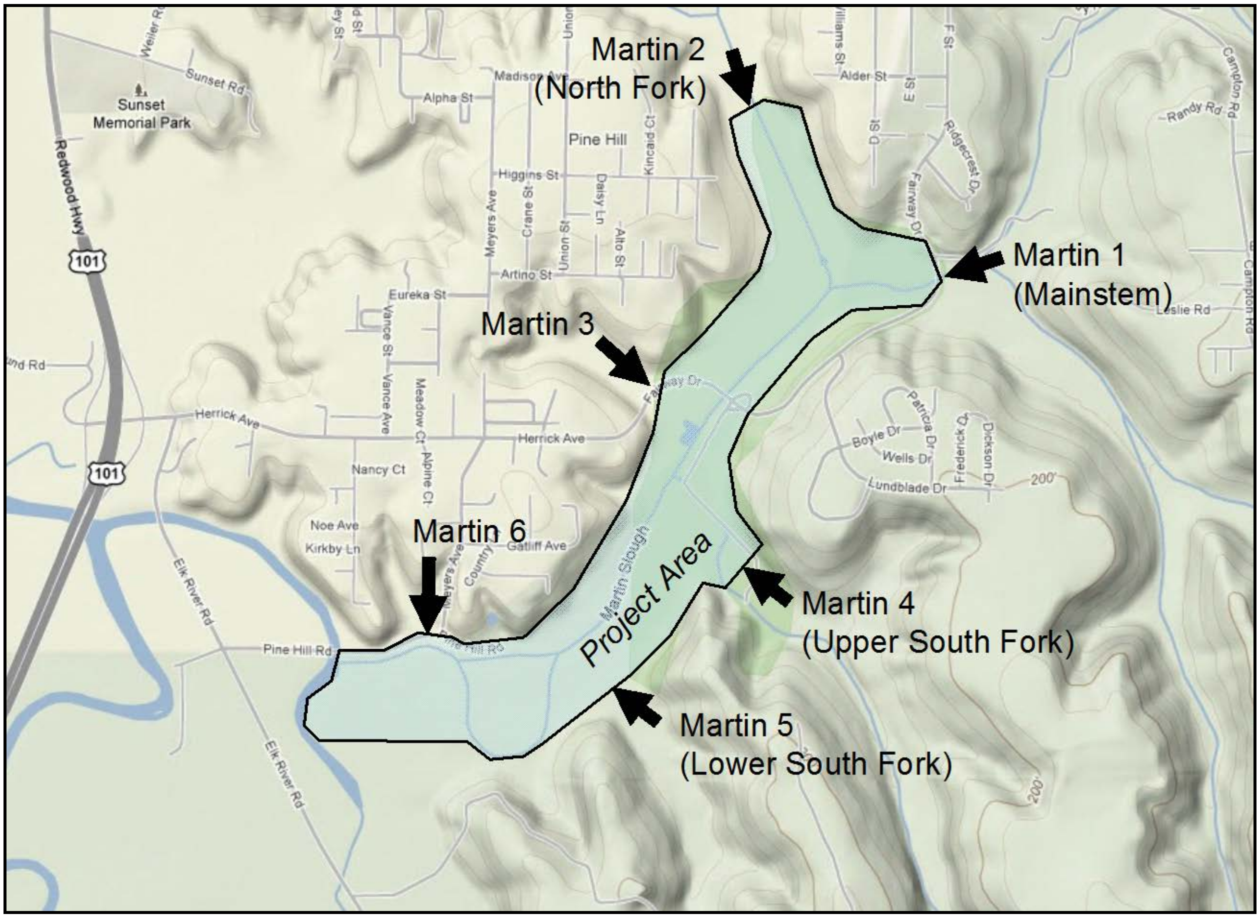


Table 3-4. General location of flow inputs to the project area. Flow inputs obtained from the HEC-HMS rainfall-runoff simulations prepared in W-K et al. (2006).

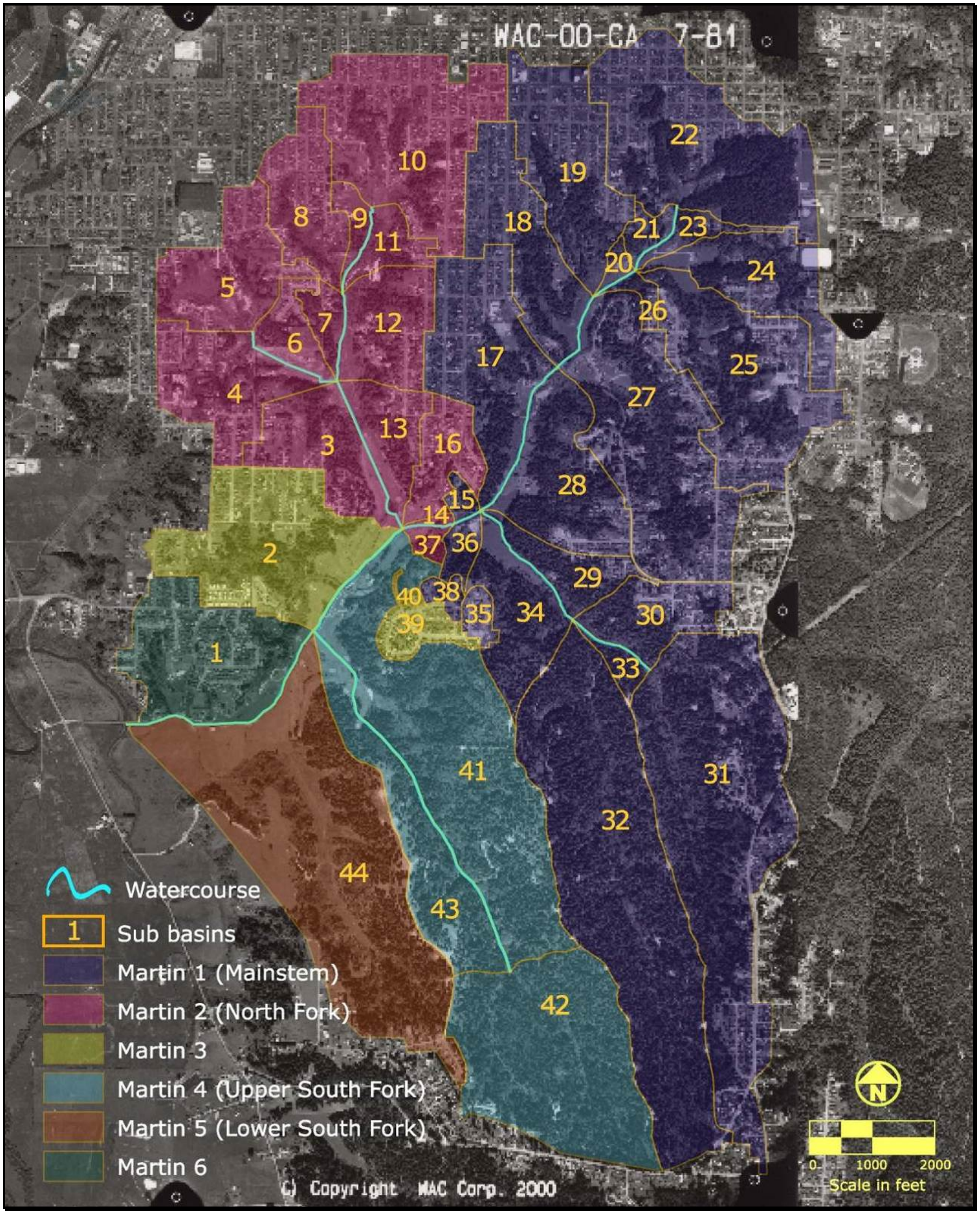


Table 3-5. Martin Slough HEC-HMS sub-basins grouped by color based on location of flow inputs to the project area. See W-K et al. (2006) for hydrologic model development.

Table 3-6. Summary of drainage areas and peak inflows to the project area obtained using HEC-HMS rainfall-runoff simulations for 24-hour precipitation events.

Inflow Location	Description	Drainage Area (mi^2)	Predicted Peak Flow		
			2-Yr 24-Hr Precipitation	10-Yr 24-Hr Precipitation	100-Yr 24-Hr Precipitation
Martin 1	Mainstem	2.76	94 cfs	207 cfs	499 cfs
Martin 2	North Fork	0.99	49 cfs	105 cfs	
Martin 3	Drainages near Lower Fairway Drive	0.26	10 cfs	21 cfs	72 cfs
Martin 4	Upper South Fork	0.82	14 cfs	33 cfs	
Martin 5	Lower South Fork	0.50	9 cfs	22 cfs	98 cfs
Martin 6	Drainages near Tide Gates	0.18	9 cfs	19 cfs	

3.3.3 Swain Slough Tides

As part of the hydrologic calibration presented in the feasibility study (W-K et al., 2006), tidal elevations were monitored in Martin Slough and Swain Slough at the confluence with Martin Slough between February 12, 2003 and February 20, 2003 (Table 3-7). At tides above 2.5 feet, the recorded Swain Slough water surface elevations closely matched corresponding tidal elevations recorded at the North Spit, Humboldt Bay (Station No. 9418767). At lower tides, water levels in Swain Slough receded slowly and did not drop below approximately 1.5 feet. This is attributed to a sill at the mouth of Elk River, most likely a persistent sandbar (Eicher, 1987).

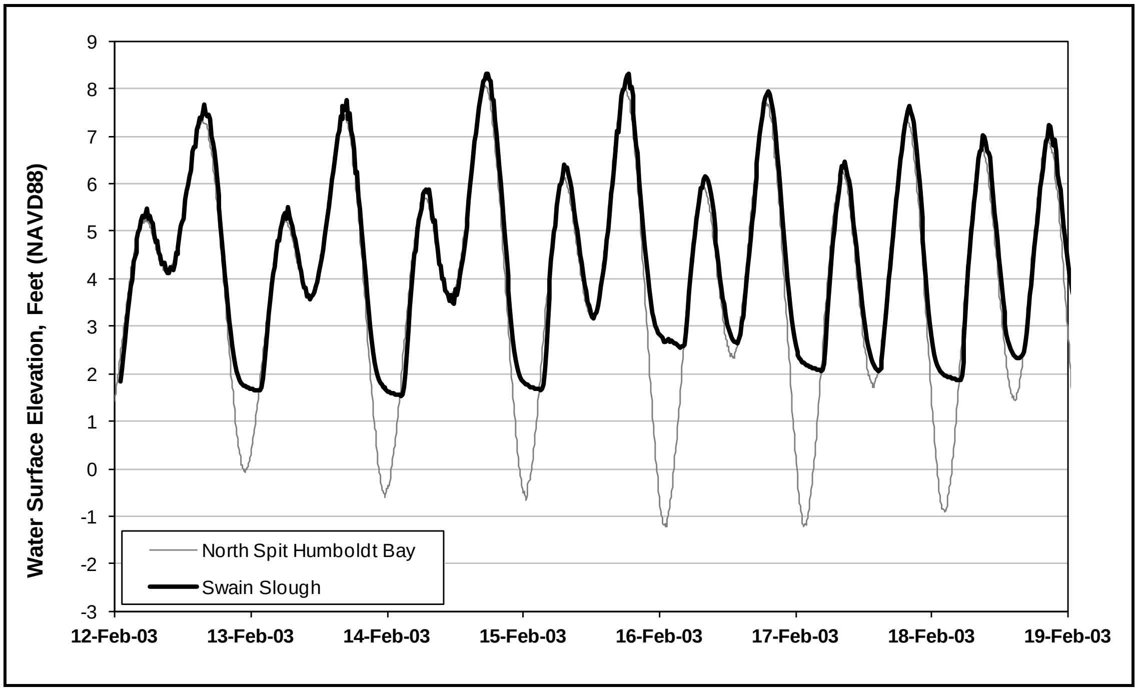


Table 3-7. Recorded tides in Swain Slough (Black) and corresponding tide recorded at North Spit, Humboldt Bay (Grey) (Station No. 9418767). A tidal sill, likely a sand bar at the mouth of Elk River, prevents Swain Slough water levels from dropping below approximately 1.5 feet.

Tidal Datums

Humboldt Bay experiences semidiurnal tides: two high tides and two low tides per day. The tidal datums of Mean Higher High Water (MHHW), Mean Lower High Water (MLHW), Mean Higher Low Water (MHLW) and Mean Lower Low Water (MLLW) are used for designing tidal restoration projects. These tidal datums can be computed from tidal records for a given time period, typically a 19 year epoch. The last complete tidal epoch extended from 1983 to 2001.

To consider changes in sea-level that may have occurred more recently than the 1983-2001 epoch, mean daily tidal datums were computed for the North Spit tidal station (No. 9418767) using years 1993 through 2010, nearly a complete epoch (Table 3-8). With the exception of MLLW, which is influenced by the tidal sill, the tidal datums in Swain Slough were assumed nearly identical to the North Spit (W-K et al., 2006).

Table 3-8. Tidal Datums for the North Spit, Humboldt Bay (Station No. 9418767) using the Period of 1993 through 2010. Swain Slough Assumed to be Identical, Except at MLLW.

	Tidal Elevation (NAVD88)	
Tidal Datum	**North Spit**	**Swain Slough**
Mean Higher High Water (MHHW)	6.65 feet	6.65 feet
Mean Lower High Water (MLHW)	5.23 feet	5.23 feet
Mean Higher Low Water (MHLW)	2.3 feet	2.3 feet
Mean Lower Low Water (MLLW)	-0.20 feet	1.5 feet*

*Approximate elevation of the tidal sill at the mouth of Elk River.

Swain Slough Annual Tide

Design of the tidal components of the project, such as design MHHW and the proposed marsh plain elevations, required a longer-term tidal record for Swain Slough than the week of gaged tides. Therefore, it was necessary to construct a long-term record using the North Spit record. Using a record length that encompasses an entire tidal epoch to simulate hydraulic conditions in the project area would be both time and computationally intensive. One year of record, if it reflects the range of tidal conditions in an epoch, is more manageable for computations and provides similar results as modeling a full epoch.

To validate this approach, exceedance frequency curves of tidal elevations at the North Spit using the long-term record (1993-2010) and only one-year of record, February of 2003 through January of 2004 were compared (Table 3-9). The selected one-year period encompasses the period-of-record of flow monitoring in Martin Slough and the one-week period of tidal monitoring in Martin Slough. It also included an extreme high tide of 9.48 feet on December 23, 2003 that has an annual probability of exceedance of approximately 12 percent (8.33-year return period), occurring only twice between 1993 and 2010.

The comparison of the 28-year tidal record and the one-year record showed that the frequency of inundation only differs by a maximum of 0.1 feet at any given elevation (Table 3-9). Therefore, it was determined that for the project design, the one-year tidal record (2003-2004) adequately represents the long-term frequency of tidal inundation that occurs in Humboldt Bay.

Using the one-year data record for the North Spit, an Annual Tide was constructed for Swain Slough (Table 3-10). It matches the North Spit tides except that tides below 1.5 feet are truncated to represent the tidal sill at the mouth of Elk River. The resulting tidal datums for Swain Slough are the same as for North Spit (Table 3-9), except for MLLW, which is equal to the presumed tidal sill elevation of 1.5 feet.

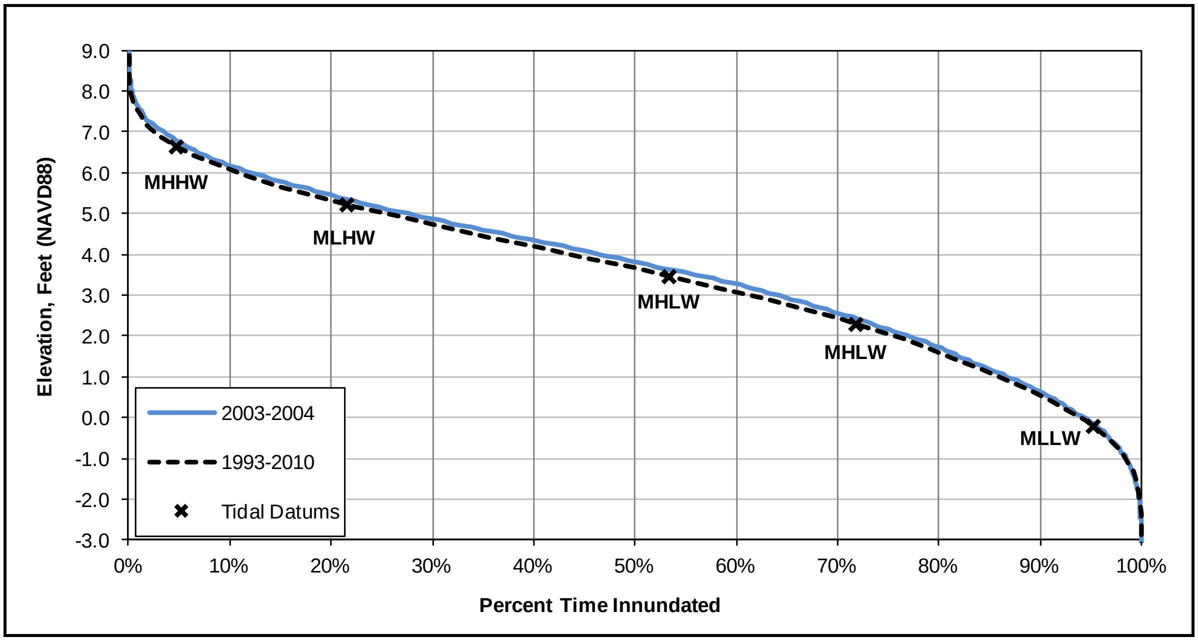


Table 3-9. Comparison of frequency of tidal inundation for North Spit, Humboldt Bay (Station 9418767) using different record lengths. Tidal datums calculated using the 1993 to 2010 record.

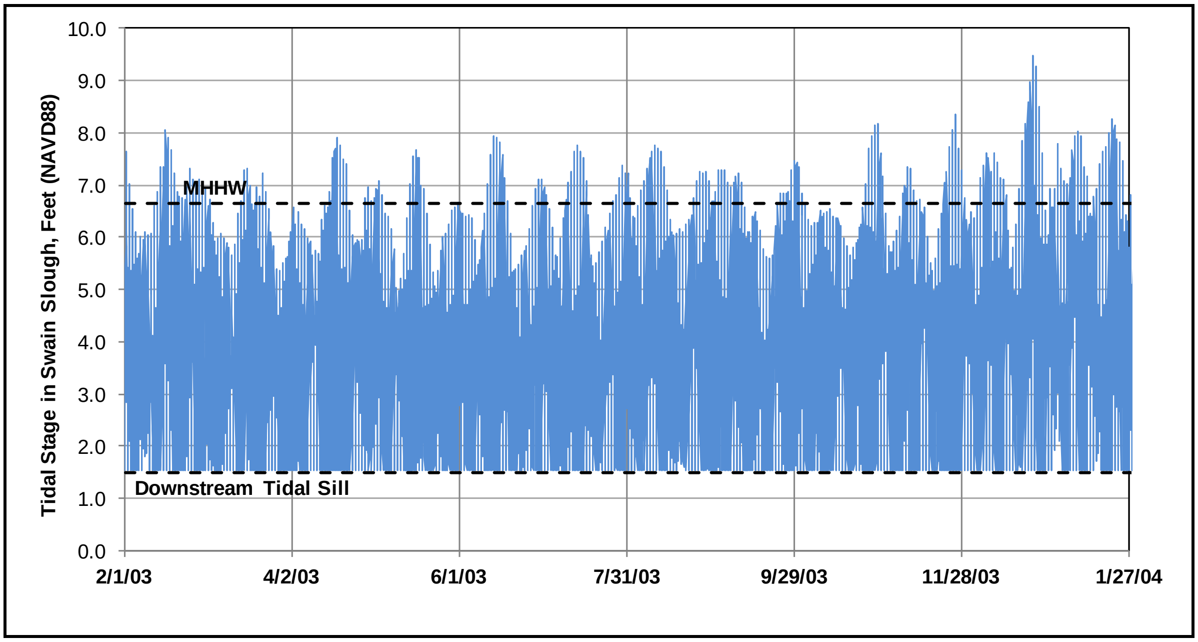


Table 3-10. One-year of tidal record for Swain Slough constructed using North Spit, Humboldt Bay tidal records (Station No. 9418767), with tide truncated at elevation 1.5 feet to account for downstream tidal sill at the mouth of the Elk River.

4.0 Design Development

Development of individual project elements was an iterative process involving (1) determining initial channel and pond dimensions, (2) simulating hydraulic conditions for the initial design, (3) reviewing, processing and synthesizing model results, (4) refining the project design, and (5) updating the hydraulic model to represent project refinements and repeating this process.

Preliminary designs for the replacement tide gate, tidal slough channel and tidal wetlands for the project were developed and presented in the feasibility study (W-K et al., 2006) and further developed in the 30% basis of design report (GHD and MLA, 2013). The new tide gate, as described in the 30% plans, was constructed in 2014. As part of the 65% design development, several substantial design refinements were required to address review comments on the 30% design and underground utility constraints. This required adjusting slough channel sizes, pond and marsh plain sizes, and re-analyzing hydraulic conditions for the revised design.

The following sections present the methods used to design the tidal slough channel and the one-dimensional hydraulic model used for the project design. Chapter 5 presents the results of the hydraulic modeling for specific project elements.

4.1 Tidal Slough Channel Design

4.1.1 Martin Slough

The project area of Martin Slough will be largely within the limits of tidal influence after project implementation. Though Martin Slough receives freshwater inflows, the hydraulic geometry of the tidal channel of Martin Slough will be governed by the daily tidal flux created by the muted tide rather than less frequent high flow events from upstream. Therefore, the channel cross section and profile design were based primarily on established tidal channel design methodologies. These methods use geomorphic relationships between stable tidal channel geometry and tidal prism to predict the channel dimensions.

The dimensions of the tidal slough channel in Martin Slough were designed using equilibrium hydraulic geometry relationships for tidal channels, which are summarized in Williams et al. (2002). Additional information is available in Coats et al. (1995) and PWA and Faber (2004). A series of three iterative regression equations are available that relate the contributing tidal prism to the channel cross sectional area, top width, and channel depth below MHHW. The final tidal channel geometry should fall within the recommended values predicted by the regression equations.

Because the tidal sill in Swain Slough prevents tide levels from falling below 1.5 feet, substantially higher than MLLW, only the regression equation that relates channel area to the contributing tidal prism was used. The iterative process used in solving the regression equations yielded a channel cross section shape and size and a longitudinal profile in equilibrium with the contributing tidal prism.

Tidal prism decreases in the upstream direction, causing the stable tidal channel geometry to decrease moving upstream. To account for this, the project channel was divided into seven reaches, numbered from downstream to upstream as 1 through 7 (Table 2-1). Reach breaks were generally located at the confluences of the proposed ponds and tributary confluences because they contribute significantly to the tidal prism of the downstream reach. The channel cross sections narrow toward the upstream end of the reach and match the existing channel width upstream of the confluence with the North Fork Tributary.

Contributing tidal prism and stable channel geometry was calculated for each reach (Table 2-5). The contributing tidal prism in the channel ranges from nearly zero at the upstream end of the project area to approximately 20 acre-feet at the Martin Slough tide gates.

4.1.2 Slough Channel in Tidal Marsh Complex C

Tidal Marsh Complex C will contain a tidal slough channel that connects Martin Slough and the existing tributary channel. The tidal channel was sized using the same relationships and process as for the mainstem slough. Pond C channel was divided into 5 reaches. Each reach was sized for the contributing tidal prism using the hydraulic geometry relationships.

4.2 Hydraulic Modeling

The channel design for the project was evaluated using the Army Corps of Engineers HEC-RAS hydraulic model. HEC-RAS was selected due to its capabilities and ease of modifying project geometry and simulating different boundary conditions. HEC-RAS performs the unsteady, gradually varied flow modeling that was necessary to evaluate the interaction of the freshwater inflow hydrographs with the changing tidal conditions. Unsteady flow simulations in HEC-RAS were used to route flow through the project area for various scenarios, including annual tidal and streamflow conditions and discrete storm events. The results of the modeling were used for the following:

- Verifying the design tidal prism and MHHW elevation,
- Identifying the elevations of design tidal marsh plains to obtain the desired vegetation diversity,
- Comparing duration and frequency of storm flow flooding for existing and design conditions,
- Assessing sediment mobility through the project area, and
- Evaluating seasonal salinity within the Martin Slough mainstream and tidal wetlands.
- Estimate water velocities and slopes for engineered log structure stability calculations.

4.2.1 HEC-RAS Model Geometries

Three model geometries representing the proposed project were used to evaluate (1) return-period storm flow conditions, (2) annual variation in flows and water levels, and (3) seasonal extents of salinity within the project area. Each model geometry is described in the following sections. Design conditions for the storm flows were compared to an existing-condition model prepared as part of the 30% design.

Storm Flow Model Geometry

Cross Sections

Cross sections were used to reflect proposed channel and overbank topography of the mainstem, North Fork Tributary and historical meander within the project area. Cross sectional geometry and spacing is used by HEC-RAS to route flow and calculate water storage within the project area at each modeling time-step.

Cross sections were spaced approximately 100 feet apart, except where the presence of ponds, confluences and the Fairway Drive road crossing required closer spacing (Table 4-1). Within the golf course portion of Martin Slough, cross sections encompass both the channel and overbank areas and extend to the adjacent valley walls. Where areas of the golf course will be raised to elevation 7.0 feet, “Blocks” were inserted into relevant cross sections to an elevation of 7.0 feet.

The existing stream channel upstream of the project area in both the North Fork Tributary and mainstem of Martin Slough were modeled using field surveyed channel cross sections.

The confluences of the Martin Slough mainstem channel with the North Fork tributary and both the upstream and downstream confluences of the meander bend with the mainstem and Southeast Tributary were simulated using “Junctions”. Pilot channels up to 1-foot wide were used where necessary to maintain model stability when portions of the project area are not inundated.

No topography was available for Swain Slough. Therefore Swain Slough was modeled as a separate reach that was 1,000 feet long, with cross sections that were 100-feet wide. Reach length and channel dimension were selected to create a stable downstream boundary condition with water levels influenced only by the tidal boundary conditions.

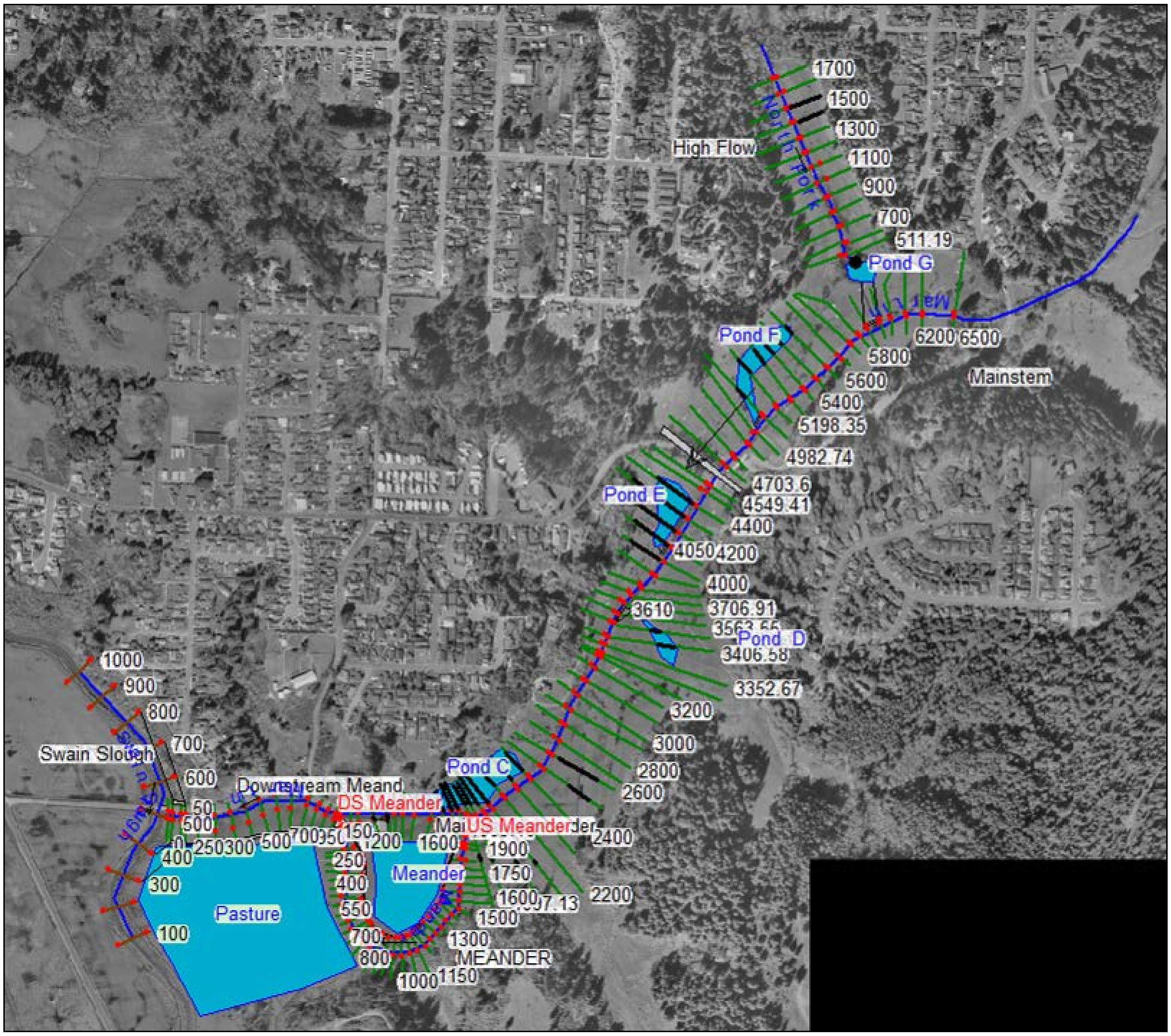


Table 4-1. Schematic of HEC-RAS cross section locations (green) and storage areas (blue). Ponds and overbank storage areas are schematic and not to scale. The black lines associated with some cross sections indicate where "blocks" are used in the cross sections.

A Manning's roughness coefficient of 0.04 was used to simulate a mature tidal channel, which includes woody debris, overhanging vegetation, and irregular banks. Overbank roughness coefficients of 0.06 were used to simulate shallow flow through the mowed or grazed grass adjacent to the channel (Chow, 1959).

Storage Areas and Lateral Structures

Ponds C, D, E, F and G were modeled as "Storage Areas" connected to the main channel using "Lateral Structures" (Table 4-1). Storage-elevation relationships were computed at 0.5-foot increments from the digital terrain model of design conditions. Storage volumes for the channels connecting the ponds to Martin Slough were included in the total pond storage. The connecting channels between the ponds and the main channel were modeled as broad crested weirs 10-feet long with the cross-sectional shape of the pond sill. The elevations of the lateral structures were set at the sill elevation of the pond. Where the proposed ponds coincide with a cross sections, the elevations of the ponds represented by the storage areas were "blocked" in the cross sections to eliminate redundant computation of storage areas.

The overbank areas of the meander bend and pasture on the NCLT property and the overbank area of the confluence of the North Fork Tributary with the Mainstem were modeled as Storage Areas connected to the main channel using Lateral Structures (Table 4-1). On the NRLT property, the planform geometry of the channel, pasture area and meander bend necessitated modeling these overbank areas as storage areas with lateral structures extending across multiple channel cross sections connecting the channel to the storage area. Therefore, the cross sections on the NRLT property only include the tidal channel and new marsh plain, with the lateral structures allowing overbank flow to enter the storage areas.

Bridge Crossings

The existing bridge crossing at Fairway Drive was modeled using the surveyed top and bottom of the bridge deck and pier locations. As-built drawings for the bridge from 1976 were used to determine pier dimensions and locations. The pier depths are not specified in the as-built drawings and are unknown.

The two proposed livestock crossings across the meander bend were simulated as 30-foot long, 4-foot diameter RCP culverts with a road crest elevation of 8 feet. The invert elevations of the downstream culvert were set at 2.5 feet. The invert elevations of the upstream culvert were set at 3.0 feet.

The proposed bridge crossings at the NRLT barn and on the golf course were not included in the hydraulic modeling. The bridge at the barn is designed to be above the 100-year water surface elevation and will have no influence on project hydraulics. It is assumed that the golf course bridges will also be perched above the 100-year water surface elevation, with cart path approaches to the bridges at grade except in close proximity to the bridges. It is not expected that the bridge approaches will substantially block floodplain flows.

Tide Gates

The new tide gate and MTR doors was modeled assuming that it is fully operational and will provide bi-directional hydraulic connectivity between Martin Slough and Swain Slough. The tide gate was included in the HEC-RAS modeling using Lateral Structures connecting Swain Slough to Martin Slough. Outgoing flow was modeled using a triple cell 6-foot high by 6-foot wide concrete box culvert with flaps that allow flow to leave Martin Slough but prevent tidal inflow from entering. The invert elevation of each cell was set to the design elevation.

During the incoming tide, tidal waters flowing from Swain Slough into the project area through the 6-foot by 6-foot MTR gate and auxiliary door were modeled as sluice gates with a discharge coefficient of 0.6. Once the water surface elevation in Martin Slough reaches the specified closing elevations of the MTR gate and auxiliary door, the gates fully close in one time-step. Though constructed as a 2-foot by 2-foot gate, the auxiliary door was modeled as a 2-foot wide gate that is 1.5-feet tall to account for the partially open nature of the top hinged gate.

Annual Variation and Sediment Transport Model Geometry

The model geometry from the Storm Flow HEC-RAS model was adapted for the annual variation modeling. To obtain more detailed results of the water level within the ponds, cross sections were incorporated into the model to represent each pond, outfall channel and pond sill, and the realigned Southeast Tributary channel and pond. Cross section spacing for these features was typically approximately 50 feet, varying with the level of complexity of the proposed feature. The lateral structures connecting the ponds to the channel were replaced with Junctions.

Pilot channels, typically use to stabilize unsteady flow models when a cross section goes dry were found to drain the ponds rather than maintain standing water at the pond sill elevation. To maintain model stability and pond standing water elevation, the “Deck/Roadway” function was used to establish the sill elevation. To maintain model stability a 0.2-foot diameter culvert placed through the pond outlet sill was used as a pilot channel to maintain negligible pond outflow when the sill was dry. Typically, flow elevations did not exceed the cross-sectional elevations representing the Meander and Pasture, thus overbank storage areas were not included in the model geometry.

Salinity Model Geometry

The water quality module of the HEC-RAS (ACOE 2010a) was used to model design-condition salinity in Martin Slough mainstem, North Fork Tributary, Southeast Tributary, and Ponds C, D, E, F and G during the range of tides and freshwater inflows that occur on an annual basis. The HEC-RAS water quality module is not compatible with lateral structures or storage areas, which were used extensively in the Storm Flow and Annual Variation HEC-RAS geometries. Therefore, it was necessary to adapt the Annual Variation HEC-RAS model geometry into a more simplified geometry for the salinity model.

The Swain Slough Reach was deleted and the lateral structures representing the tide gate at the downstream end of the model was replaced with a stage and flow hydrograph at the location of the tide gate. The stage and flow hydrographs served as the downstream boundary condition and was obtained from the Annual Variation HEC-RAS model results at the cross section just upstream of the tide gate. This hydrograph reflects tidal stage and flow into and out of the project area through the new tide gates.

Existing Condition Geometry

Existing condition HEC-RAS geometry was based on models prepared for the design of the new tide gates, and described in the 30% basis of design report (GHD and MLA, 2013). The existing condition model was prepared using the tide gate that was present before the new tide gate was installed in 2014.

Model Boundary Conditions

Several different freshwater inflow and Swain Slough tidal conditions were developed for evaluating various scenarios in HEC-RAS.

Stormflow Hydrographs and Tides

The hydrographs for the 24-hour 2-year, 10-year and 100-year precipitation events predicted using HMS (Section 3.2.3) were used as the stormflow hydrographs to evaluate high flows (Table 4-2).

The recorded tidal elevations in Swain Slough between February 13 and 19, 2003 (Section 3.3.3) were used for the corresponding tidal boundary condition, as shown in Table 4-2.

Annual Inflow Hydrographs and Tides

Project design required simulating conditions throughout an entire year. Annual inflow hydrographs for each of the six inflow locations (Martin 1 through 6) were constructed using the gaged flow record for the mainstem of Martin Slough at the upper Fairway Drive crossing scaled to the contributing drainage area for inflow location. The resulting annual inflow hydrographs extend from February 12, 2003 to January 9, 2004, approximately 11 months. The gaging record had a gap between July 22 and November 7, when flows were not gaged. Because this period is dominated by baseflow conditions, a total baseflow of 1.0 cfs at the tide gates was assumed. The total baseflow was then scaled by drainage area to arrive at the baseflow for each inflow location. This yielded a baseflow of 0.5 cfs at Lower Fairway Drive, which is slightly lower than the flows gaged in 2003, but higher than flows gaged in 2008 (Section 3.3.1).

The 2003-2004 Annual Tide constructed for Swain Slough based on North Spit Humboldt Bay records (Section 3.3.3) was used as the corresponding tidal conditions for the Annual Hydrograph.

Stormflow Hydrographs and Tides

The hydrographs for the 24-hour 2-year, 10-year and 100-year precipitation events predicted using HMS (Section 3.2.3) were used as the stormflow hydrographs to evaluate high flows (Table 4-2).

The recorded tidal elevations in Swain Slough between February 13 and 19, 2003 (Section 3.3.3) were used for the corresponding tidal boundary condition.

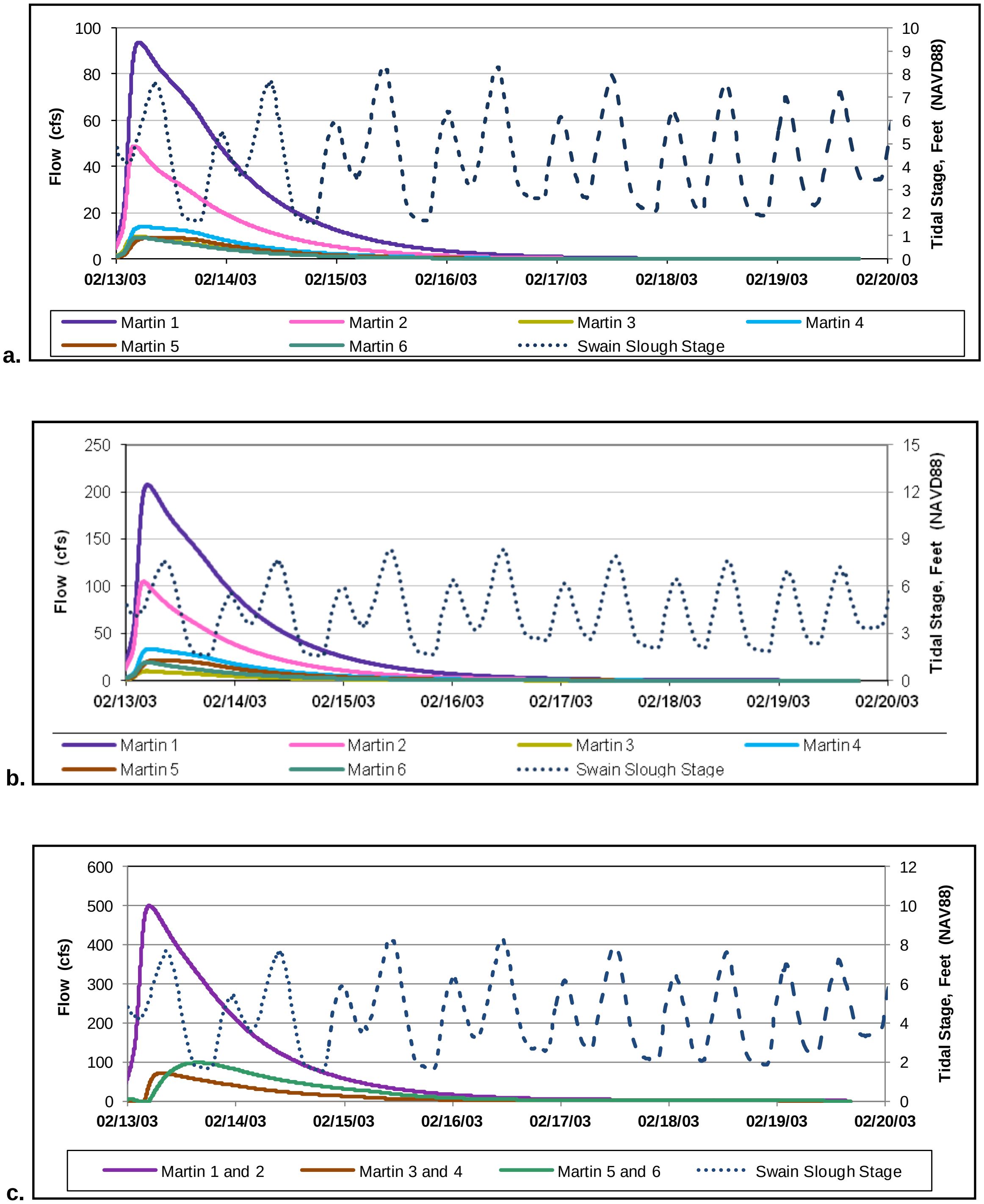


Table 4-2. HEC-HMS predicted runoff hydrographs for sub basins draining into the project area resulting from a 24-hour rainfall events with return periods of (a) 2 -years, (b) 10 years, and (c) 100 years. The dotted line shows the measured tidal stage in Swain Slough.

Annual Inflow Hydrographs and Tides

Project design required simulating conditions throughout an entire year. Annual inflow hydrographs for each of the six inflow locations (Martin 1 – 6) were constructed using the gaged flow record for the mainstem of Martin Slough at the upper Fairway Drive crossing scaled to the contributing drainage area for inflow location. The resulting annual inflow hydrographs extend from February 12, 2003 to January 9, 2004, a total of 331 days. The gaging record had a gap between July 22 and November 7, when flows were not gaged. Because this period is dominated by baseflow conditions, a total baseflow of 1.0 cfs at the tide gates was assumed. The total baseflow was then scaled by drainage area to arrive at the baseflow for each inflow location. This yielded a baseflow of 0.5 cfs at Lower Fairway Drive, which is slightly lower than the flows gaged in 2003, but higher than flows gaged in 2008 (Section 3.3.1).

The 2003-2004 Annual Tide constructed for Swain Slough based on North Spit Humboldt Bay records (Section 3.3.3) was used as the corresponding tidal conditions for the Annual Hydrograph.

4.2.2 Salinity Modeling

The water quality module of HEC-RAS uses the results of the hydraulic modeling to compute advection and dispersion of various water quality constituents, including user-defined constituents. Salinity can be modeled in the water quality module as a user-defined constituent with concentrations defined in the boundary conditions of the model.

Salinity Boundary Conditions

The salinity model was prepared using the gaged Annual Inflow Hydrographs prepared for the project. The model was executed from February 12, 2003 to December 6, 2003, approximately 10 months, after which it became unstable.

Salinity was modeled with a fixed concentration at the model boundary conditions. For all freshwater boundary conditions, a concentration of 0.1 mg/l ([0.0001 parts per thousand (ppt)) was used. Waters in Swain Slough were assumed fully saline for this analysis. A value of 32,000 mg/l (32 ppt) was used as the downstream boundary conditions where the fully saline flows from Swain Slough enter the project area through the tide gates. This boundary condition neglects that during extended periods of high flow in Elk River, salinity concentrations become substantially lower in Swain Slough.

Dispersion Coefficient

Literature values for dispersion coefficients in tidal channels and streams of similar size to Martin Slough vary from less than 10 feet^2/second to over 100 feet^2/sec. (Vallino and Hopkinson, 1997, Kashefipour and Falconer, 2002; Ralston and Stacey, 2005). Dispersion values were found to decrease with decreasing channel/estuary size and distance from the tidal boundary as well as with decreased water velocities and increased water depths. Vallino and Hopkinson (1997) used field measurements to calculate a dispersion coefficient of approximately 36 feet^2/second in a tidal channel upstream of an estuary similar in size to Martin Slough.

Vallino and Hopkinson (1997) found that measured dispersion values were similar to values predicted using equations developed by Fischer et al. (1979). Dispersion values computed using Fischer et al. (1979) are a function of flow velocity, flow depth, channel width and slope. This set of equations is available in the HEC-RAS water quality module and are computed for each time-step that the model is run.

Dispersion coefficients were computed internally by HEC-RAS with an allowable range limitation of 0.1 to 500 feet^2/second. Model computed values averaged 0.3 feet^2/second.and ranged from 0.09 to 46.5 feet^2/second. Computed dispersion coefficient values increased with increased freshwater inflow and with proximity to the tidal effects of Swain Slough. Although some of the dispersion coefficients computed for the mainstem are higher than literature values, they typically occurred during a few discrete timesteps and did not appear to affect the results of the modeling. Salinity results where anomalously high dispersion coefficients computed were not used.

A constant water temperature of 15°C was used in the salinity modeling.

4.2.3 Modeled Scenarios

HEC-RAS model simulations were performed for a variety of boundary conditions dependent on the modeling purpose. Table 4-3 presents the various scenarios modeled and their use in the project design and evaluation process.

For each simulation, computations were performed at 1-minute time-steps. Modeling for short-term stormflow events was reported at 10-minute intervals. Due to file size limitations in HEC-RAS, results for the year-long modeling events were reported at 20-minute intervals.

Table 4-3. Scenarios for which HEC-RAS modeling was performed. The results of the modeling were used to design various project elements as indicated.

Scenario	Purpose
Scenarios 1-3: Existing Condition Stormflow Geometry: Existing Conditions (with original tide gate) Freshwater Inflow: Stormflow Hydrographs for 24-hr 2-yr, 10-yr, & 100-yr Precipitation Swain Slough Tidal: February 2003 Recorded Stages Duration of Simulations: 7 days	• Evaluate flood extents and duration of out-of-bank flows for existing conditions • Evaluate existing conditions sediment transport competence
Scenarios 4-6: Design Condition Stormflow Geometry: Proposed Storm Flow Freshwater Inflow: Stormflow Hydrographs for 24-hr 2-yr, 10-yr, & 100-yr Precipitation Swain Slough Tidal: February 2003 Recorded Stages Duration of Simulations: 7 days	• Evaluate channel capacity • Evaluate extents and duration of out-of-bank flooding • Evaluate sediment transport competence • Establish minimum bridge bottom and sheet pile top elevations
Scenario 7: Annual Variation Geometry: Proposed Annual Variation Freshwater Inflow: Gaged Annual Inflow Hydrographs with constant baseflow of 1 cfs in Summer Swain Slough Tidal: Annual Tide Duration of Simulation: February 2003 to January 2004	• Establish tidal datums in project area resulting from tidal muting • Characterize frequency of inundation to set salt marsh and emergent wetland vegetation elevations. • Evaluate sediment transport competence
Scenario 8: 2-Year Flow Geometry: Proposed Annual Variation Freshwater Inflow: 2-Year Flows Swain Slough Tidal: Annual Tide Duration of Simulation: February 2003 to January 2004	• Evaluate sediment transport competence • Evaluate flow velocities across pond sills.
Scenario 9: Salinity Distribution Geometry: Salinity Modeling Freshwater Inflow: Gaged Annual Inflow Hydrographs with constant baseflow of 1 cfs in Summer Swain Slough Tidal: Results of Annual Variation Modeling at Tide gate Duration of Simulations: February 2003 to December 2003	• Evaluate extent and concentration of salinity in project area throughout the year • Predict salinity for aquatic habitat and vegetation communities

4.3 Engineered Log Structure Stability Calculations

The proposed engineered logs structures (ELS) were designed and their stability evaluated in accordance with guidelines presented in NRCS, 2005 and 2007; and D'aoust and Miller, 2000. The overall approach was adapted from NRCS Technical Supplement 14J (2007). A factor of Safety (FS) for buoyancy is defined as the ratio of Resisting Moments to Driving Moments and is considered adequate when equal or greater than 2.0 (NRCS 2007). A minimum FS value of 2.0 against buoyancy and overturning was used to determine the necessary anchoring required for each structure. Details are provided in Appendix C

4.3.1 Uplift Forces

Each structure experiences two types of uplifting forces that contribute to the overturning moment: buoyancy and lift.

Buoyancy

In all cases, the logs in the ELS are fully submerged during high flows. An average value for specific gravity for Douglas fir and redwood of 0.51 was used (D'oust 1998, USFS 1999) and represents "seasoned" logs. The resulting net buoyant force is in the upward direction. After construction, as the log is fully or partially submerged daily, the density of the log will increase and the resulting buoyant force will decrease.

Lift

Lift is an upward force resulting from water velocities traveling over or around the log. The maximum water velocities predicted by the HEC-RAS model were used to calculate lift. In general, the force imposed by lift is negligible in comparison to the buoyant force.

4.3.2 Vertical Resisting Forces

The ELS were designed to remain in place when completely submerged and subject to maximum anticipated water velocities. The proposed large wood structures are anchored using (1) soil placed on top of the buried portions of logs, (2) by vertical log piles, and (3) soil anchors.

Weight of Soil on Logs

Some of the log members in the ELS will be buried into the channel banks. The weight of the soil on top of the log helps resist uplift and overturning of the log. The length of buried log and mean burial depth were used to calculate the weight of the soil. In general, minimal depths were used to be conservative. In all situations the soil was assumed to be submerged, thus reducing its effective weight due to buoyancy of the soil. A silty clay soil type was assumed based on the project geotechnical report (SHN 2013 - Appendix A). For submerged soil, a specific gravity of 1.63 and angle of internal friction of 20 degrees were applied. For simplicity, the weight was applied to the center of the buried section of the log member. This was considered an acceptable assumption given that the burial depth was estimated conservatively.

Log Pile Anchors

Log piles are driven below the potential scour line and project above the finished structure. Log piles will be used to counteract the upward force of buoyancy and lift from flowing water. The number and embedment depth of the log piles was computed using standard skin friction capacity for wood piles in non-cohesive material.

The available anchoring force for each pile depends on material properties of density, diameter, buried depth, and soil properties of unit weight of soil, cohesion, and soil internal angle of friction. Soil properties used in the pile calculations and soil anchor holding capacity are derived from the project Geotechnical Report (SHN, 2013 – Appendix A). Multiple borings show that the soils are composed of sandy silts, silty sands, silty clays, and peat. Internal angles of friction for these soil types range from 18° to 27°, with larger angles resulting in greater skin friction capacity. A conservative internal angle of friction of 20° was selected for calculations of pile skin friction. Conservative values were used for properties of cohesion (0.1 lb/sf) and unit weight of soil (62.4 lb/cf). An average value for specific gravity for Douglas fir of 0.51 was used (D'oust 1998, USFS 1999). Buoyant forces were computed using diameter of the log, length of the log, the submerged specific gravity of the log, gravitational acceleration, and density of material to compute the total upward buoyant force acting on a log structure.

Skin friction capacity for piles was calculated for buried depths of 6, 7, 8, 9, and 10 feet. Assuming a total pile length of 15 feet, a portion of the skin friction capacity was applied to each pile such that the pile itself would have a factor of safety of 2.0 against buoyancy when completely submerged. The remaining skin friction capacity was then applied to resist uplifting forces of the log structure members. A moment analysis was conducted for the log structure members to determine the required depth to bury the pile logs and prevent overturning with a factor of safety of at least two.

Soil Anchors

The available holding capacity for soil anchors depends on soil type and is provided by Manta Ray for each anchor that they manufacture. The selected soil anchor model (MR-2) was selected based on soil type, the expected size of the root wad or log member, and attaining a factor of safety of at least two. Based on manufacture's recommendations, the anchor will be installed a minimum of 6 feet deep.

5.0 Proposed Project Conditions

The following sections characterize the predicted physical conditions of the proposed project based on conducted hydraulic, sediment mobility, and water quality analyses.

5.1 Muted Tide Characteristics

Table 5-1 presents a typical portion of the Scenario 7 hydraulic modeling results during a period of low stream flows and spring tides in Swain Slough. These conditions are expected to produce the maximum concentrations and extent of salinity within Martin Slough. Table 5-2 presents a typical portion of the Scenario 7 hydraulic modeling during the wetter months during which rain events and freshwater inflow are at their peak.

Table 5-1 indicates that the new 6-foot by 6-foot (6x6) Muted Tide Regulator (MTR) gate, when open on a flood tide, is adequately sized to allow Martin Slough to rise at the same rate as Swain Slough. Once the flood tide causes Martin Slough water level to reach an elevation of 4.0 feet, the 6x6 MTR gate closes but in-flow continues through the MTR equipped auxiliary door. The auxiliary door was sized to restrict inflow, slowing the rate that tidal water levels rise in Martin Slough relative to Swain Slough. This mimics but mutes the natural tidal patterns in Swain Slough, which is necessary to maintain tidal marsh vegetation zonation and diversity (Section 5.2).

The elevation in Martin Slough at which the auxiliary door shuts was established to prevent tidal flooding of low-lying areas on the golf course and to help ensure that saline waters do not reach the elevation of the root-zone of golf course turf. The minimum elevation of golf course turf within the golf course will be at approximately 7 feet, after several low areas within the golf course are raised. Assuming approximately 1-foot of capillary action may occur, the maximum elevation for saltwater was targeted at approximately 6 feet in elevation.

If the auxiliary door was left open during large spring tides, the tide elevation in Martin Slough would slightly exceed the threshold of 6.0 feet. Therefore, it is equipped with an MTR system that can be set to shut the gate when incoming flows into Martin Slough reach 5.7 feet. During average and neap tides, the high tide within Martin Slough will be less than 5.7 feet and the auxiliary door will not shut. During spring tides, the Martin Slough tidal levels reach 5.7 feet and the auxiliary door will close to prevent saltwater elevations within Martin Slough from reaching an elevation of 6 feet (Table 5-1). The tide gate system was designed such that the elevations at which both the 6x6 MTR gate and auxiliary door shut on incoming tides can be fine-tuned and the float-switch adjusted as needed.

Table 5-2 presents typical water levels in Martin Slough during the wetter winter months. After the auxiliary door shuts, freshwater inflow causes water levels in Martin Slough to rise above an elevation of 6 feet. Because these are freshwater flows, they will stratify on top of the water column. Once water levels within Martin Slough are higher than in Swain Slough, all three 6x6 gates and the auxiliary door will open to allow water to drain from Martin Slough.

5.2 Marsh Plain Design

Marsh Plains A and B and Tidal Marsh Complex C (Pond C) are expected to be brackish to saline most of the year and are expected to support tidal marsh vegetation, thus were designed specifically to support salt marsh plant communities.

5.2.1 Salt Marsh Plant Community Distribution by Elevation

The composition and function of tidal marshes are highly dependent on site-specific dynamics of the tide cycle. The duration that soil is inundated by saltwater is influential in what plant species, if any, become established. With this information, tidal wetlands can be designed with predictable species composition. Also, careful selection of constructed wetland elevations can sometimes be used to hinder colonization by a targeted invasive species such as *Spartina densiflora.*

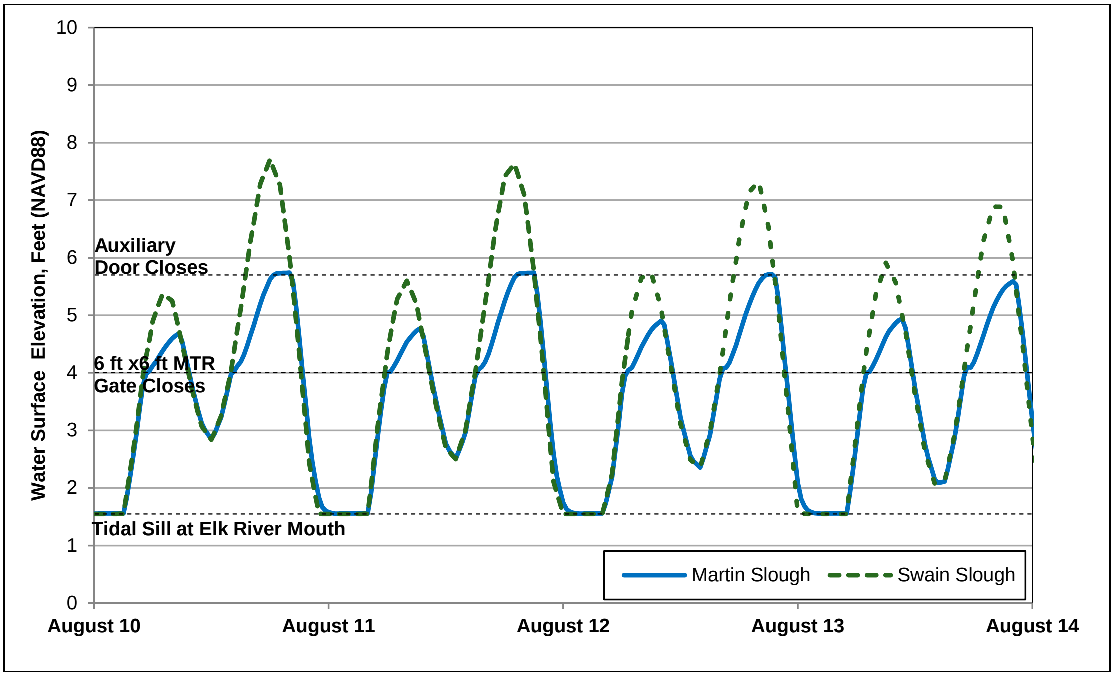


Table 5-1. Simulated muted tide in Martin Slough during a period of low streamflows and spring tides in Swain Slough. During an incoming tide the 6-foot by 6-foot MTR gate will close when Martin Slough rises to elevation 4.0 ft and the MTR equipped auxiliary door remains open until Martin Slough reaches elevation 5.7 feet.

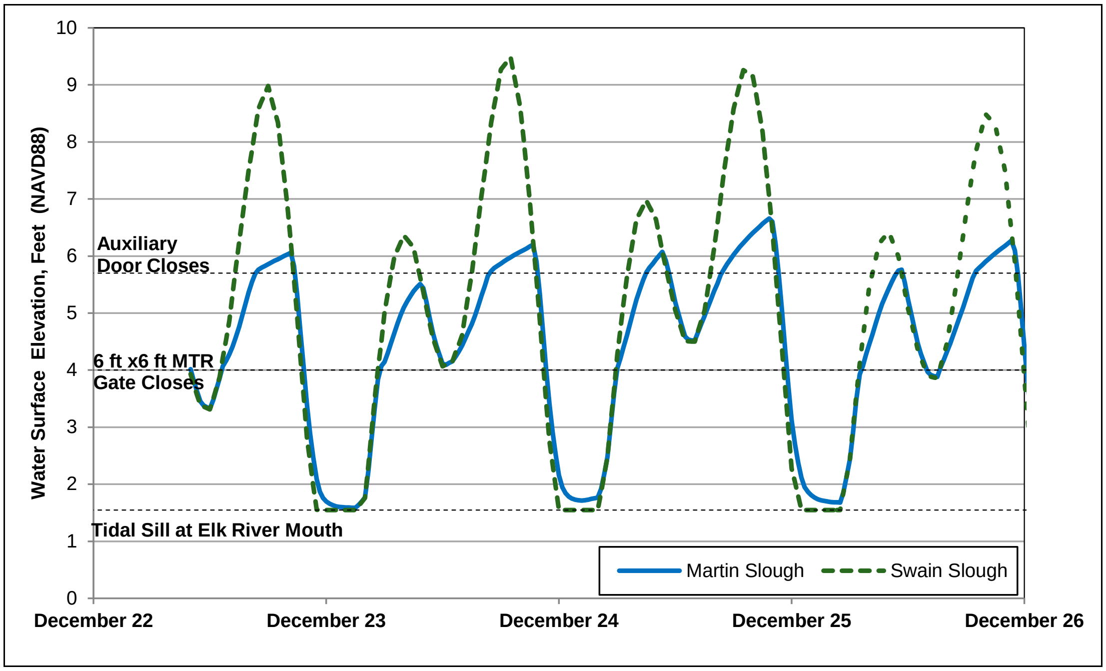
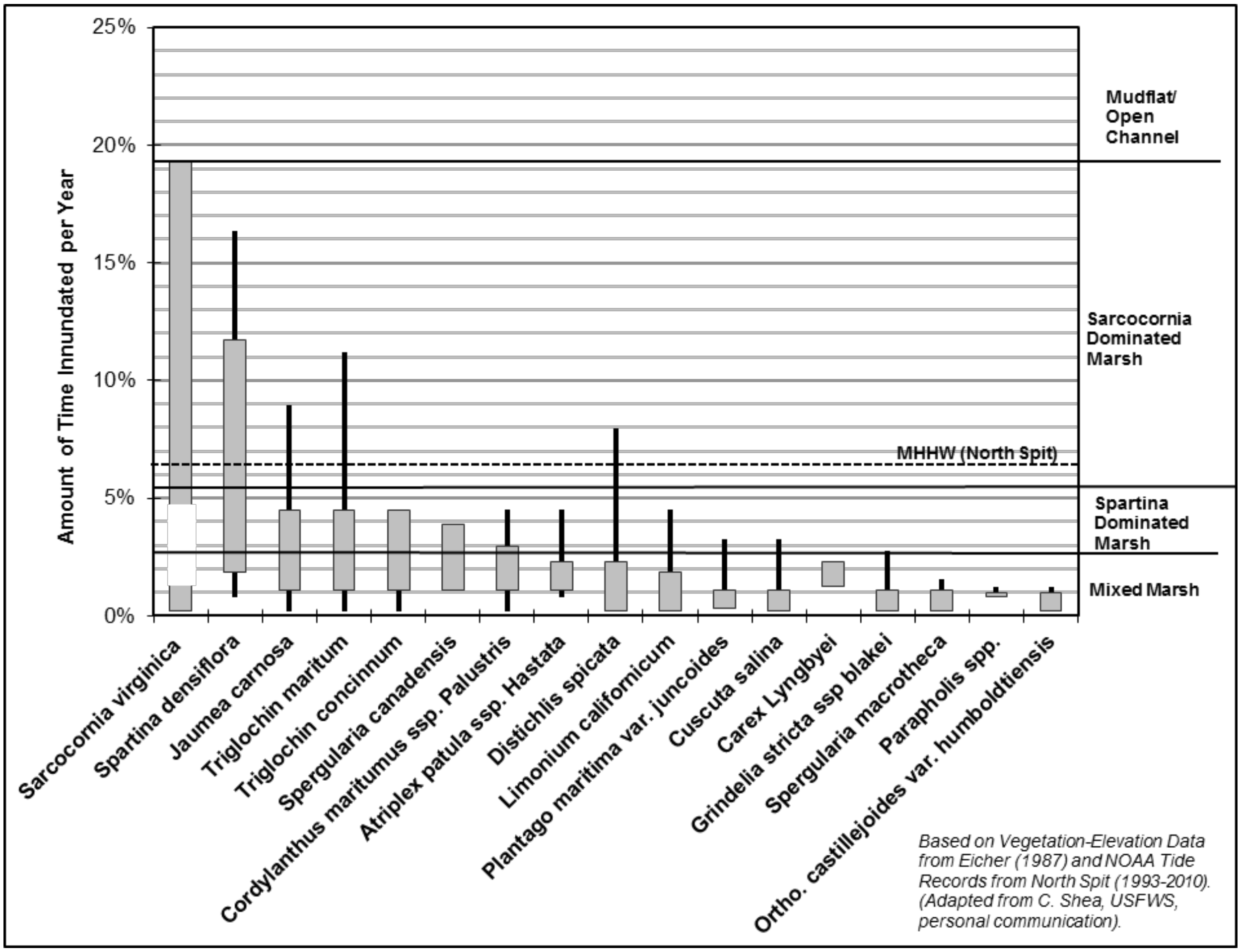


Table 5-2. Simulated muted tide in Martin Slough during the wet season and winter tides in Swain Slough. Freshwater inflow results in an increasing water surface elevation on incoming tides after both MTR gates close.

Eicher (1987) performed a survey of vascular plants within the salt marshes of Humboldt Bay and related the distribution of commonly found species and marsh communities to tidal elevation in Humboldt Bay. Using tidal data from the North Spit, the salt marsh plant species and communities identified by Eicher (1987) can be plotted by amount of time, on an annual basis, that the ground elevation where they are present is flooded by the tide (Table 5-3).

Mudflats and tidal channels are inundated over 19 percent of the time and no salt marsh species are present at these low elevations. Sarcocornia dominated marshes are inundated between 5 and 19 percent of the time. Sarcocornia dominated marshes are characterized with the presence of only four other species. Spartina dominated marshes, at a slightly higher elevation, is inundated between approximately 3 and 5 percent of the time and up to 10 other marsh species are present, though Spartina dominates. Mixed marshes, inundated less than 3 percent of the time, have the greatest species diversity with the presence of 22 species, with no individual species dominating. Elevations inundated less than 0.2 percent of the time are characterized by freshwater plant species. Sarcocornia is present in the Mixed marshes, but not present in the Spartina dominated marshes. Eicher (1987) speculated that the invasive Spartina out-competes Sarcocornia, resulting in a gap in its representation at middle elevations.



Based on Vegetation-Elevation Data from Eicher (1987) and NOAA Tide Records from North Spit (1993-2010). (Adapted from C. Shea, USFWS, personal communication).

Table 5-3. Salt marsh plant species and communities identified by Eicher (1987) plotted by the amount of time per year they are inundated by tidal fluctuations.

The inundation frequency and elevation of specific salt marsh plant species and marsh types identified by Eicher (1987) can be used during design to predict ground elevations where specific types of salt marsh species can be expected to occur. Table 5-4 indicates that salt marsh plants in Humboldt Bay are found between approximately 5.5 feet and 8 feet where non-muted, natural tidal fluctuations occur. Sarcocornia dominated marsh can be found between approximately elevations of 5.5 feet to 6.5 feet, Spartina dominated marshes between 6.5 feet and 7 feet, and Mixed Marsh between 7 feet and 8 feet.

The relationship between ground elevation, inundation, and salt marsh species can be used to predict where salt marsh species will be expected to occur under muted tidal conditions in Martin Slough. Table 5-4 shows the results of the Scenario 7 (one year of flows and tides) modeling in the Martin Slough Mainstream near the outlet of Tidal Marsh Complex C. The muted tide in Martin Slough mimics the inundation frequencies of the natural tide in Humboldt Bay, but at a lower elevation. In Martin Slough, Sarcocornia dominated marshes are expected to occur between approximate elevations of 4.8 and feet to 5.5 feet, Spartina dominated marshes between 5.5 feet and 5.7 feet, and a mixed marsh between 5.7 feet and 7 feet. The range in elevations where Spartina dominates is narrow in Martin Slough and may reduce the potential for this invasive species to become well established in the project area.

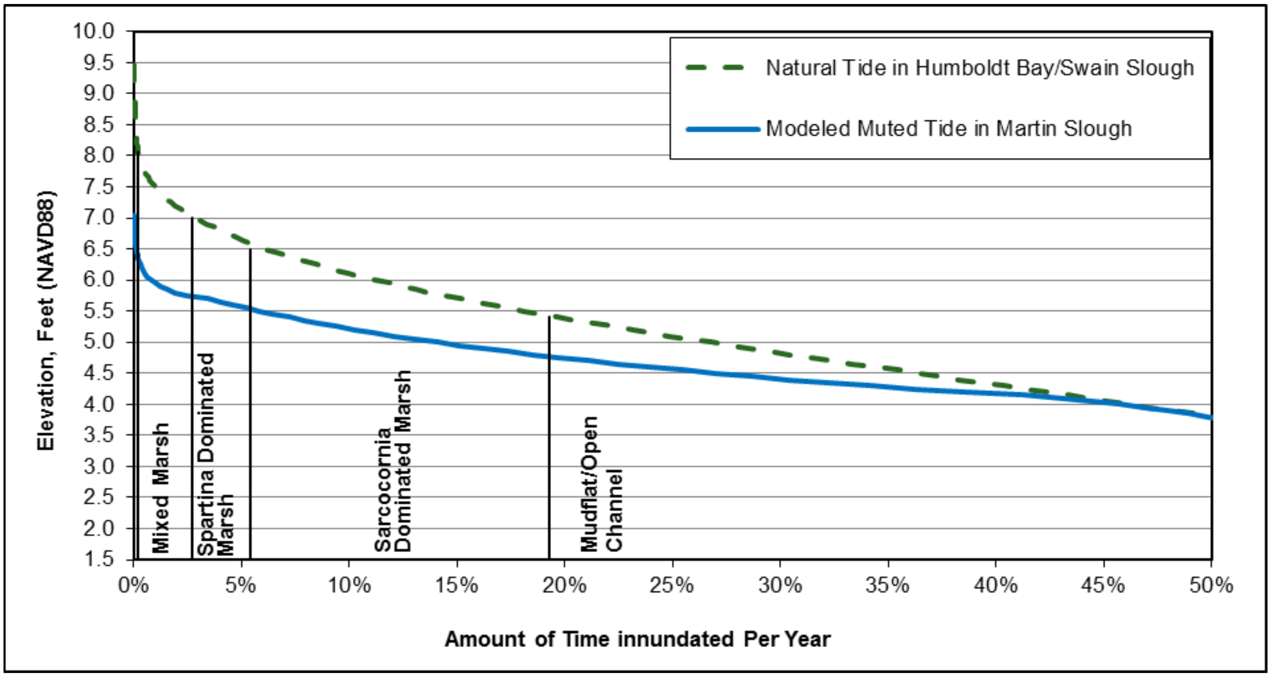


Table 5-4. Inundation frequency and elevation of specific salt marsh plant species and marsh types identified by Eicher (1987) that occur under natural tidal conditions of Humboldt Bay and can be expected to occur under the muted tidal conditions of Martin Slough.

5.2.2 Tidal Marsh Plains A and B

Approximately 600 feet of tidal marsh plain will be constructed along the south bank of the tidal channel reach downstream of the historical meander, and 1,400 feet along the north bank of the historical meander on the NRLT property. The design marsh plain will range in elevation from 4.8 to 6.0 feet, with varying elevations both in cross section and along the channel length. The design grading will create small drainage areas and concentrate flow to form small 1st order tidal channels. This range in elevations is expected to support a variety of salt marsh plant species. As shown in Table 5-4, elevations below 4.5 feet in Martin Slough are not expected to support salt marsh vegetation and will be open channel or mudflat. Elevations between 4.8 and 6 are expected to support marsh communities including Sarcocornia dominated marsh and mixed marsh communities. Brackish and freshwater vegetation is expected to grow at the back of the marsh plain.

5.2.3 Tidal Marsh Complex C (Pond C)

Approximately 1.4 acres of marsh plain will be constructed adjacent to the tidal slough channel in Pond C. The marsh plain will vary in elevation ranging from 4.8 feet to 6.0 feet with gentle slopes of 1% to 2% slopes to allow drainage towards the main channel and minimize salt panne formation (Zedler, 1984; Eicher, 1987). The range of marsh plain and upland elevations were designed to support a full suite of low to high salt marsh vegetation with freshwater vegetation on the higher elevations (Table 5-4). Several “fingers” of higher ground will project into the marsh plain, where freshwater species will grow close to the Pond C slough channel.

5.2.4 Tidal Pond Outfall Design

The historical meander and Ponds D, E, F, and G will be connected to Martin Slough through an elevated pond inlet/outlet channel, referred to as the outfall. The upstream and downstream ends of the meander will be controlled by the invert elevation of replacement reinforced concrete pipes (RCPs). The pond outfalls at Ponds E, F, and G were designed as broad crested earthen weirs that will be at a higher elevation than the upstream pond bottom and downstream channel. Pond D will have a log weir controlling the pond outfall elevation and backwatering the gas line protection concrete matting immediately upstream.

The proposed pond outfall elevations are provided in Table 5-5. The RCP invert elevations in the historical meander are designed to match the existing RCP inverts to prevent channel incision in the meander due to the presence of the two gas line crossings. Pond outfall elevations in Ponds D, E, F, and G were established with the objective of limiting saltwater intrusion while keeping the pond hydraulically connected to the channel under most tidal conditions. The design elevations will ensure frequent backwatering by the tides, which will allow aquatic organism ingress and egress and ensure frequent water exchange and flushing between the pond and main channel. Additionally, each pond outfall was set at a different elevation to create a diversity of off-channel conditions and habitats.

The elevations of pond outfalls were also established to minimize entry of bedload sediments from the main channel into the ponds. Some accretion of fine material will occur from smaller grained sediments suspended within the water column during flood events. However, a substantial volume of the water in the ponds will be flushed twice daily by tidal action, decreasing the amount of time for settlement of smaller silts and colloidal particles.

Table 5-5. Summary of pond outfall elevations for the tidal wetlands within the Martin Slough project area.

Pond	Outfall Crest Elevation (NAVD 88)
A	NA
B	NA
Historical Meander	2.5 feet (Downstream)
	3.0 feet (Upstream)
Southeast Tributary	6.7 feet
C	NA
D	4.25 feet
E	3.0 feet (Downstream)
	4.25 feet (Upstream)
F	3.0 feet
G	3.8 feet

5.3 Flood Conditions

HEC-RAS modeling Scenarios 1 through 6 were used to evaluate existing and design condition 2-, 10-, and 100-year flood elevations and the amount of time that the floodplain and golf course will be inundated during a flood event. The results of the peak 100-year flood elevations and velocities can also be used by the golf course to establish the minimum bottom elevation for new cart path bridges. The existing condition model results are for the pre-project tide gates. Note that these results are based on assuming Swain Slough is fully tidal and neglects the potential for elevated water levels in the slough resulting from high flows in Elk River at the confluence with Swain Slough.

5.3.1 Flood Elevations

Table 5-6 presents existing (E) and design condition (N) peak 2-, 10- and 100-year water surface elevations plotted along the (N) channel alignment. The propose project results in lower peak 2-, 10- and 100-year water surface elevations. The drop in water surface elevations is a combination of the lower design channel bottom, larger channel cross sectional area, increased outflow capacity of the new tide gates, and instead flood storage in the ponds. Proposed conditions also reduces the magnitude of backwatering that occurs at the Lower Fairway Drive Bridge during (E) conditions. Appendix B provides a summary of HEC-RAS modeling results of peak flow elevations for the flow events assessed.

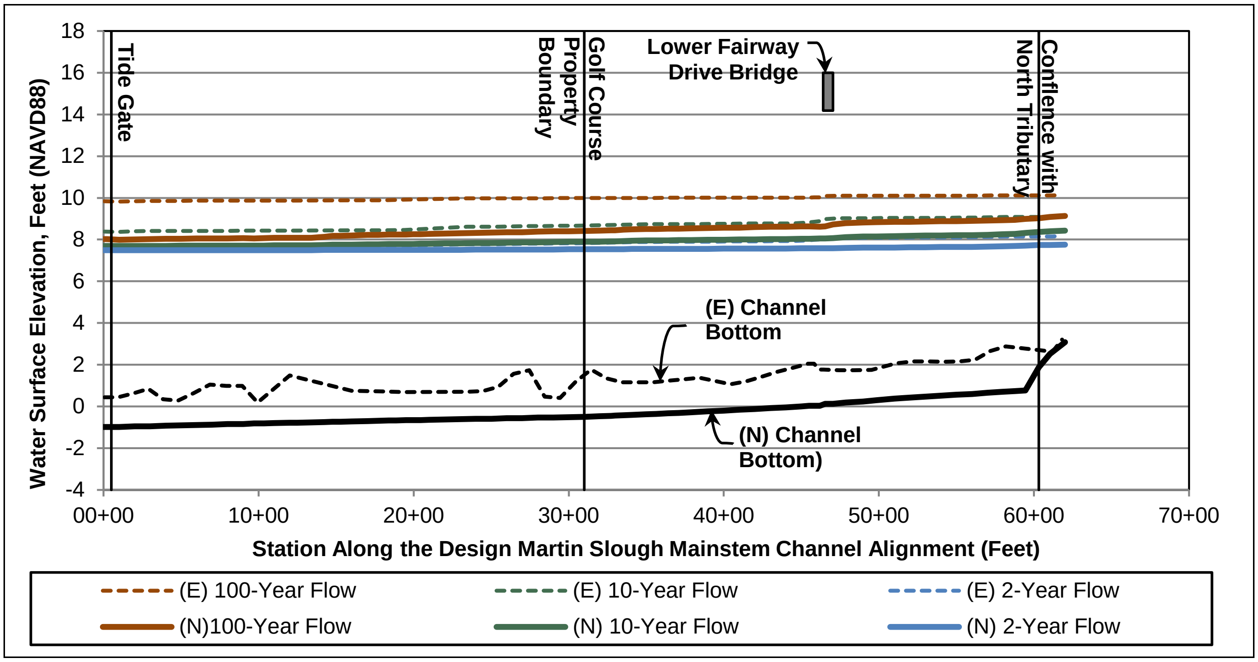


Table 5-6. The 2, 10, and 100-year water surface profiles and channel profiles in Martin Slough for existing (E) and new (N) design conditions. Stationing is along the (N) channel alignment.

5.3.2 Flood Flow Velocities

Peak flow velocities in the channel during baseflow and the 2-, 10-, and 100-year storm events were evaluated to assess channel stability around utility crossings and at the constricted channel near the Barn on the NRLT property. Additionally, baseflow and 2-year peak velocities were assessed at each of the pond outfall sills to evaluate the long-term stability of the sill. Table 2-3 summarizes peak flow velocities at various locations within the project area. The model indicates that peak velocities in the main channels occur typically during outflowing tides. Peak flow velocities are expected to be fairly similar during both inflow and outflow over the pond sills, except for Pond D, which will have a series of log weirs forming a steep connecting channel that will experience the highest velocities during outflows from the pond.

In locations where gas lines cross the proposed channels and ponds, the channel bottom and streambanks will be protected with an articulated concrete mat (Contech Armorflex Mat or equivalent). Permissible velocities of over 15 feet per second are listed by the manufacturer. The peak velocity that is expected to occur over a mat is 2.8 feet per second, which would occur doing a 100-year flow event (Table 2-3). Because the expected peak velocities are much less than the manufacturer-recommended permissible velocity, it is expected the articulated mats will fully protect the gas line from scour and any channel erosion.

The channel constriction created by the sheet pile walls near the NRLT barn will only cause negligible increases in water velocities compared to the adjacent channel. Therefore, it is expected that the channel will remain stable in the reaches containing sheet pile.

Table 5-7. Peak flow velocities in feet per second (fps) for the summer baseflow, 2, 10, and 100-year flow events at various locations in the Martin Slough Project area. Negative values represent inflows into the ponds.

Location	Summer Baseflow (Scenario 7)	2-Year (Scenario 8)	10-Year (Scenario 5)	100-Year (Scenario 6)
Martin Slough 9+69 (6 inch Gas Line Crossing)	1.4 fps	2.0 fps	2.6 fps	2.8 fps
Martin Slough 12+49 (Downstream of Sheet Pile)	1.3 fps	1.9 fps	2.5 fps	2.6 fps
Martin Slough 13+39 (Sheet Pile at Barn)	1.4 fps	2.1 fps	2.7 fps	3.2 fps
Martin Slough 14+25 (Sheet Pile at Barn)	1.4 fps	2.1 fps	2.7 fps	3.2 fps
Martin Slough 14+76 (Upstream of Sheet Pile)	1.3 fps	1.9 fps	2.5 fps	2.6 fps
Meander 6+50 (12 inch Gas Line Crossing)	0.3 fps	0.4 fps	0.7 fps	0.5 fps
Meander 18+76 (Gas Line Crossing)	0.6 fps	0.7 fps	0.8 fps	0.9 fps
Pond C Outfall	-0.8 fps	-0.8 fps	-	-
Pond D Outfall Log Weir	2.5 fps	2.1 fps	-	-
Pond E Outfall Sill	-1.8 fps	-1.9 fps	-	-
Pond F Outfall Sill	-2.2 fps	-2.3 fps	-	-
Pond G Outfall Sill	0.9 fps	-0.9 fps	-	-

To evaluate the stability of the pond sills, a stability analysis was prepared using the allowable velocity method in USDA (2007). The allowable velocity method is used to evaluate whether the bed and banks of an earthen channel without vegetative stabilization will erode or remain stable during a given flow event. The analysis is based on relative channel sediment load, the plasticity index and void ratio of the materials forming the channel, and planform geometry. The allowable velocity method recommends that to maintain a stable channel, water velocities for the design storm flow do not exceed the computed allowable velocities.

The pond sills will be constructed of compacted cohesive soils salvaged from the project area. The project geotechnical report indicated that three locations within the project area contain layers approximately 3-5 feet thick of soils with moderate plasticity indices within the limit of excavation (SHN, 2013, Appendix A). Soil boring MS-5 contains a clayey sand (SC), HB-9 contains a high plasticity silt (MH), and HB-12 contains a silt (ML). Allowable velocities for all three of these materials exceed 3.4 feet per second, assuming the more conservative sediment free flow condition (Appendix D). Therefore, because the actual velocities expected across the sills are less than the allowable velocities, it can be expected that pond sills constructed using these materials will remain stable.

It is unknown if sufficient quantities of suitable materials will available for the pond sill construction. The project geotechnical report indicates that the majority of the excavated soils in the project area will be overly wet, with a moisture content over optimum for compaction and are slow to dry. If insufficient suitable material can be salvaged for the sills, then imported material or use of large wood may be needed to stabilize the sills.

5.3.3 Duration of Overbank Flooding

Currently, the golf course has numerous low areas on the overbank floodplain that do not drain after high-flow events. Additionally, the original tide gate, replaced in 2014, had very limited outflow capacity that limited the rate that floodwaters could drain out of Martin Slough, resulting in long durations of floodplain inundation.

As part of design conditions, the low areas within the golf course that hold standing water will be raised to an approximate elevation of 7.0 feet and sloped to drain toward the channel. Additionally, the new tide gates provide nearly three times more conveyance area, allowing floodwaters to drain unimpeded by the tide gate structure.

Table 5-8 presents the amount of time that 2, 10 and 100-year flows are above 7.0 feet in elevation for existing and design conditions. The amount of time that the golf course will be inundated will substantially decrease with design conditions for the three flow events assessed.

5.4 Sediment Transport Competence

Sediment transport competence in Martin Slough and the North Fork Tributary was assessed for both existing and design conditions using peak shear stresses for the flows generated from the 2-year, 24-hour precipitation event (Scenarios 1 and 4, Section 4.2.3). Peak shear stresses occur on outgoing flows. Transport competence during winter baseflow was also assessed on the Martin Slough mainstem for design conditions using results from HEC-RAS modeling Scenario 7. Estimation of sediment transport capacity, or volume of sediment transported, was not assessed because there is no information regarding total sediment load for Martin Slough nor the North Fork Tributary.

Sediment transport competency is a measurement of a flow’s ability to mobilize a given size sediment particle and is typically evaluated by comparing shear stress from the flow through a channel with the critical shear stress, or entrainment shear stress for the particle. If the shear stress is greater than the critical shear stress of the particle, the flow has the competence to move a particle of that size. Channel shear stress is a function of the channel hydraulic radius and slope of the energy grade line, and can be obtained from HEC-RAS results. The entrainment shear stress for a given particle can be computed using the Shields Equation and a value of critical dimensionless shear stress.

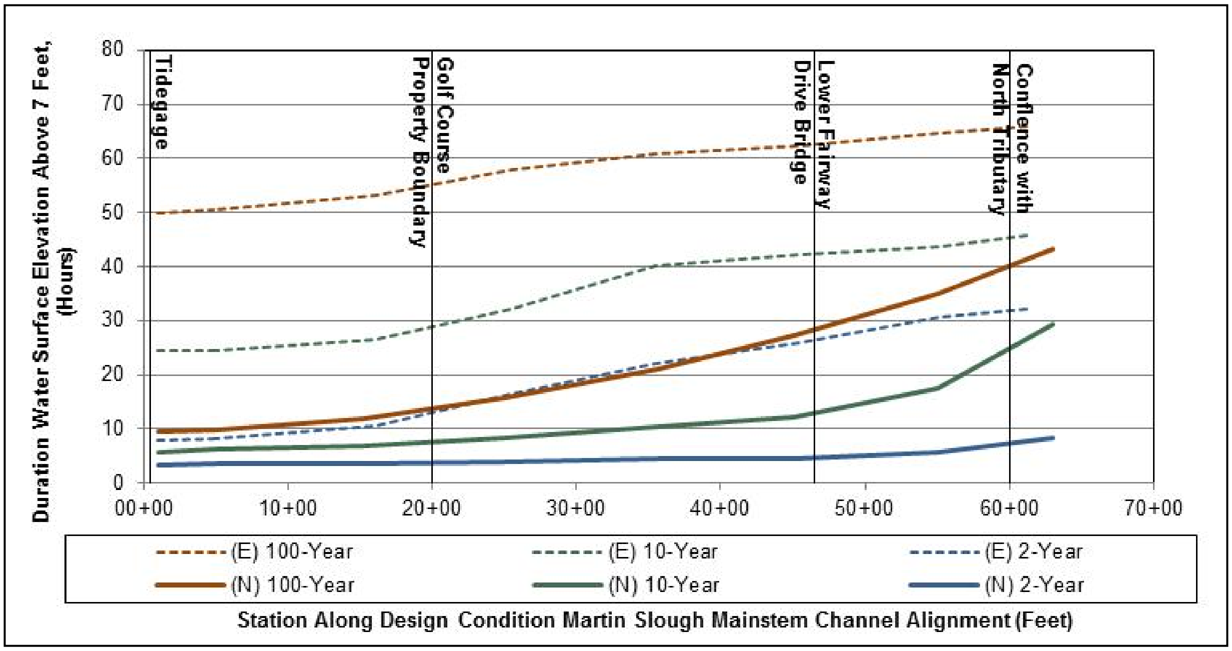


Table 5-8. Duration that flood flow are above elevation 7.0 feet (NAVD88) for existing (E) and design conditions (N) along the mainstem Martin Slough.

Critical dimensionless shear stress is a function of grain size, angle of repose of the grain, kinematic viscosity of the water. A grab sample of channel sediment in the Martin Slough mainstem channel indicates that the channel sediment consists of sands, silts and clays (W-K et al., 2006). Sand sizes ranged from coarse silt (0.05 mm) to medium sand (0.5 millimeters). Critical dimensionless shear stress values were computed using Julien (1998) for grain sizes ranging from coarse clay to medium sand. These values were used to compute critical shear stress for each particle size (Table 5-9).

Table 5-9. Critical shear stresses for grain sizes in the Martin Slough streambed.

Grain Size Category	Grain Size (mm)	Critical Shear Stress (psf)
Coarse Clay	0.003	0.0004
Very Fine Silt	0.006	0.0007
Fine Silt	0.012	0.0014
Medium Silt	0.024	0.0021
Coarse Silt	0.05	0.0028
Very Fine Sand	0.09	0.0037
Fine Sand	0.2	0.0049
Medium Sand	0.5	0.0072

5.4.1 Martin Slough Mainstem Channel

In the Martin Slough mainstem channel upstream of the NRLT Property, peak shear stresses during a 2-year 24-hour flow event are increased substantially from existing conditions (Table 5-10). This is a result of the increased channel capacity and improved flood conveyance throughout the project area, which results in a steeper water surface slope. Although peak channel shear stresses are lower under design conditions than existing within some reaches on the NRLT Property, the 2-year peak shear stresses continue to have the competence to transport medium sands through the project area to the tide gate. Medium sands are the largest particle sizes found in the streambed, therefore, long-term deposition is not expected to occur in the design condition Martin Slough mainstem channel. During winter baseflow conditions, the Martin Slough mainstem also has the competence to transport particle sizes up to medium sands.

5.4.2 North Fork Tributary

The flow competence of the North Fork Tributary under design conditions is substantially improved from existing conditions for the 2-year flow event both within and upstream of the project area (Table 5-11). The 2-year shear stresses have the competence to move medium sands through all but the most upstream reaches of the project area. Medium sands are the largest particle sizes found in a grab sample in the mainstem of Martin Slough. The gradation of sediment in the North Fork Tributary is unknown, but is similar or finer. Therefore, it can be expected that sediment delivered to the project area will be transported through the North Fork and into the mainstem.

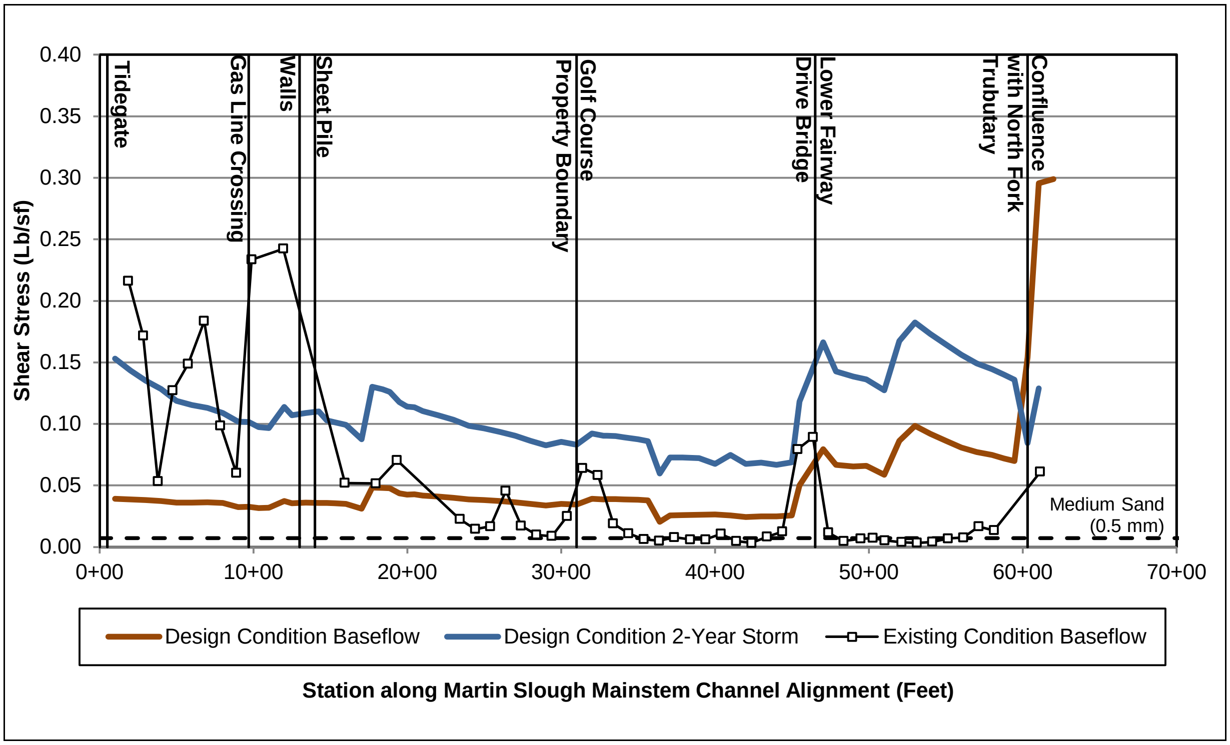


Table 5-10. Computed peak shear stresses along the Martin Slough Mainstream for existing and design conditions.

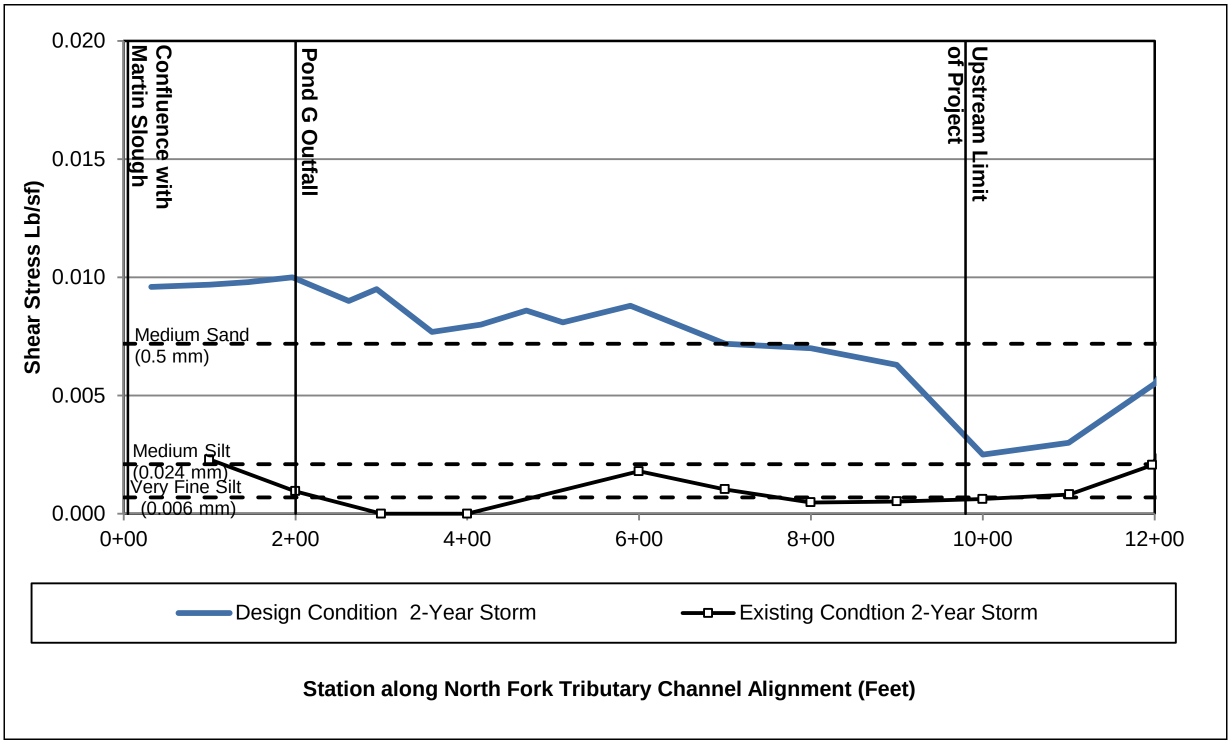


Table 5-11. Computed peak shear stresses along North Fork Martin Slough for existing and design conditions.

5.5 Design Condition Salinities

Salinity concentrations within the proposed project with varying tidal and freshwater inflow conditions were evaluated using HEC-RAS Scenario 9 (Section 4.2.3). The results of this model reflect salinity conditions in Martin Slough during the wet season and as the rains end and baseflow recedes in the late spring and summer. The model computations are based on mass-balance flow mixing, and does not compute horizontal freshwater/saltwater stratification. Stratification is expected to occur during low-flow periods, which results in a layer of freshwater on top of the water column.

Table 5-12 through Table 5-14 present the results of the salinity modeling along select mainstem cross sections, the North Fork Tributary, Southeast Tributary, historical meander, and the ponds. As expected, the salinity modeling indicated that salinity concentrations fluctuate with the tide and with freshwater inflows. Salinity increases in the downstream direction, with rising tides, and with decreasing freshwater inflows. Conversely, salinity decreases during freshwater inflow events and when the tide is falling.

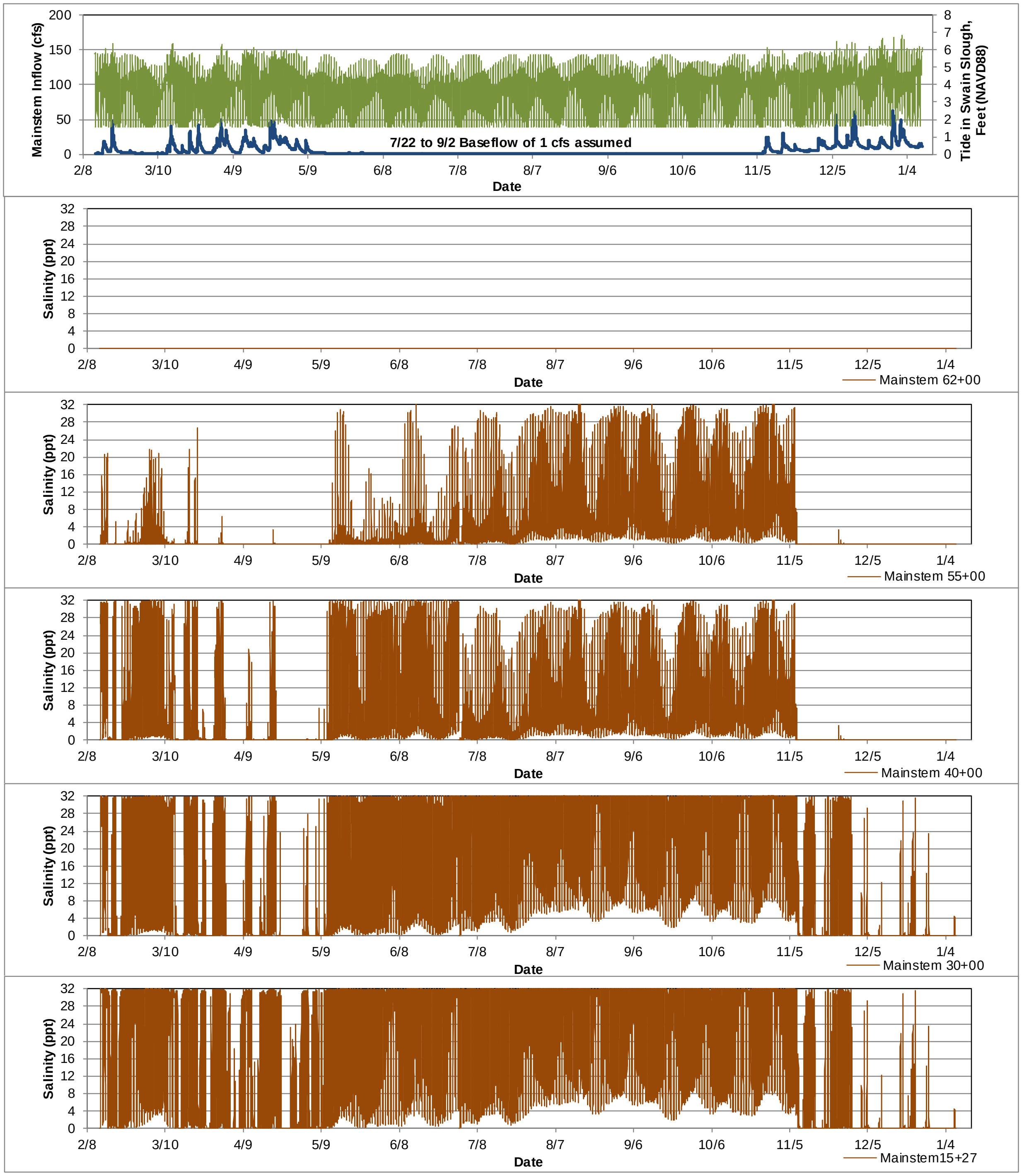


Table 5-12. Predicted salinity concentrations at various locations along the Martin Slough mainstem. Tidal elevations and freshwater inflows are shown in the top plot.

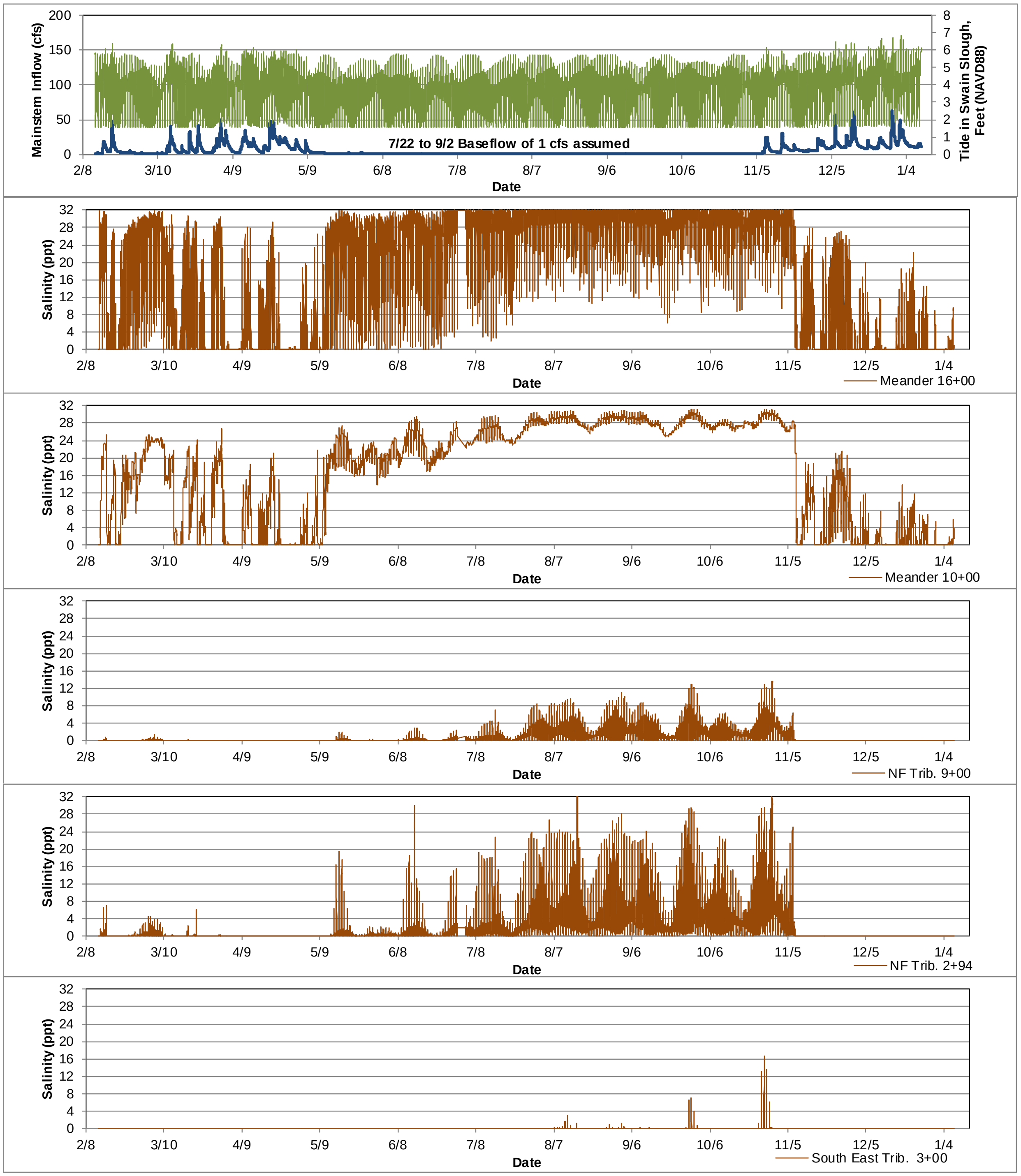


Table 5-13. Predicted salinity concentrations in the North Fork Tributary, Southeast Tributary, and the Meander. Tidal elevations and freshwater inflows are shown in the top plot.

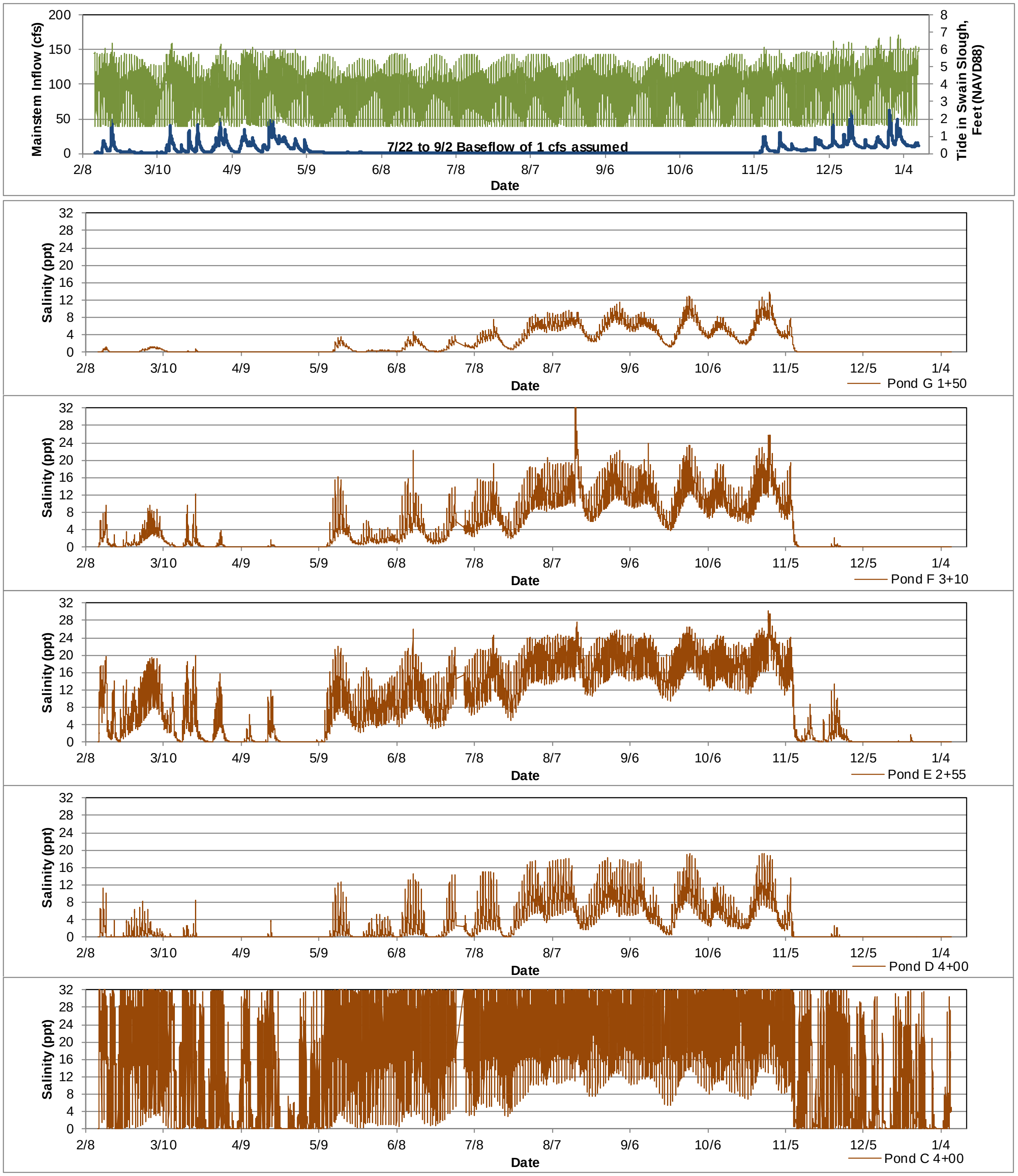


Table 5-14. Predicted salinity concentrations in each of the Ponds in the Martin Slough project. Tidal elevations and freshwater inflows are shown in the top plot.

5.5.1 Wet Season

Table 5-15 presents a graphical representation of predicted depth-averaged salinity concentrations within Martin Slough at a high tide between rain events in December, which represents average wet season baseflow conditions. Inflow into the upstream end of the Martin Slough Mainstem is approximately 8 cfs. This figure represents the predicted extent and concentrations of salinity into Martin Slough between small rain events. During rain events, salinity concentrations throughout the entire project area will decrease substantially and often effectively becoming a freshwater environment extending into Swain Slough (Table 5-12 through Table 5-14).

Through the wet season, depth-averaged salinity concentrations greater than 8 ppt are predicted to extend upstream in the Martin Slough mainstem to Pond E during high tides. Pond E may become slightly brackish during high tides, but a freshwater lens is expected to remain on the pond surface. The mainstem, North Fork Tributary, and Ponds F and G are expected to remain fresh throughout the wet season during average rainfall. The Southeast Tributary and Pond D are also expected to remain fresh during the wet season.

The meander will experience fluctuating salinities due the freshwater inflow from the Southeast Tributary and brackish inflows at the downstream end of the meander bend. It is expected that a persistent lens freshwater will remain on the surface of the meander water surface due to freshwater inflows from the Southeast Tributary.

Pond C and the Martin Slough channel downstream of Pond C are expected to experience fluctuating freshwater and saline conditions, dependent on both the tide and freshwater inflows.

Note that Swain Slough was modeled assuming fully saline conditions. However, salinity measurements in Swain Slough indicate that the slough experiences salinities ranging from 0.1 ppt in the winter months to 34 ppt in early summer (Wallace, 2015). Therefore, actual salinities in Martin slough may be lower than predicted during the wet season because incoming flows from Swain Slough have a lower salinity concentration than in the marine environment.

5.5.2 Dry Season

Table 5-16 presents a graphical representation of predicted depth-averaged salinities within Martin Slough reflecting a high tide in early September, which represents late summer dry-season conditions when freshwater baseflow is lowest. This figure represents the expected furthest influx and highest concentrations of salinity into Martin Slough during the dry season. During falling tides, salinity concentrations throughout the project area will be decrease slightly (Table 5-12 through Table 5-14).

During the dry season fully saline conditions are expected to extend upstream in the Martin Slough mainstem to the confluence with the North Fork Tributary. Pond C is expected to remain fully saline. Pond D is expected to become brackish a freshwater inflows decrease through the dry season. Ponds E and F are also expected to remain brackish and stratified. Pond G is predicted to remain brackish to fresh.

The proposed transition channel with log steps on the Martin Slough mainstem upstream of the confluence with the North Fork Tributary prevents brackish water from extending upstream. Similarly, the upper reaches of the proposed channel and pond on the Southeast Tributary will remain fresh through the dry season.

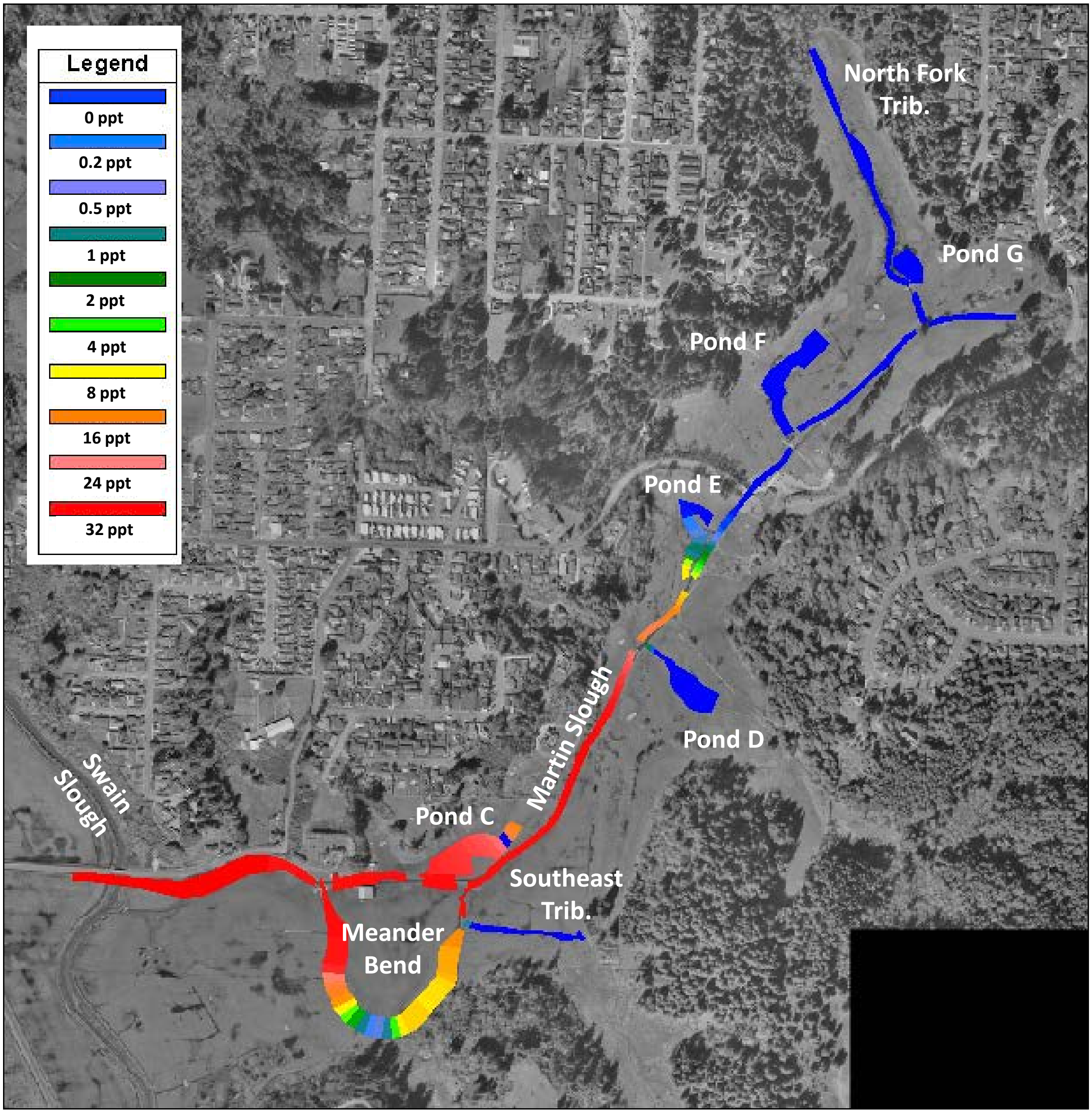


Table 5-15. Modeled salinity in Martin Slough at a high tide between rain events in December (December 23, 2003 09:00), which represents average low-flow conditions during the wet season. Inflow into Martin Slough mainstem is approximately 8 cfs.

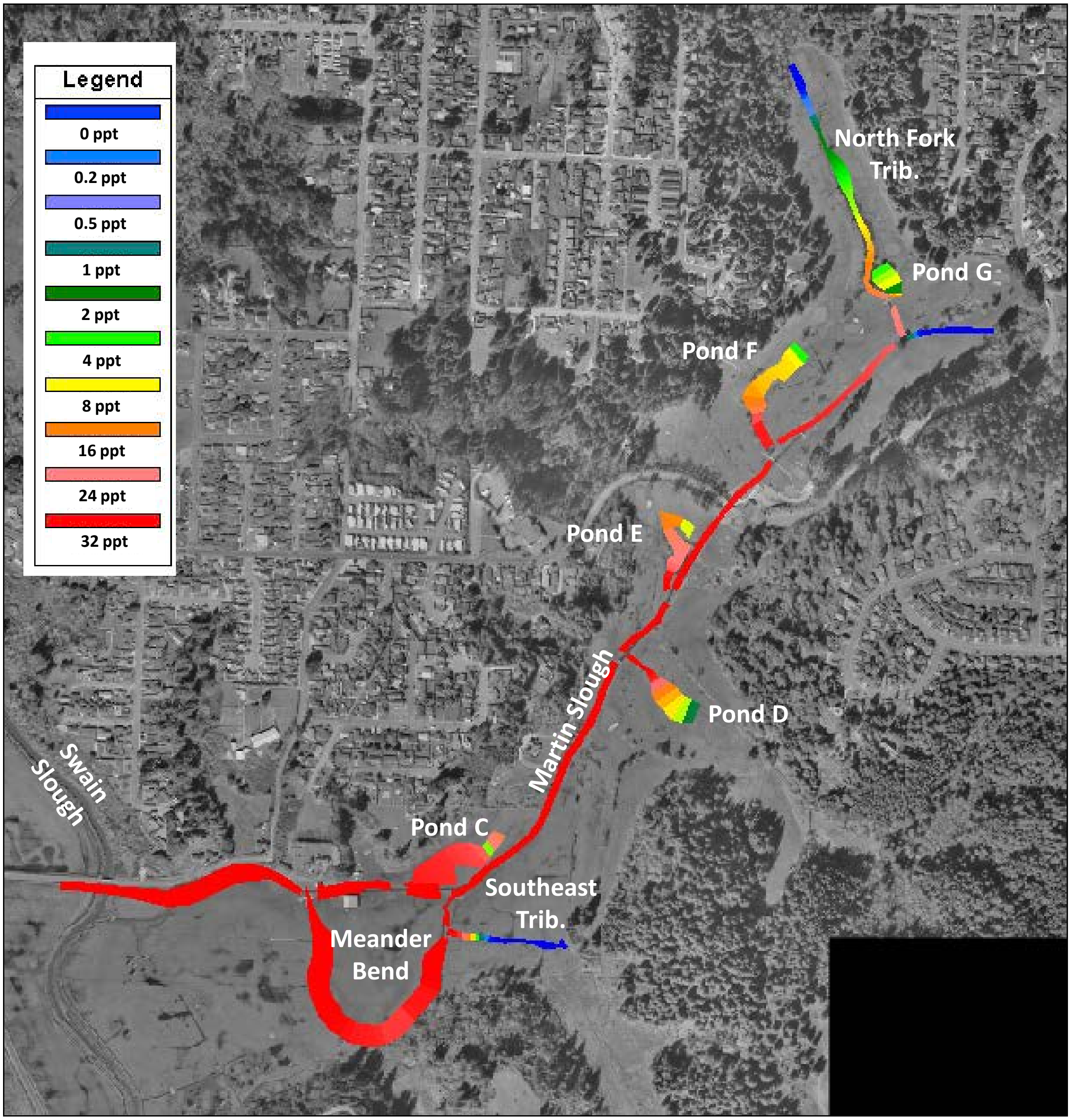


Table 5-16. Modeled salinity in Martin Slough at a high tide in September (September 05, 2003 02:15), which represents late summer dry-season conditions when baseflows have receded. Total inflow into the Martin Slough project area is a constant 1 cfs.

6.0 Construction

6.1.1 Sediment Reuse

The construction of new ponds and expanded channels will result in a large volume of excavated material. It is beneficial to the project and environment to reuse as much sediment on-site as feasible. Primary reuse areas would be to repair, maintain, and raise existing dikes, and for beneficial use on land adjacent to the channel and ponds. The excavated material will include some topsoil, some silts and sands, and some clay materials. In some locations, such as deeper excavations and excavations nearer to Swain Slough, we expect to see excess soluble salts (salinity) which would not be desirable to spread on land used for agricultural uses or other uses where non-salt-marsh vegetation is desired. Clay materials excavated on the site would be reused for the pond outfall sills. Any excess clay, and soil with excess soluble salts, may be reused in the dike repairs. Excess sediment that is not reused on-site, may be hauled to off-site reuse areas such as White Slough on the Humboldt Bay Wildlife Refuge, or other permitted sites.

6.1.2 Fill for Beneficial Purposes on Adjacent Land

Some of the excavated material is suitable for spreading within the project area adjacent to the channel and ponds on existing agricultural lands, the golf course, or upland vegetation areas. Application of the excavated sediments on adjacent lands will be similar to natural floodplain depositional processes. The primary concern with reuse of the materials excavated from the project area is the presence of excess soluble salts (salinity), which can inhibit plant growth. Testing of soils within the project would be conducted to evaluate which excavated materials have excess soluble salts or are essentially non-saline.

Fill for beneficial reuse on adjacent land will generally be spread in a thin (approximately 3- to 4-inch thick) uncompacted lift on unprepared surfaces to minimize detrimental effects to existing vegetation and overland drainage patterns. Thicker application of materials may occur in locations identified as low spots that are not wetlands, in wetlands where fill would not change the wetland classification, or in areas where drainage improvements are desired to help reduce the potential for stranding of salmonids during overbank events and to improve other beneficial uses. Compaction efforts on thin fill areas are not proposed.

Depending upon the location of the reuse area, the material will be transported to the adjacent land reuse site likely in either a lightweight off-highway dump truck or highway legal belly-dump or end-dump truck. To improve construction efficiency, the material could be placed directly onto the agricultural lands and then spread with a grader, bulldozer or loader. Alternatively, soil could be placed initially in windrows within the pasture and spread later in the growing season during a time compatible with the landowner's operations and grazing rotation. Depending upon the crop and grazing rotation specific to the landowner's operation and the time during which the material is delivered, the spreading of the material will occur within an 18-month period from when the material is delivered and consistent with the crop agronomy and operational use of the land. Temporary erosion control Best Management Practices (BMPs) including seeding, mulching, and perimeter control practices will be applied to the windrows to minimize wind and rain-induced erosion prior to spreading. These BMPs will be maintained until the windrowed material is spread.

6.1.3 Construction Techniques and Temporary Disturbance

The primary excavation methods that will likely be utilized include track-mounted excavators, and bull-dozers. Excavated material will be loaded into either off-highway dump trucks, belly-dump trucks, or end-dump trucks and hauled to the reuse areas or offsite as necessary. It will be the responsibility of the contractor to ensure the haul trucks are street legal and that local speed and weight limits are obeyed. The Contractor will also be responsible for developing and submitting for review by the Construction Manager a Traffic Control Plan prior to construction commencement. Hauling the excavated material from the project area to reuse sites will require multiple dump trucks operating continuously during the excavation activities. Table 3 shows the range of project construction equipment estimates for any given construction season.

Temporary construction areas will be needed to stage equipment, store material and transport material. Temporary construction areas will be located within locations already identified as permanent impact areas such as excavation areas or areas within close proximity as depicted on the 65% Design Plans. Temporary construction activities outside permanent impact areas will be limited to temporary construction buffers, haul routes, material and equipment staging/stockpiling areas, and temporary egress/ingress areas adjoining City and County Roads and as shown on the 65% Design Plans. Locations identified as temporary construction areas will be restored to pre-construction conditions once construction is complete. Temporary haul roads and other high traffic areas will be de-compacted and restored back to pre-construction soil densities. Restoration of temporary construction disturbance areas will be specified in the final design drawings and specifications.

Table 6-1. Estimates of equipment necessary for project construction

Equipment Type	Estimated Quantity
Excavators	1-5
Dozers	1-5
Loaders	2-4
Dump Trucks	2-10
Small Tractors	1-3
Compactors	1-3
Graders	1-2
Water Trucks	1-3
Small Crane	1

6.1.4 Temporary Haul Roads

The construction of temporary haul roads will be required to transport excavated materials from the channel corridor to other on-site reuse areas, or to City, County, and State Roads depending upon the final reuse areas. Haul roads will also provide stable working and staging areas for excavation and loading activities. Haul road construction will depend on subgrade suitability, the size of the transport equipment to be used, the intensity of use, excavation/reuse locations, and identification of sensitive habitats and species. Temporary haul road construction could include proof-rolling native subgrade to provide a non-yielding surface or placement of crushed rock or river-run gravel over woven or non-woven geotextile fabric. Locations of anticipated temporary haul roads will be within the limits of temporary construction disturbance as depicted on the 65% Design Plans. Should regulatory restrictions or soft subgrade conditions require additional, or more robust, temporary haul roads, the geotechnical report provides additional information and options for temporary haul roads in the project area.

6.1.5 Construction Erosion and Sediment Control BMPs

Prior to project construction, a Storm Water Pollution Prevention Plan (SWPPP) will be developed and submitted to the North Coast Regional Water Quality Control Board (RWQCB) and implemented during construction. As part of the SWPPP, Best Management Practices (BMPs) for controlling soil erosion and the discharge of construction-related contaminants will be developed and monitored for successful implementation. BMPs that would be implemented as part of the SWPPP would likely include:

- Coffer dams or other temporary fish barriers/water control structures will be placed in the channel during low tide, and will only be removed during low tide (if possible), after work is completed.
- Because coffer dams will be installed and the channel will be dewatered prior to excavation, equipment will not be operated directly within tidal waters or stream channels of dewatered streams after fish removal efforts have been completed.
- Silt fences and or silt curtains will be deployed in the vicinity of the coffer dams, at excavations of sloughs, and at culvert installation and removal areas to prevent any sediment from flowing into the creek or wetted channels. If the silt fences are not adequately containing sediment, construction activity will cease until remedial measures are implemented that prevents sediment from entering the waters below.
- Sediment sources will be controlled using fiber rolls, sediment basins, and/or check dams that will be installed prior to or during grading activities and removed once the site has stabilized.
- Erosion control may include seeding, mulching, erosion control blankets, plastic coverings, and geotextiles that will be implemented after completion of construction activities.
- Excess water will be pumped into the surrounding fields to prevent sediment-laden water from entering the stream channel. When internal sloughs are connected to the mainstem Martin Slough, excavation will occur during a rising tide so that water flows into the marsh and sediment has a chance to settle out, allowing impacts of turbid water generated from excavations necessary for connection of the sloughs to the mainstem to be minimized by settlement and dilution.
- Appropriate energy dissipation devices will be utilized to reduce or prevent erosion at discharge end of dewatering activity.
- Turbidity and pH monitoring will be conducted in Martin Slough throughout the site stabilization period to ensure that water quality is not being degraded. Turbid water will be contained and prevented from being transported in amounts that are deleterious to fish, or in amounts that could violate state pollution laws. Silt fences or water diversion structures will be used to contain sediment. If sediment is not being contained adequately, as determined by visual observation, the activity will cease.
- Construction materials, debris, and waste will not be placed or stored where it can enter into or be washed by rainfall into waters of the U.S./State.
- Upland areas will be used for equipment refueling. If equipment must be washed, washing will occur where wash water cannot flow into wetlands or waters of the U.S./State.
- Operators of heavy equipment, vehicles, and construction work will be instructed to avoid sensitive habitat areas. To ensure construction occurs in the designated areas and does not impact environmentally sensitive areas, the boundaries of the work area will be fenced or marked with flagging.
- Equipment when not in use will be stored outside of the slough channel and above high tide elevations.
- All construction equipment will be maintained to prevent leaks of fuels, lubricants or other fluids into the slough. Service and refueling procedures will not be conducted where there is potential for fuel spills to seep or wash into the slough.
- Extreme caution will be used when handling and/or storing chemicals and hazardous wastes (e.g., fuel and hydraulic fluid) near waterways, and any and all applicable laws and regulations will be followed. Appropriate materials will be on site to prevent and manage spills.
- All trash and waste items generated by construction or crew activities will be properly contained and remove from the project area.

- After work is completed, project staff will be on site to ensure that the area is recontoured as per approved specifications. If necessary, restoration work (including revegetation and soil stabilization) will be performed in conformance with the Revegetation and SWWP plans.

6.1.6 Construction Dewatering and Phasing

During excavation within the channel and replacement of the tide gate, management of the stream flow from Martin Slough tributaries will be required through the construction period. Preventing inflow into the active work zones (both tidal and freshwater) will be required to prevent aquatic and non-aquatic organisms from entering the construction site, to reduce the water to be managed in the active work area, and to reduce moisture content in the excavated soils. Inflow control practices include placement of temporary cofferdams to isolate active work zone. The cofferdams may be comprised of native material, washed gravel encased with an impermeable geotextile or visqueen liner in combination with ecology blocks, water bladders, and/or sheetpiles. A combination of pumped and or gravity diversion pipes will be used to route flow around the active work areas. Fish screens will be installed immediately upstream from the cofferdams to prevent aquatic organisms from being transported into the bypass pipe as well as downstream of the bypass pipe outlet.

Ponded storm or groundwater in construction areas will not be dewatered by project contractors directly into adjacent surface waters or to areas where they may flow to surface waters unless authorized by a permit from the North Coast RWQCB. In the absence of a discharge permit, ponded water (or other water removed for construction purposes), will be pumped into adjoining fields to infiltrate if suitable, baker tanks or other receptacles. If determined to be of suitable quality, some of this water may be used on-site for dust control purposes. The Contractor will be required to submit for review and approval by the Construction Manager a Dewatering and Creek Diversion Plan that shall include the proposed dewatering and diversion techniques and schedule of operations. Construction is anticipated to occur in three phases over three years. The following construction phases and associated dewatering and diversions activities are proposed to occur in the order presented below:

Phase 1: Pond C and the Lower Martin Slough Channel:

Phase 1 construction will be limited to the NRLT property. Coffer dams will be placed at the upstream and downstream end of the restoration area. Stream flow will be pumped, gravity piped or ditched and conveyed around the active work zone. Prior to placement of temporary coffer dams, a qualified biologist will utilize seines to corral fish out of the construction limits and into adjoining waters. In the event temporary coffer dams temporarily eliminate tidal exchange into the upper reaches of Martin Slough, a temporary gravity bypass pipe shall be implemented adjacent to the construction area to allow tidal flow exchange to sustain brackish conditions in the upstream reaches of Martin Slough during the construction period. If a gravity bypass pipe is not feasible, prior to tidal flow to the upstream reach, water temperature, pH and conductivity monitoring shall be conducted in hole 17 pond (Pond E) and where salmonids and tidewater gobies are known to persist. Adjustments to maintain appropriate water quality shall be made if necessary and by means of water pumped from Swain Slough.

Phase 2: Ponds D and E and the Martin Slough Channel downstream of Lower Fairway Drive

Phase 2 construction will occur on the golf course downstream of the Lower Fairway Drive bridge crossing. Coffer dams will be placed at the upstream and downstream end of the restoration area. Diverted flow will be pumped, gravity piped or ditched and conveyed downstream of the active work zone. Prior to placement of temporary coffer dams, a qualified biologist will utilize seines to corral fish out of the construction limits and into adjoining waters. Fish that cannot be corralled to areas outside of the construction limits will be captured and relocated.

Phase 3: Ponds F and G and the Martin Slough Channel upstream of Lower Fairway Drive

Phase 3 construction will occur on the golf course upstream of the Lower Fairway Drive bridge crossing. Prior to placement of temporary coffer dams, a qualified biologist will utilize seines to corral fish to areas out of the construction limits and into adjoining waters including the newly constructed Ponds C, D, and E. Fish that cannot be corralled to areas outside of the construction limits will be captured and relocated.

6.1.7 Revegetation

The 65% Design Plans include the planting areas and species densities for the project area. The goal is to create native, forested riparian, wetland and tidal marsh habitats along the Martin Slough channel and expanded ponds. The excavated reaches of Martin Slough and expanded ponds will be revegetated with low growing brackish and freshwater wetland (sedges and rushes) and riparian forest (Sitka spruce, willow, wax myrtle, and alder). Plant material, to the extent feasible, will be salvaged from the project impact footprint. All areas disturbed during grading and other construction activities will be treated with erosion control seeding with native grasses, forbs and shrubs. Active planting is currently proposed however natural recruitment of native plant species would be desirable to augment the active planting activities. Exclusion fencing will be constructed around the perimeter of the riparian areas to protect the plantings in the pasture. Fencing is not needed nor proposed on the golf course (City) property as no cattle are allowed on the City property.

Active vegetation maintenance will be regularly performed to ensure that the target riparian forest habitat develops along the riparian corridor areas. Options for limiting undesirable vegetation include intermittent controlled flash grazing (assumed to be by cattle, but could also be by goat or sheep), manual removal, and mechanical removal. Special attention will be given to non-native invasive species such as dense-flowered cordgrass, and maintenance activities will be coordinated with regional eradication programs, including both timing and methods for removal of specific species. If grazing is employed, exclusion fencing will be placed to protect channel banks, newly establishing revegetation plantings, and areas of naturally recruiting desirable native plants. Flash grazing may be carefully employed to control weed cover in active planting areas and natural recruitment areas but will be managed to avoid excessive damage to native plantings and recruits.

7.0 References

ACOE. 2010a. HEC-RAS River Analysis System Version 4.1.0. U.S. Army Corps of Engineers Hydrologic Engineering Center. Davis, California.

ACOE. 2010b. HEC-RAS River Analysis System User’s Manual Version 4.1. U.S. Army Corps of Engineers Hydrologic Engineering Center. Davis, California.

California Data Exchange Center (CDEC). 2011. Rainfall data for Woodley Island, Eureka, CA. http://cdec.water.ca.gov/.

CDFG. 2002. Culvert criteria for fish passage. Appendix A in California Salmonid Stream Habitat Restoration Manual 3rd edition. California Department of Fish and Game.

Chow. V. T. 1959. Open Chanel Hydraulics. McGraw Hill. 680 pp.

Coats, R.N., P.B. Williams, C.K. Cuffe, J.B. Zedler, and D. Reed. 1995. Design Guidelines for Tidal Channels in Coastal Wetlands. Prepared for U.S. Army Corps of Engineers, Waterways Experiment Station by Philip Williams & Associates, Ltd.

D’aoust, S. and R G. Millar. 2000. Stability of Ballasted Woody Debris Habitat Structures. Journal of Hydraulic Engineering, November, 2000.

Eicher, A. 1987. Salt Marsh Vascular Plant Distribution in Relation to Tidal Elevation, Humboldt Bay, California. M.A. Thesis, Humboldt State University.

Fischer, H.B. J.E. List, C.R. Koh J. Imberger, and N.H. Brooks. 1979. Mixing in inland and coastal waters. Elsevier, 302 pp.

GHD and Michael Love & Associates, 2013. Basis of Design Report: Martin Slough Enhancement Project, Eureka, CA. 30% Design Submittal. Prepared for the Natural Resources Division of Redwood Community Action Agency. 184 pp.

Julien, P.Y. 1998. Erosion and Sedimentation, Cambridge University Press, Cambridge, UK.

Kashefipour, S. M. and R. A. Falconer. 2002. Longitudinal dispersion coefficients in natural rivers. Water Research, 36: 1596-1608.

Michael Love & Associates. 2010. Technical Memorandum: Monitoring of Summer Low Flows in Martin Slough, Eureka CA. Prepared for Redwood Community Action Agency, Natural Resources Division.

NMFS. 2001. Guidelines for salmonid passage at stream crossings. NOAA Fisheries, NMFS SW Region.

NRCS. 2002. National Engineering Handbook: Part 630 Hydrology. USDA – Natural Resources Conservation Service. Last revised: 1992.

NRCS. 2007. Use of Large Woody Material for Habitat and Bank Protection. Technical Supplement J of the National Engineering Handbook.

Philip Williams and Associates, and P.M. Faber. 2004. Design Guidelines for Tidal Wetland Restoration in San Francisco Bay. Prepared for The Bay Institute and California State Coastal Conservancy by Philip Williams & Associates, Oakland, CA.

Ralston, D. K. an M. T. Stacey. 2005. Longitudinal dispersion and lateral circulation in the intertidal zone. Journal of Geophysical Research, 110: C07015.

SHN. 2013. Geologic Report Martin Slough Enhancement Project. Prepared for Redwood Community Action Agency.

USDA. 2007. Chapter 8: Threshold Channel Design. Part 654 Stream Restoration Handbook, National Engineering Handbook.

USFS. 1999. Wood handbook: wood as an engineering material. General Technical Report, FPL-GTR-113.

Vallino, J. J. and C. S. Hopkinson, Jr. 1998. Estimation of dispersion and characteristic mixing times in Plum Island Sound Estuary. Estuarine, Coastal and Shelf Science, 46: 333-350.

Wallace, M. 2015. Field Note: Martin Slough, Thence Swain Slough, Thence Elk River, Thence Humboldt Bay. October 2014 through June 2015. California Department of Fish and Wildlife.

Williams, P.B., M.K. Orr, and N.J. Garrity.2002. Hydraulic Geometry: A Geomorphic Tool for Tidal Marsh Channel Evolution in Wetland Restoration Projects. Restoration Ecology 10(3):577-590.

Winzler and Kelly, Michael Love and Associates and Coastal Analysis, LLC. 2006. Martin Slough Enhancement Feasibility Study, Eureka, CA. Prepared for the Natural Resources Division of Redwood Community Action Agency.

Zedler, J. 1984. Salt Marsh Restoration, a Guidebook for Southern California. A California Sea Grant College Program Publication.

Appendix A

Project Geotechnical Investigations

Geotechnical Report

Martin Slough Enhancement Project

Prepared for:

Redwood Community Action Agency

Consulting Engineers & Geologists, Inc.

812 W. Wabash Ave.
Eureka, CA 95501-2138
707-441-8855

May 2013
013035

Reference: 013035

Geotechnical Report
Martin Slough Enhancement Project

Prepared for:

Redwood Community Action Agency

JASON P. BUCK
exp. 7/31/14
No. 8723

ROLAND S. JOHNSON, JR.
exp 12/31/14
No. 1120

JOHN H. DAILEY
No. 256
EXP. 3/31/14

Jason P. Buck, PG

Roland Johnson, CEG

John Dailey, PE, GE

Prepared by:
SHN
Consulting Engineers & Geologists, Inc.
812 W. Wabash Ave.
Eureka, CA 95501-2138

May 2013

QA/QC: GDS gds

\\Eureka\Projects\2013\013035-MrtnSlghEnhnc\PUBS\rpts\20130510-GeotchRpt.doc

Table of Contents

Appendices

List of Illustrations

Figures **Follows Page**

Tables **Page**

Abbreviations and Acronyms

pcf	pounds per cubic foot
psf	pounds per square foot

ASTM	American Society for Testing and Materials-International	
CEQA	California Environmental Quality Act	
CPT	cone penetrometer test	
H:V	horizontal to vertical (ratio)	
HB-#	hand boring designation	
HDPE	high-density polyethylene	
MLA	Michael Love & Associates	
NAVD88	North American Vertical Datum, 19888	
NR	no reference	
OSHA	United States Occupational Safety and Health Administration	
PVC	polyvinyl chloride	
RCAA	Redwood Community Action Agency	
SCP	standard cone penetrometer	
SHN	SHN Consulting Engineers & Geologists, Inc.	
USCS	Unified Soil Classification System, where:	
	CH	high plasticity clays
	CL	clay with lower plasticity
	MH	high plasticity silts
	ML	low plasticity silts
	SM	silty sand
	SC	clayey sand
USGS	United States Geological Survey	

1.0 Introduction

1.1 General

This report provides the results of field and laboratory investigations conducted by SHN Consulting Engineers & Geologists, Inc. (SHN), and includes geotechnical recommendations for design development and construction of the Martin Slough Enhancement project. The Martin Slough Enhancement Project is a restoration project within the Martin Slough Valley in the southwestern portion of Eureka, California (Figure 1). The stated goals of the project are to improve fish habitat and access, to restore and enhance the former tidal salt/brackish marsh and freshwater wetlands in the lower Martin Slough floodplain, and to reduce the duration of flooding in the valley.

Our scope of work was developed from the request for proposals provided by Redwood Community Action Agency (RCAA) and included field and laboratory testing, analysis of results, development of recommendations, and the preparation of this report. A discussion of the project's geologic setting intended to be used in support of the California Environmental Quality Act (CEQA) compliance documentation has been provided under separate cover.

1.2 Project Location

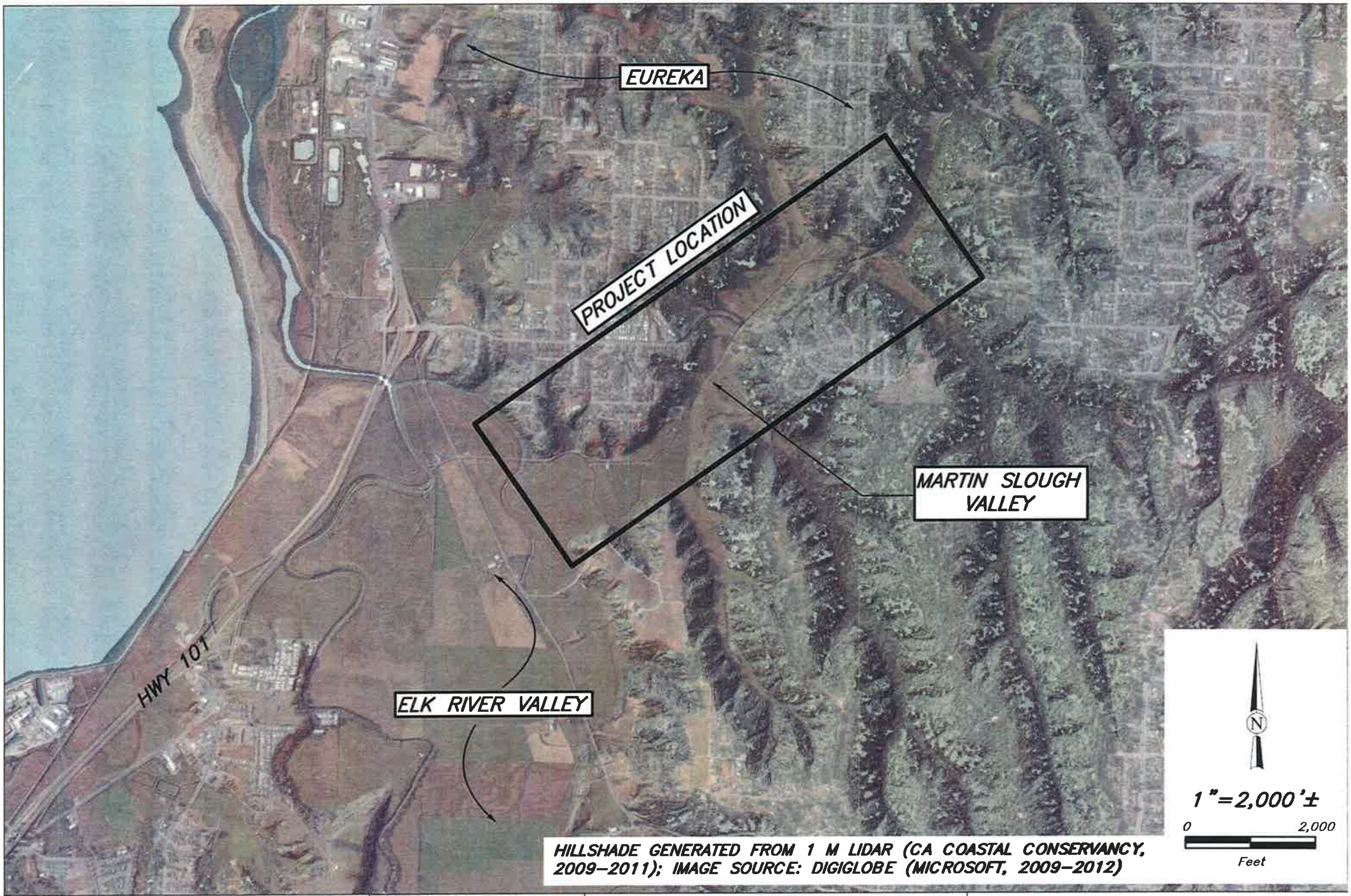
The project is located within the Martin Slough Valley, a coastal drainage that borders the southern part of the City of Eureka (Figure 1). The area is surrounded by unincorporated uplands. Martin Slough flows to Swain Slough downstream of the project area; Swain Slough is a tributary of the Elk River, which subsequently flows to Humboldt Bay west of the project area in southwest Eureka. The project area is within Sections 3, 4, 9 and 10, Township 4N, Range 1W, on the Eureka 7.5-minute United States Geological Survey (USGS) quadrangle.

1.3 Previous Work

SHN's experience going into this study includes previous geotechnical and construction observation projects within the Martin Slough Valley. Of these, one of the most relevant is the Martin Slough Interceptor project, a large sewer improvement project in which a sewer main was installed down the axis of the eastern portion of the valley. Many subsurface investigations were conducted for this project. The findings from our geotechnical studies are included in our 2003 *Geotechnical Study, Proposed Martin Slough Interceptor Sewer Project* (SHN, 2003) and our 2009 *Geotechnical Baseline Report, Phases I and II, Martin Slough Interceptor Project* (SHN, 2009). The excavations for the pipeline and the pump station (just south of the Fairview Drive Bridge) ranged from 8 to 25 feet in depth. SHN's construction observation experience during Phase I of the interceptor project was invaluable. The lessons learned about the limitations of the equipment, the condition of the excavated soils, and the difficulties with excavation are directly applicable to the Martin Slough Enhancement Project.

SHN has also been involved in the geotechnical investigation for the replacement of the Pine Hill Road Bridge over Swain Slough (in process) at the south end of the valley. Our investigation for that project included one boring and four cone penetration tests (CPT) to depths ranging from 60 to 105 feet. The boring for this project was placed very near the proposed new tide gate structure and extended to a total depth of 90 feet below grade.

Path: \\Eureka\Projects\2013\013035-MrtnSlghEnhnc\GIS\PROJ_MXD\Figure1.mxd



HILLSHADE GENERATED FROM 1 M LIDAR (CA COASTAL CONSERVANCY, 2009–2011); IMAGE SOURCE: DIGIGLOBE (MICROSOFT, 2009–2012)

SHN
Consulting Engineers
& Geologists, Inc.

Redwood Community Action Agency
Martin Slough Enhancement Project
Eureka, California

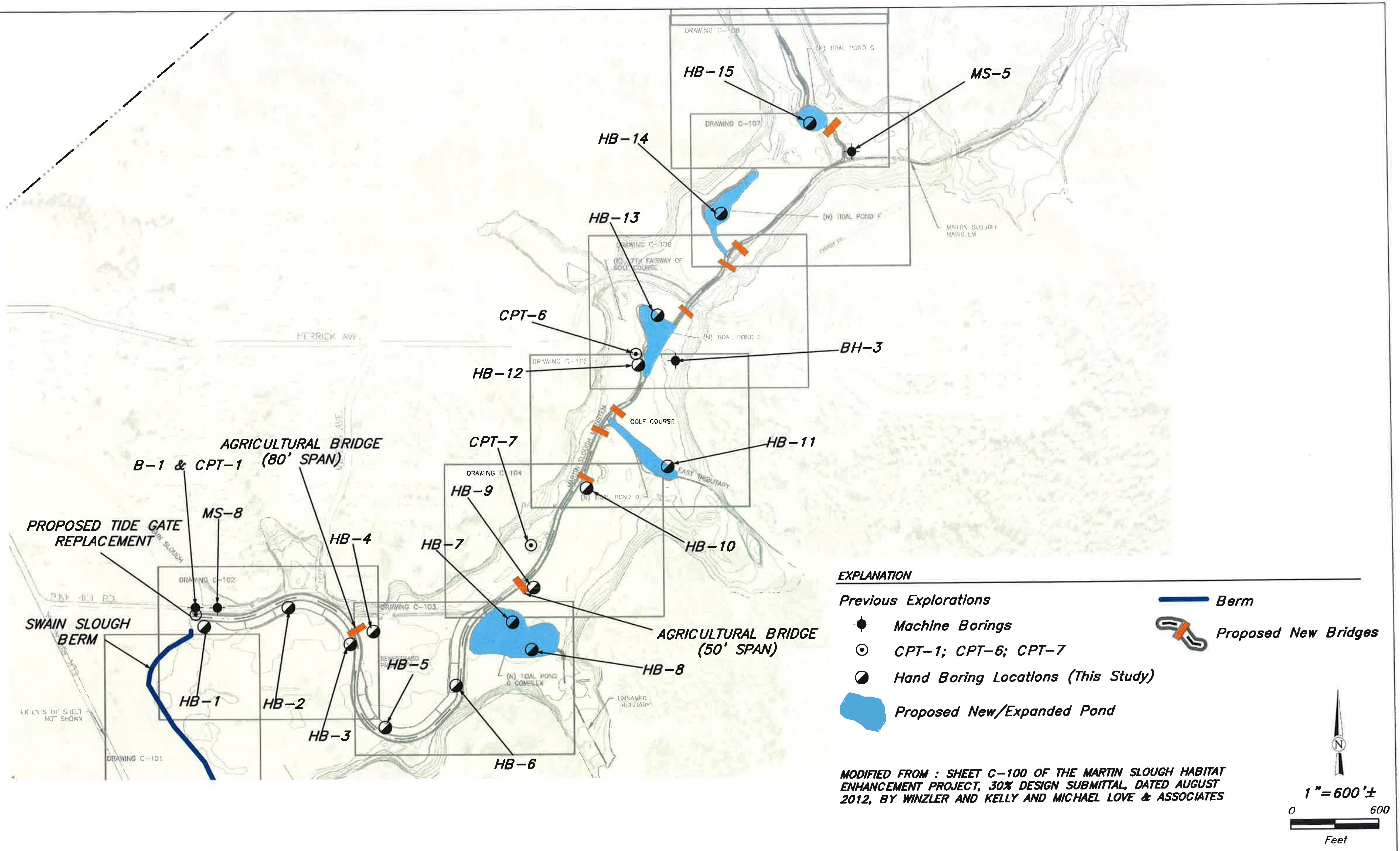
Project Location

SHN 013035

May 2013

Figure1

Figure 1



MODIFIED FROM : SHEET C-100 OF THE MARTIN SLOUGH HABITAT ENHANCEMENT PROJECT, 30% DESIGN SUBMITTAL, DATED AUGUST 2012, BY WINZLER AND KELLY AND MICHAEL LOVE & ASSOCIATES

Path: \\Eureka\Projects\2013\013035-MrtnSlghEnhnc\GIS\PROJ_MXD\SitePlan_w_Boring_Locations.mxd

SHN
Consulting Engineers
& Geologists, Inc.

Redwood Community Action Agency
Martin Slough Enhancement Project
Eureka, California

Site Plan Showing Project Elements
and Exploration Locations
SHN 013035

May 2013
SitePlan_w_Boring_Locations
Figure 2

We have included selected exploration logs from previous investigations for reference in Appendix A. Locations of these explorations are noted on Figure 2.

2.0 Project Description

2.1 Project Understanding

Our understanding of the scope of the Martin Slough Enhancement Project is based on information provided in the request for proposals, a pre-bid site walk, our review of the 30% design plans prepared by GHD, Inc. (formerly Winzler & Kelly) and Michael Love & Associates (MLA), dated August 2012, the *Martin Slough Enhancement Feasibility Study* (Winzler & Kelly and MLA, 2006) and our consultation with the design team, RCAA, GHD, and MLA.

2.2 Project Elements

The Martin Slough Enhancement Project consists of enlarging and recontouring the drainage network within the axis of the valley, including the development of a series of ponds, and as proposed will include a substantial amount of earthwork. Between the channel widening and construction of new ponds, the project includes an estimated 123,000 cubic yards of excavation. The project also includes infrastructural improvements (such as, the replacement of the tide gate at the Swain Slough junction and the construction of new agricultural access bridges). The specific project elements that we address in this report are described below. The locations of these project elements are shown on Figure 2.

Channel Widening/Realignment. The Martin Slough mainstem (7,300 lineal feet) and portions of the east tributary (600 lineal feet), the north fork tributary (1,100 lineal feet) and 700 lineal feet of an unnamed tributary will be widened and deepened. The final configuration of the channel varies greatly.

Construction and Expansion of Tidal Ponds. There are five tidal ponds that will be constructed. Some of these are expansions of existing ponds, while others are totally new. The ponds have been designed with variable floor elevations and strategically placed wood structures.

Replacement of Tide Gate. The existing tide gates (48-inch culverts with flap gates) at the confluence of the Martin Slough and Swain Slough are to be removed and replaced with a single concrete tide gate structure. The new tide gate planned for use is a 24-foot by 30-foot concrete box structure with four wing walls extending from each corner. The base of the structure will be founded at a depth of approximately 10 feet below grade.

New Bridges. Many of the existing golf cart bridges will need to be replaced once the channel has been widened. The project also includes the construction of two “agricultural” bridges that will provide access for agricultural equipment and emergency vehicles.

Enhancement of the Existing Berm along Swain Slough. The berm along the east side of Swain Slough is to be raised to an elevation of 9.5 feet (approximately 1.5 to 2 feet above existing grade).

Miscellaneous Grading. The project includes filling abandoned channels and loosely compacted fill areas in various locations on the golf course. Generally, these graded fill areas are broad and are called out to be approximately 1-foot thick.

3.0 Project Geologic Setting

The project is located within Martin Slough, an estuarine stream that drains a coastal valley that opens into the eastern shore of Humboldt Bay at the southern margin of the City of Eureka. The Humboldt Bay region occupies a complex geologic environment characterized by very high rates of active tectonic deformation and seismicity. The geomorphic landscape of the Humboldt Bay region is largely a manifestation of the active tectonic processes and the setting in this dynamic coastal environment.

Martin Slough and other coastal valleys around Humboldt Bay represent sediment-filled estuaries that reflect the late Quaternary history of sea level changes and tectonic deformation (uplift and subsidence). Sea level apparently reached its current high level in the mid-Holocene, about 6,000 years ago. As such, at least the uppermost part of the sediment filling the Martin Slough Valley would be anticipated to be mid-Holocene in age, or younger.

A comprehensive discussion of the geologic setting, including a description of geologic hazards associated with the project location, is provided under separate cover.

4.0 Field Investigation and Laboratory Testing

SHN conducted geotechnical investigations to evaluate representative subsurface soil conditions, and to provide foundation design criteria and site development recommendations for the project elements described above. Our field investigation was limited to reconnaissance of the project site and the drilling and sampling of 15 widely spaced exploratory borings.

The borings were advanced to depths ranging from of 5 to 15 feet below the ground surface. The borings were logged in general accordance with the Unified Soil Classification System (USCS). (See Figure 2 for boring locations, and Appendix A for subsurface exploration logs.) The borings were advanced using hand augers. Samples were collected using a 2.5-inch diameter thin-walled tube, driven using a slide hammer sampler.

Penetration resistance tests were conducted in the field using a static cone penetrometer (SCP). Tests using the SCP were focused on the upper 4 feet of the soil profile and results are shown on the logs.

Selected undisturbed and disturbed samples were collected, and laboratory tests were conducted. Laboratory testing for index properties included in-place moisture content, dry density, unconfined compressive strength (in lab, and using hand-held penetrometer), percent fines, and Atterberg Limits (plasticity). Triaxial tests were also conducted, and the results are presented on plates in Appendix B. Ad hoc testing was done to evaluate the shrinkage potential of selected soil samples.

For characterization of soils for agricultural purposes, selected samples were submitted to A & L Western Agricultural Laboratories, Inc. in Modesto, California. The results of these tests are provided in Appendix C.

See the attached subsurface exploration logs (Appendix A) for detailed soil descriptions, the penetration resistance test results, and laboratory index test results.

5.0 Site Conditions

5.1 Artificial Fill

Artificial fill was not encountered within our borings. Fill is expected to be encountered within the berm alignment, at the tide gate, and at various locations within the golf course area. Fill materials are generally anticipated to be thin and are not expected to be a significant factor in the proposed project.

5.2 Native Soils

Sediment filling Martin Slough is generally fine-grained (silt and clay). The material is primarily derived from alluvial sources (overbank/floodplain deposits) in the upper part of the canyon, and estuarine sources (tidal marine deposits) in the lower reaches of the valley nearest the bay. Evidence of marine influence (deposits with marine shells for example) decreases as you move up the valley. We did not encounter shell fragments within our borings upstream of the Fairway Drive bridge. In this report, we refer to the alluvium and estuarine deposits together as "valley fill sediments." Valley fill sediments are young, unconsolidated materials that contain wood fragments, and other organic materials. Sandy deposits are present, and generally consist of fine sands interbedded with silt. Naturally occurring coarse materials were not encountered during subsurface investigations and are not expected to be encountered during construction operations.

The topsoil within the project area is generally thin with a surficial grass/root mat of 4 to 6 inches and a root zone that extends to 12 to 18 inches below grade. The agricultural characteristics of the upper 2 feet were characterized by A&L Laboratories. The results of the agricultural testing are provided as Appendix C.

Using the USCS system, textures in the valley fill sediments below the topsoil included silt (ML), clay (CL), sandy silt (ML), silty sand (SM), with less common lenses of fat clay (CH), elastic silt (MH) and clayey sand (SC).

From a geotechnical standpoint, the fine-grained valley fill sediments encountered in subsurface excavations are typically soft to very soft, only locally demonstrating higher strength to a level considered to be medium stiff. In previous investigations, blow counts (N-values) in these materials rarely exceeded 10 blows/foot, and were commonly less than 5. Where granular sediments were encountered, consistency ranged from very loose to medium dense. Blow counts in the less frequent granular materials were generally in the 4 to 12 blows/foot range. The upper 2 feet of the soil profile can be the most competent, simply because it has the benefit of the root structures, and the materials are slightly more consolidated from the seasonal wetting and drying cycle. Especially during the dry season, the upper 1 to 2 feet forms a "crust" of more competent soils. Once this crust is removed or disrupted (excavation, vehicle traffic, etc.) the ground strength is significantly reduced. This will be an important consideration in planning excavations and developing haul roads.

In general, fine-grained valley fill sediments within the upper 10 feet are associated with low dry density values (85 pounds per cubic foot [pcf] or less) and high relative moisture (25 to 45%). Shear strength of the soils, based on triaxial shear testing ranges from 200 to 300 pounds per square foot (psf).

5.3 Groundwater Conditions

Subsurface investigations conducted in the Martin Slough Valley bottom and other low-lying areas encountered a uniformly high groundwater table. Many of the subsurface investigations in low-lying areas were conducted, by necessity, near the end of the dry season, and generally encountered groundwater within 6 feet of the ground surface. Groundwater levels adjacent to the mainstem in the lower part of the Martin Slough Valley are influenced by tidal fluctuations, such that the water table rises during high tides. During the rainy season, water frequently ponds at the ground surface throughout the Martin Slough Valley.

Intense and long duration precipitation, modification of topography, and cultural activities, such as irrigation, water well usage, onsite waste disposal systems, and water diversions, can contribute to fluctuations in groundwater levels. Although the depth to groundwater can vary throughout the year and from year to year, a shallow groundwater condition persists throughout the year.

Groundwater elevations encountered within our borings during our field investigation for this project (March 21 and 22, 2013) are provided in the Table 1, below. At four of the boring locations, a slotted polyvinyl chloride (PVC) pipe was installed and left for 5 days to allow groundwater to stabilize. Measurements reported in Table 1 with a piezometer designation were taken on March 26, 2013. All other values within the "Depth of Stabilized Groundwater" column were measured the same day, after the borehole had remained open for a few hours.

Table 1
Groundwater Elevation Data

Location	Depth Groundwater Initially Encountered	Depth of Stabilized Groundwater
HB-1	5.0 feet	6.75 feet
HB-2	3.0 feet	2.36 feet (piezometer)
HB-3	1.75 feet	1.76 feet (piezometer)
HB-4	6.0 feet	-
HB-5	5.5 feet	2.24 feet (piezometer)
HB-6	4.5 feet	-
HB-7	1.25 feet	-
HB-8	-	1.71 feet (piezometer)
HB-9	4.0 feet	-
HB-10	3.5 feet	6.5 feet
HB-11	2.75 feet	1.5 feet
HB-12	3.0 feet	2.5 feet
HB-13	3.0 feet	0.75 feet
HB-14	2.0 feet	1.0 feet
HB-15	not encountered	>7 feet

The groundwater elevation data provided above is specific to the dates on which the measurements were taken. Because of the slow movement of water through the native soils, only the stabilized measurements taken from piezometers should be considered as actual groundwater elevations.

Groundwater should be expected to be encountered within most of the proposed excavations for this project. It should be noted, however, that although groundwater levels are generally shallow, the permeability of the fine-grained soils are typically low. Because of this, groundwater generally seeps into excavations at a relatively low rate. In past excavations associated with the interceptor project, for instance, rapid infiltration of groundwater was generally only observed when lenses of sandy or woody material were encountered.

6.0 Conclusions and Discussion

Based on the results of our field and laboratory investigations, it is our opinion that the project site can be developed as proposed, provided that our recommendations are followed, and that noted conditions and risks are acknowledged.

Soils will be easy to excavate and can be done so with most any equipment. Excavated soils will have over-optimum moisture content and will be difficult to dry out. Groundwater should be anticipated within all but the very shallowest excavations.

The primary geotechnical site consideration is the pervasive, soft, saturated soil conditions. Due to the weak, compressible soils, and the volume of materials planned for excavation and off-hauling, the construction operations will present the greatest geotechnical challenge to the project. Access roads will need to be robust to remain functional and minimize impacts to the natural grounds. We strongly encourage careful planning of the haul roads layout.

Permanent structures (such as, the tide gate and the bridges) that are supported on shallow soils are anticipated to be susceptible to settlement. The risks associated with settlement and the cost/ benefit of mitigation measures should be considered in the design of these structures. We recommend that the tide gate structure implement some form of deeper support beyond what is shown on the 30% design plans. Implementing deep support for the bridges, however, is likely not necessary to meet project objectives and would not be cost effective. We would recommend designing the bridges and their abutments to accommodate some settlement. We provide foundation design criteria recommendations for these structures below.

7.0 Recommendations

7.1 Site Preparation and Grading

A significant part of the enhancement project is associated with grading.

7.1.1 General Fill Areas

The project plans show multiple areas where fill materials will be loosely placed in a thin layer (approximately 1 foot) over broad areas. Abandoned channel segments will be filled in. In these areas, the fill placement methods are not considered critical. If necessary, performance criteria could be developed for fills.

- If possible, we recommend targeting the driest soils for re-use as fill. Stockpiling the upper 1 to 1.5 feet of soil for reuse in these general fill areas would not only ensure that the driest soils are being used, but the existing organics may help with establishing new vegetation.

7.1.2 Temporary Cut Slopes

Temporary cut slopes are anticipated for excavations associated with the installation of the tide gate, construction entrances, cofferdams, and (possibly) other project elements. The stability of a cut slope depends upon the soil type, the groundwater conditions (or soil moisture conditions), and the angle of the cut. Most of the soils encountered in excavations will be silts and clays, which tend to be moderately cohesive, especially under unsaturated conditions, but with seeping groundwater, the stable angle of a cut decreases dramatically.

Relatively small temporary cut slopes (less than 4 feet) where the soil profile has had time to dewater, or where only a minor amount of water is present may hold a 1:1 horizontal to vertical (1H:1V) orientation, for a few days.

- Construction equipment should be excluded from within 5 feet of the edge of temporary cut slopes that are 1H:1V.
- As a general guide we recommend that the angle of temporary cut slopes higher than 4 feet, or where groundwater seepage is present, be limited to a 1.5H:1V cut. However, even some 1.5H:1V cuts in very soft soils may fail within a few hours of excavation. Ultimately, field conditions will dictate the appropriate angle.

7.1.3 Swain Slough Berm

The project includes reconstructing the existing berm along Swain Slough. It is our understanding that the berm will be raised slightly and widened toward the east side. The design elevation shown on the 30% plans is at 9.5 feet, though we understand the final design may be up to 12 feet using the North American Vertical Datum, 1988 (NAVD88). The planned crest width is approximately 6 feet. Currently, the upper surface of the berm is irregular, ranging in elevation from 7 to 8.5 feet.

The berm is to be constructed using soils excavated from other areas of the project. It should be expected that excavated soils will be fine-grained (silt and clay) and have an over-optimum moisture condition. Excavated soils will be slow to dry out and may need to be staged to allow moisture conditioning. Our recommendations provided below assume that the berm is not intended to be a certified flood control structure and that the objectives of the reconstruction are to enhance the ability of the berm to serve as a temporary water barrier and maintaining stable side slopes. Our understanding is that the upper surface of the berm will not be required to serve as a road surface.

- If possible, we recommend targeting the driest soils for re-use in the berm construction. Soils immediately below the organics, but above the groundwater table will most likely be in the best condition for re-use. Soils below the water table will be saturated and difficult to place and compact.
- The berm will be accessed from a single location, so careful consideration of construction methods should be made to minimize the number of trips in and out. Using lightweight equipment should also be considered. Installing a temporary access road may be necessary. Ideally, the footprint of the berm can serve as the access route for importing materials; however, if the soils become too soft for travel, then a temporary road adjacent to the berm may be necessary.
- To prepare the berm for fill placement, the footprint of the new berm should be stripped of the existing organic layer. Just the vegetation and the root system should be removed. If

debris or other deleterious material is encountered, it should also be removed. Care should be taken at this stage to minimize over-excavation. The deeper the excavation extends, the less suitable the operating surface will become. Organic-rich materials should be stockpiled nearby for reuse as the final cover layer.

- Once the organics have been removed from the footprint of the berm, the subgrade surface should be leveled or benched if necessary. If conditions allow, the surface should be rolled with a small sheep's-foot roller or equivalent. The berm should be constructed in lifts no greater than 12 inches. Compaction effort should be made on each lift using track-equipment or a small sheep's-foot roller as soil conditions allow. Side slopes on the Martin Slough side should be constructed at a gradient of 2H:1V. Side slopes on the Swain Slough side should be constructed at a gradient of 3H:1V.
- For poor soil conditions (such as, those at this site), we recommend developing a performance-based criteria for compaction that is feasible, yet meets the objectives of the project. Compaction criteria (such as, a percent of maximum dry density) is not considered appropriate for the type of soils that will be used or necessary for the project objectives.
- Once design grades have been achieved, the stockpiled organic rich materials should be spread over the bare soils and tamped into place so that vegetation can be reestablished. Alternatively, covering the berm with an erosion control blanket and seeding could be used to reestablish vegetation.

7.2 Seismic Design

We recommend that proposed bridges and the tide gate structure be designed and built to withstand strong seismic shaking. As in all of Humboldt County, the site is subject to strong ground motion from seismic sources.

The 2010 California Building Code requires the following information for seismic design. Based on our knowledge of subsurface and geologic conditions, we estimate a Site Class E (soft soil profile) for the project. Based on the Site Class and the latitude and longitude, we calculated the design spectral response acceleration parameters S_S, S_1, F_a, F_v, S_{MS}, S_{M1}, S_{DS} and S_{D1} using the USGS seismic calculator program, “Seismic Hazard Curves, Response Parameters, Design Parameters: Seismic Hazard Curves, and Uniform Hazard Response Spectra”, v. 5.1.0, dated February 10, 2011. Calculated values are presented in the following Table 2, Seismic Design Criteria.

Table 2
Seismic Design Criteria

Latitude	40.752144
Longitude	-124.178327
Site Class	E
S_S	2.57
S_1	1.00
F_a	0.9
F_v	2.40
S_{MS}	2.31
S_{M1}	2.40
S_{DS}	1.54
S_{D1}	1.60
Occupancy Category	II
Seismic Design Category	E

7.3 Foundations

7.3.1 General Design for Shallow Foundations

The primary consideration for the design and construction of shallow foundations is the low bearing capacity of the soils which is constrained by the high settlement potential. Some settlement of the structures placed on shallow foundations should be anticipated (2 to 6 inches) over time. Traditional deep foundations for non-critical structures are not considered cost effective because of the significant depths to good "bearing soils."

- Shallow foundations are proposed for supporting the new bridges. Assuming some settlement (2 to 6 inches) is acceptable, the abutments may be constructed on a shallow support system. Minimizing the weight of the foundation and incorporating allowances for settlement are recommended. The use of gravel ramps on the approaches should make adjustments to the transitions easy. If tilting is to be avoided, then adding provisions that allow for re-leveling at a later date would be advised.
- For general design criteria, we recommend that shallow foundations not exceed an allowable bearing capacity of 1,000 psf for dead plus live loads. A horizontal friction coefficient of 0.30 may be used for the footing/soil contact. Frictional resistance may be calculated in conjunction with an allowable lateral passive pressure represented by an equivalent fluid weighing 150 pcf for short-term loadings, such as lateral foundation resistance in response to wind or earthquake loadings. Lateral passive pressure can be calculated where footings bear laterally against undisturbed native subsoils or structural fill.
- Foundation embedment should remain as shallow as feasible. As discussed in Section 5.0, the upper 1 to 2 feet of soils are generally the strongest, so deeper embedment does not equate to stronger soils, as is usually the case. It is only necessary to remove the organics. Also, the deeper the excavation, the more difficult the working conditions will be for establishing a stable subgrade, setting forms for concrete, etc.
- Where new channel banks are constructed on 1.5H:1V slopes adjacent to bridge abutments, the base of the abutment closest to the channel should be constructed on or behind a sloping plane of 2H:1V starting at the edge of the channel bottom.

Below we provide a discussion of the general types of bridges proposed and our foundation design and construction recommendations for each.

7.3.2 Golf Cart Bridges

The existing golf cart bridges will be replaced, in some cases with longer spans, as a consequence of the channel being widened. The new golf cart bridges are anticipated to be similar in design to the existing. Two of the bridges, one on each side of the Fairview Drive bridge, are planned to accommodate heavier traffic, including emergency vehicles.

- Shallow, reinforced concrete abutments like those currently in use should be adequate for both of these bridge types that are less than 30 feet in length, provided they meet the design criteria specified in Section 7.3.1, above.
- For bridges with spans larger than 30 feet, we recommend using bridge abutments similar to those discussed below for the agricultural bridges.
- Ramp fills shall be no thicker than 2 feet considering the design criteria provided in Section 7.3.1.

7.3.3 Agricultural Bridges

There are two free-span steel bridges proposed within the agricultural areas south of the golf course: a 50-foot span and an 80-foot span (Figure 2). It is our understanding that the bridges will only be used for ranch trucks, agricultural equipment, or other light duty use. The anticipated maximum loads on the abutments of the 80-foot-span bridge are assumed to be on the order of 62 kips.

- For bridge spans 30 feet and longer, we recommend the use of a two-part system, which includes a stabilization mat and the bridge footing itself. Figure 3 presents a schematic drawing of this concept.

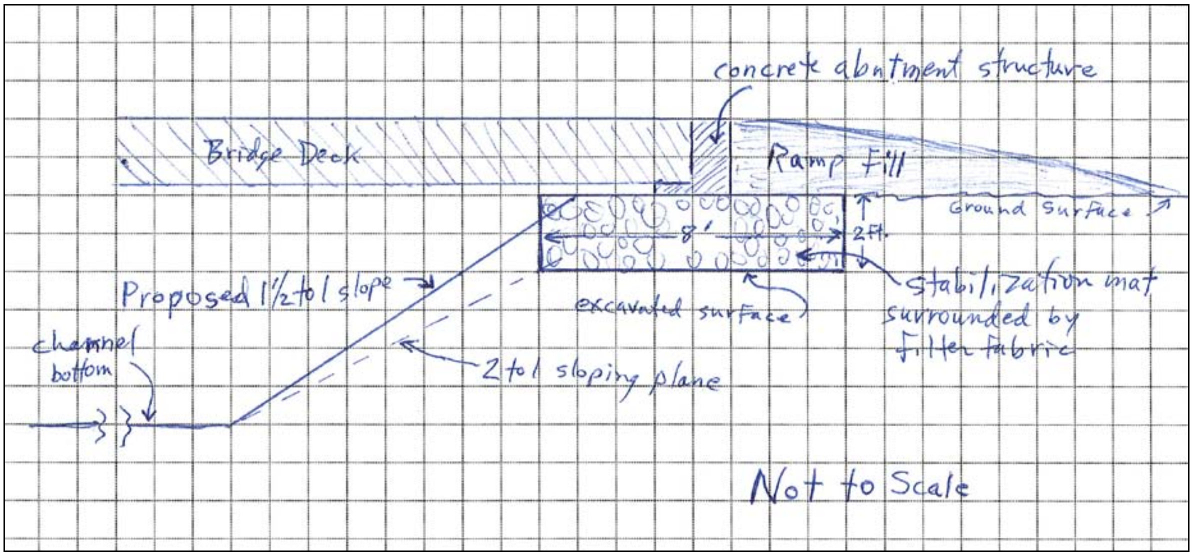


Figure 3. Schematic Drawing of Foundation System for Bridges with Spans Greater Than 30 feet
(actual dimensions will vary)

The purpose of the stabilization mat is to distribute the load of the bridge footing through a flexible, low density, laterally constrained structure that will maintain its integrity while undergoing significant differential settlement.

- We suggest the use of welded wire gabions for this, because it will result in minimal excavation, a relatively easy installation process, and low-cost compared with reinforced concrete. Other alternatives for a stabilization mat may include a laterally constrained multi-layered bed of crushed aggregate and geogrid or interlaced wood beams.
- The stabilization mats should be designed for equivalent basal footing loads of 750 psf or less.
- The bridge footing load should be centered on the stabilization mat structure and should not exceed a footing load of 2,500 psf.
- The thickness of the stabilization mat should be at a ratio of 1:4 with the basal width. For example, an 8-foot basal-width stabilization mat would be at least 2 feet thick. In this example, the overlying concrete abutment footing would need to have a minimum basal width of 2 feet.
- Under no condition should the stabilization mat be less than 6 feet wide or be embedded less than 1.5 feet below original ground surface.
- Where new channel banks are constructed on 1.5H:1V slopes adjacent to bridge abutments, the base of the stabilization mat closest to the channel should be constructed on or behind a sloping plane of 2H:1V starting at the edge of the channel bottom.

- All backfill overlying the bridge abutment footing systems should be low density and provisions should be made to prevent saturation. Ramp fills shall be no thicker than 2.5 feet considering the above design criteria.

7.3.4 Tide Gate Structure

The project includes a 24-foot by 30-foot concrete tide gate with wing walls extending out from each corner. The plans show the structure to have a 1-foot-thick reinforced slab foundation throughout the main part of the structure, with wing walls supported by 4-foot-wide spread footings. As discussed above, the soils at the foundation-bearing depth of this structure are soft, and there is, therefore, a moderate to high settlement potential.

- To minimize differential settlement, we recommend two alternatives for increasing support for the tide gate structure;
  1) sheet piles, and/or
  2) driven piles.

  These options could be used alone or in combination.

Currently, the 30% plans specify sheet piles installed on both the upstream and downstream edges of the structure including along the wing walls.

- Although the purpose of the sheet piles is to provide a groundwater cutoff, if the sheet piles could get extended to a depth of 20 feet below slab grade, then they would also provide support for the structure and reduce the settlement potential.
- Alternatively, or in concert, driven piles could be used to support the slab and wing walls. Driven piles that extend to "solid ground" are not likely cost effective, so piles, if used, should derive their support from friction. Friction piles may need to be extended to 50+ feet below grade, depending upon the loads, and if they are used in combination with the sheet piles. Further evaluation should be conducted to develop specific recommendations.

7.4 Temporary Roads for Construction Access

The temporary roads are a critically important part of the successful completion of the project. As discussed in Section 5.0, the soil conditions in the Martin Slough Valley are soft and saturated at a very shallow depth.

- All heavy equipment and truck traffic should be conducted on temporary roads. Only in rare cases (light vehicles and/or few trips) will vehicles be able to navigate across ground that is not reinforced. Careful consideration of the temporary roads and the layout will be necessary to maintain a functioning access system and minimize the environmental impacts.

Based on the volume of material planned for removal, the highest demand on the temporary road system is likely going to be traffic associated with off hauling the spoils.

- Special attention should be made during laying out the temporary road network and access points in order to minimize disturbance to the project area, maximize the use of temporary materials, and strike the right balance between the number of trips for offhaul and the load of each haul.

Below, we provide recommendations for two types of temporary roads:

1) a mat system, and
2) a geocell system.

Each has its advantages and disadvantages regarding cost/benefit. The specific details of each option may be amended based on the intended use of the particular roadway. In general high volume roadways will require more robust roads than short-term or light duty roads.

7.4.1 Mat System

This option uses interlocking composite road mats placed on a bed of reinforced gravel. The road should be underlain by a medium-weight non-woven filter fabric to act as a separation layer. The bed of gravel should be approximately 2 to 4 inches thick and should consist of crushed rock or equivalent gravel. A medium-grade geogrid should be used at the base of the gravel bed.

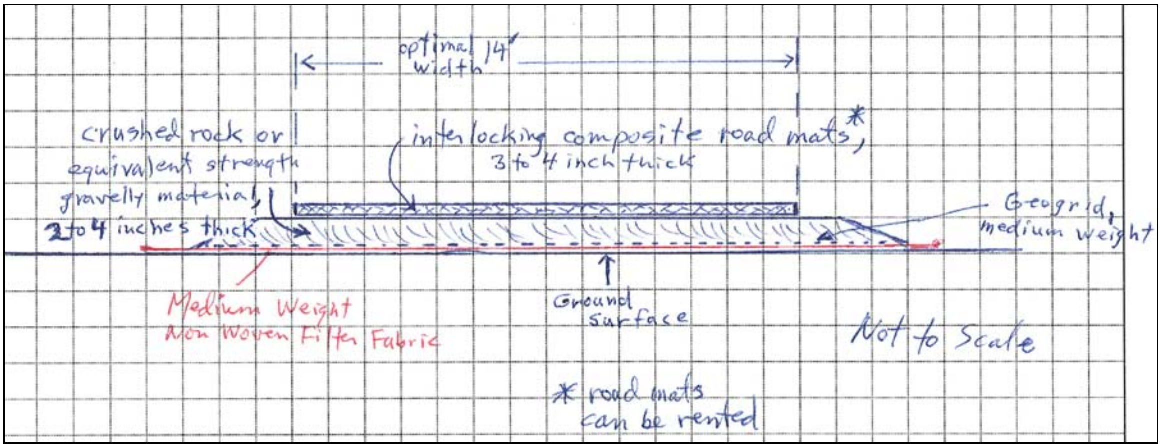


Figure 4. Schematic Drawing of a Temporary Haul Road Using a Mat System
(actual dimensions will vary)

Mats can be rented and will likely drive the cost of using this system. The mats can be pulled and placed with greater ease than some other road systems. Because of the interlocking nature of the mats, curved roads are not easily accommodated with this type of system. From our experience, the optimal width for a road like this is 14 feet.

7.4.2 Geocell System

This option uses a cellular confinement system, also known as geocells. The system is made of an expandable honey-comb-like structure (typically high-density polyethylene [HDPE]) which can be filled with sand and gravel, creating a strong, stiff, cellular mattress. When the soil contained within a geocell is subjected to pressure, it causes lateral stresses on perimeter cell walls. This type of system can be placed directly on the separation layer (woven filter fabric). Figure 5 depicts a schematic drawing of a typical geocell system.

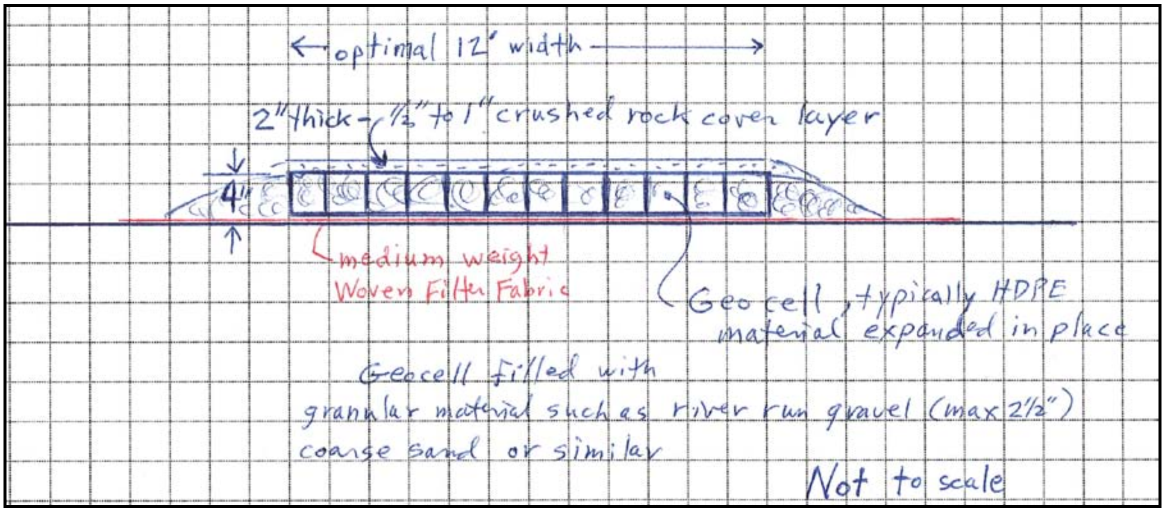


Figure 5. Schematic Drawing of a Temporary Haul Road Using a Geocell System
(actual dimensions will vary)

The material used to fill the cells is not as critical as in other applications, so most any coarse granular material will work. The geocell should be capped with a 2-inch layer of crushed rock. This type of system can more easily accommodate a curved road alignment. Pulling and reuse of this system is more difficult, because the HDPE structure is susceptible to damage.

7.5 Construction-Phase Monitoring

In order to assess construction conformance with the intent of our recommendations, it is important that a representative of our firm review the foundation excavations for the new tide gate and the large-span bridges.

This construction-phase monitoring is important because it provides the owner and SHN the opportunity to verify anticipated site conditions, and recommend appropriate changes in design or construction procedures if site conditions encountered during construction vary from those described in this report. It also allows SHN to recommend appropriate changes in design or construction procedures if construction methods adversely affect the competence of onsite soils to support the structural improvements.

Because of the variable conditions (generally poor) and the large area of the overall project, the project will be a "see as you go" type of endeavor. Various recommendations provided in this report are general, and depend upon the site conditions of the specific project at the time of construction. In many cases, the most appropriate approach cannot be evaluated until the work has begun.

- SHN should be included early on in the various phases of construction to verify the appropriateness of our recommendations and make adjustments if necessary.

8.0 Construction Considerations

This section presents construction considerations that are intended to aid in project planning. These considerations are not intended to be comprehensive; other issues may arise that would require coordination between the owner, the engineer, and the contractor's construction means and methods and capabilities.

Construction considerations for this project include the following:

1. The groundwater is characteristically shallow throughout the year. Based on recent excavation projects in the Martin Slough Valley, groundwater inflow is usually slow and easily managed with pumps. It is important to note, however that even small quantities of persistent seepage may substantially complicate construction operations where excavations extend below areas of saturated soil.
2. Following even minimal site stripping of the upper 1 to 2 feet of soil (the "crust"), exposed soil subgrade will likely be too soft and wet for heavy equipment to traverse. Compaction of the soil subgrade, or achieving a firm soil subgrade surface will be difficult or impractical.
    - If equipment access on excavated areas is necessary, special provisions should be developed, following review of subgrade conditions.
    - To avoid complications with soft subgrade, careful planning of the excavations, particularly those that cover a large area (such as the ponds), is encouraged.
3. We anticipate a vast majority of the excavated soils will be cohesive silty and clayey soils with a moisture content over optimum for compaction. These soils are typically not suitable for use as fill material to be compacted into place, because they will likely be overly wet, slow-drying due to their plasticity, and thus difficult to properly moisture condition and compact.
    - Spreading the soils out and repeatedly turning/disking may be necessary to enhance the usability of the soils.
4. OSHA Type C soils are indicated, requiring excavation side slopes of 1.5H:1V for excavations up to 10 feet in depth, or shoring. However, even at 1.5H:1V some slope failure may occur, particularly where saturated conditions are encountered. Compliance with safety regulations is the responsibility of the contractor.
    - OSHA trench and excavation safety regulations should be acknowledged and followed.

9.0 Plan and Specification Review

- We recommend communications be maintained during the design phase, between the design team and SHN, to optimize compatibility between the design and soil and groundwater conditions.
- We also recommend that we be retained to review those portions of the plans and specifications that pertain to earthwork and foundations. The purpose of this review is to confirm that our earthwork and foundation recommendations have been properly interpreted and implemented during design.

10.0 Closure and Limitations

The analyses, conclusions, and recommendations contained in this report are based on site conditions that we observed at the time of our investigation, data from our subsurface explorations and laboratory tests, our current understanding of proposed project elements, and on our experience with similar projects in similar geotechnical environments. We have assumed that the information obtained from our limited subsurface explorations is representative of subsurface conditions throughout the site.

We recommend that a representative of our firm confirm site conditions during the construction phase. If subsurface conditions differ significantly from those disclosed by our investigation, we should be given the opportunity to re-evaluate the applicability of our conclusions and recommendations. Some alteration of recommendations may be appropriate.

If the scope of the proposed construction, including the proposed loads, grades, or structural locations, changes from that described in this report, our recommendations should also be reviewed.

If there is a substantial lapse of time between the submission of our report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, we should review our report to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse. This report is applicable only to the project and site studied.

The conclusions and recommendations presented in this report are professional opinions derived in accordance with current standards of professional practice. Our recommendations are tendered on the assumption that design of the improvements will conform to their intent. No representation, express or implied, of warranty or guarantee is included or intended.

The field and laboratory work was conducted to investigate the site characteristics specifically addressed by this report. Assumptions about other site characteristics, such as, hazardous materials contamination, or environmentally sensitive or culturally significant areas, should not be made from this report.

11.0 References

California Building Standards Commission. (2010). *2010 California Building Code–Title 24 Part 2*, Two-Volumes. Based on International Building Code (2009) by the International Code Council. Sacramento, CA:California Building Standards Commission.

SHN Consulting Engineers & Geologists, Inc. (2003). *Geotechnical Study, Proposed Martin Slough Interceptor Sewer Project*. Eureka, CA:SHN.

---. (2009). *Geotechnical Baseline Report, Phases I and II, Martin Slough Interceptor Project*. Eureka, CA:SHN.

U.S. Geologic Survey. (February 10, 2011). "Seismic Hazard Curves, Response Parameters, Design Parameters: Seismic Hazard Curves, and Uniform Hazard Response Spectra," v. 5.1.0. NR:USGS.

Winzler & Kelly and Michael Love & Associates. (August 2012). "Martin Slough Habitat Enhancement Project" (Plan set). Eureka,CA:Winzler & Kelly.

---. (2006). *Martin Slough Enhancement Feasibility Study*. Eureka,CA:Winzler & Kelly.

Appendix A

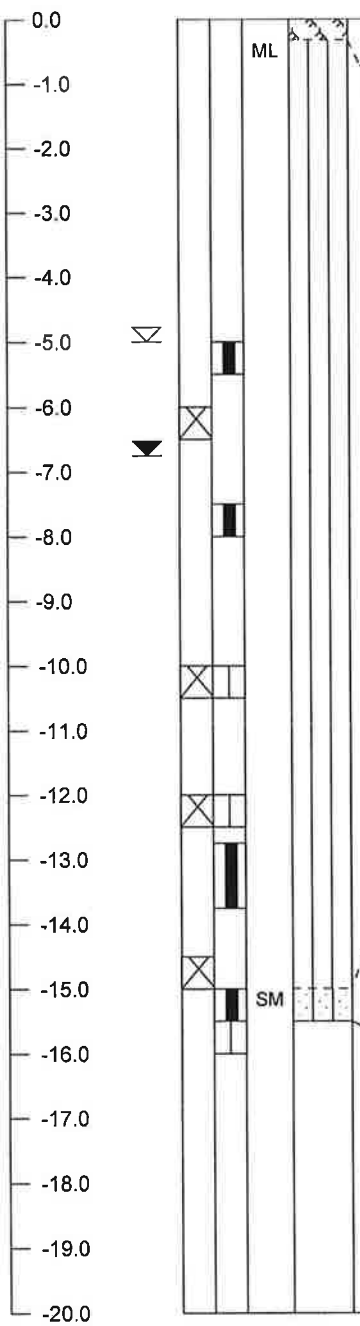
Subsurface Exploration Logs

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: Tide Gate
GROUND SURFACE ELEVATION: 11 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/21/13
TOTAL DEPTH OF BORING: 15.5 feet
SAMPLER TYPE: 2.5" O.D. brass shelby tube; hand hammer drive

BORING NUMBER HB-1



USCS	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits		% Dry Shrinkage	REMARKS
							Liquid Limit	Plastic Index		
ML	Grass, roots to 4".									
	SILT: Brown, soft, damp.									
	becomes weakly mottled. grades brown to brownish-gray.									
	grades bluish-gray and brown. slight increase clay content, very soft, moist to wet.									
	becomes saturated. grades bluish-gray.	28								Decomposed organic odor
		35	86				37		3	
	clamshells observed.									
	becomes very soft.									
	sand content increases. becomes soft. clamshells and wood fragments.	40	85			990				
SM	SILTY SAND; Bluish-gray, loose, saturated, clam shells.	22	103	25					0	
	Boring terminated at a depth of 15.5 feet. Groundwater initially encountered at a depth of 5.0 feet; stabalized at a depth of 6.75 feet.									

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: South side of slough channel, ~ Sta. 6+00
GROUND SURFACE ELEVATION:
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/21/13
TOTAL DEPTH OF BORING: 7.0 feet
SAMPLER TYPE: Bulk

BORING NUMBER HB-2

DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits		% Dry Shrinkage	REMARKS
											Liquid Limit	Plastic Index		
0.0	☒		ML		Grass, roots to 4". SILT WITH SAND; Brown, soft, damp to moist, minor fine sand.				13					
-1.0	☒								7					
-2.0 ▼ ▽	☒					26			5				5	
-3.0									9.5					
-4.0			ML/ SM		SANDY SILT; Yellowish-brown to grey (weakly mottled), soft, wet, fine sand, occasional wood fragments and decomposed organics.									
-5.0			ML		SILT; Grey, soft, wet, shell fragments.									
-6.0	☒													
-7.0					Boring terminated at a depth of 7.0 feet. Groundwater encountered at a depth of 3 feet. Piezometer installed following completion. Stabilized groundwater elevation at a depth of 2.36 feet on 3/26/13.									
-8.0														
-9.0														
-10.0														

LOG OF BORING

Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: 80' Span Agricultural Bridge
GROUND SURFACE ELEVATION: 16 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/21/13
TOTAL DEPTH OF BORING: 10.5 feet
SAMPLER TYPE: 2.5" O.D. brass shelby tube; hand hammer drive

BORING NUMBER HB-3

DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits		% Dry Shrinkage	REMARKS
											Liquid Limit	Plastic Index		
0.0			ML		Grass, roots to 4".									
-1.0					SILT; Brown to dark brown, medium stiff, moist, trace fine sand.				12					
▽														
-2.0		■	ML		SILT; Yellowish-brown to light olive grey (weak mottles), soft, wet.	49	72							Direct push from 2.25' to 2.75'
-3.0					sand content increases.				18					
-4.0					SANDY SILT; Yellowish-brown to grey (weakly mottled), soft, wet.									
-5.0	☒	■			SILT; Yellowish-brown to light olive grey (weak mottles), soft, wet.	43	80	79						Direct push from 5.0' to 5.5'
-6.0					decomposed wood and plant and shell fragments									
-7.0		■												Direct push from 7.0' to 7.5'
-8.0														
-9.0					becomes soft to very soft.									
-10.0		■				28	83							Direct push from 10' to 10.5'
-11.0					Boring terminated at a depth of 10.5 feet. Groundwater initially encountered at a depth of 1.76 feet. Piezometer installed following completion. Stabilized groundwater elevation measured at a depth of 1.76 feet on 3/26/13.									
-12.0														
-13.0														
-14.0														
-15.0														
-16.0														
-17.0														
-18.0														
-19.0														
-20.0														

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: 80' Span Agricultural Bridge
GROUND SURFACE ELEVATION: 16 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/21/13
TOTAL DEPTH OF BORING: 10 feet
SAMPLER TYPE: 2.5" O.D. brass shelby tube; hand hammer drive

BORING NUMBER HB-4

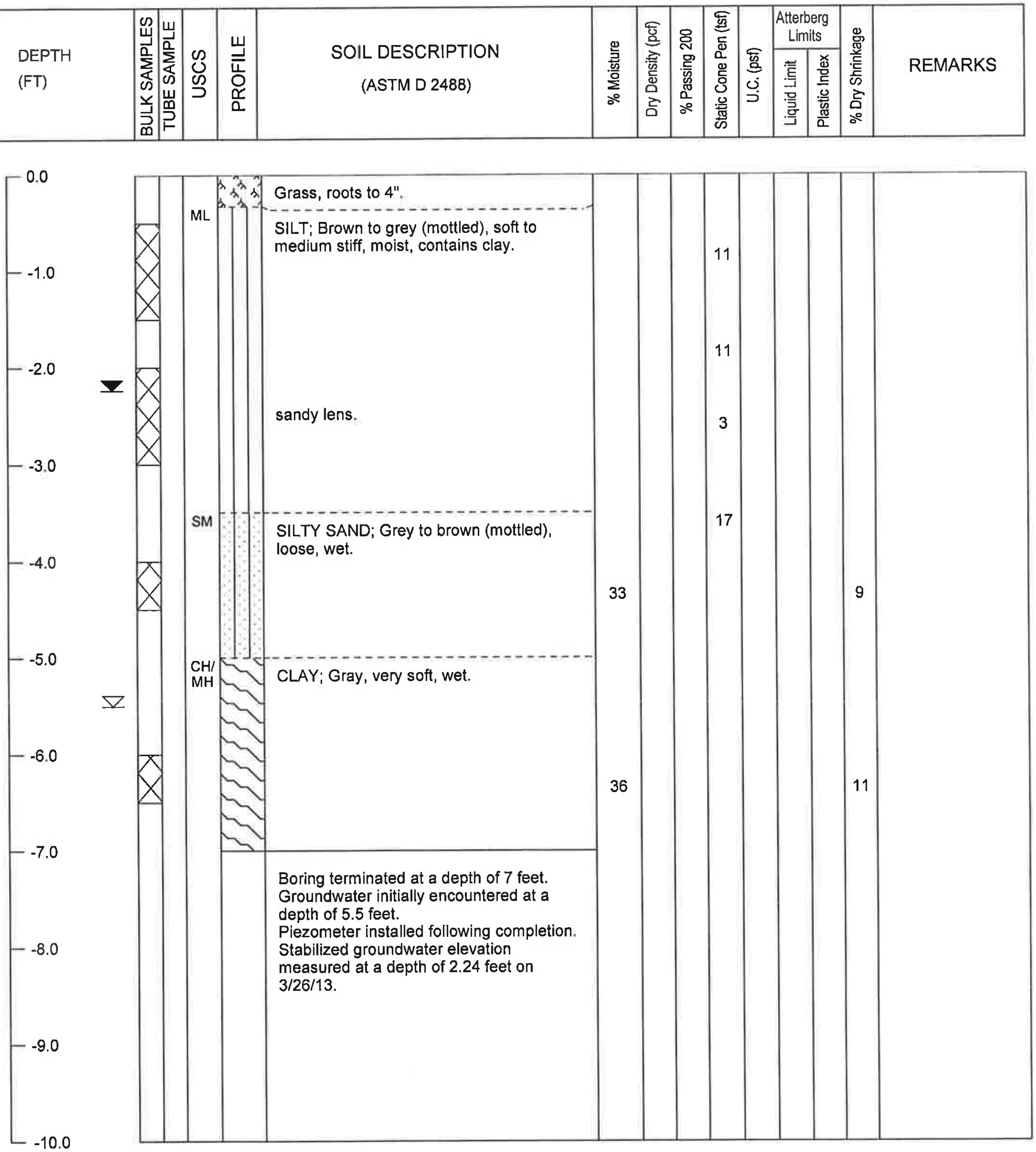
DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
0.0			ML		Grass, roots to 4".									
-1.0					SILT; Dark brownish-grey to brown (weakly mottled), soft to medium stiff, damp.				20					
-2.0									9					
-3.0									5					
-4.0		▮				36	86							
-5.0					grades grey. wood fragments and decomposed organics.									
-6.0 ▼		▮			becomes wet.					760				Direct push from 6.1' to 6.6'
-7.0					becomes soft.									
-8.0		▯												
-9.0		▮				38	80							Direct push from 8.6' to 9.1'
-10.0					Boring terminated at a depth of 10 feet. Groundwater encountered at a depth of 6 feet.									
-11.0														
-12.0														
-13.0														
-14.0														
-15.0														
-16.0														
-17.0														
-18.0														
-19.0														
-20.0														

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

LOG OF BORING

PROJECT: Martin Slough Enhancement Project
JOB NUMBER: 013035
LOCATION: East of south end of west limb of overflow
DATE DRILLED: 03/21/13
GROUND SURFACE ELEVATION: 16 feet
TOTAL DEPTH OF BORING: 7 feet
EXCAVATION METHOD: Hand Auger
SAMPLER TYPE: Bulk
LOGGED BY: AC, JMA

BORING NUMBER
HB-5



DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
0.0					Grass, roots to 4".									
			ML		SILT; Brown to grey (mottled), soft to medium stiff, moist, contains clay.				11					
-1.0														
									11					
-2.0														
					sandy lens.				3					
-3.0														
			SM		SILTY SAND; Grey to brown (mottled), loose, wet.				17					
-4.0						33							9	
-5.0														
			CH/ MH		CLAY; Gray, very soft, wet.									
-6.0						36							11	
-7.0					Boring terminated at a depth of 7 feet. Groundwater initially encountered at a depth of 5.5 feet. Piezometer installed following completion. Stabilized groundwater elevation measured at a depth of 2.24 feet on 3/26/13.									
-8.0														
-9.0														
-10.0														

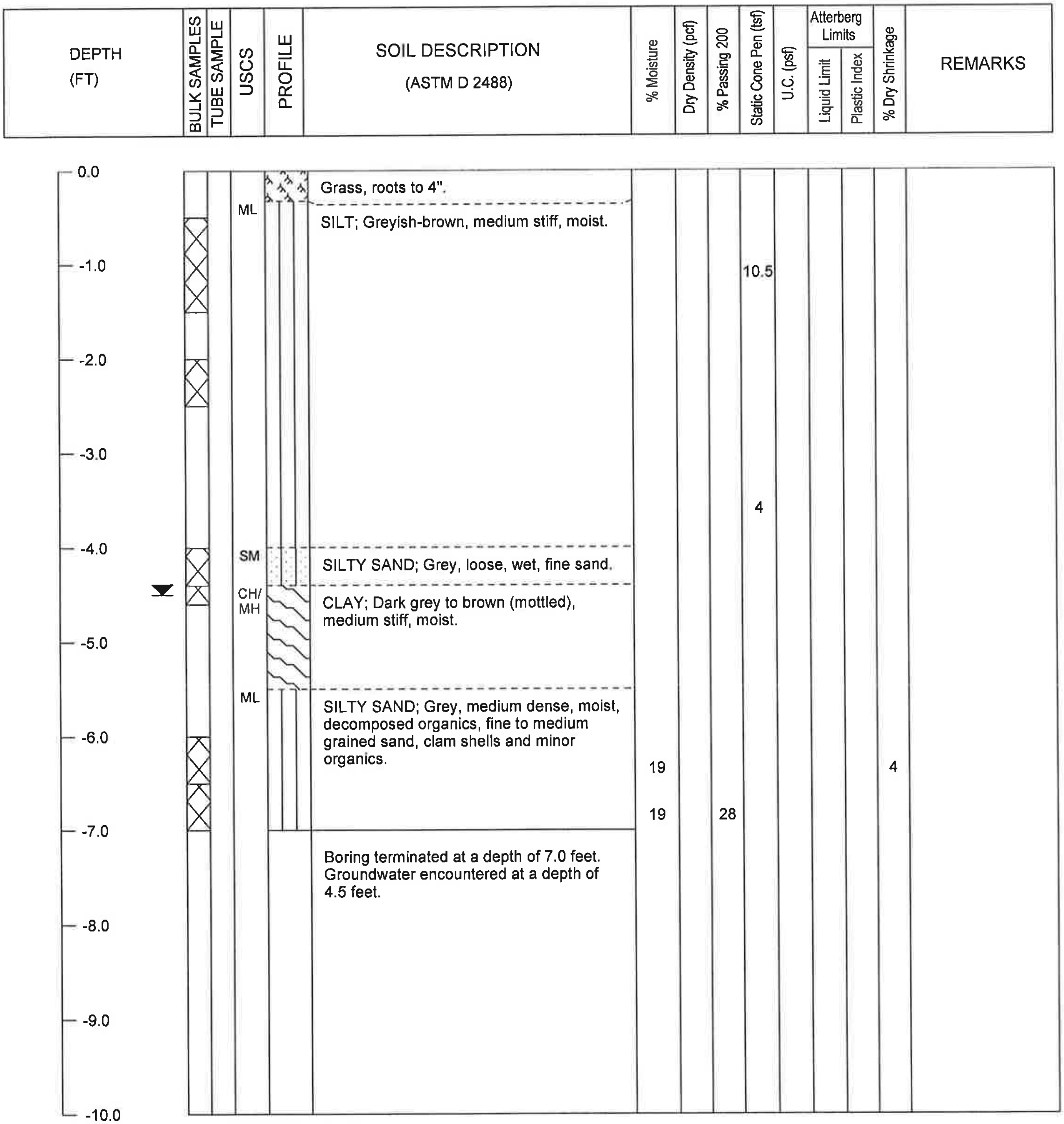
LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: East limb of overflow
GROUND SURFACE ELEVATION: 16 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/21/13
TOTAL DEPTH OF BORING: 7 feet
SAMPLER TYPE: Bulk

BORING NUMBER
HB-6



DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
0.0			ML		Grass, roots to 4". SILT; Greyish-brown, medium stiff, moist.									
-1.0									10.5					
-2.0														
-3.0									4					
-4.0			SM CH/MH		SILTY SAND; Grey, loose, wet, fine sand. CLAY; Dark grey to brown (mottled), medium stiff, moist.									
-5.0			ML		SILTY SAND; Grey, medium dense, moist, decomposed organics, fine to medium grained sand, clam shells and minor organics.									
-6.0						19 19		 28					4	
-7.0					Boring terminated at a depth of 7.0 feet. Groundwater encountered at a depth of 4.5 feet.									
-8.0														
-9.0														
-10.0														

LOG OF BORING

Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: Tidal pond C Complex
GROUND SURFACE ELEVATION: 16 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/21/13
TOTAL DEPTH OF BORING: 5 feet
SAMPLER TYPE: Bulk

BORING NUMBER
HB-7

DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits		% Dry Shrinkage	REMARKS
											Liquid Limit	Plastic Index		
0.0					Grass, roots to 4".									
			ML		SILT; Brown to brownish-grey to yellowish-brown (mottled), soft to medium stiff, moist, slightly clayey, some wood fragments.				7					
-1.0														
▼									7.5					
-2.0	☒								5					
-3.0									5					
-4.0	☒		ML		SILT; Dark grey to brown (mottled), medium stiff, moist, slightly clayey, minor organics.	30							6	
-5.0					Boring terminated at a depth of 5.0 feet. Groundwater encountered at a depth of 1.25 feet.									
-6.0														
-7.0														
-8.0														
-9.0														
-10.0														

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

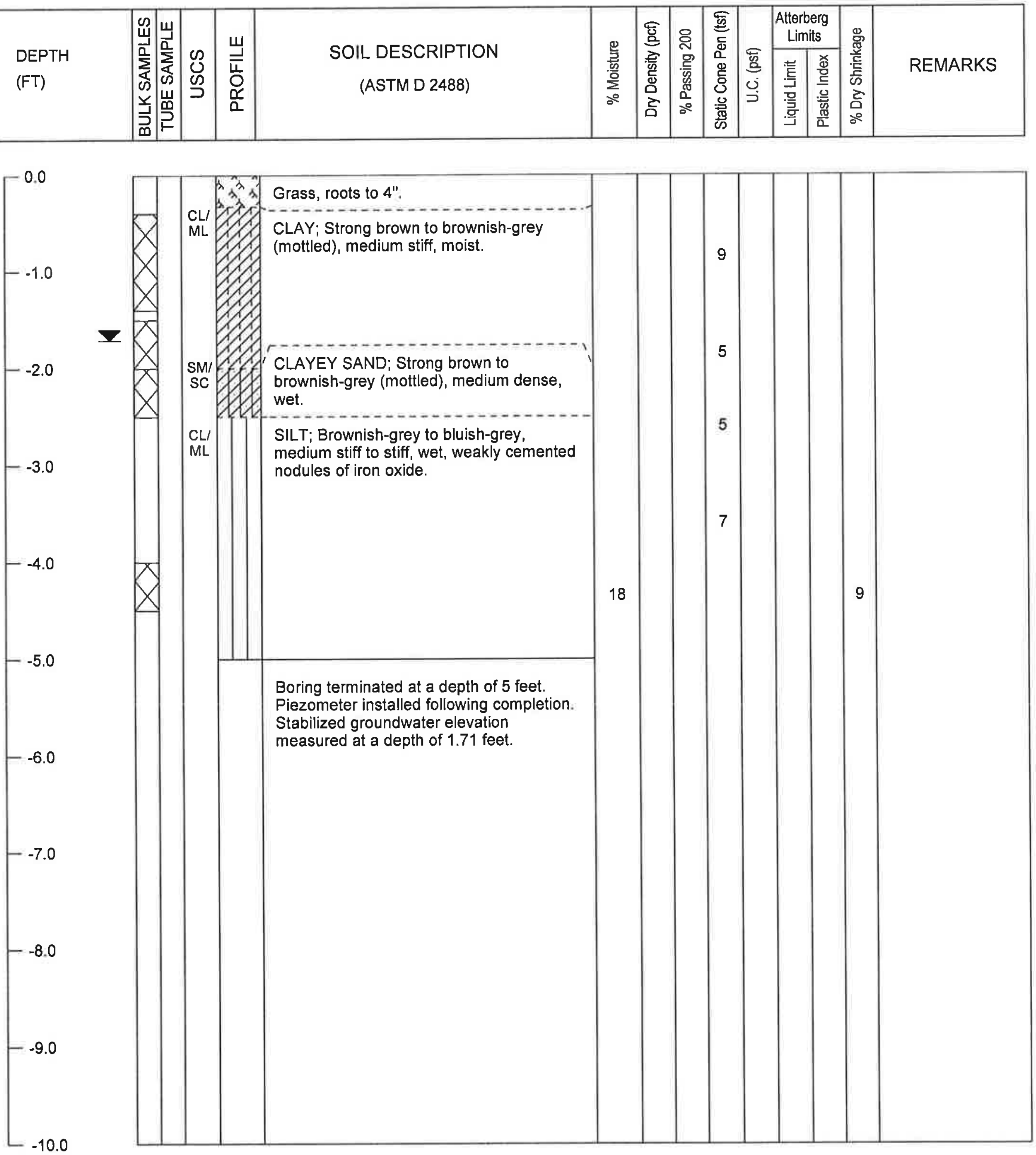
LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: Tidal pond C Complex
GROUND SURFACE ELEVATION: 17 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/21/13
TOTAL DEPTH OF BORING: 5 feet
SAMPLER TYPE: Bulk

BORING NUMBER HB-8



DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
					Grass, roots to 4".									
			CL/ ML		CLAY; Strong brown to brownish-grey (mottled), medium stiff, moist.				9					
			SM/ SC		CLAYEY SAND; Strong brown to brownish-grey (mottled), medium dense, wet.				5					
			CL/ ML		SILT; Brownish-grey to bluish-grey, medium stiff to stiff, wet, weakly cemented nodules of iron oxide.	18			5 7				9	

Boring terminated at a depth of 5 feet. Piezometer installed following completion. Stabilized groundwater elevation measured at a depth of 1.71 feet.

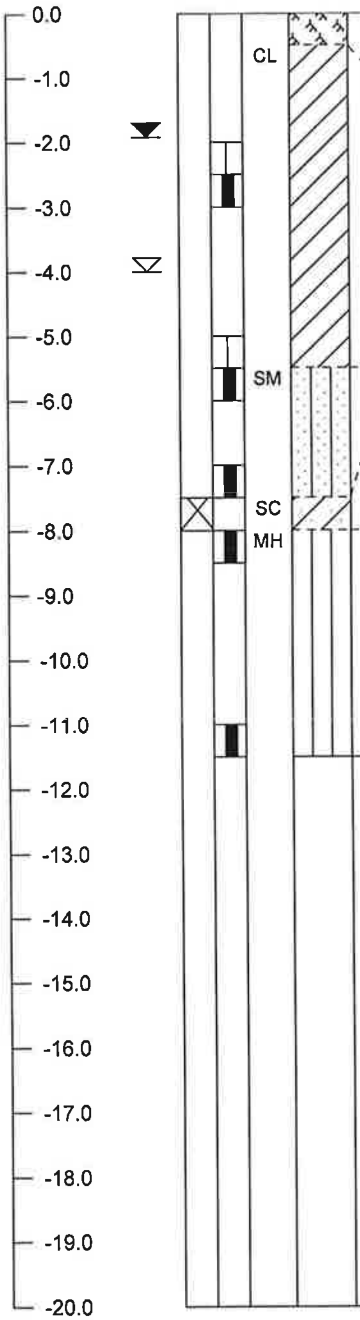
LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: 50' Span Agricultural Bridge
GROUND SURFACE ELEVATION: 16 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/21/13
TOTAL DEPTH OF BORING: 10 feet
SAMPLER TYPE: Bulk

BORING NUMBER HB-9



USCS	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits		% Dry Shrinkage	REMARKS
							Liquid Limit	Plastic Index		
	Grass, roots to 4".									
CL	SILTY CLAY; Brownish-grey to strong brown (mottled), medium stiff, moist.				14					
					16					
					8					
		31	90			1350	38	14		
SM	SANDY SILT; Greyish-brown, medium stiff, moist, trace fine sand, clamshells.	38	80	76					8.5	
SC	CLAYEY SAND; Bluish-grey, medium dense, wet, abundant clamshells.	25	100						14	
MH	SILT; Bluish-grey, soft to medium stiff, wet, decomposed organics.	34	86	34		340	57	25		
		41	80				46	25	2.5	

Boring terminated at a depth of 11.5 feet. Groundwater initially encountered at a depth of 4.0 feet. Piezometer installed following completion. Stabilized groundwater elevation measured at a depth of 1.92 feet on 3/26/13.

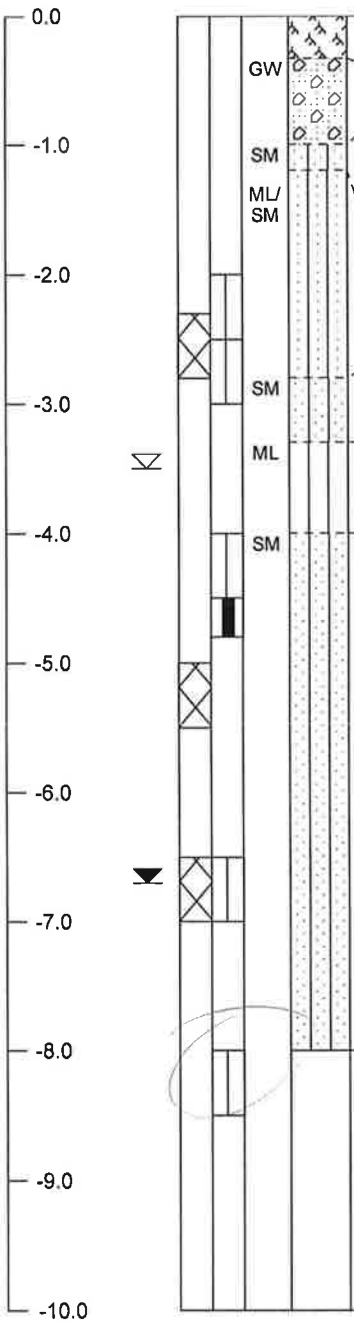
LOG OF BORING

Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: East side of slough channel; ~Sta. 44+00
GROUND SURFACE ELEVATION: 16 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/22/13
TOTAL DEPTH OF BORING: 8 feet
SAMPLER TYPE: Bulk

BORING NUMBER HB-10



USCS	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	% Dry Shrinkage	REMARKS
	Grass, roots to 4".									
GW	FILL: SANDY GRAVEL; Grey, rounded, medium dense, damp.									
SM	SILTY SAND; Yellowish-brown to grey (mottled), loose to medium dense, damp.									
ML/ SM	SILT WITH SAND; Grey, medium stiff, damp to moist.									
SM	SAND; Grey, loose, wet, trace silt, fine sand.									
ML	SANDY SILT; Grey, very soft, wet, very fine sand.									
SM	SILTY SAND; Grey, loose, wet. occasional lense of shells and shell hash.	25		30						

8.5'?

Boring terminated at a depth of 8 feet.
Groundwater initially encountered at a depth of 3.5 feet.
Stabilized groundwater elevation measured at a depth of 6.5 feet.

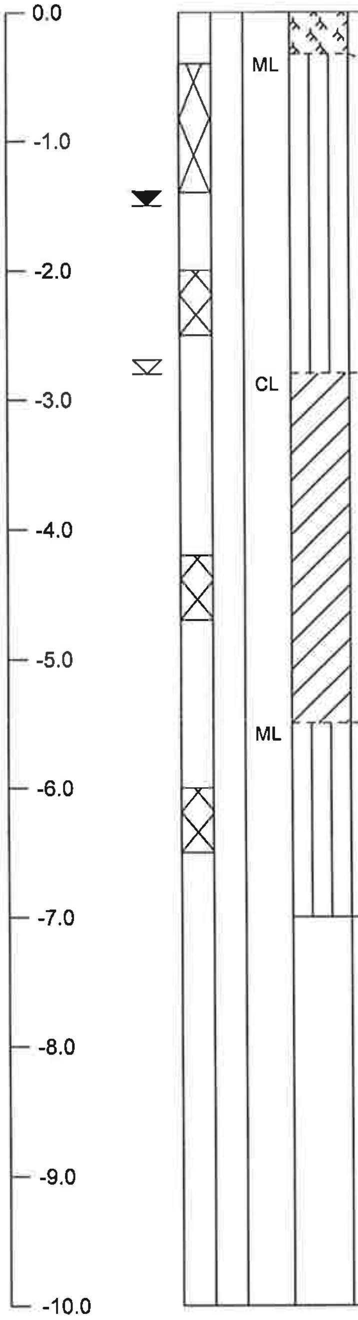
LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: Tidal Pond D
GROUND SURFACE ELEVATION: 16 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/22/13
TOTAL DEPTH OF BORING: 7 feet
SAMPLER TYPE: Bulk

BORING NUMBER
HB-11



DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits		% Dry Shrinkage	REMARKS
											Liquid Limit	Plastic Index		
					Grass, roots to 4".									
			ML		SANDY SILT; Brown to gray (mottled), soft, damp to moist, contains clay.	46			8.5 8				7	
			CL		SILTY CLAY; Grey to olive brown (weakly mottled), soft, wet.	42							12	
			ML		SILT; Grey, soft, wet, organics and peat filaments.									

Boring terminated at a depth of 7 feet.
Ground water initially encountered at a depth of 2.75 feet.
Stabilized groundwater elevation measured at a depth of 1.5 feet.

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.

812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project

LOCATION: Tidal Pond E

GROUND SURFACE ELEVATION: 16 feet

EXCAVATION METHOD: Hand Auger

LOGGED BY: AC, JMA

JOB NUMBER: 013035

DATE DRILLED: 03/22/13

TOTAL DEPTH OF BORING: 8 feet

SAMPLER TYPE: Bulk

BORING NUMBER HB-12

DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
0.0					Grass, roots to 4".									
			ML		SILT; Brown, soft, damp, contains clay.									
-1.0														
-2.0		■				42	75						10	
▼														
-3.0 ▽														
-4.0														
					grades bluish-grey.									
-5.0		■				45	73				46	19		
			SM		SAND; Grey, loose, saturated.									
-6.0														
-7.0														
			ML		SILT; Grey, soft, wet, few clamshells, trace fine sand.									
-8.0					Boring terminated at a depth of 8 feet. Groundwater initially encountered at a depth of 3.0 feet. Stabilized groundwater elevation measured at a depth of 2.5 feet.									
-9.0														
-10.0														

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: Tidal Pond E
GROUND SURFACE ELEVATION: 16 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/22/13
TOTAL DEPTH OF BORING: 7 feet
SAMPLER TYPE: Bulk

BORING NUMBER
HB-13

DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
0.0					Grass, roots to 4".									
▼	☒		CL		SILT; Yellowish-brown to grey (mottled), soft to medium stiff, damp, slightly clayey.									
-1.0	☒								28					
-2.0	☒					31							5	
▽ -3.0									7					
-4.0	☒													
-5.0			ML/ SM		SANDY SILT; Grey, soft, wet, interfingered sandy, clayey, and silty lenses, occasional clam shells.								5	
-6.0	☒					23								
-7.0					Boring terminated at a depth of 7 feet. Groundwater initially encountered at a depth of 3.0 feet. Stabilized groundwater elevation measured at a depth of .75 feet.									
-8.0														
-9.0														
-10.0														

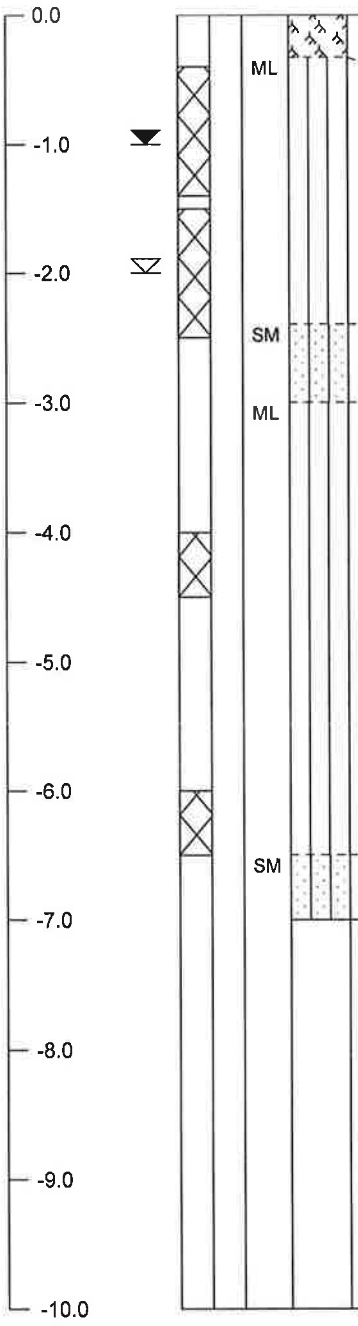
LOG OF BORING

Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough Enhancement Project
LOCATION: Tidal Pond F
GROUND SURFACE ELEVATION: 17 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/22/13
TOTAL DEPTH OF BORING: 7 feet
SAMPLER TYPE: Bulk

BORING NUMBER
HB-14



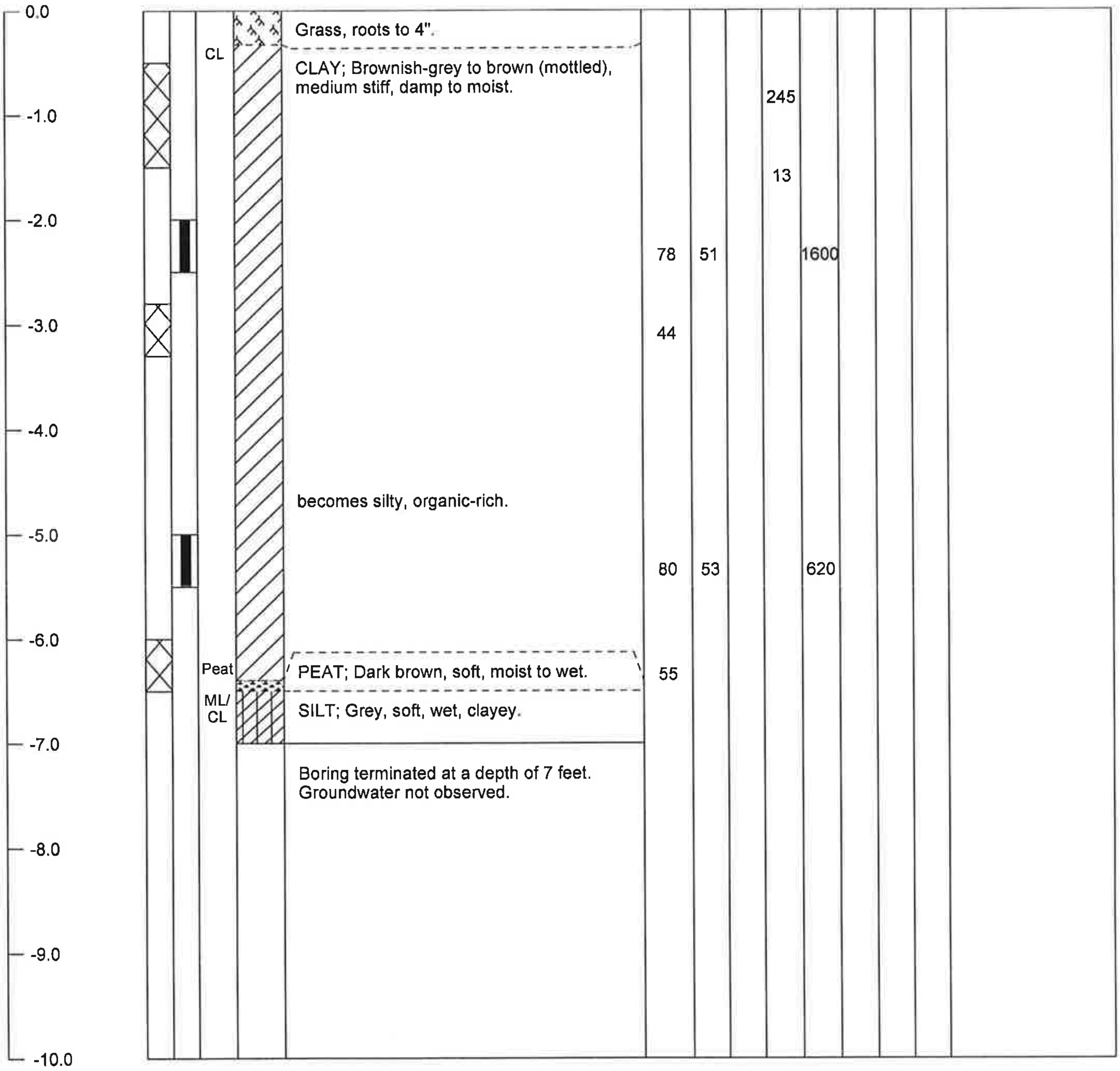
DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
					Grass, roots to 4".									
			ML		SILT; Yellowish-brown to gray (mottled), soft, damp to moist.				6 5					
			SM		SILTY SAND; Brownish-grey, loose, saturated, flowing, fine sand.				5					
			ML		SILT; Grey, soft, wet, organics, decomposed plant fragments.	27			14				8	
			SM		SILTY SAND; Grey, loose, wet.									
					Boring terminated at a depth of 7 feet. Groundwater initially encountered at a depth of 2.0 feet. Stabilized groundwater elevation measured at a depth of 1.0 feet.									

LOG OF BORING

PROJECT: Martin Slough Enhancement Project
LOCATION: Tidal Pond G
GROUND SURFACE ELEVATION: 17 feet
EXCAVATION METHOD: Hand Auger
LOGGED BY: AC, JMA

JOB NUMBER: 013035
DATE DRILLED: 03/22/13
TOTAL DEPTH OF BORING: 7 feet
SAMPLER TYPE: Bulk

BORING NUMBER
HB-15



DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
0.0					Grass, roots to 4".									
			CL		CLAY; Brownish-grey to brown (mottled), medium stiff, damp to moist.									
									245					
									13					
						78	51			1600				
						44								
					becomes silty, organic-rich.									
						80	53			620				
			Peat		PEAT; Dark brown, soft, moist to wet.	55								
			ML/ CL		SILT; Grey, soft, wet, clayey.									
					Boring terminated at a depth of 7 feet. Groundwater not observed.									

LOG OF BORING

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough
LOCATION: Pine Hill Road, Eureka, CA
GROUND SURFACE ELEVATION: --
EXCAVATION METHOD: Solid Stem Flight Auger (4")
LOGGED BY: SMB

JOB NUMBER: 001283.320
DATE DRILLED: 9/25/02
TOTAL DEPTH OF HOLE: 31.5 feet
SAMPLER TYPE: 2.5" I.D. Calif. Split Spoon, 140 lb telescoping hammer, 30" drop

HOLE NUMBER MS-8

DEPTH (FT)	BULK SAMPLES SS SAMPLES	SPT BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Comp. (psf)	% Passing 200	Atterberg Limits		REMARKS
										Liquid Limit	Plastic Index	
0.0			ML-CL	▨	SILT, clayey, very sandy, fine, with few to moderate organics, soft, dry, very dark grey.							
∇												
-5.0		1 2 2			Becomes wet.							
		1 2 2				41.6	75			30.5	11	Peak @ 6.5-7.0' C = 0.35 ksf Phi = 30.6 deg. Residual C = 0.20 ksf Phi = 32.8 deg.
					SILT, slightly clayey, slightly sandy, fine to medium, with few to no organics, soft, wet, very dark grey.							
-10.0		1 2 2				43.4	77			40.6	16	
-15.0		1 2 2			No organics.							
					SILT, slightly clayey to clayey, few to no organics, medium stiff, moist to wet, very dark grey.							
-20.0		1 2 3										
			ML	▥	SILT, slightly clayey to clayey, very slightly sandy, fine, medium stiff, wet, very dark grey.							ML-CL/ML Contact inferred.
-25.0		3 4 5										
					With shells.							
-30.0		2 3 5										
					Bottom of boring at 31.5 feet.							

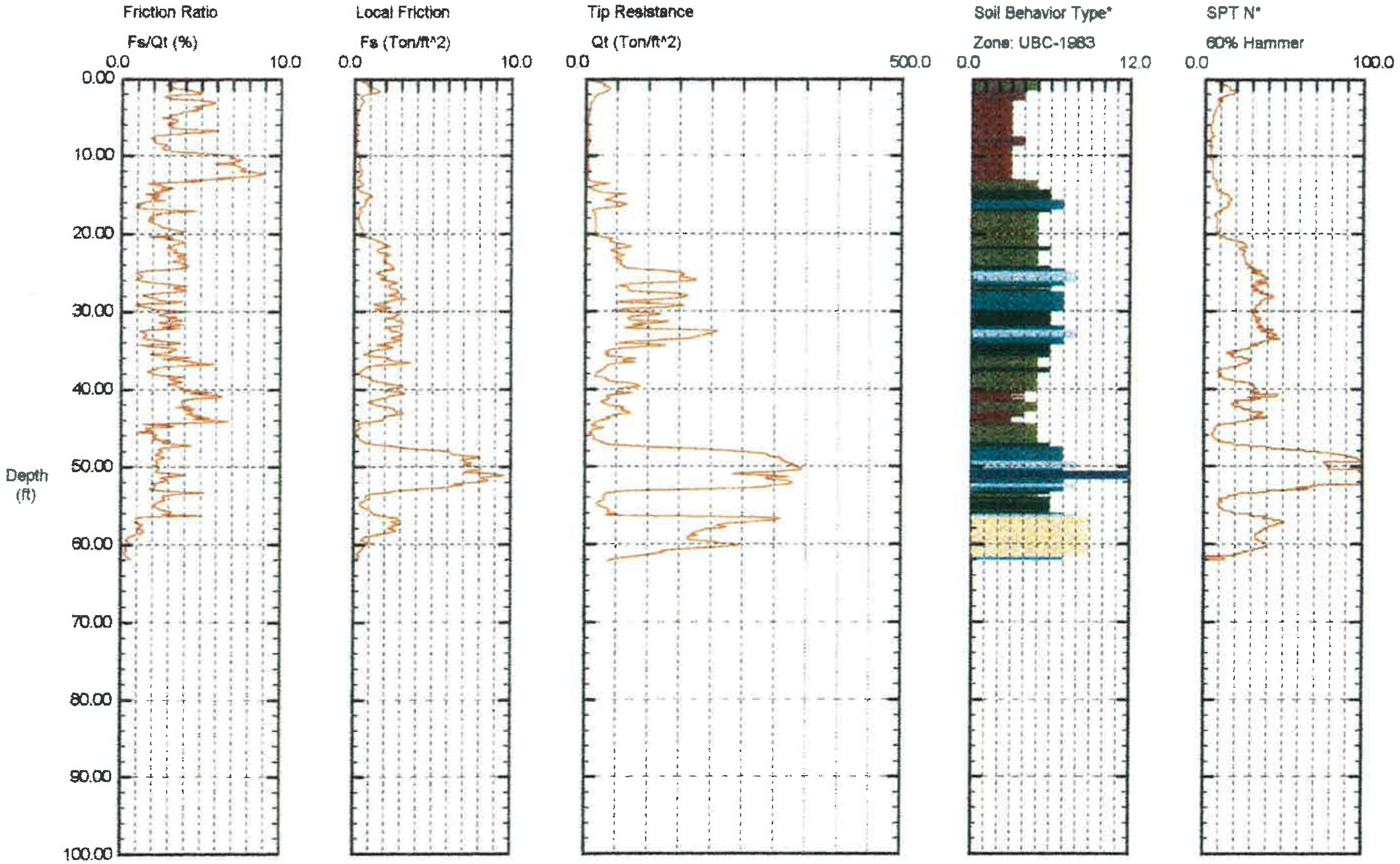
FIELD BORING LOG

behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

Operator: MIKE JONES
Sounding: 02W324
Cone Used: HO752TC-U2

CPT Date/Time: 09-25-02 09:26
Location: CPT-7
Job Number: MARTIN SLOUGH



Maximum Depth = 62.17 feet

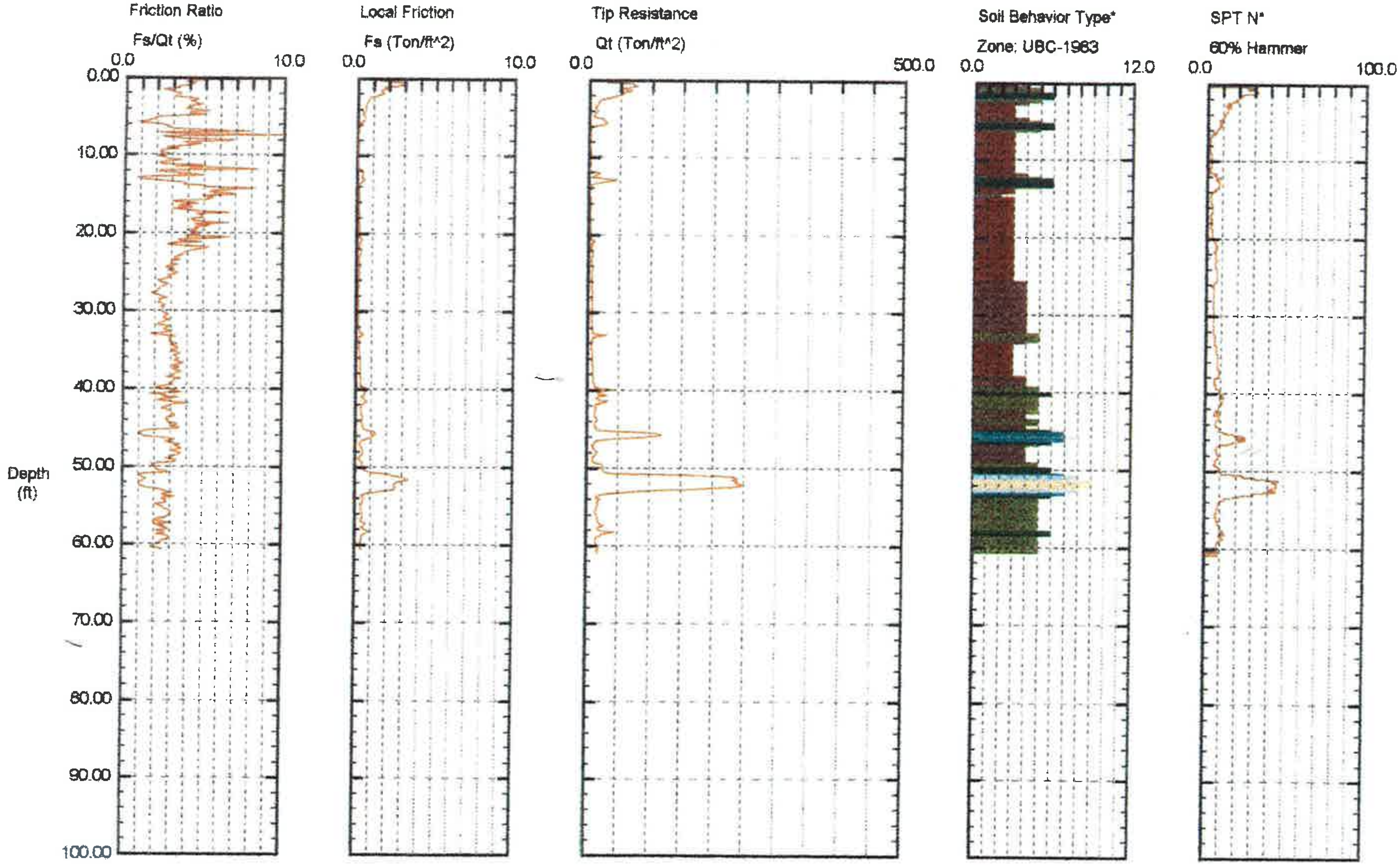
Depth Increment = 0.16 feet

⬜ 1 sensitive fine grained
⬛ 2 organic material
🟥 3 clay
🟫 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟦 7 silty sand to sandy silt
🟦 8 sand to silty sand
🟨 9 sand
🟧 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

VBI In-Situ Testing

Operator: MIKE JONES
Sounding: 02W321
Cone Used: HO752TC-U2

CPT Date/Time: 09-24-02 11:41
Location: CPT-6
Job Number: MARTIN SLOUGH



Maximum Depth = 61.02 feet

Depth Increment = 0.16 feet

■ 1 sensitive fine grained
■ 2 organic material
■ 3 clay
■ 4 silty clay to clay
■ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
■ 7 silty sand to sandy silt
■ 8 sand to silty sand
■ 9 sand
■ 10 gravelly sand to sand
■ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA 95501 ph. (707) 441-8855 fax. (707) 441-8877

PROJECT: Martin Slough
LOCATION: Pump House
GROUND SURFACE ELEVATION: --
EXCAVATION METHOD: Rotary Wash 6"
LOGGED BY: SMB

JOB NUMBER: 001283.675
DATE DRILLED: 5/21/08
TOTAL DEPTH OF BORING: 51.5 feet
SAMPLER TYPE: Shelby, SPT

BORING NUMBER BH-3

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	REMARKS
0.0													
-1.0													
-2.0													
-3.0													
-4.0 ▼													
-5.0				SM		FINE SAND, silt, with clay, loose, saturated, grey.	89.0	47.2					Shelby tube encountered wod, no return
-6.0						Gas smell.							
-7.0			4										
			4										Modified Cal. sampler driven through wood fragment
-8.0			4										
-9.0													Loss of drilling fluids due to hydraulic fracture; casing placed to 10 feet
-10.0			2	SM		FINE SAND, silty with clay, wood, loose, saturated, grey							
			3										Sample description based on shoe material
-11.0			3										Drilling fluid lost again due to hydraulic fracture; casing extended to 15 feet
			7	ML/SC		SILT, clay, with trace fine sand, and organics, medium stiff, wet, grey to							
-12.0													

The log and data presented are a simplification of actual conditions encountered at the time of drilling at the drilled location. Subsurface conditions may differ at other locations and with the passage of time.

FIELD LOG

PROJECT: Martin Slough
LOCATION: Pump House
GROUND SURFACE ELEVATION: --
EXCAVATION METHOD: Rotary Wash 6"
LOGGED BY: SMB

JOB NUMBER: 001283.675
DATE DRILLED: 5/21/08
TOTAL DEPTH OF BORING: 51.5 feet
SAMPLER TYPE: Shelby, SPT

BORING NUMBER
BH-3

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	REMARKS
-12.0			7 8			FINE SAND, clayey, with silt and organics, medium dense, wet, grey.							
-13.0													Sampled with CAT
-14.0													
-15.0			5 6 8	SC		FINE SAND, clayey, with silt, medium dense, wet, grey.							Cased to 15 feet
-16.0													
-17.0				MH		SILT, clayey, with fine sand, organics, soft, wet, dark grey.				71.3	48	22	
-18.0			Push										
-19.0													
-20.0				ML		SILT, clayey, with sand, soft to medium stiff, wet, dark gray.	33.2	87.7					Direct Shear: Peak @ 20' to 22.5' C = 2.53 psf Phi = 19.8 deg
-21.0			Push										
-22.0													
-23.0													
-24.0													

PROJECT: Martin Slough

LOCATION: Pump House

GROUND SURFACE ELEVATION: --

EXCAVATION METHOD: Rotary Wash 6"

LOGGED BY: SMB

JOB NUMBER: 001283.675

DATE DRILLED: 5/21/08

TOTAL DEPTH OF BORING: 51.5 feet

SAMPLER TYPE: Shelby, SPT

BORING NUMBER **BH-3**

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	REMARKS
-24.0													
-25.0				ML		SILT, fine sand, wilt clay, medium stiff, moist to wet, grey.	14.9	117.2	4850				
-26.0			Push										
-27.0													
-28.0													
-29.0													
-30.0			4 6	SM		FINE SAND, with silt, medium dense, moist, grey.							
-31.0			6										
-32.0													
-33.0													
-34.0													
-35.0			4 5	SC		FINE SAND, clay, silt, medium dense, moist, grey.							
-36.0													

PROJECT: Martin Slough
LOCATION: Pump House
GROUND SURFACE ELEVATION: --
EXCAVATION METHOD: Rotary Wash 6"
LOGGED BY: SMB

JOB NUMBER: 001283.675
DATE DRILLED: 5/21/08
TOTAL DEPTH OF BORING: 51.5 feet
SAMPLER TYPE: Shelby, SPT

BORING NUMBER
BH-3

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits		REMARKS
											Liquid Limit	Plastic Index	
-36.0			10										
-37.0													
-38.0													
-39.0													
-40.0			4 8 8	SM/ SP		FINE SAND, with silt, medium dense, moist, grey.							
-41.0													
-42.0													
-43.0													
-44.0													
-45.0			4 6 8	SM/ SP		FINE SAND, with silt, medium dense, moist, grey.							
-46.0													
-47.0													
-48.0													

FIELD LOG

PROJECT: Martin Slough

LOCATION: Pump House

GROUND SURFACE ELEVATION: --

EXCAVATION METHOD: Rotary Wash 6"

LOGGED BY: SMB

JOB NUMBER: 001283.675

DATE DRILLED: 5/21/08

TOTAL DEPTH OF BORING: 51.5 feet

SAMPLER TYPE: Shelby, SPT

BORING NUMBER
BH-3

DEPTH (FT)	BULK SAMPLES	SHELBY TUBE	BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Com. (psf)	% Passing 200	Atterberg Limits		REMARKS
											Liquid Limit	Plastic Index	
-48.0													
-49.0													
-50.0			12	SM		FINE SAND, with silt, medium dense, moist, grey.							
			12										
-51.0			16										
						Bottom of Boring at 51.5 feet.							
-52.0													
-53.0													
-54.0													
-55.0													
-56.0													
-57.0													
-58.0													
-59.0													
-60.0													

FIELD LOG

SHN Consulting Engineers & Geologists, Inc.
812 West Wabash, Eureka, CA ph. (707) 441-8855 fax. (707) 441-8877

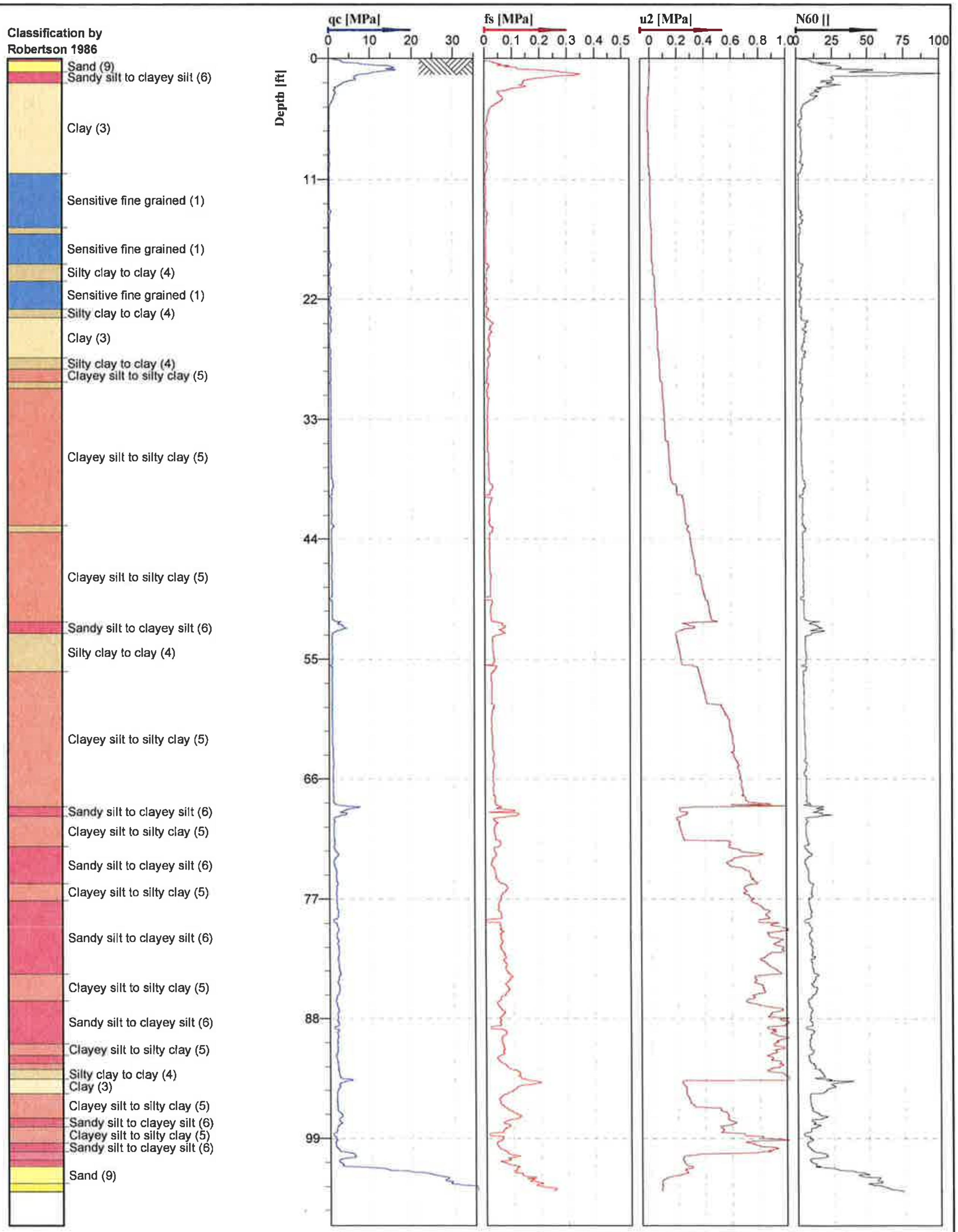
PROJECT: Martin Slough
LOCATION: Golf Course, Eureka, CA
GROUND SURFACE ELEVATION: –
EXCAVATION METHOD: Solid Stem Flight Auger (4")
LOGGED BY: SMB

JOB NUMBER: 001283.320
DATE DRILLED: 9/24/02
TOTAL DEPTH OF HOLE: 21.5 feet
SAMPLER TYPE: 2.5" I.D. Calif. Split Spoon, 140 lb telescoping hammer, 30" drop

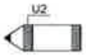
HOLE NUMBER
MS-5

DEPTH (FT)	BULK SAMPLES	SS SAMPLES	SPT BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Comp. (psf)	% Passing 200	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	REMARKS
0.0				CL		CLAY, silty, medium stiff, dry, yellow brown, with distinct mottles.							
-5.0 ▽			P 1 2 1 1 2			Becomes moist to wet.							
-10.0			1 2 4 3 7	SC		SAND, fine to medium, clayey, slightly silty, with rare organics, medium dense, wet, dark grey.	91.4	52			50.4	26	Peak @ 8.5-9.0' C = 0.44 ksf Phi = 27.2 deg. Residual @ 8.5-9.0' C = 0.15 ksf Phi = 33.9 deg.
			7	SM		SAND, fine to medium, slightly silty, medium dense, wet, dark yellowish brown.	18.8	112					
-15.0			5 10 6 1 2	SP-SM		SAND, fine to medium, slightly silty, with few organics, medium dense, wet.	122.6	42			86.4	39	SM/SP-SM Contact inferred Woody debris in sampling shoe
			2	OH		SILT, sl. clayey to clayey, sl.sandy, fine, with many organics, soft, wet, dark yellowish brown.							
-20.0			3 3	SM-SP		SAND, fine to medium, slightly silty, medium dense, wet, dark yellowish brown.							OH/SM-SP Contact inferred
			3	ML		SILT, sandy, fine, slightly clayey, with few organics, medium stiff, wet, dark yellowish brown.							
-25.0						Bottom of boring at 21.5 feet.							

FIELD BORING LOG



Fisch Drilling



Cone No: 4057
Tip area [cm2]: 10
Sleeve area [cm2]: 150

Location: Eureka, Ca	Position: X: 0.00 ft, Y: 0.00 ft	Ground level: 0.00	Test no: CPT-1
Project ID: G3904	Client: SHN	Date: 9/28/2012	Scale: 1 : 150
Project: Pine Hill Bridge		Page: 1/1	Fig:
		File: shn pine hill1.cpt	

Project: Pine Hill Road Bridge Replacement
Project Location:
Project Number: 012163

Log of Boring B-1
Sheet 1 of 1

Date(s) Drilled	10/16/12	Logged By	JHD	Checked By	
Drilling Method	Hollow-stem auger	Drill Bit Size/Type		Total Depth of Borehole	90.5 Feet
Drill Rig Type	0-40' Rotary-wash 40-91'	Drilling Contractor	Taber Drilling	Approximate Surface Elevation	
Groundwater Level and Date Measured		Sampling Method(s)	Shelby Tube	Hammer Data	Automatic
Borehole Backfill	Cement grout	Location	SE corner of bridge 17' east of CPT 1		

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
	0				SP		Brown GRAVELLY SAND medium dense, moist	
					CL/OL		Blue Gray Lean CLAY, very soft, wet to saturated, minor sand, minor organics	
	5		NR	2 2 2				
	10		NR	P				
			SHELBY					consol TXCU $w=31.4\%$ $\gamma_d=89pcf$
	15							
	20						minor decomposing organics	TXCU LL= 32, PI = 12 $w=34.6\%$ $\gamma_d= 85pcf$
	25							
	30							

Project:
Project Location: SEE PAGE 1
Project Number: 012163

Log of Boring B-1
Sheet 2 of ~~1~~ 3

Date(s) Drilled		Logged By		Checked By	
Drilling Method		Drill Bit Size/Type		Total Depth of Borehole	
Drill Rig Type		Drilling Contractor		Approximate Surface Elevation	
Groundwater Level and Date Measured		Sampling Method(s)		Hammer Data	
Borehole Backfill		Location			

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
	30				MH/OH		Grey CLAYEY SILT, soft, saturated, minor organics and clam shells	Consol TXCU W= 37.1% γ_d= 82pcf
	35							
	40				MH/OH		Grey CLAYEY SILT, soft to medium stiff, saturated, minor sand and organics	txcu LL=53, PI= 24 W= 39.7% γ_d= 80pcf
	45							
							Brownish-gray SILTY SAND medium dense, saturated, few organics and shell fragments	
	50				SM			TXCU W= 42.3% γ_d= 77psf
					MH/OH		Gray CLAYEY SILT, soft to medium stiff, saturated, few organics and shell fragments	
	55							
					SM		Brownish-gray SILTY SAND medium dense, saturated	
			4				layer of decomposing roots w/ Few rounded gravels to 1/2"	
	60							

Project:	SEE PAGE 1	**Log of Boring** B-1
Project Location:		**Sheet** ~~1~~ **of** ~~1~~ 3 of 3
Project Number: 012163		

Date(s) Drilled		Logged By		Checked By	
Drilling Method		Drill Bit Size/Type		Total Depth of Borehole	
Drill Rig Type		Drilling Contractor		Approximate Surface Elevation	
Groundwater Level and Date Measured		Sampling Method(s)		Hammer Data	
Borehole Backfill		Location			

Elevation (feet)	Depth (feet)	Sample Type	Sample Number	Sampling Resistance, blows/ft	USCS Symbol	Graphic Log	MATERIAL DESCRIPTION	REMARKS AND OTHER TESTS
	60	▨		6 6	CL		layer of Reddish-brown SANDY CLAY w/ few gravels and organics Grey SILTY CLAY soft to medium stiff, saturated	UC w= 50.8% γ_d= 69 pcf
	65							
	70	▨	NR	4 6 9	SC		Grey CLAYEY SAND medium dense, saturated, Few organics	Pushed another sample w/ catcher -200 = 45.4%
	75							
					CL		Grey SILTY CLAY very stiff, saturated	
	80	▨		11 18 30	SC		Grey CLAYEY SAND dense, saturated	collected sample using catcher -200 = 27.8%
	85							
	90	▨		35 50/4"	SP		SAND, very dense, saturated, medium-grained BOH 90½'	collected sample using catcher -200 = 12.9%

Appendix B

ASTM Laboratory Test Results

CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name:	Martin Slough Enhancement	**Project Number:**	**013035**
Performed By:	**JMA**	**Date:**	**4/9/2013**
Checked By:	DL	**Date:**	4/16/13
Project Manager:	**JPB**		

Lab Sample Number	13-240	13-241	13-242		
Boring Label	HB9	HB10	HB12		
Sample Depth (ft)	11-11.5	4.5-4.8	2-2.5		
Diameter of Cylinder, in	2.38		2.38		
Total Length of Cylinder, in.	7.45		7.95		
Length of Empty Cylinder A, in.	0.00	disturbed	0.00		
Length of Empty Cylinder B, in.	4.70	sample	5.10		
Length of Cylinder Filled, in	2.75		2.85		
Volume of Sample, in^3	12.23		12.68		
Volume of Sample, cc.	200.48		207.77		
Pan #	s29	ss7	s26		
Weight of Wet Soil and Pan	509.9	477.9	521.1		
Weight of Dry Soil and Pan	405.4	421.5	416.2		
Weight of Water	104.5	56.4	104.9		
Weight of Pan	148.6	193.0	165.5		
Weight of Dry Soil	256.8	228.5	250.7		
Percent Moisture	**40.7**	**24.7**	**41.8**		
Dry Density, g/cc	1.28		1.21		
Dry Density, lb/ft^3	**80.0**		**75.3**		
Shrinkage Percentage	2.5		1		

SHN CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

DENSITY BY DRIVE- CYLINDER METHOD (ASTM D2937)

Project Name: Martin Slough Enhancement **Project Number:** 013035
Performed By: JMA **Date:** 4/9/2013
Checked By: **Date:** 4/16/13
Project Manager: JPB

Lab Sample Number	13-226	13-228	13-232	13-237	13-238
Boring Label	HB1	HB1	HB3	HB9	HB9
Sample Depth (ft)	7.5-8.0	15-15.5	10-10.5	7-7.5	5.5-6
Diameter of Cylinder, in	2.38	2.38	2.38	2.38	2.38
Total Length of Cylinder, in.	7.93	9.70	7.90	7.92	7.95
Length of Empty Cylinder A, in.	0.00	0.00	0.00	4.90	4.73
Length of Empty Cylinder B, in.	4.52	7.33	2.32	0.38	0.00
Length of Cylinder Filled, in	3.41	2.37	5.58	2.64	3.22
Volume of Sample, in^3	15.17	10.54	24.82	11.74	14.33
Volume of Sample, cc.	248.60	172.78	406.80	192.46	234.75
Pan #	s22	s27	s22	s27	ss12
Weight of Wet Soil and Pan	616.1	502.5	844.3	537.9	609.9
Weight of Dry Soil and Pan	495.7	438.6	694.2	460.2	495.5
Weight of Water	120.4	63.9	150.1	77.7	114.4
Weight of Pan	151.2	152.7	151.3	152.9	194.4
Weight of Dry Soil	344.5	285.9	542.9	307.3	301.1
Percent Moisture	**34.9**	**22.4**	**27.6**	**25.3**	**38.0**
Dry Density, g/cc	1.39	1.65	1.33	1.60	1.28
Dry Density, lb/ft^3	**86.5**	**103.3**	**83.3**	**99.7**	**80.1**
Shrinkage Percentage	2.9	0	3.7	14	8.4

SHN
CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

Moisture Content (ASTM D 2216)

Job Name: Martin Slough Enhancment
Job Number: 013035
Performed By: JMA
Date: 4/12/2013
Checked By: Dh
Date: 4/16/13
Project Manager: JPB

Lab Sample Number	13-224	13-225	13-249	13-255	13-261
Job Sample Number	HB15 @2.8	HB15@ 6.5	HB1@ 6'	HB2 @ 2-3	HB5 @ 4-4.5
A. Pan #	a7	a8	a5	a3	a9
B. Weight of Wet Soil and Pan	241.1	240.6	266.3	266.2	262.1
C. Weight of Dry Soil and Pan	194.2	186.4	227.0	229.0	219.5
D. Weight of Water	46.9	54.2	39.3	37.2	42.6
E. Weight of Pan	86.7	87.5	86.9	85.3	88.9
F. Weight of Dry Soil	107.5	98.9	140.1	143.7	130.6
G. Percent Moisture (D/F)	43.6	54.8	28.1	25.9	32.6

SHRINKAGE CALCULATIONS

Original Dia 2.42"	2.12	2.07	2.31	2.31	2.20
Original Height 1.00"	1.06	0.86	0.99	0.97	0.94
Percent Shrinkage DIA	12.4	14.5	4.5	4.5	9.1
Percent Shrinkage Height	-6.0	14.0	1.0	3.0	6.0

SHN
CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

Moisture Content (ASTM D 2216)

Job Name: Martin Slough Enhancment
Performed By: JMA
Checked By: Dh
Project Manager: JPB

Job Number: 013035
Date: 4/12/2013
Date: 4/16/13

Lab Sample Number	13-281	13-284	13-286	13-290	
Job Sample Number	HB11 @ 4.2-4.	HB13 @ 2-2.	HB13 @ 6-6.5	HB14 @ 4-4.5	
A. Pan #	s25	s26	s8	s29	
B. Weight of Wet Soil and Pan	302.3	338.5	343.7	329.7	
C. Weight of Dry Soil and Pan	256.4	297.9	309.4	290.8	
D. Weight of Water	45.9	40.6	34.3	38.9	
E. Weight of Pan	146.2	165.9	161.2	148.6	
F. Weight of Dry Soil	110.2	132.0	148.2	142.2	
G. Percent Moisture (D/F)	41.7	30.8	23.1	27.4	

SHRINKAGE CALCULATIONS

Original Dia 2.42"	2.13	2.31	2.30	2.24	
Original Height 1.00	0.89	0.98	0.99	0.98	
Percent Shrinkage DIA	12.0	4.5	5.0	7.4	
Percent Shrinkage Height	11.0	2.0	1.0	2.0	

SHN
CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	**Martin Slough Enhancement**	**Project Number:**	**013035**
Performed By:	**JMA**	**Date:**	**4/15/2013**
Checked By:		**Date:**	4/16/13
Project Manager:	**JPB**		

Lab Sample Number	13-228	13-230	13-238	13-239	13-241
Boring Label	HB1	HB3	HB9	HB9	HB10
Sample Depth (ft)	15-15.5	5-5.5	5.5-6.0	8-8.5	4.5-4.8
Pan Number	ss15	ss11	ss12	ss8	ss7
Dry Weight of Soil & Pan	295.8	303.5	284.2	317.8	300.2
Pan Weight	194.4	192.8	194.4	193.0	193.0
Weight of Dry Soil	101.4	110.7	89.8	124.8	107.2
Soil Weight Retained on #200&Pan	271.0	216.0	216.3	275.7	267.9
Soil Weight Passing #200	24.8	87.5	67.9	42.1	32.3
Percent Passing #200	**24.5**	**79.0**	**75.6**	**33.7**	**30.1**

Lab Sample Number	13-268				
Boring Label	HB6				
Sample Depth (ft)	6.5-7				
Pan Number	ss3				
Dry Weight of Soil & Pan	375.2				
Pan Weight	197.2				
Weight of Dry Soil	178.0				
Soil Weight Retained on #200&Pan	324.7				
Soil Weight Passing #200	50.5				
Percent Passing #200	**28.4**				

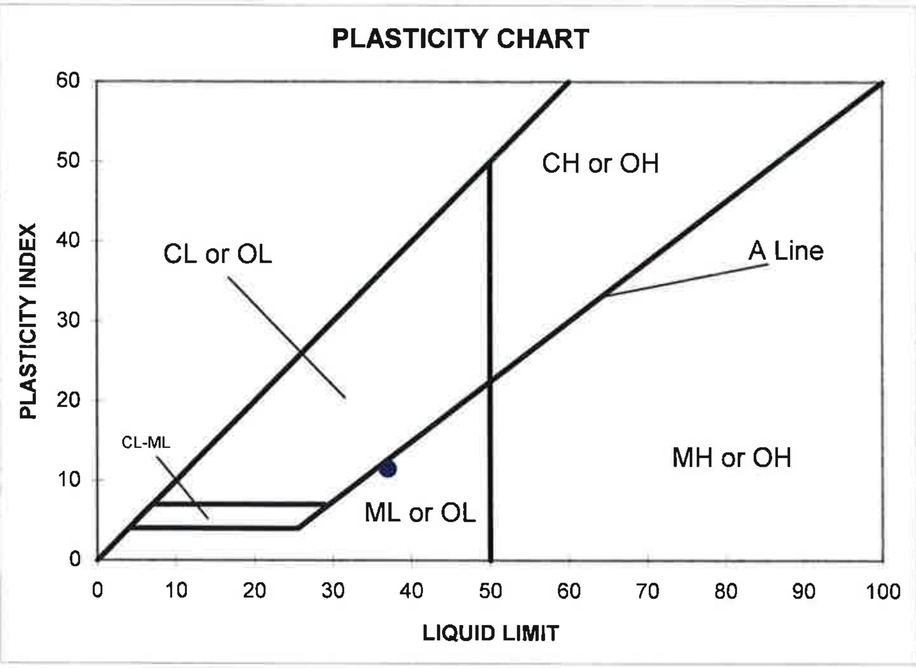
SHN CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

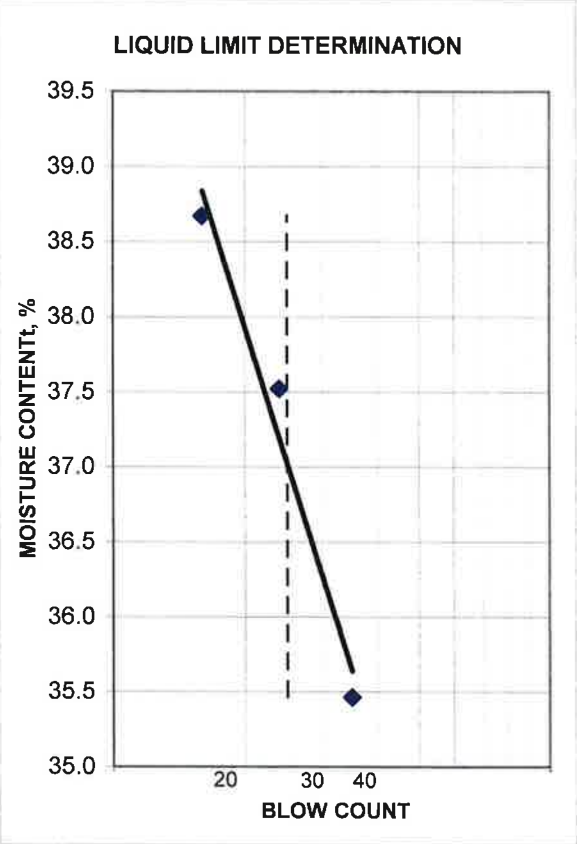
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME: **Martin Slough Enhancement** JOB #: **013035** LAB SAMPLE #: **13-226**
SAMPLE ID: **HB1 @ 7.5-8.0** PERFORMED BY: **JMA** DATE: **4/15/2013**
PROJECT MANGER: **JPB** CHECKED BY: Dh DATE: 4/16/13

LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	13	14	7	8	9
B	PAN WT. (g)	22.170	19.950	29.010	29.180	28.740
C	WT. WET SOIL & PAN (g)	28.640	25.950	35.580	37.060	36.270
D	WT. DRY SOIL & PAN (g)	27.330	24.730	33.860	34.910	34.170
E	WT. WATER (C-D)	1.310	1.220	1.720	2.150	2.100
F	WT. DRY SOIL (D-B)	5.160	4.780	4.850	5.730	5.430
G	BLOW COUNT	--	--	35	24	16
H	MOISTURE CONTENT (E/F*100)	25.4	25.5	35.5	37.5	38.7

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
37	**12**	25





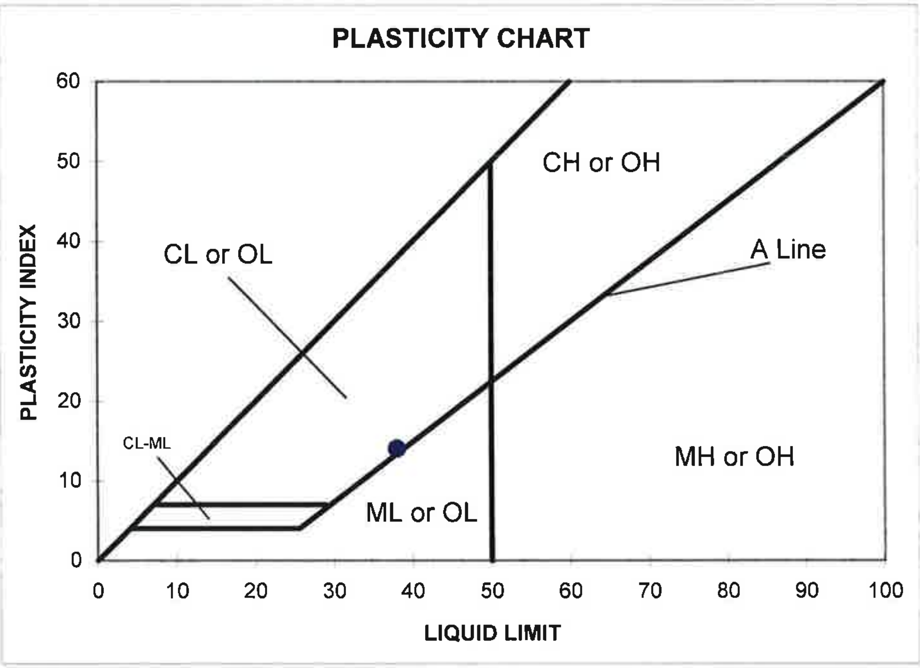
SHN CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

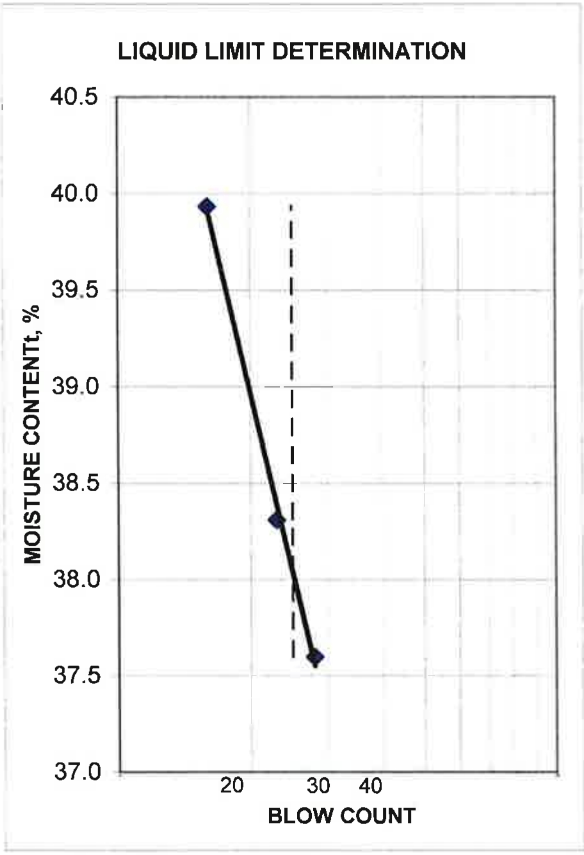
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME: **Martin Slough Enhancement** JOB #: **013035** LAB SAMPLE #: **13-236**
SAMPLE ID: **HB9 @ 2.5-3.0** PERFORMED BY: **JMA** DATE: 4/15/2013
PROJECT MANGER: **JPB** CHECKED BY: Dh DATE: 4/16/13

LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	22	23	a	b	c
B	PAN WT. (g)	17.230	16.960	29.360	29.610	28.700
C	WT. WET SOIL & PAN (g)	23.230	23.480	38.070	37.300	37.320
D	WT. DRY SOIL & PAN (g)	22.070	22.220	35.690	35.170	34.860
E	WT. WATER (C-D)	1.160	1.260	2.380	2.130	2.460
F	WT. DRY SOIL (D-B)	4.840	5.260	6.330	5.560	6.160
G	BLOW COUNT	--	--	28	23	16
H	MOISTURE CONTENT (E/F*100)	24.0	24.0	37.6	38.3	39.9

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
38	**14**	24





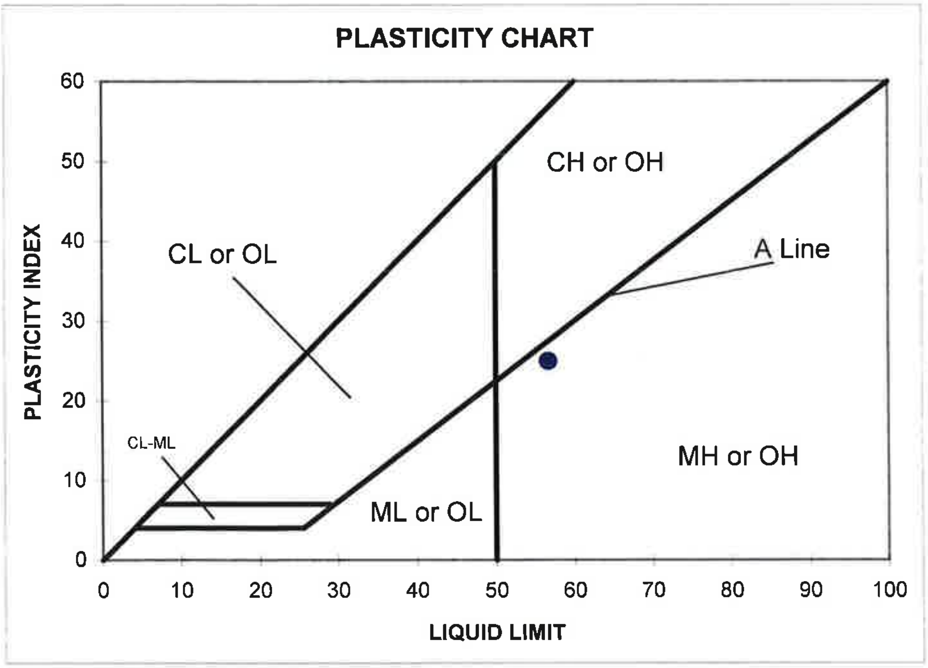
CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

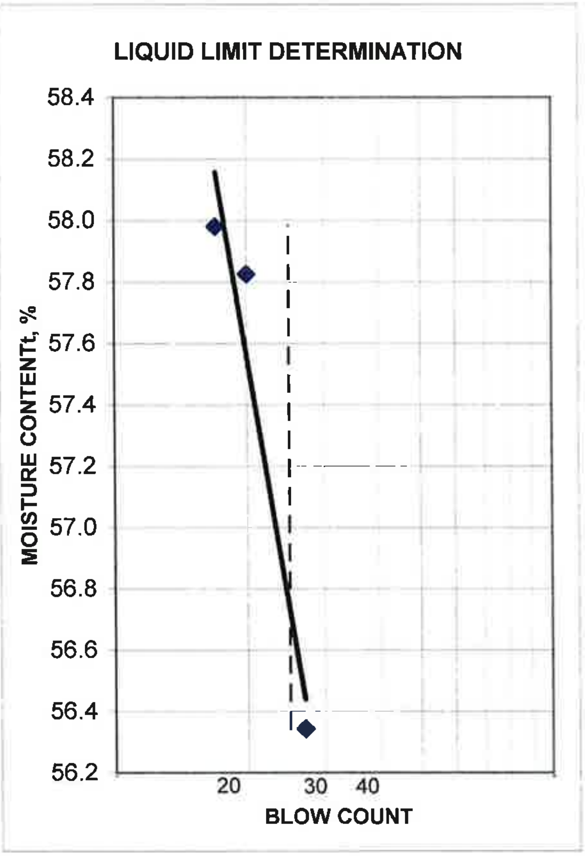
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME:	Martin Slough Enhancement	JOB #:	13035	LAB SAMPLE #:	13-239
SAMPLE ID:	HB9 @ 8.5-9	PERFORMED BY:	JMA	DATE:	4/15/2013
PROJECT MANGER:	JPB	CHECKED BY:	Sh	DATE:	4/16/13

LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	17	18	1	2	3
B	PAN WT. (g)	20.300	20.260	29.830	29.130	29.180
C	WT. WET SOIL & PAN (g)	26.840	27.310	37.100	36.390	35.910
D	WT. DRY SOIL & PAN (g)	25.260	25.620	34.480	33.730	33.440
E	WT. WATER (C-D)	1.580	1.690	2.620	2.660	2.470
F	WT. DRY SOIL (D-B)	4.960	5.360	4.650	4.600	4.260
G	BLOW COUNT	--	--	27	20	17
H	MOISTURE CONTENT (E/F*100)	31.9	31.5	56.3	57.8	58.0

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
57	**25**	32





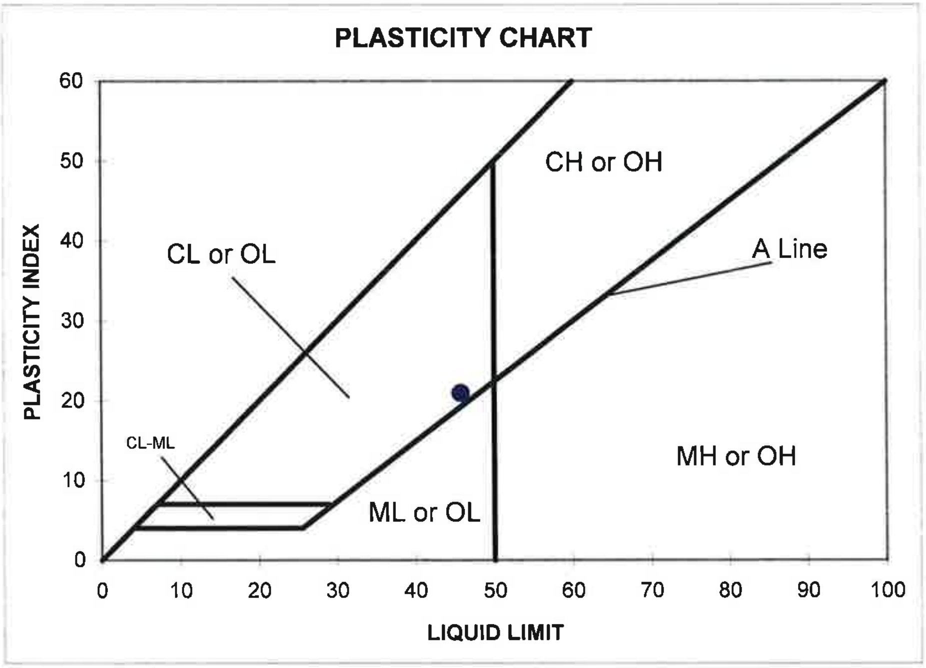
SHN CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

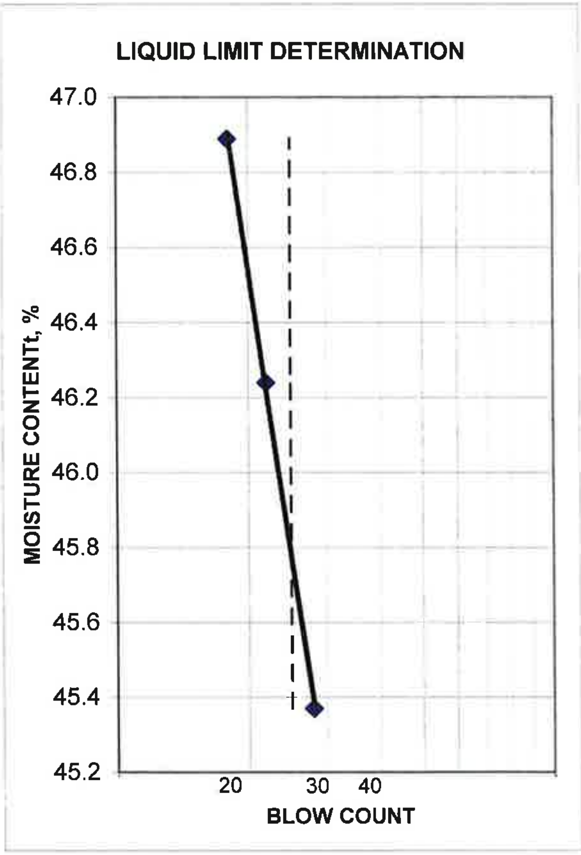
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME: **Martin Slough Enhancement** JOB #: **013035** LAB SAMPLE #: **13-240**
SAMPLE ID: **HB9 @ 11-11.5** PERFORMED BY: **JMA** DATE: 4/15/2013
PROJECT MANGER: **JPB** CHECKED BY: Dh DATE: 4/16/13

LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	23	22	A	B	C
B	PAN WT. (g)	16.960	17.220	29.360	29.610	28.730
C	WT. WET SOIL & PAN (g)	23.050	24.350	38.780	37.390	37.940
D	WT. DRY SOIL & PAN (g)	21.820	22.900	35.840	34.930	35.000
E	WT. WATER (C-D)	1.230	1.450	2.940	2.460	2.940
F	WT. DRY SOIL (D-B)	4.860	5.680	6.480	5.320	6.270
G	BLOW COUNT	--	--	28	22	18
H	MOISTURE CONTENT (E/F*100)	25.3	25.5	45.4	46.2	46.9

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
46	**21**	25





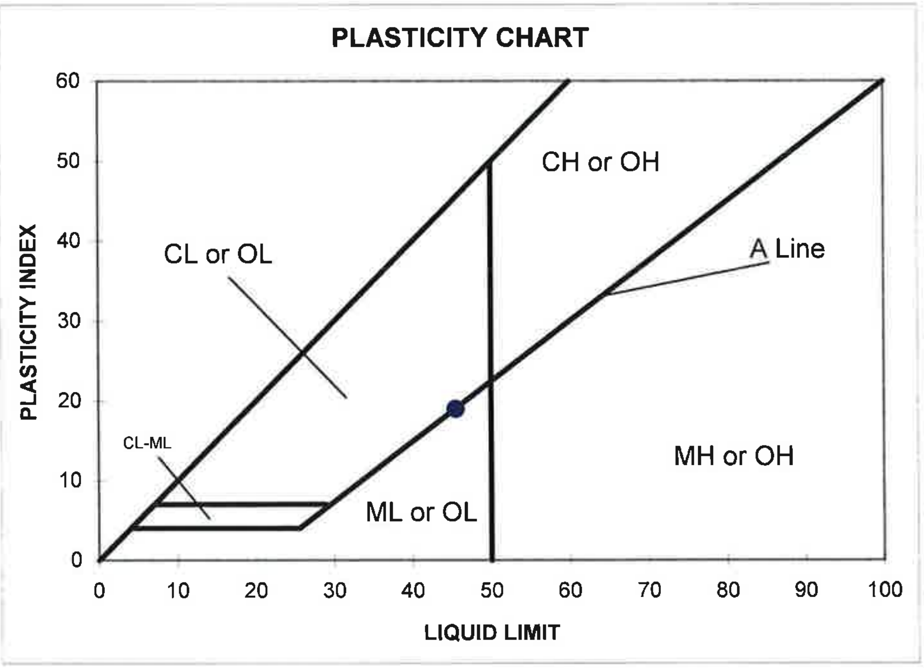
CONSULTING ENGINEERS & GEOLOGISTS, INC.
812 W. Wabash Eureka, CA 95501-2138 Tel: 707/441-8855 FAX: 707/441-8877 E-mail: shninfo@shn-engr.com

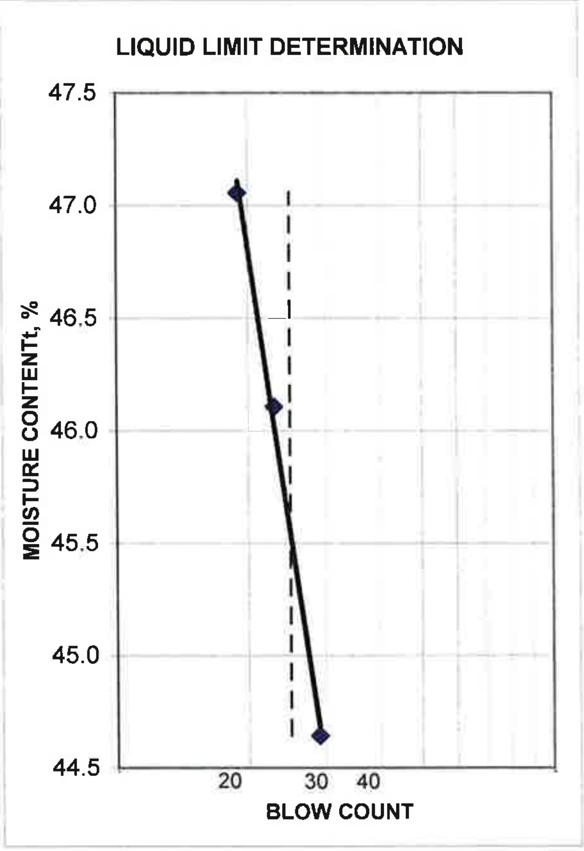
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

JOB NAME: **Martin Slough Enhancement** JOB #: **013035** LAB SAMPLE #: **13-243**
SAMPLE ID: **HB 12 @ 5-5.5** PERFORMED BY: **JMA** DATE: 4/16/2013
PROJECT MANGER: **JPB** CHECKED BY: DATE: 4/16/13

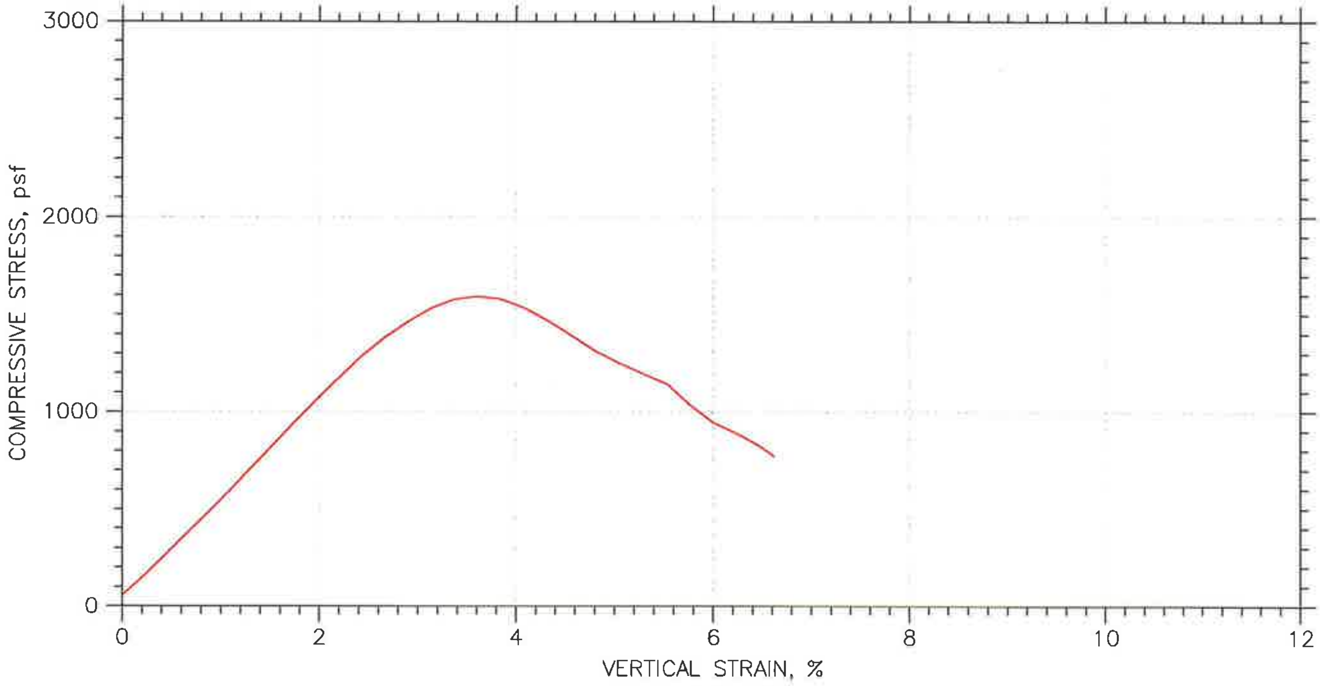
LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	13	14	7	8	9
B	PAN WT. (g)	22.170	19.950	29.000	29.140	28.710
C	WT. WET SOIL & PAN (g)	31.390	26.720	36.290	36.460	34.710
D	WT. DRY SOIL & PAN (g)	29.460	25.300	34.040	34.150	32.790
E	WT. WATER (C-D)	1.930	1.420	2.250	2.310	1.920
F	WT. DRY SOIL (D-B)	7.290	5.350	5.040	5.010	4.080
G	BLOW COUNT	--	--	29	23	19
H	MOISTURE CONTENT (E/F*100)	26.5	26.5	44.6	46.1	47.1

LIQUID LIMIT	**PLASTIC INDEX**	PLASTIC LIMIT
46	**19**	27





UNCONFINED COMPRESSION TEST REPORT



	Symbol				
	Test No.	13-244			
Initial	Diameter, in	2.38			
	Height, in	5.39			
	Water Content, %	78.04			
	Dry Density, pcf	51.309			
	Saturation, %	92.98			
	Void Ratio	2.2243			
	Unconfined Compressive Strength, psf	1595.6			
	Undrained Shear Strength, psf	797.81			
	Time to Failure, min	3.7539			
	Strain Rate, %/min	1			
	Specific Gravity	2.65			
	Liquid Limit	0			
	Plastic Limit	0			
	Plasticity Index	0			
	Failure Sketch				

Project: Martin Slough Enhancement

Location: Eureka

Project No.: 013035

Boring No.: HB15@2

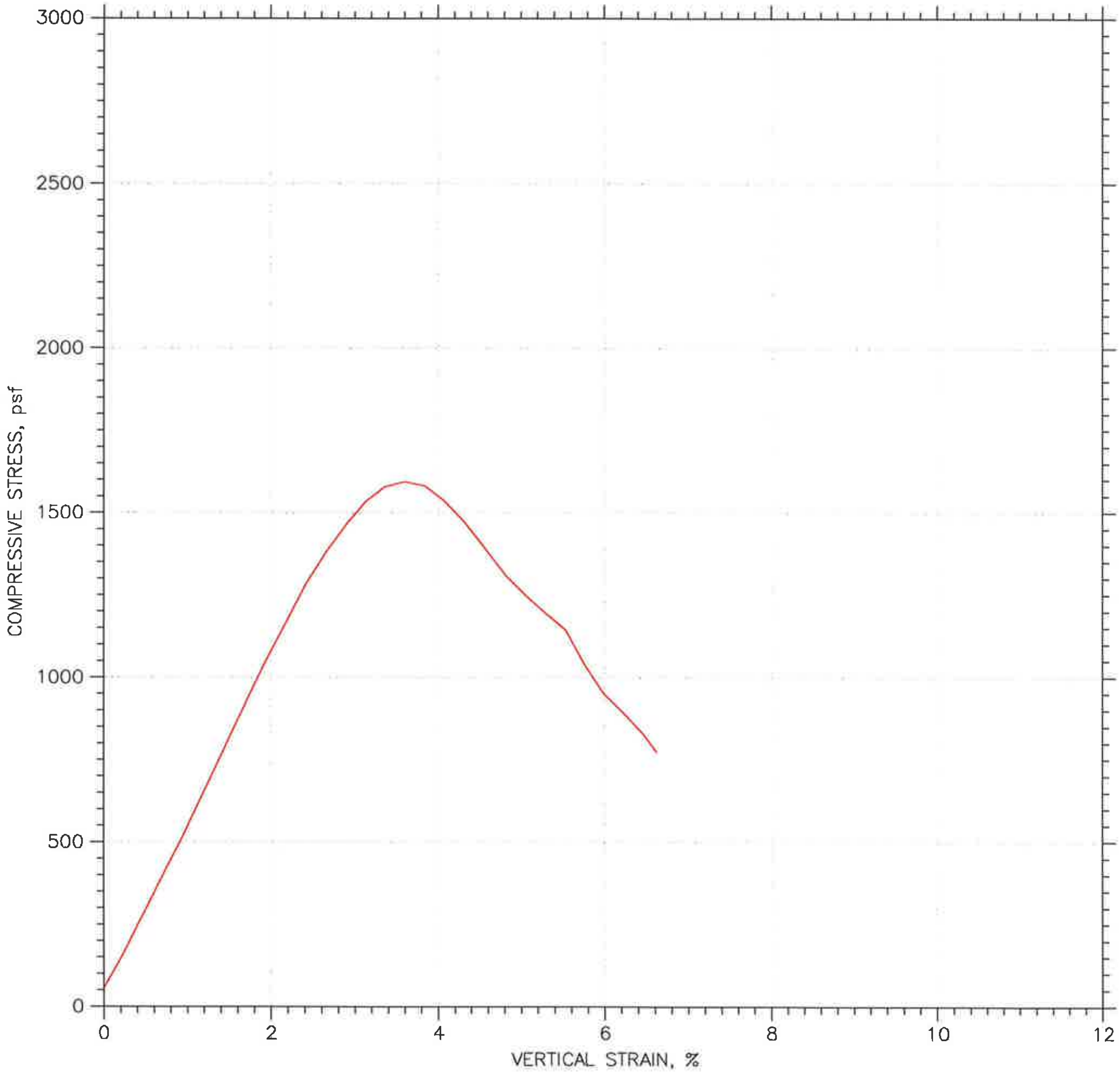
Sample Type: 2.5"shelby

Description: Strong Brown SILT

Remarks: Organics in specimen

Wed, 10-APR-2013 14:15:46

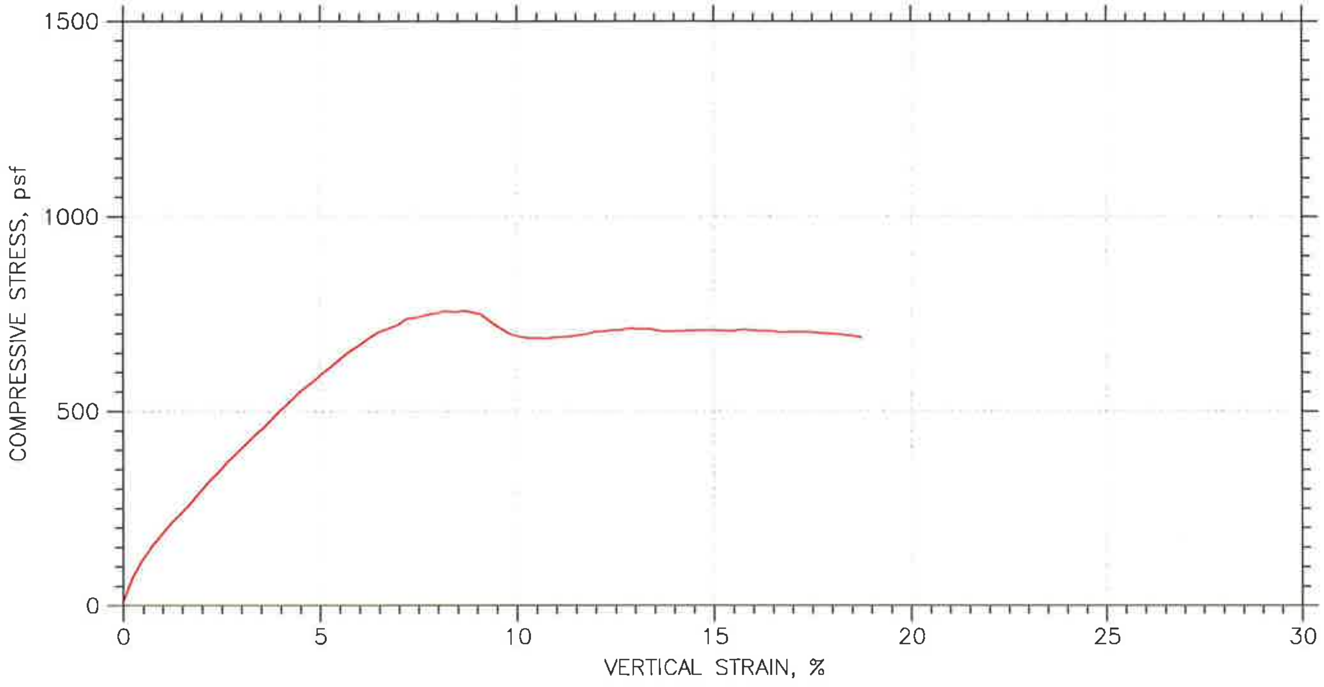
UNCONFINED COMPRESSION TEST REPORT



Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB15@2	Tested By: JMA	Checked By: DL 4/18/13
Sample No.: 13-244	Test Date: 4/9/12	Depth: 2-2.5
Test No.: 13-244	Sample Type: 2.5"shelby	Elevation:
Description: Strong Brown SILT		
Remarks: Organics in specimen		

Wed, 10-APR-2013 14:15:46

UNCONFINED COMPRESSION TEST REPORT

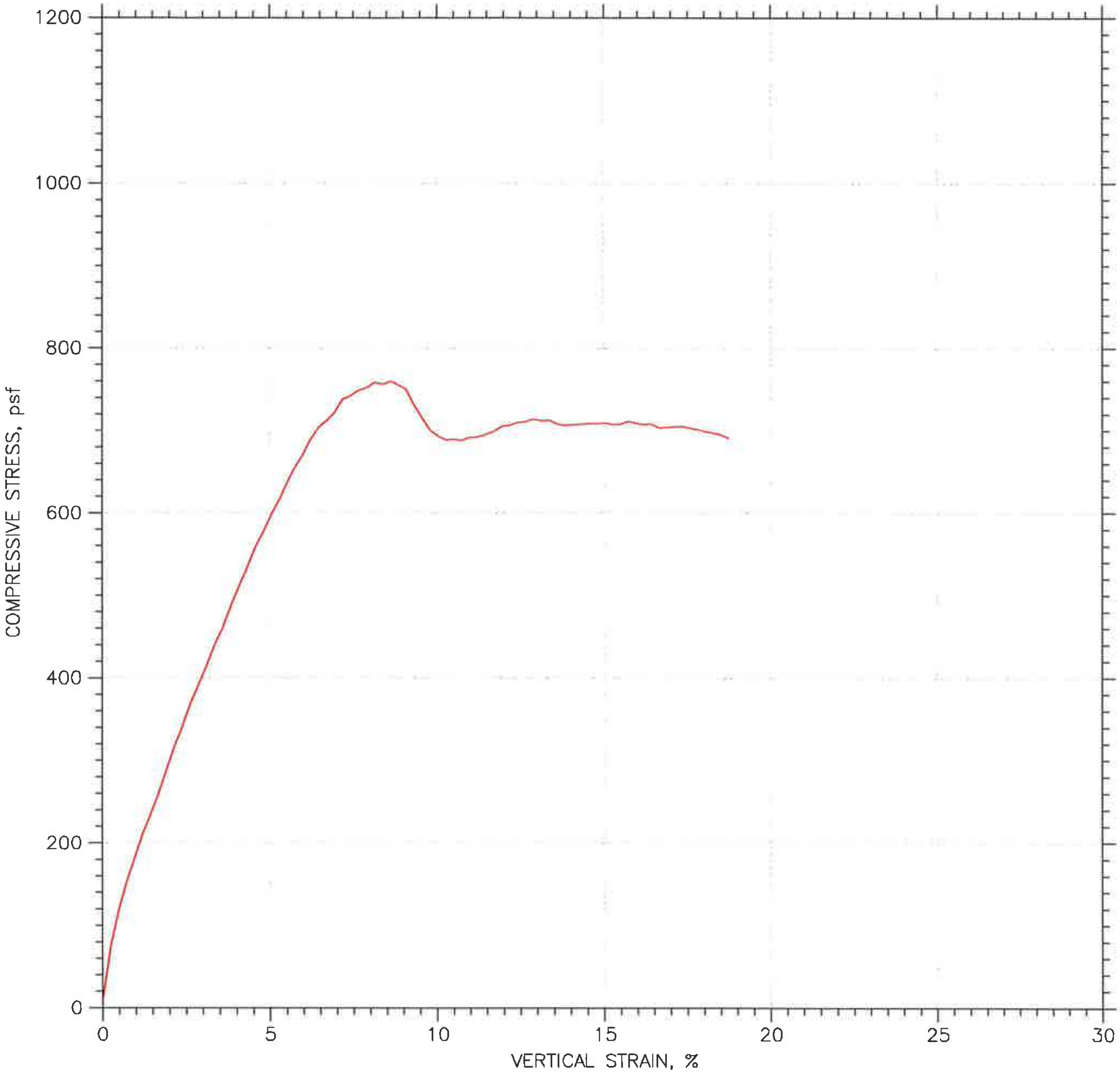


Symbol					
Test No.		13-234			
Initial	Diameter, in	2.38			
	Height, in	5.8			
	Water Content, %	66.67			
	Dry Density, pcf	58.507			
	Saturation, %	96.68			
	Void Ratio	1.8276			
Unconfined Compressive Strength, psf		760.18			
Undrained Shear Strength, psf		380.09			
Time to Failure, min		9.0011			
Strain Rate, %/min		1			
Specific Gravity		2.65			
Liquid Limit		0			
Plastic Limit		0			
Plasticity Index		0			
Failure Sketch					

Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB4@6.1
Sample Type: 2.5''shelby
Description: Gray SILT
Remarks:

Thu, 18-APR-2013 12:21:05

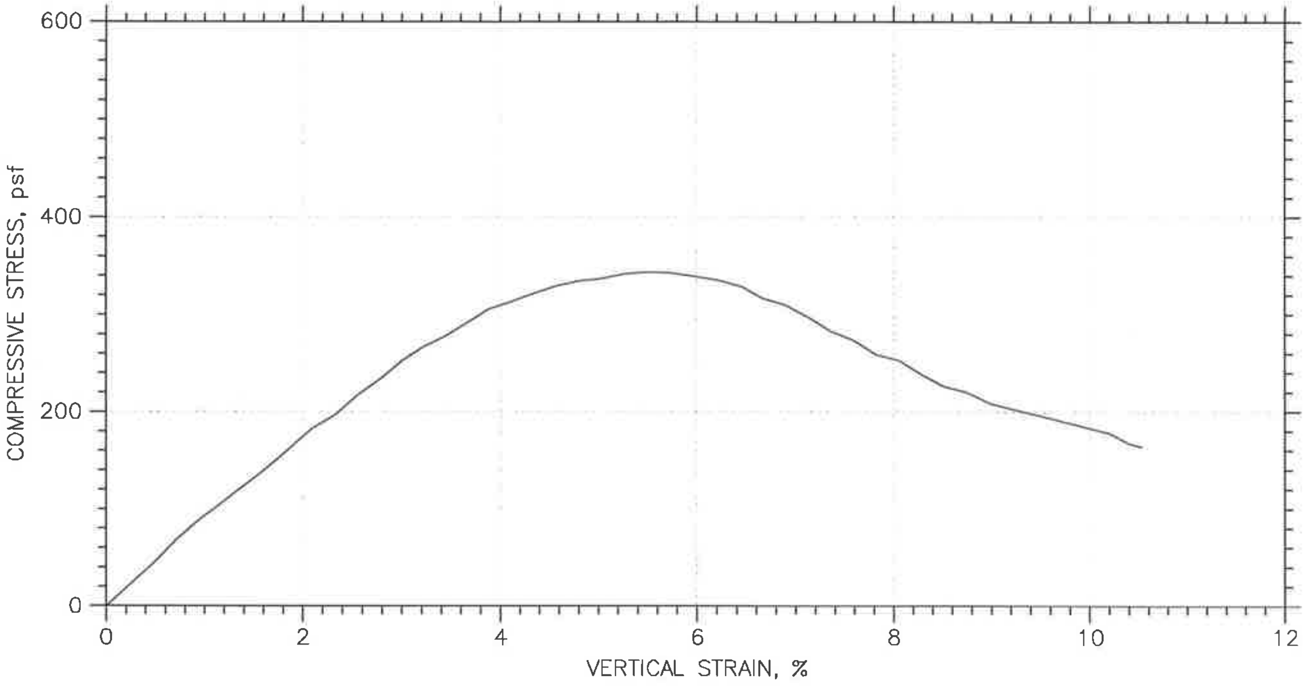
UNCONFINED COMPRESSION TEST REPORT



Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB4@6.1	Tested By: JMA	Checked By: DL 4/18/13
Sample No.: 13-234	Test Date: 4/9/12	Depth: 6.1-6.6
Test No.: 13-234	Sample Type: 2.5''shelby	Elevation:
Description: Gray SILT		
Remarks:		

Thu, 18-APR-2013 12:21:05

UNCONFINED COMPRESSION TEST REPORT

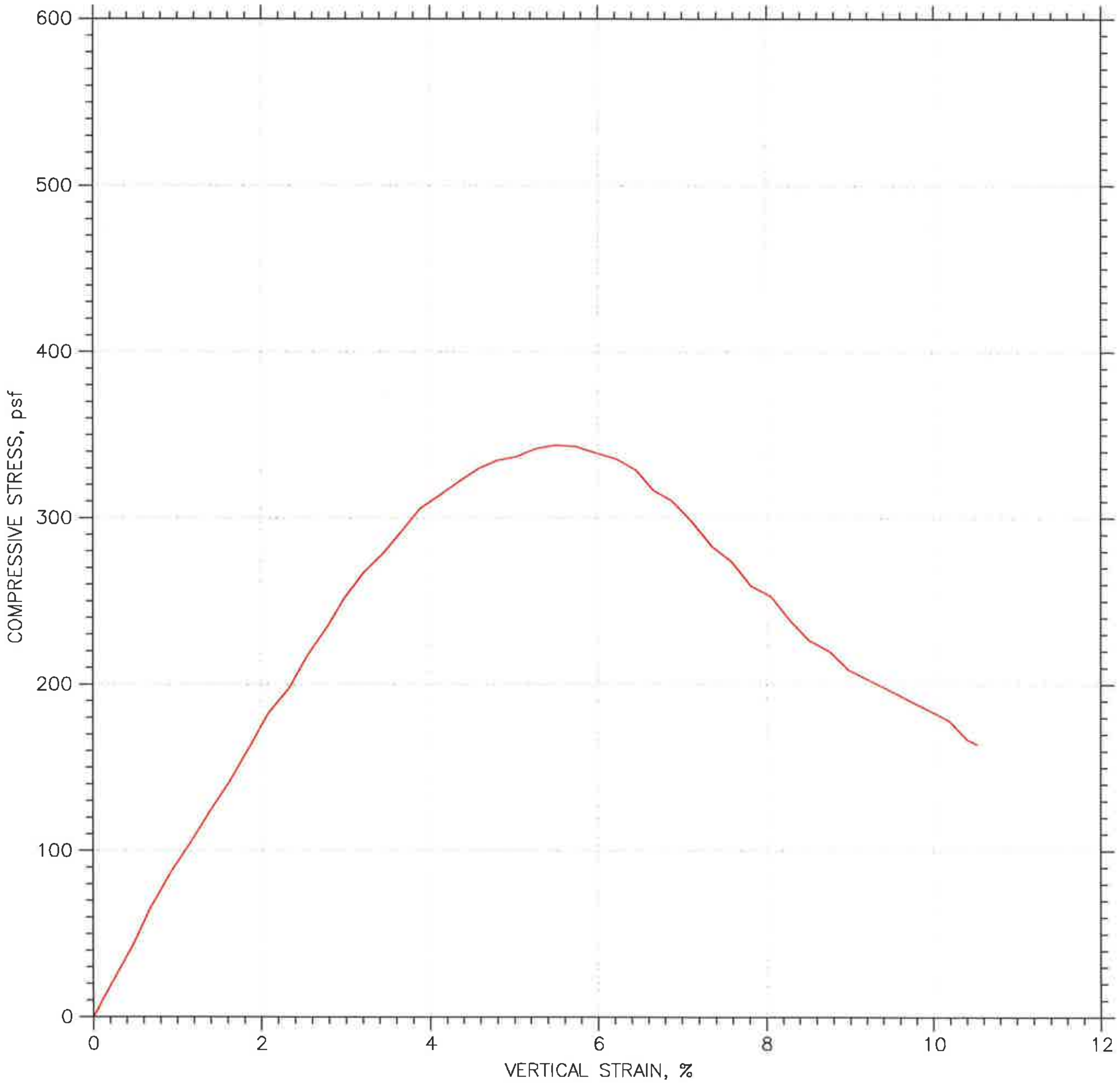


Symbol					
Test No.		13-239			
Initial	Diameter, in	2.38			
	Height, in	5.09			
	Water Content, %	33.63			
	Dry Density, pcf	86.153			
	Saturation, %	96.83			
	Void Ratio	0.92023			
Unconfined Compressive Strength, psf		344.04			
Undrained Shear Strength, psf		172.02			
Time to Failure, min		6.0021			
Strain Rate, %/min		1			
Specific Gravity		2.65			
Liquid Limit		0			
Plastic Limit		0			
Plasticity Index		0			
Failure Sketch					

Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB9@8.5
Sample Type: 2.5"shelby
Description: Strong Brown SILT
Remarks:

Wed, 10-APR-2013 14:05:57

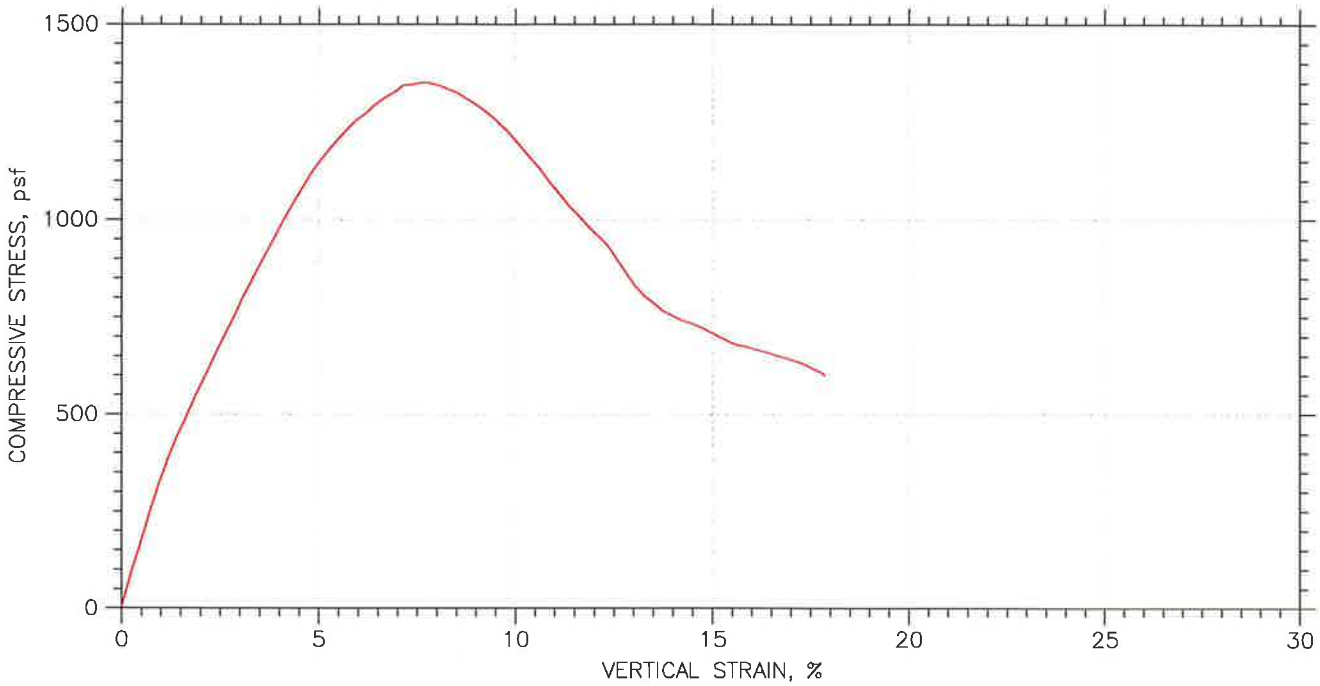
UNCONFINED COMPRESSION TEST REPORT



Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB9@8.5	Tested By: JMA	Checked By: DL 4/18/13
Sample No.: 13-239	Test Date: 4/9/12	Depth: 8.5-9.0
Test No.: 13-239	Sample Type: 2.5''shelby	Elevation:
Description: Strong Brown SILT		
Remarks:		

Wed, 10-APR-2013 14:05:58

UNCONFINED COMPRESSION TEST REPORT

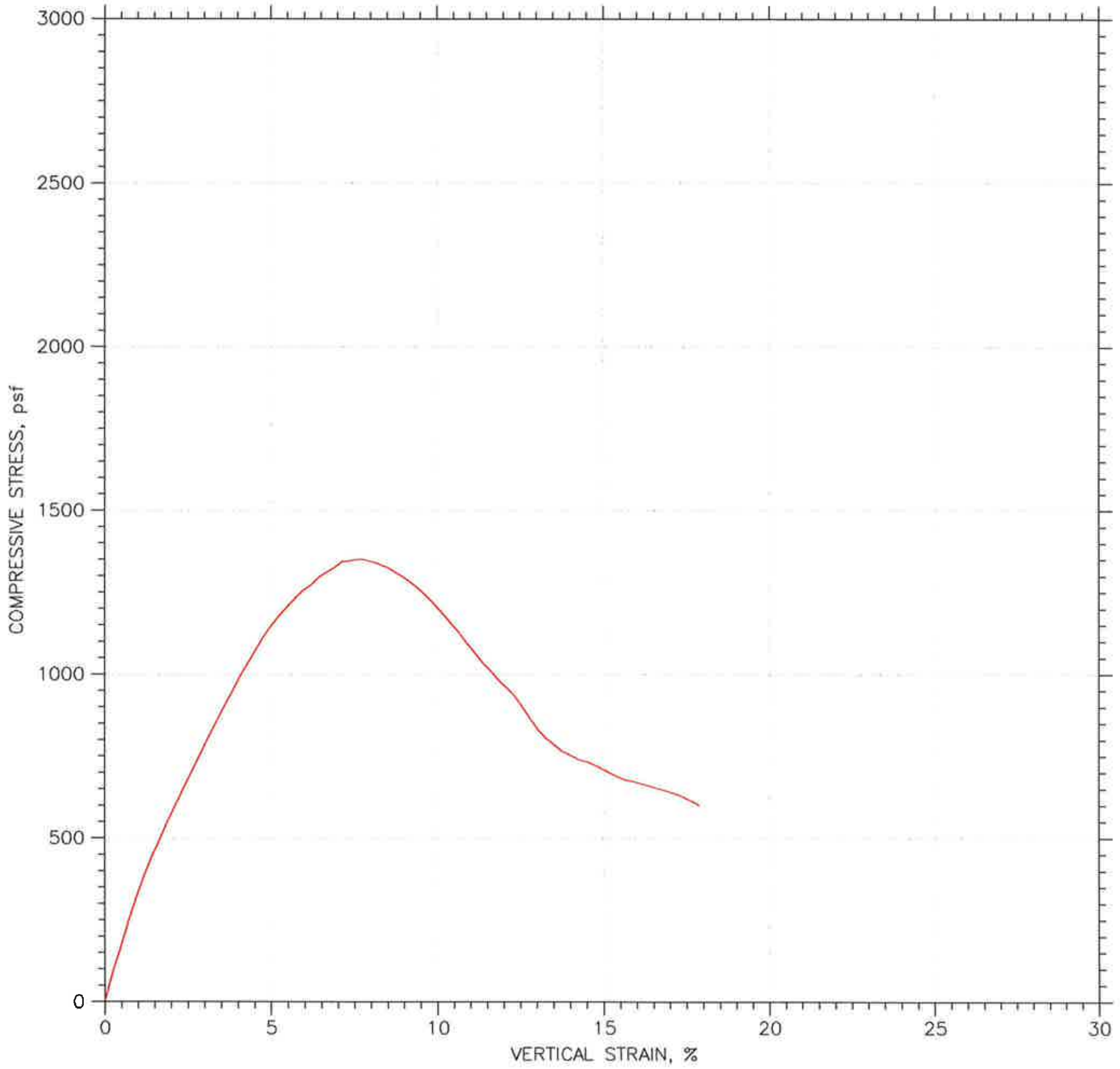


Symbol					
Test No.		13-236			
Initial	Diameter, in	2.38			
	Height, in	5.08			
	Water Content, %	30.82			
	Dry Density, pcf	90.149			
	Saturation, %	97.78			
	Void Ratio	0.83511			
Unconfined Compressive Strength, psf		1351.7			
Undrained Shear Strength, psf		675.84			
Time to Failure, min		8.2505			
Strain Rate, %/min		1			
Specific Gravity		2.65			
Liquid Limit		0			
Plastic Limit		0			
Plasticity Index		0			
Failure Sketch					

Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB9@2.5
Sample Type: 2.5"shelby
Description: Strong Brown SILT
Remarks:

Wed, 10-APR-2013 13:56:38

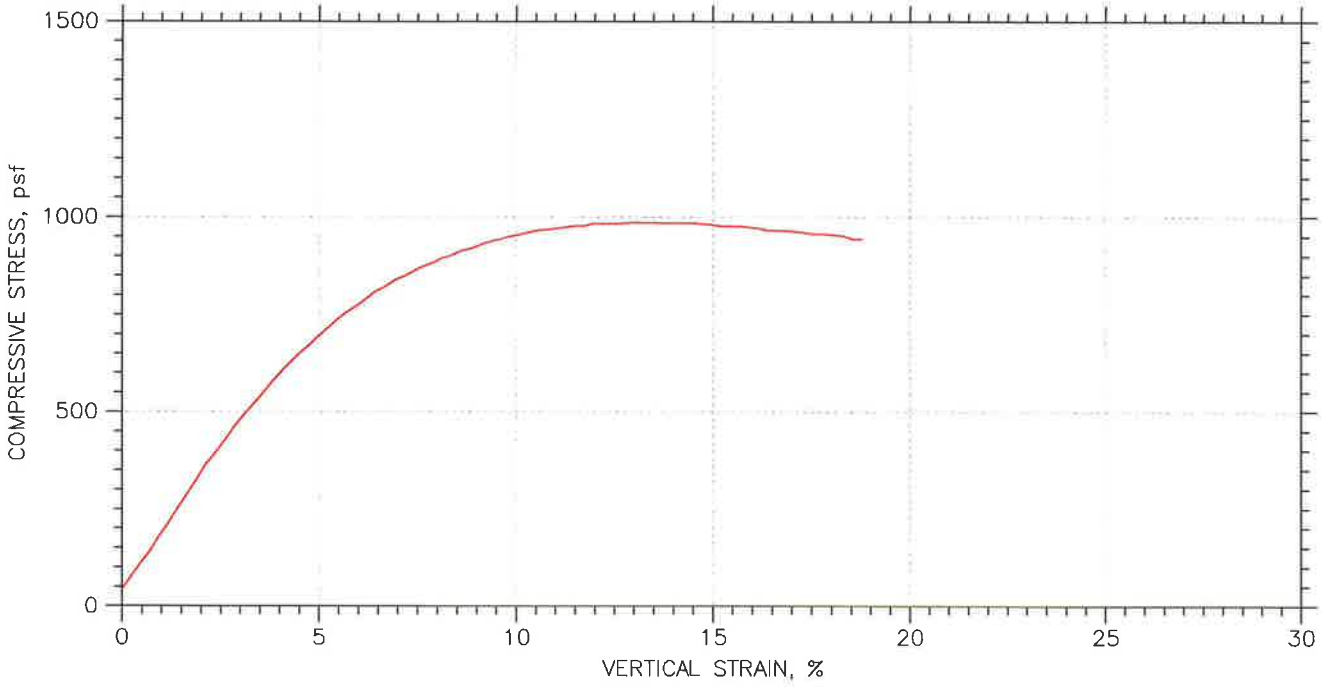
UNCONFINED COMPRESSION TEST REPORT



Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB9@2.5	Tested By: JMA	Checked By: Dh 4/18/13
Sample No.: 13-236	Test Date: 4/9/12	Depth: 2.5-3.0
Test No.: 13-236	Sample Type: 2.5"shelby	Elevation:
Description: Strong Brown SILT		
Remarks:		

Wed, 10-APR-2013 13:56:38

UNCONFINED COMPRESSION TEST REPORT

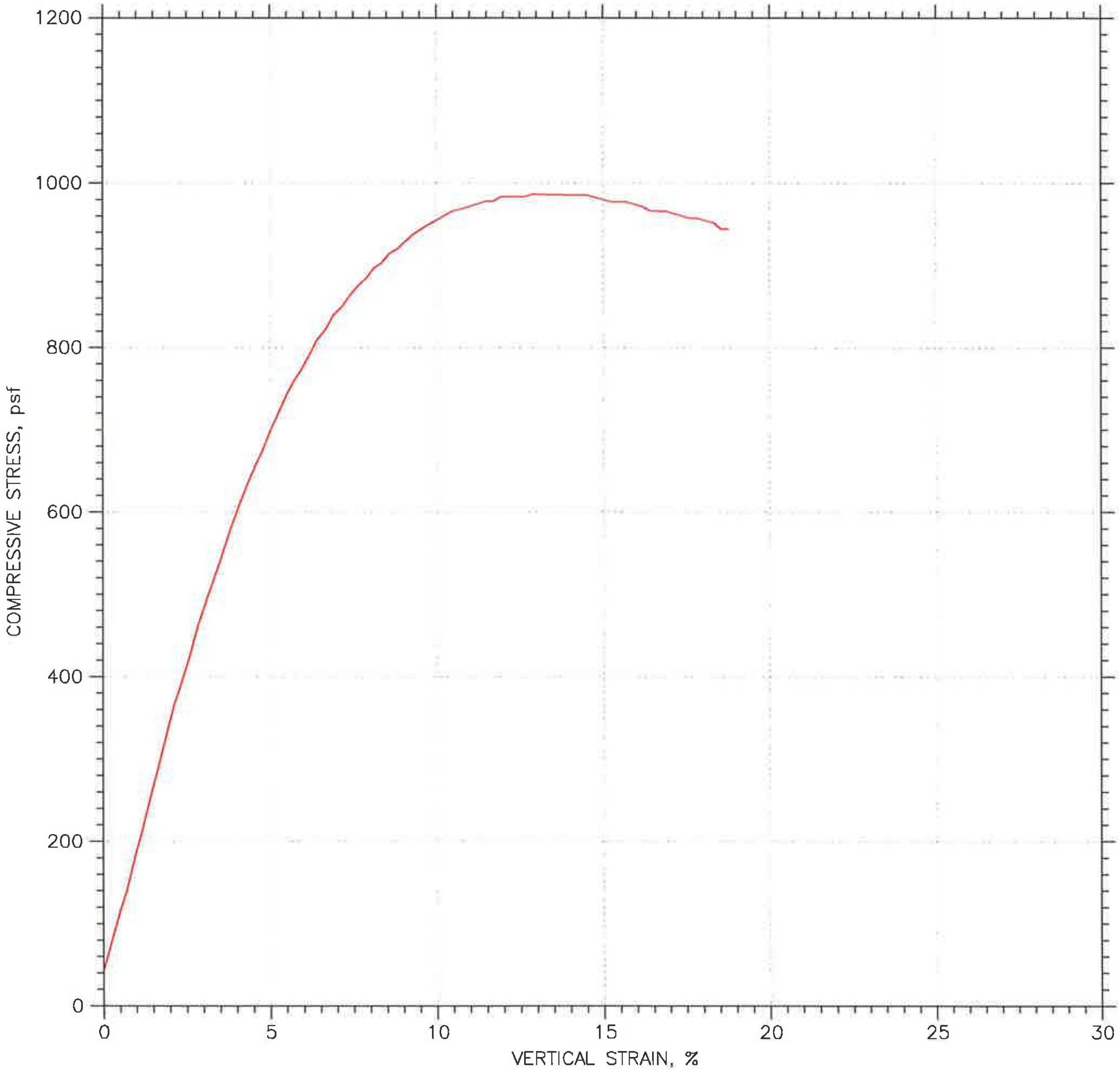


Symbol					
Test No.		13-227			
Initial	Diameter, in	2.38			
	Height, in	5.95			
	Water Content, %	40.10			
	Dry Density, pcf	85.476			
	Saturation, %	113.61			
	Void Ratio	0.93545			
Unconfined Compressive Strength, psf		986.42			
Undrained Shear Strength, psf		493.21			
Time to Failure, min		13.752			
Strain Rate, %/min		1			
Specific Gravity		2.65			
Liquid Limit		0			
Plastic Limit		0			
Plasticity Index		0			
Failure Sketch					

Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB1@12.75
Sample Type: 2.5"shelby
Description: Gray SILT
Remarks:

Wed, 10-APR-2013 13:45:49

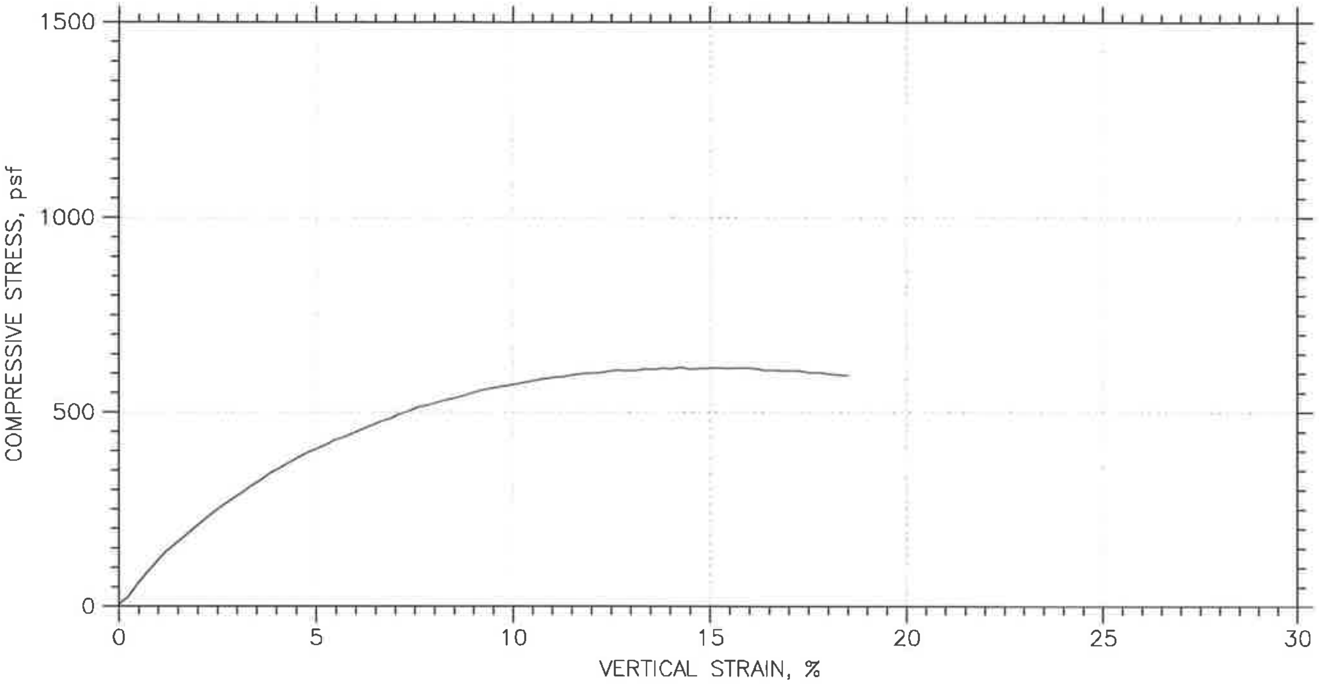
UNCONFINED COMPRESSION TEST REPORT



Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB1@12.75	Tested By: JMA	Checked By: Dh 4/16/13
Sample No.: 13-227	Test Date: 4/8/12	Depth: 12.75-13.25
Test No.: 13-227	Sample Type: 2.5"shelby	Elevation:
Description: Gray SILT		
Remarks:		

Wed, 10-APR-2013 13:45:50

UNCONFINED COMPRESSION TEST REPORT

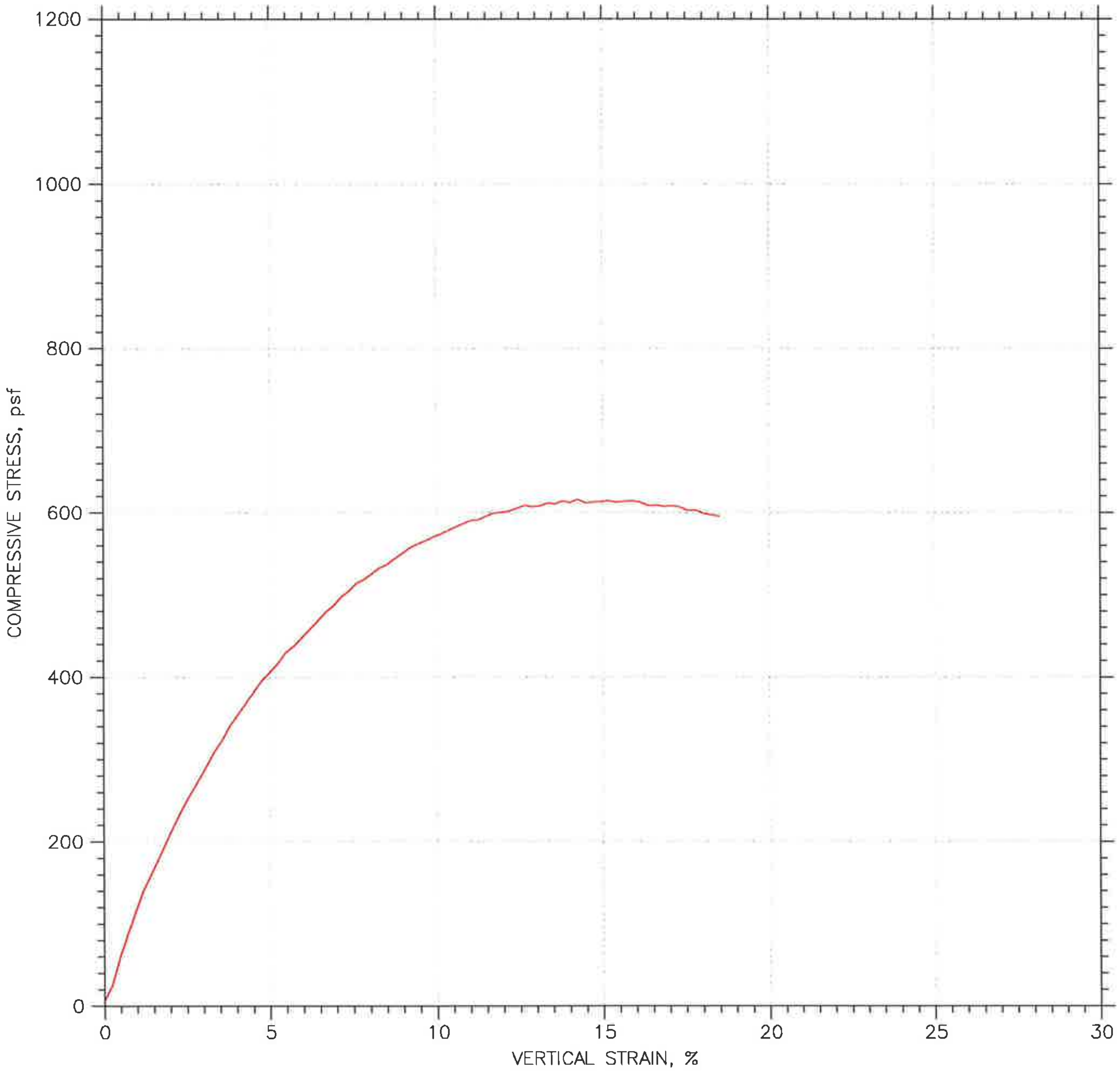


Symbol					
Test No.		13-245			
Initial	Diameter, in	2.38			
	Height, in	5.42			
	Water Content, %	79.99			
	Dry Density, pcf	52.518			
	Saturation, %	98.59			
	Void Ratio	2.15			
Unconfined Compressive Strength, psf		616.41			
Undrained Shear Strength, psf		308.2			
Time to Failure, min		15.253			
Strain Rate, %/min		1			
Specific Gravity		2.65			
Liquid Limit		0			
Plastic Limit		0			
Plasticity Index		0			
Failure Sketch					

Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB15@5'
Sample Type: 2.5''shelby
Description: Strong Brown SILT
Remarks: Organics in specimen

Thu, 18-APR-2013 12:22:12

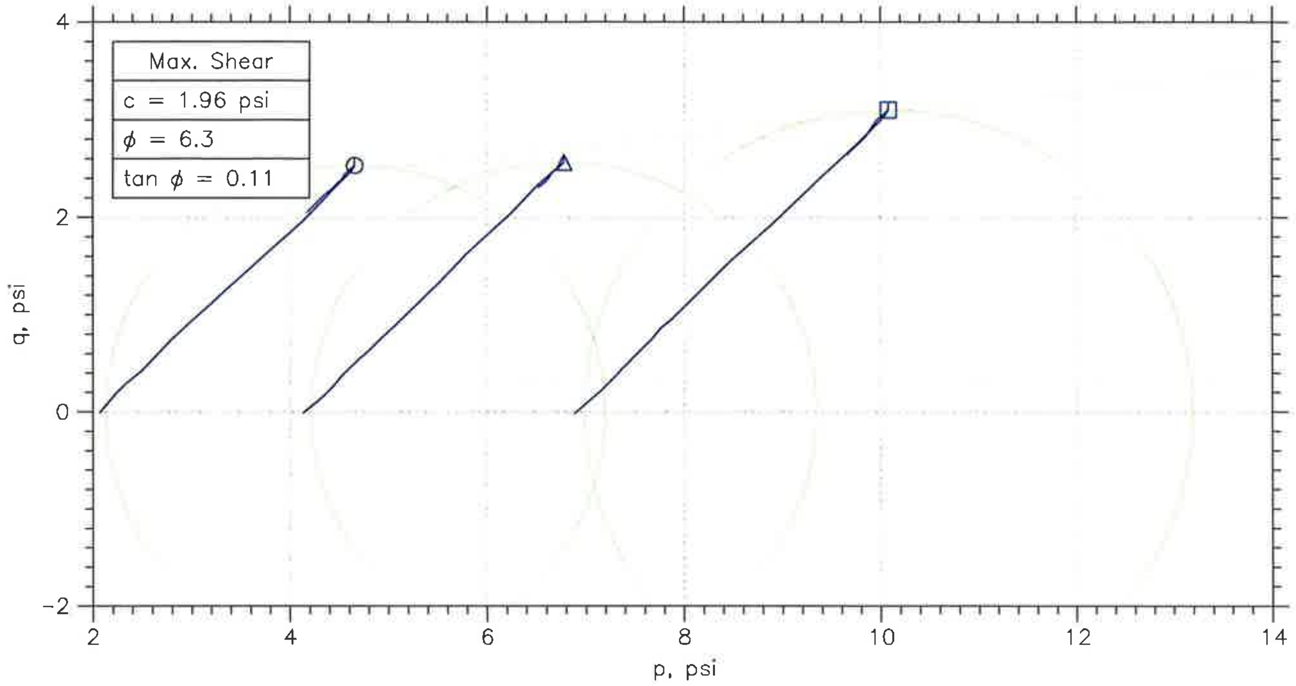
UNCONFINED COMPRESSION TEST REPORT

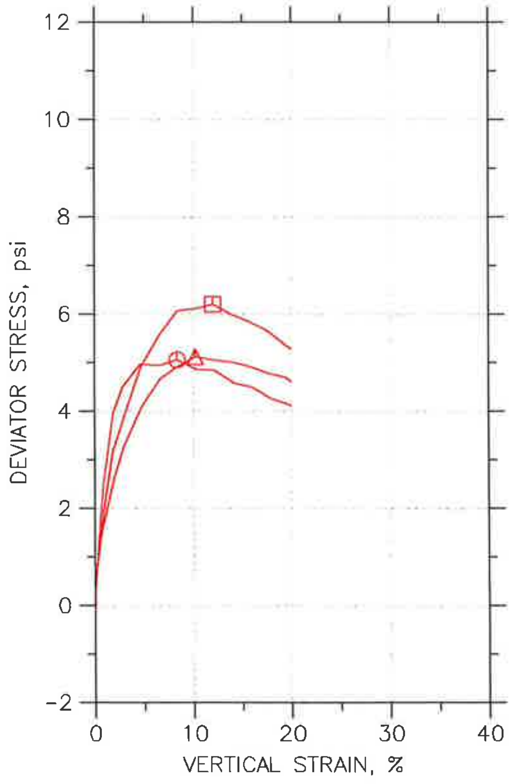


Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB15@5'	Tested By: JMA	Checked By: Dh 4/18/13
Sample No.: 13-245	Test Date: 4/9/12	Depth: 5-5.5
Test No.: 13-245	Sample Type: 2.5"shelby	Elevation:
Description: Strong Brown SILT		
Remarks: Organics in specimen		

Thu, 18-APR-2013 12:22:12

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





	Symbol	○	△	□	
	Sample No.	HB3@2.75	HB3@5'	HB4@8.6'	
	Test No.	13-229	13-230	13-235	
	Depth	2.25-2.75	5-5.5	8.6-9.1	
Initial	Diameter, in	2.38	2.38	2.38	
	Height, in	4.79	5.5	5.65	
	Water Content, %	48.5	43.0	38.3	
	Dry Density, pcf	71.94	79.49	79.71	
	Saturation, %	99.9	106.5	95.3	
	Void Ratio	1.27	1.06	1.05	
Before Shear	Water Content, %	48.5	40.3	38.3	
	Dry Density, pcf	72.06	79.58	80.15	
	Saturation*, %	100.0	100.0	96.4	
	Void Ratio	1.27	1.06	1.04	
	Back Press., psi	.E-17	.E-17	.E-17	
	Ver. Eff. Cons. Stress, psi	2.075	4.158	6.924	
	Shear Strength, psi	2.535	2.564	3.103	
	Strain at Failure, %	8.38	10.2	12	
	Strain Rate, %/min	1	1	1	
	B-Value	---	---	---	
	Estimated Specific Gravity	2.62	2.62	2.62	
	Liquid Limit	---	---	---	
	Plastic Limit	---	---	---	

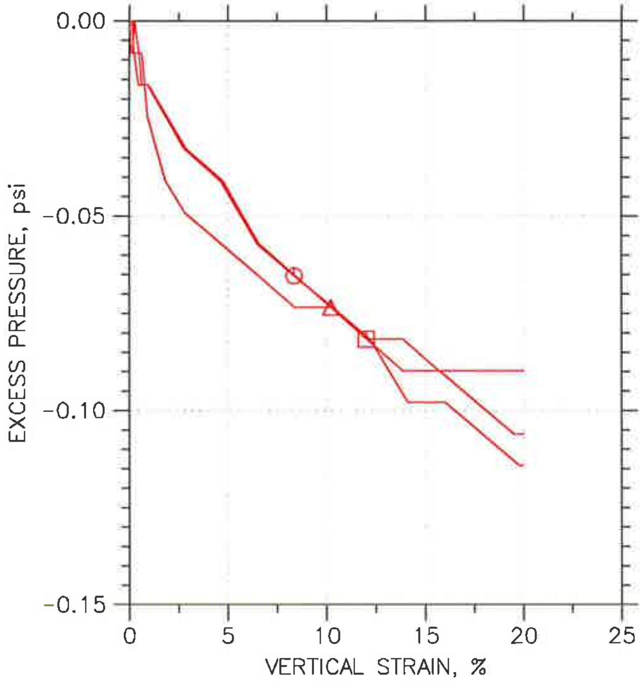
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB3 & HB4
Sample Type: 2.5"calbrl
Description: SILT
Remarks: Unconsolidated Undrained

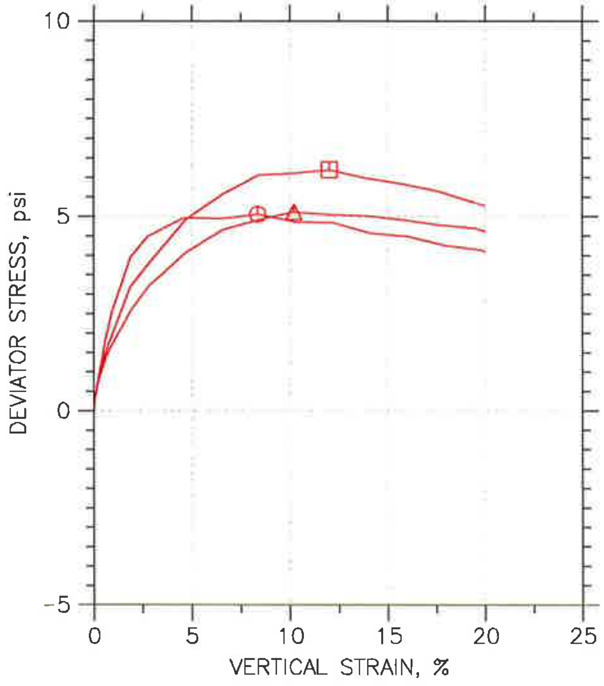
Thu, 18-APR-2013 12:09:29

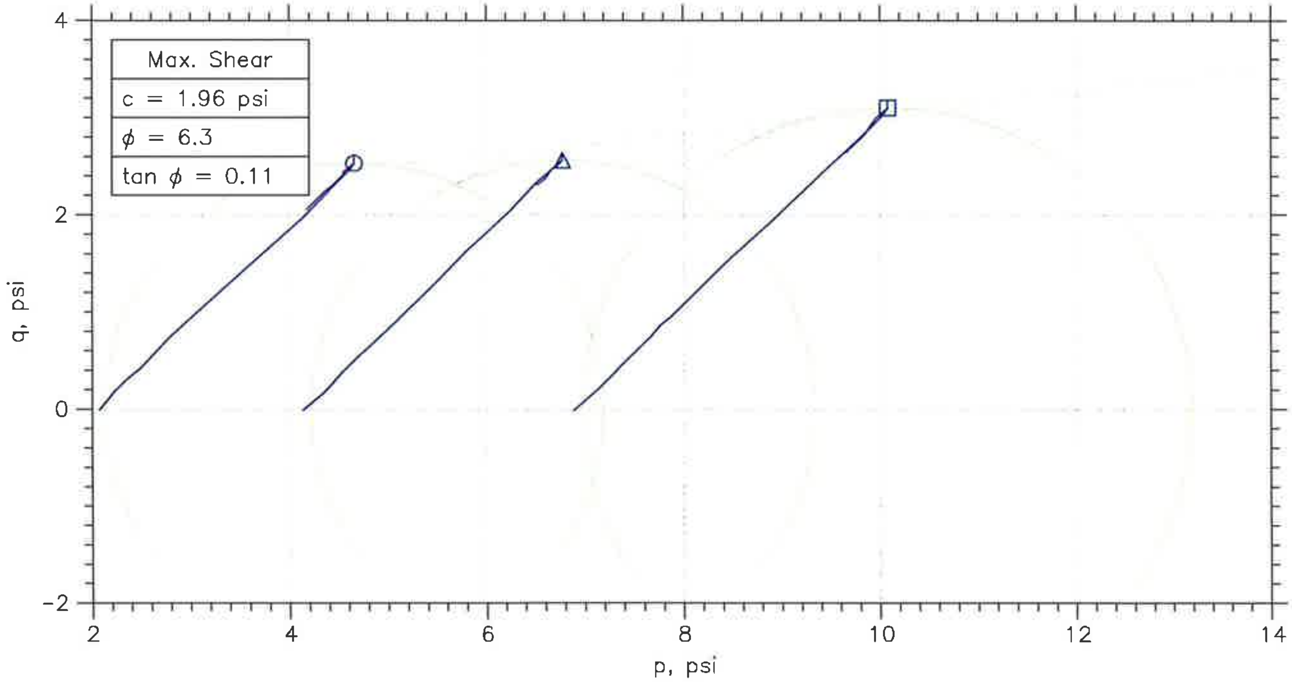
Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





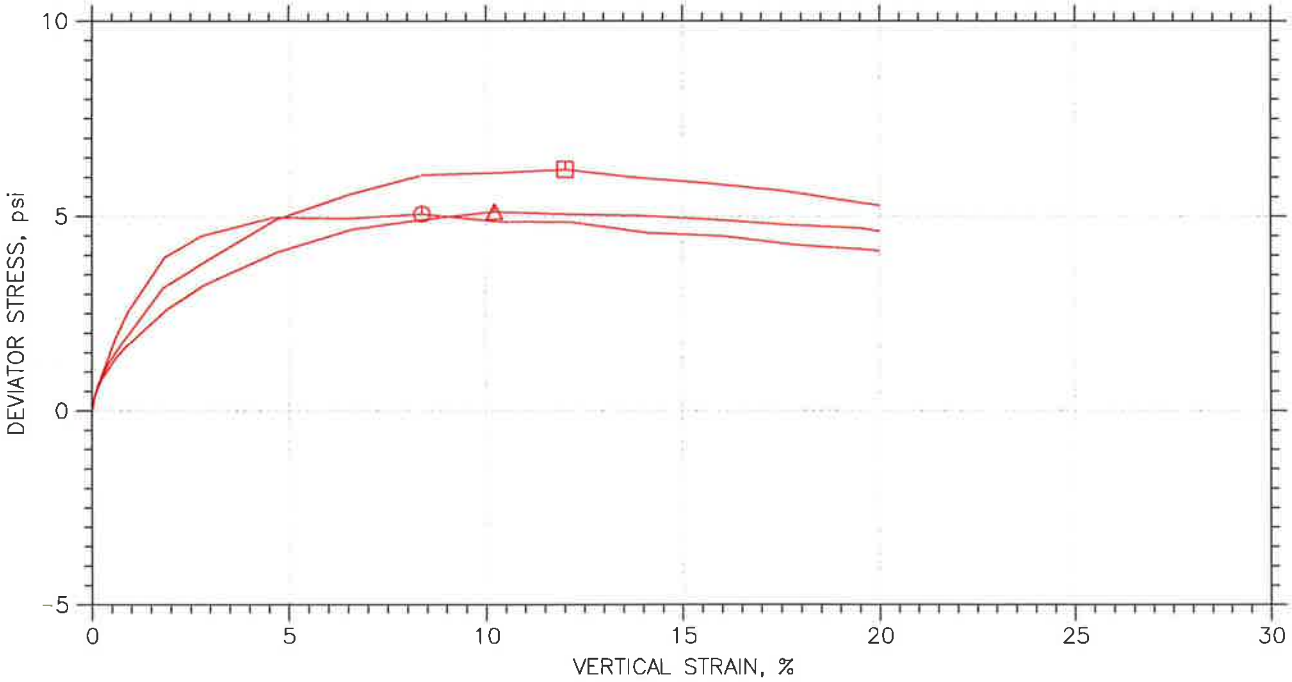


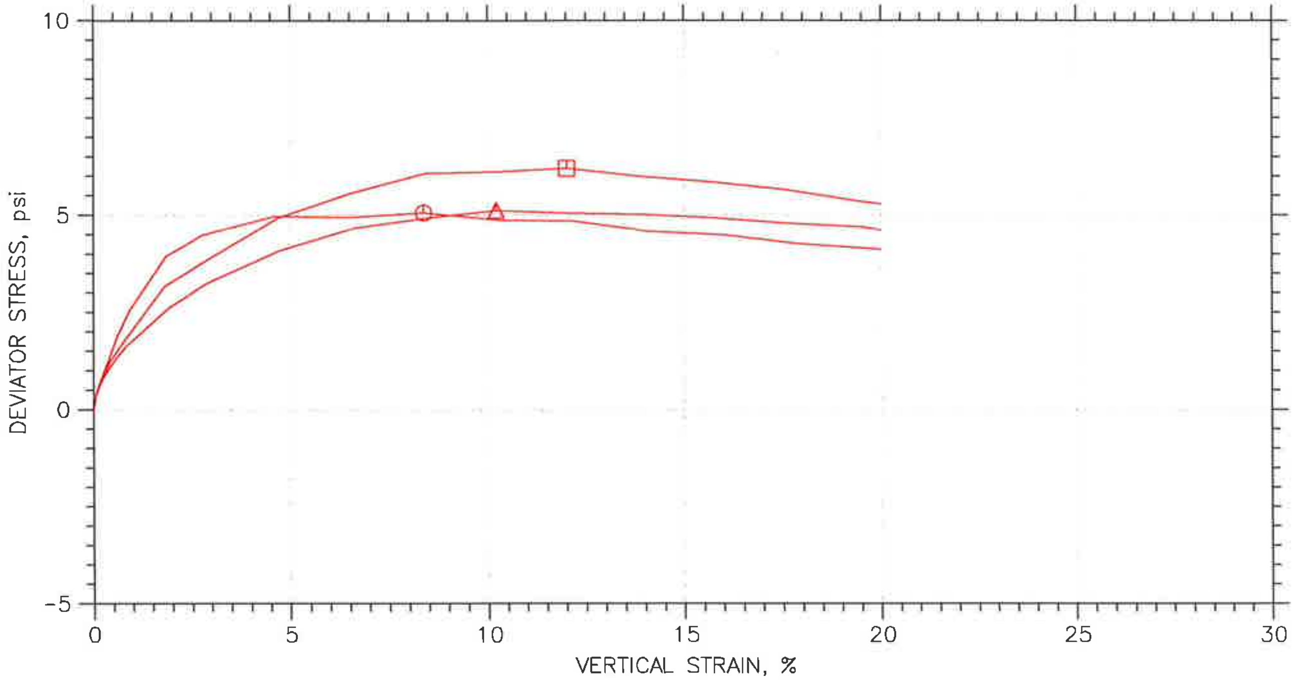
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	HB3@2.75	13-229	2.25-2.75	JMA	4/9/13			13-229 MSE.dat
△	HB3@5'	13-230	5-5.5	JMA	4/10/13			13-230 MSE.dat
□	HB4@8.6'	13-235	8.6-9.1	JMA	4/10/13			13-235 MSE.dat

Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB3 & HB4	Sample Type: 2.5"calbrl	
Description: SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:09:30

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST



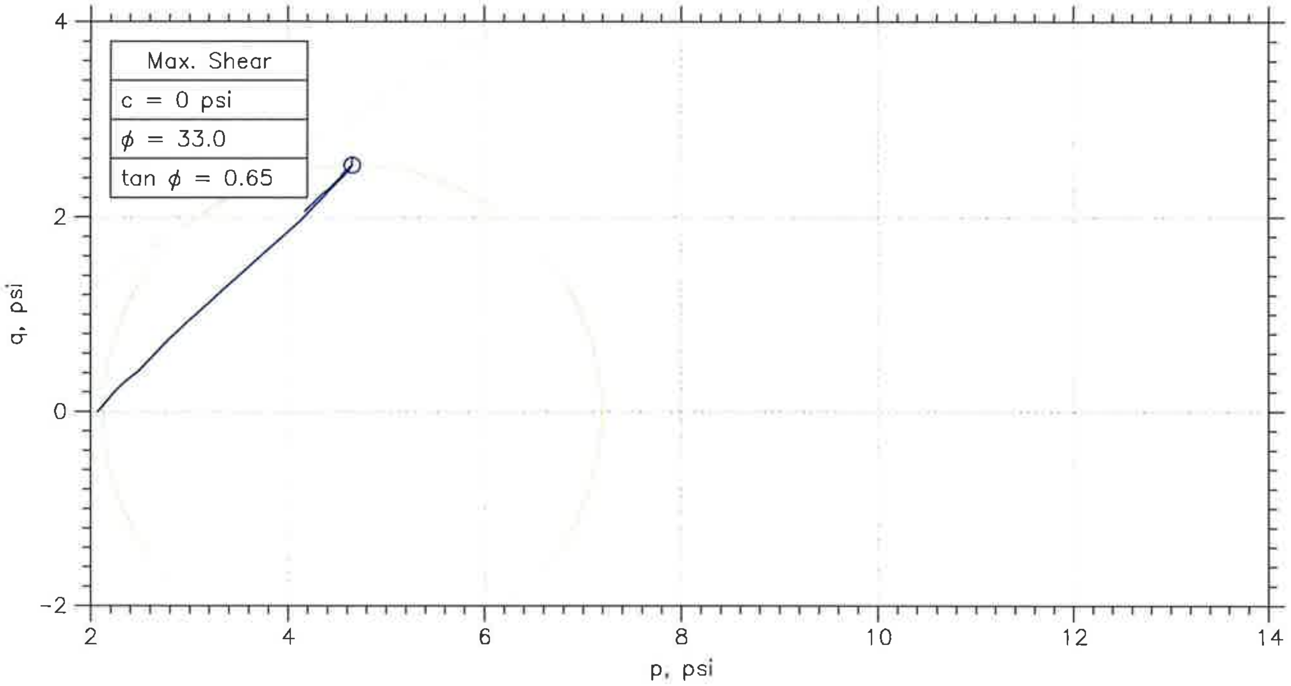


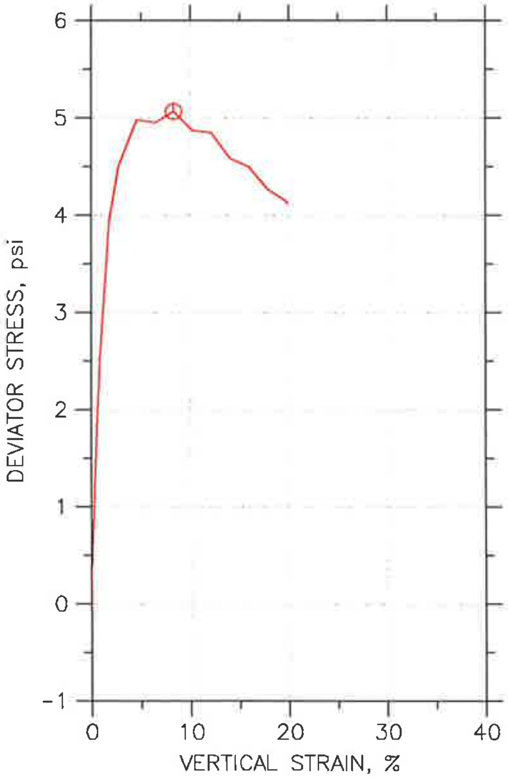
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB3@2.75	13-229	2.25-2.75	JMA	4/9/13			13-229 MSE.dat
△	HB3@5'	13-230	5-5.5	JMA	4/10/13			13-230 MSE.dat
◫	HB4@8.6'	13-235	8.6-9.1	JMA	4/10/13			13-235 MSE.dat

Project: Martin Slough Enhanceme	Location: Eureka	Project No.: 013035
Boring No.: HB3 & HB4	Sample Type: 2.5"calbrl	
Description: SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:09:30

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





Symbol		○			
Sample No.		HB3@2.75			
Test No.		13-229			
Depth		2.25-2.75			
Initial	Diameter, in	2.38			
	Height, in	4.79			
	Water Content, %	48.5			
	Dry Density, pcf	71.94			
	Saturation, %	99.9			
	Void Ratio	1.27			
Before Shear	Water Content, %	48.5			
	Dry Density, pcf	72.06			
	Saturation*, %	100.0			
	Void Ratio	1.27			
	Back Press., psi	.E-17			
Ver. Eff. Cons. Stress, psi		2.075			
Shear Strength, psi		2.535			
Strain at Failure, %		8.38			
Strain Rate, %/min		1			
B-Value		---			
Estimated Specific Gravity		2.62			
Liquid Limit		---			
Plastic Limit		---			

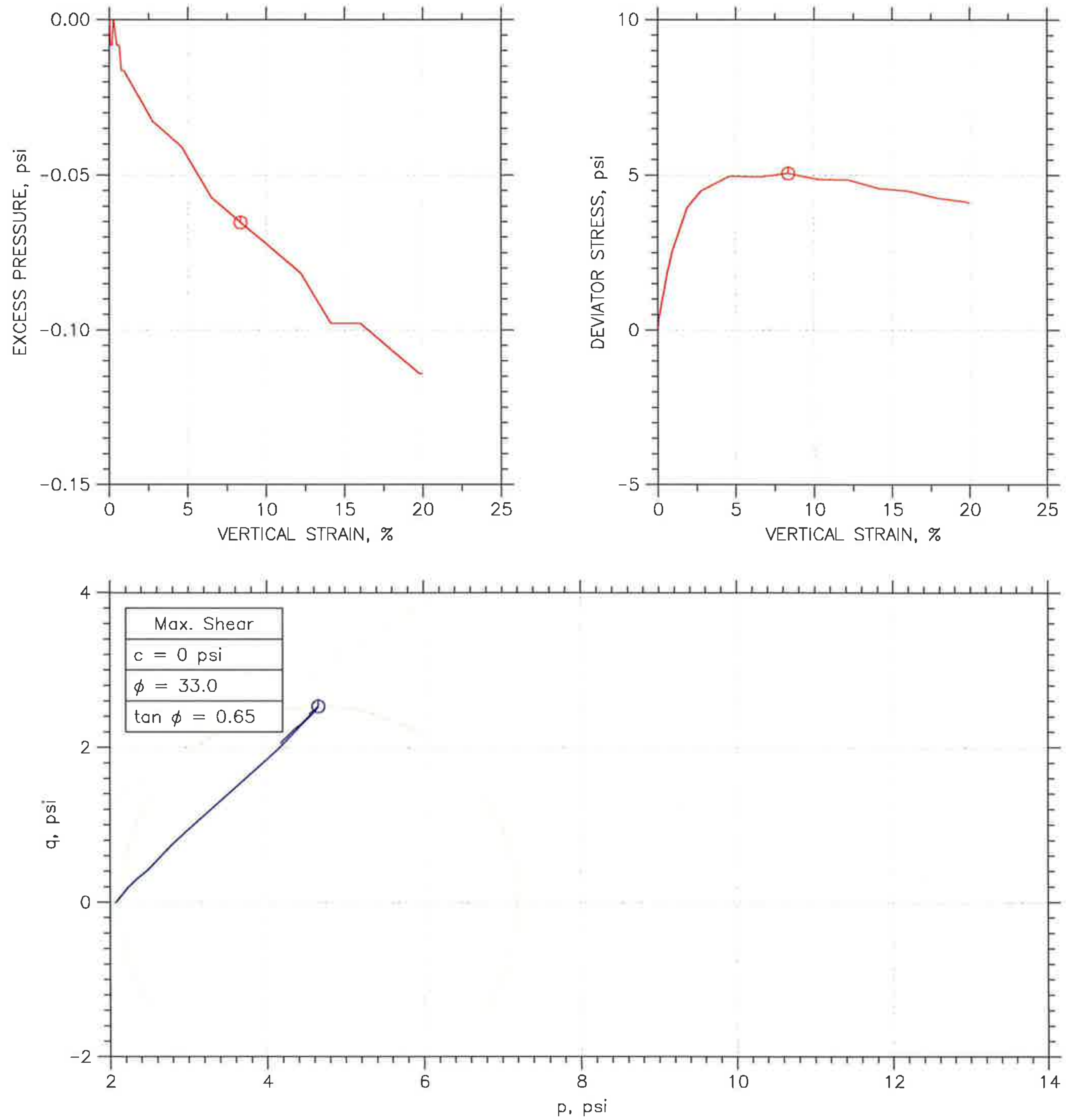
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB3@2.25
Sample Type: 2.5"calbrl
Description: SILT
Remarks: Unconsolidated Undrained

Thu, 18-APR-2013 12:15:54

Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST



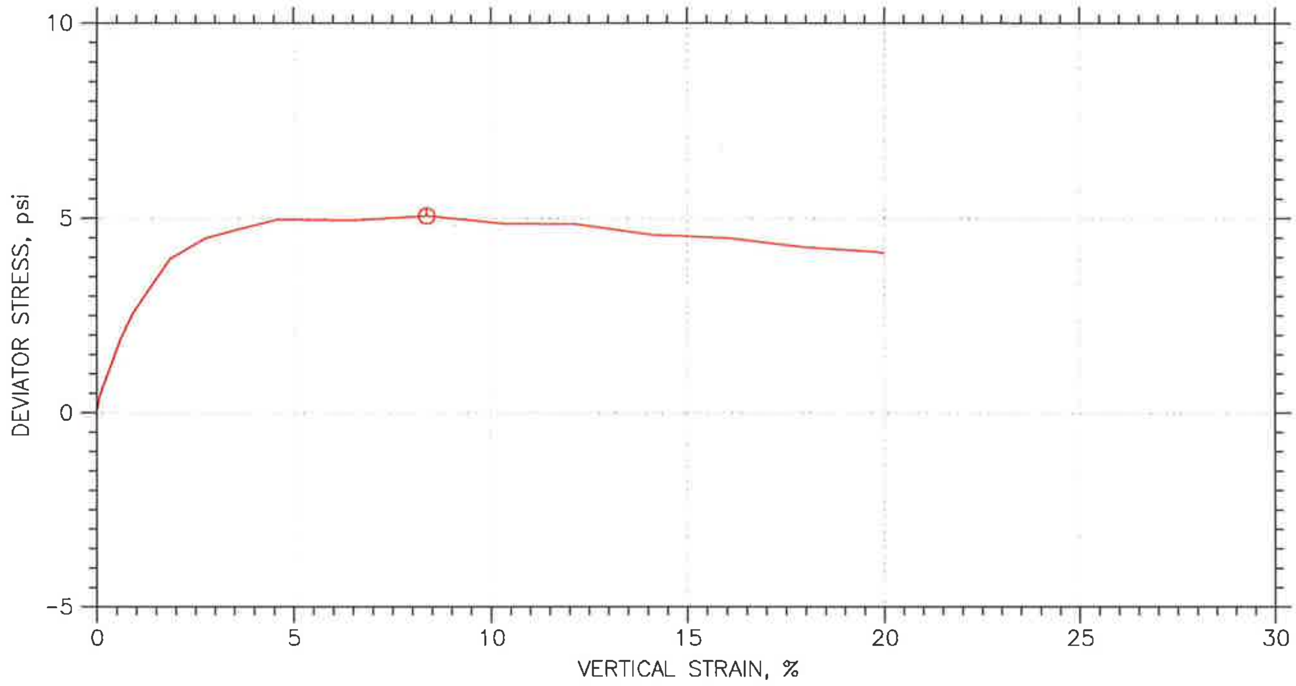
Max. Shear
c = 0 psi
ϕ = 33.0
tan ϕ = 0.65

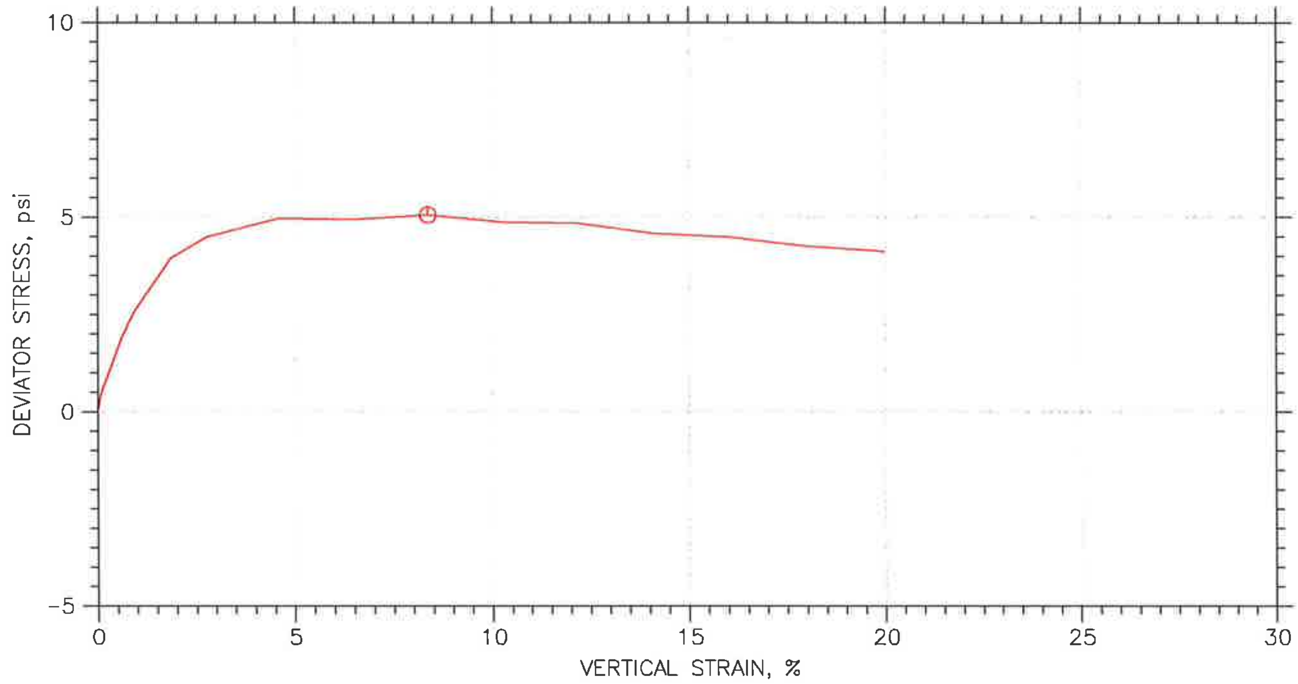
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⌽	HB3@2.75	13-229	2.25-2.75	JMA	4/9/13			13-229 MSE.dat

Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB3@2.25	Sample Type: 2.5"calbrl	
Description: SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:15:55

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST



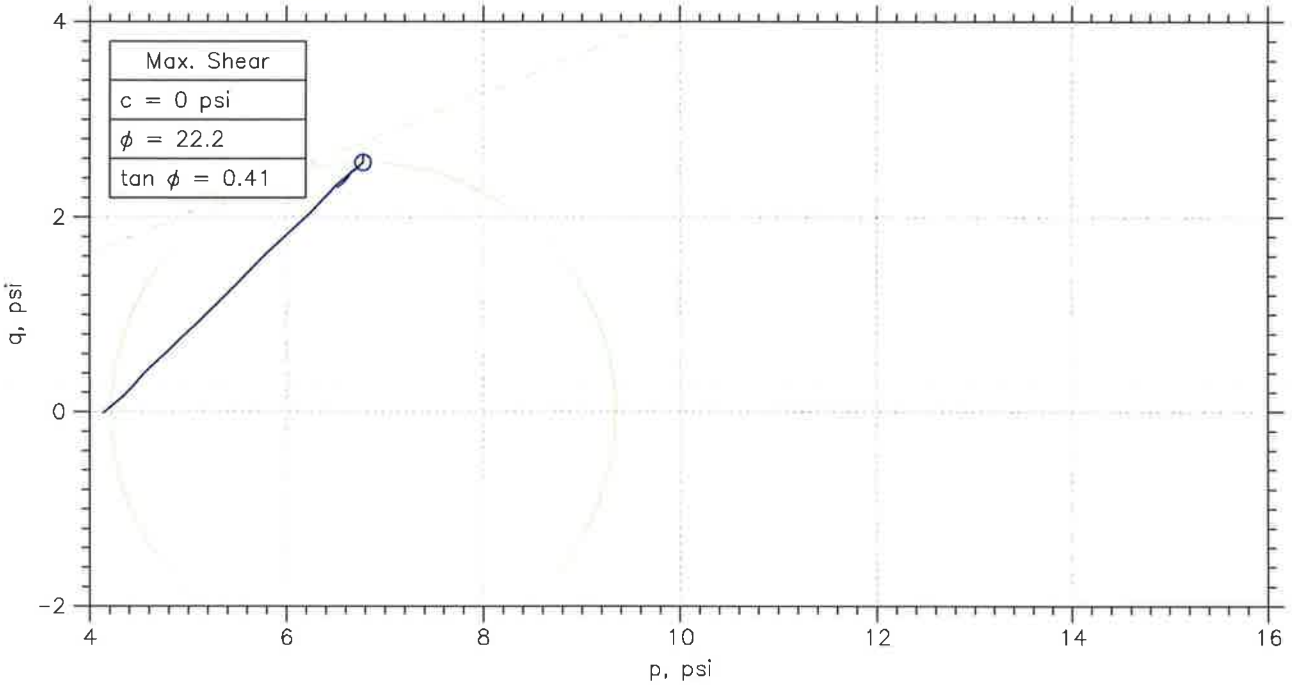


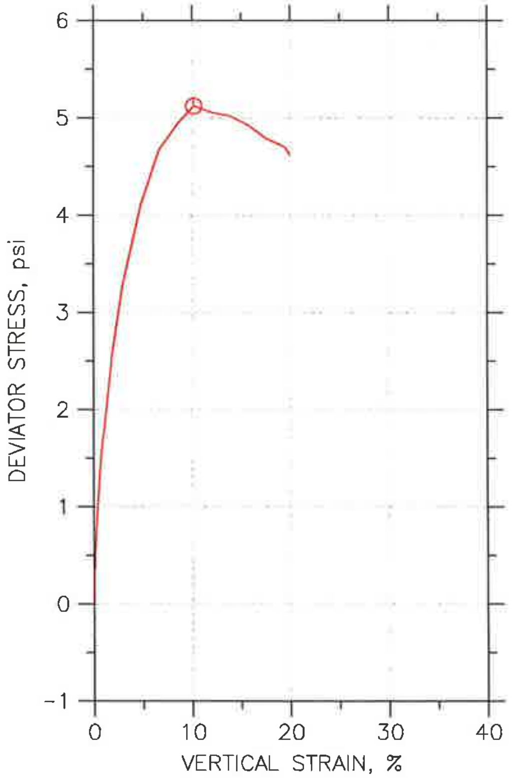
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB3@2.75	13-229	2.25-2.75	JMA	4/9/13			13-229 MSE.dat

Project: Martin Slough Enhancem	Location: Eureka	Project No.: 013035
Boring No.: HB3@2.25	Sample Type: 2.5"calbrl	
Description: SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:15:55

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





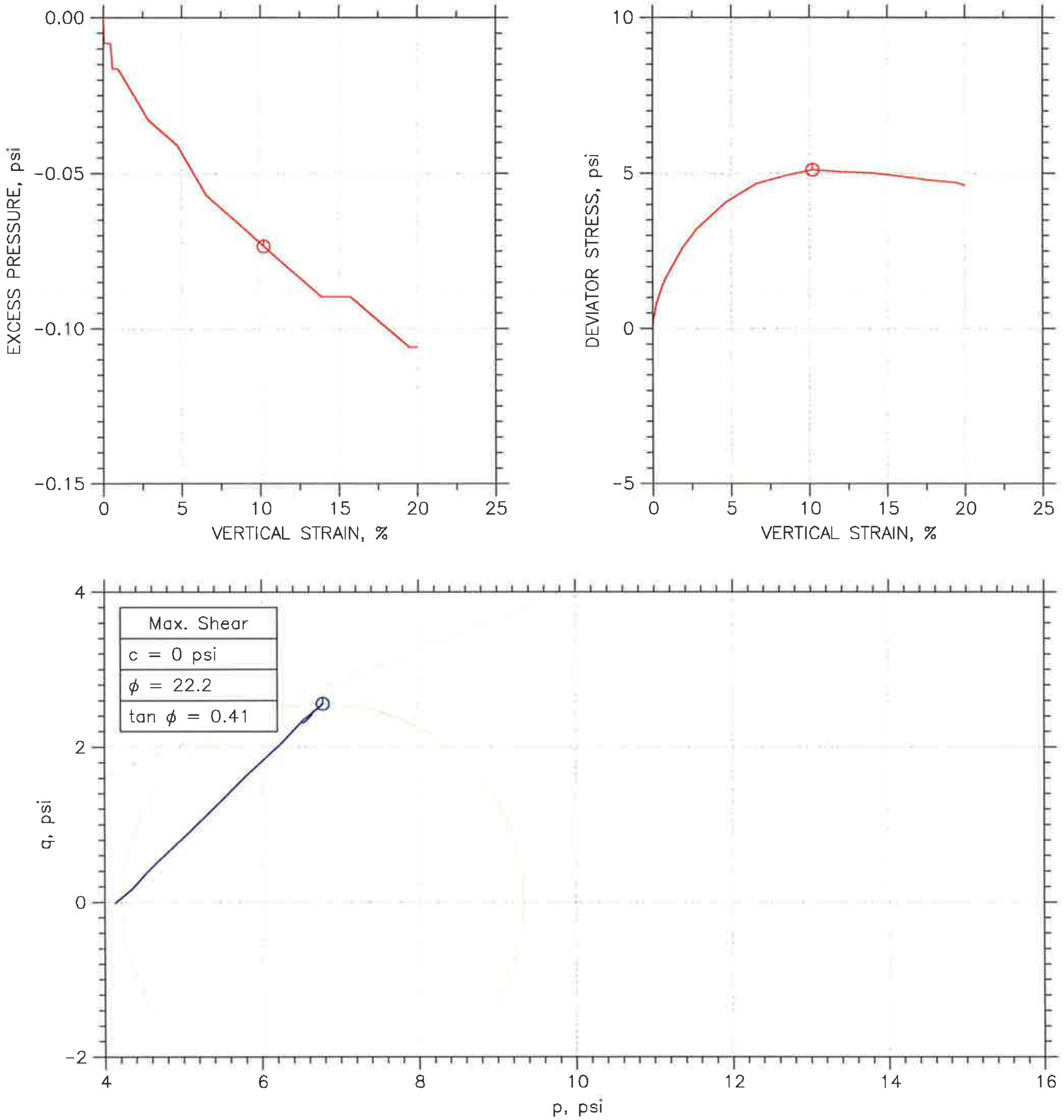
Symbol		⦶			
Sample No.		HB3@5'			
Test No.		13-230			
Depth		5-5.5			
Initial	Diameter, in	2.38			
	Height, in	5.5			
	Water Content, %	43.0			
	Dry Density, pcf	79.49			
	Saturation, %	106.5			
	Void Ratio	1.06			
Before Shear	Water Content, %	40.3			
	Dry Density, pcf	79.58			
	Saturation*, %	100.0			
	Void Ratio	1.06			
	Back Press., psi	.E-17			
Ver. Eff. Cons. Stress, psi		4.158			
Shear Strength, psi		2.564			
Strain at Failure, %		10.2			
Strain Rate, %/min		1			
B-Value		---			
Estimated Specific Gravity		2.62			
Liquid Limit		---			
Plastic Limit		---			

Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB3@5'
Sample Type: 2.5"calbrl
Description: SILT
Remarks: Unconsolidated Undrained

Phase calculations based on start and end of test.
* Saturation is set to 100% for phase calculations.

Thu, 18-APR-2013 12:19:50

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

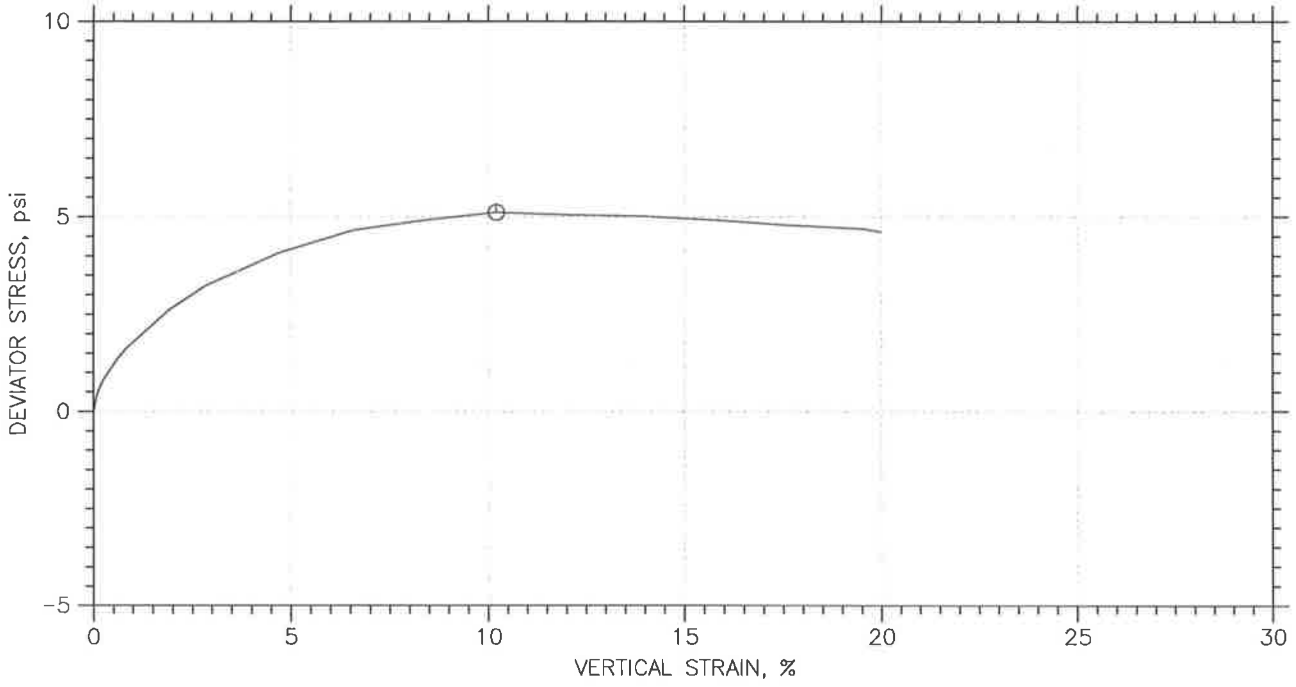


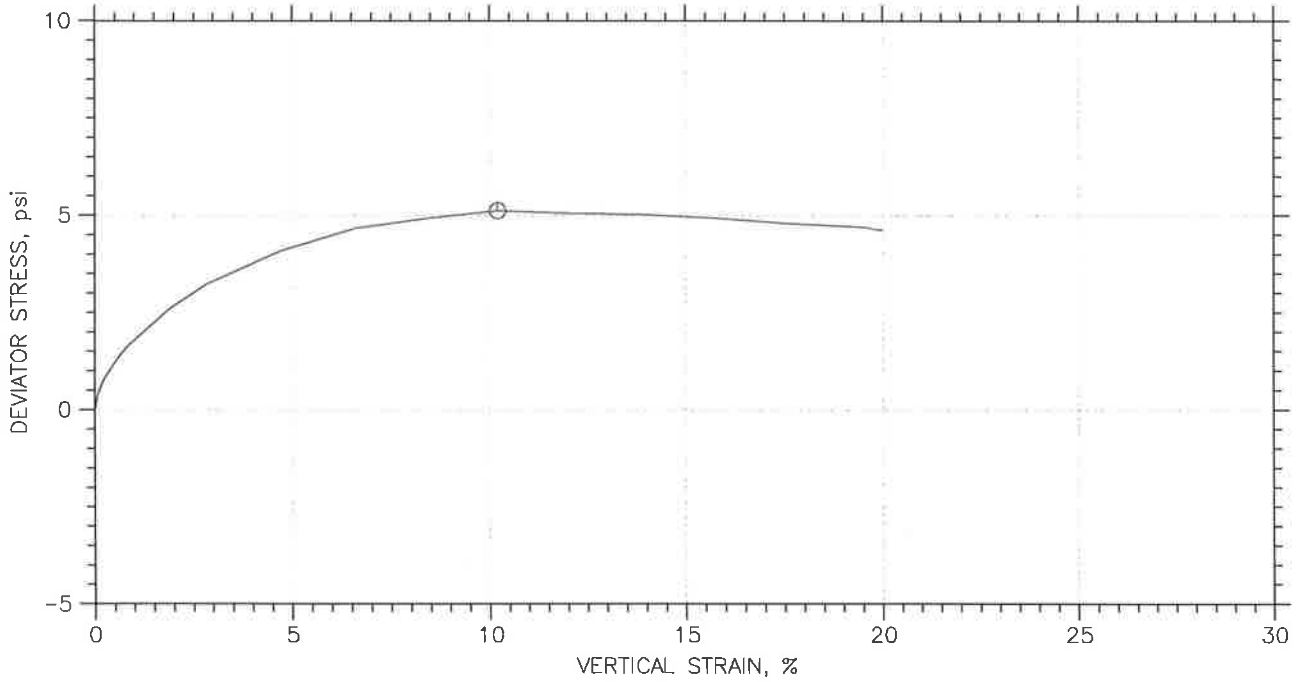
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	HB3@5'	13-230	5-5.5	JMA	4/10/13			13-230 MSE.dat

Project: Martin Slough Enhanceme	Location: Eureka	Project No.: 013035
Boring No.: HB3@5'	Sample Type: 2.5"calbrl	
Description: SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:19:50

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST



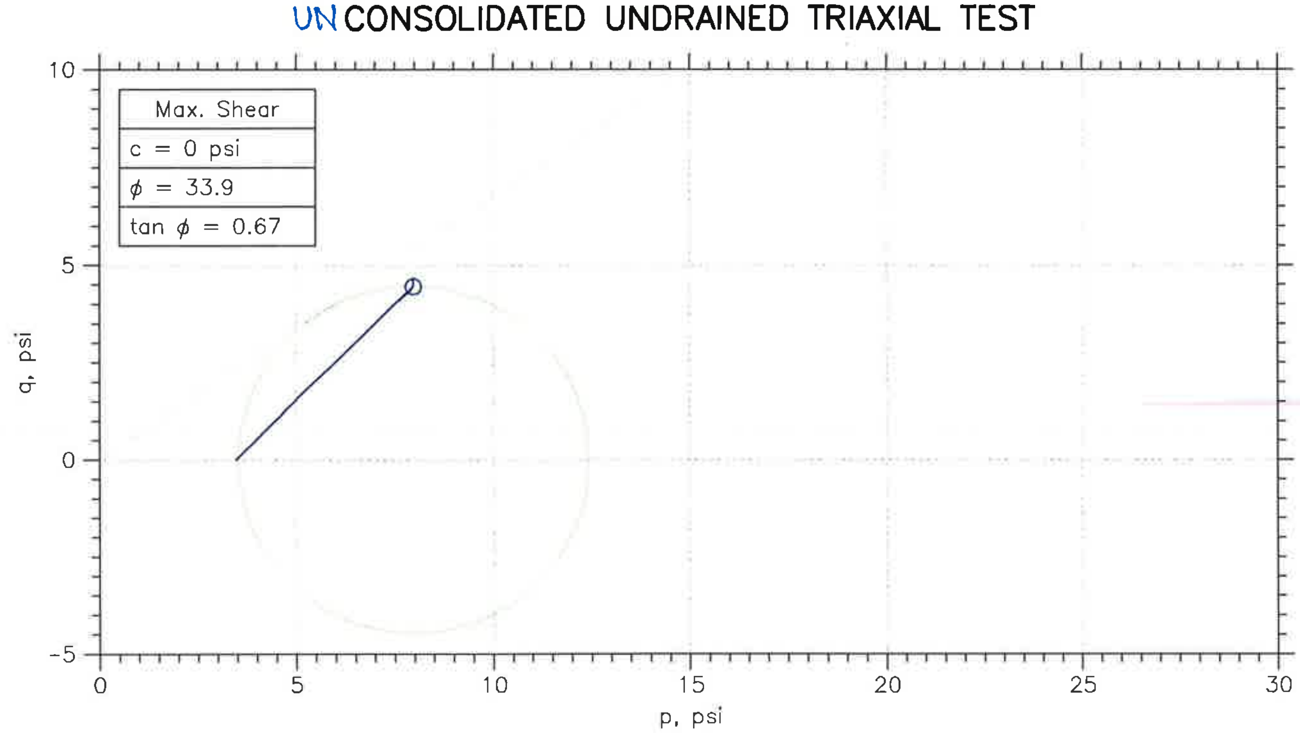


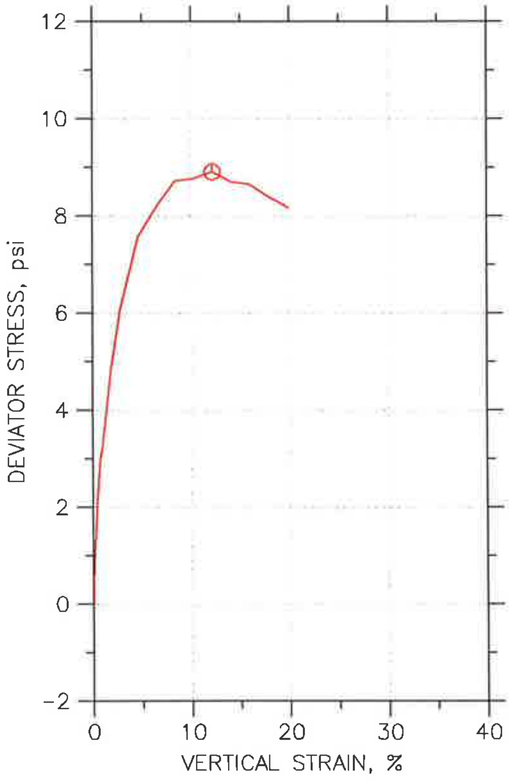
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
①	HB3@5'	13-230	5-5.5	JMA	4/10/13			13-230 MSE.dat

Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB3@5'	Sample Type: 2.5"calbrl	
Description: SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:19:51

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





Symbol		⌽			
Sample No.		HB4@3.2			
Test No.		13-233			
Depth		3.2-3.7			
Initial	Diameter, in	2.38			
	Height, in	4.8			
	Water Content, %	36.0			
	Dry Density, pcf	86.22			
	Saturation, %	102.9			
	Void Ratio	0.933			
Before Shear	Water Content, %	34.8			
	Dry Density, pcf	86.42			
	Saturation*, %	100.0			
	Void Ratio	0.929			
	Back Press., psi	.E-17			
Ver. Eff. Cons. Stress, psi		3.474			
Shear Strength, psi		4.456			
Strain at Failure, %		12.2			
Strain Rate, %/min		1			
B-Value		---			
Estimated Specific Gravity		2.67			
Liquid Limit		---			
Plastic Limit		---			

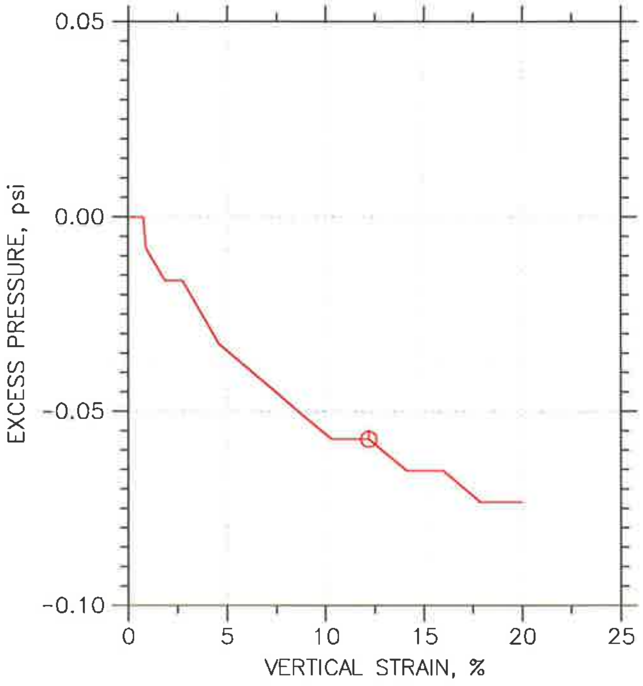
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB4@3.2-3.7
Sample Type: 2.5"calbrl
Description: SILT
Remarks: Unconsolidated Undrained

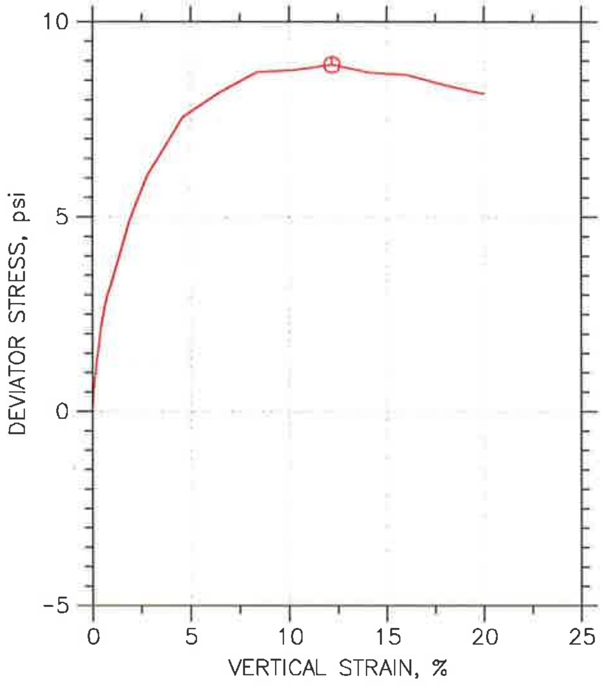
Thu, 18-APR-2013 12:24:31

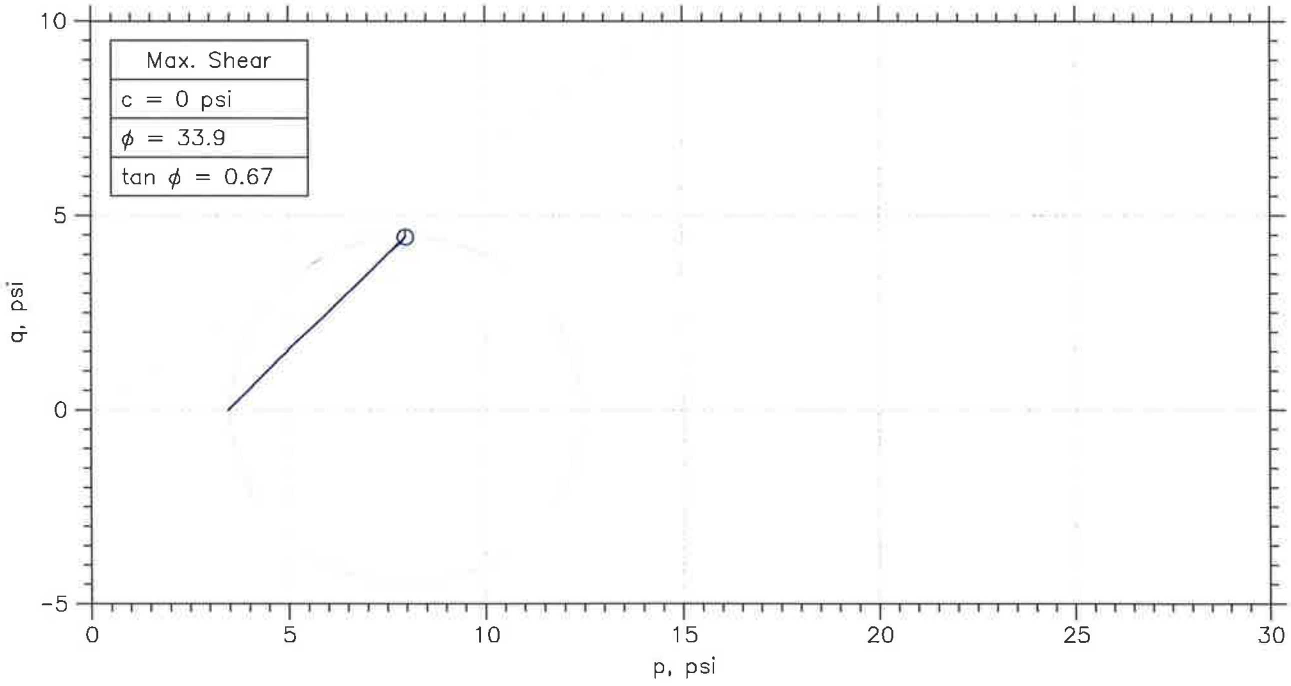
Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

UN CONSOLIDATED UNDRAINED TRIAXIAL TEST





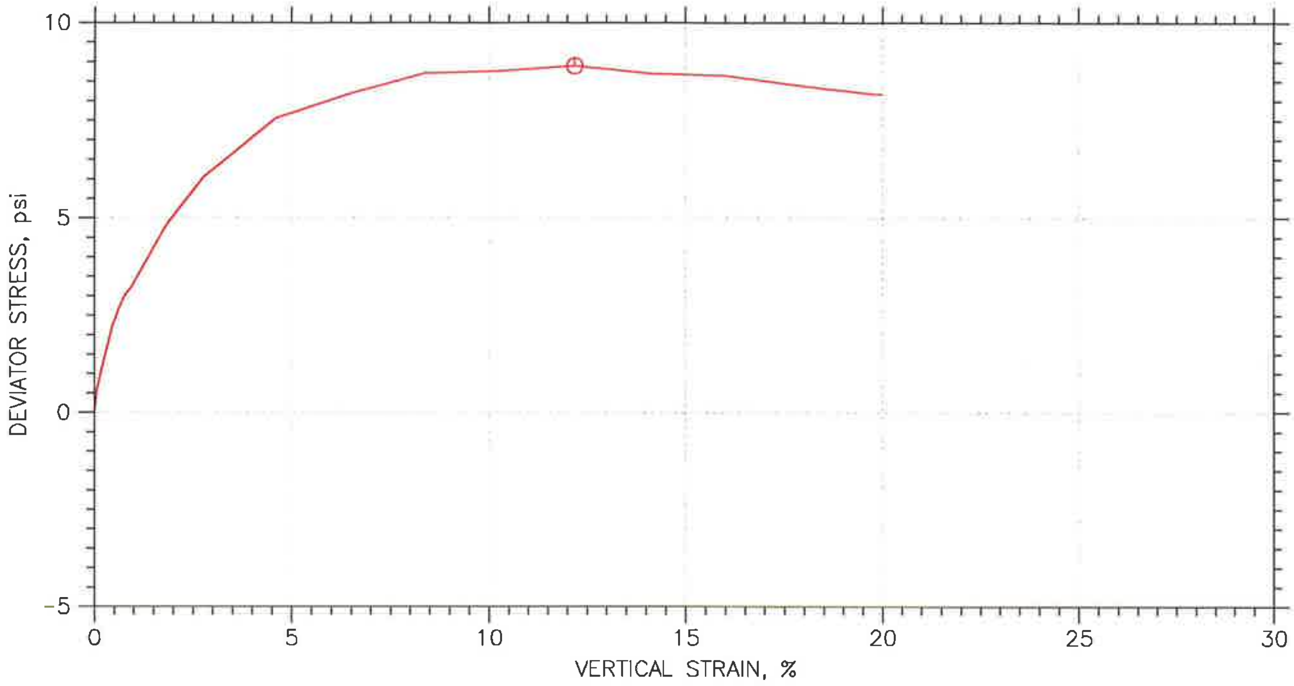


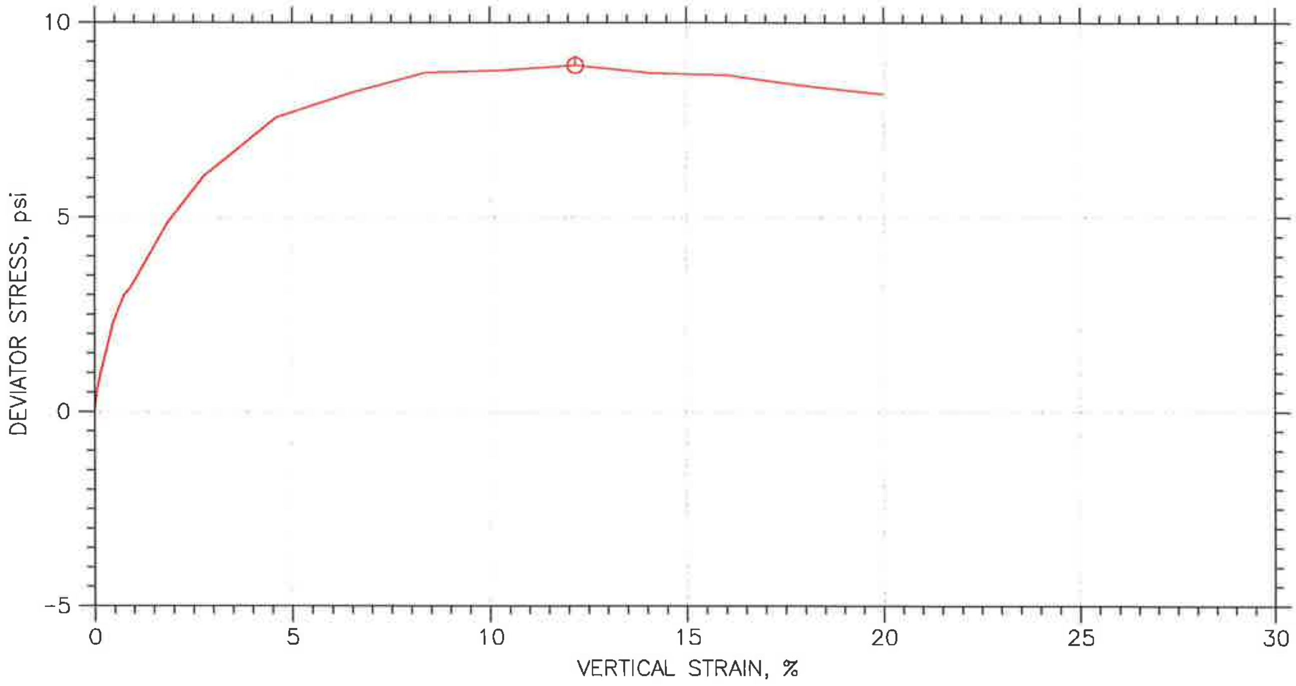
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB4@3.2	13-233	3.2-3.7	JMA	4/10/13			13-233 MSE.dat

Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB4@3.2-3.7	Sample Type: 2.5"calbrl	
Description: SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:24:32

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST



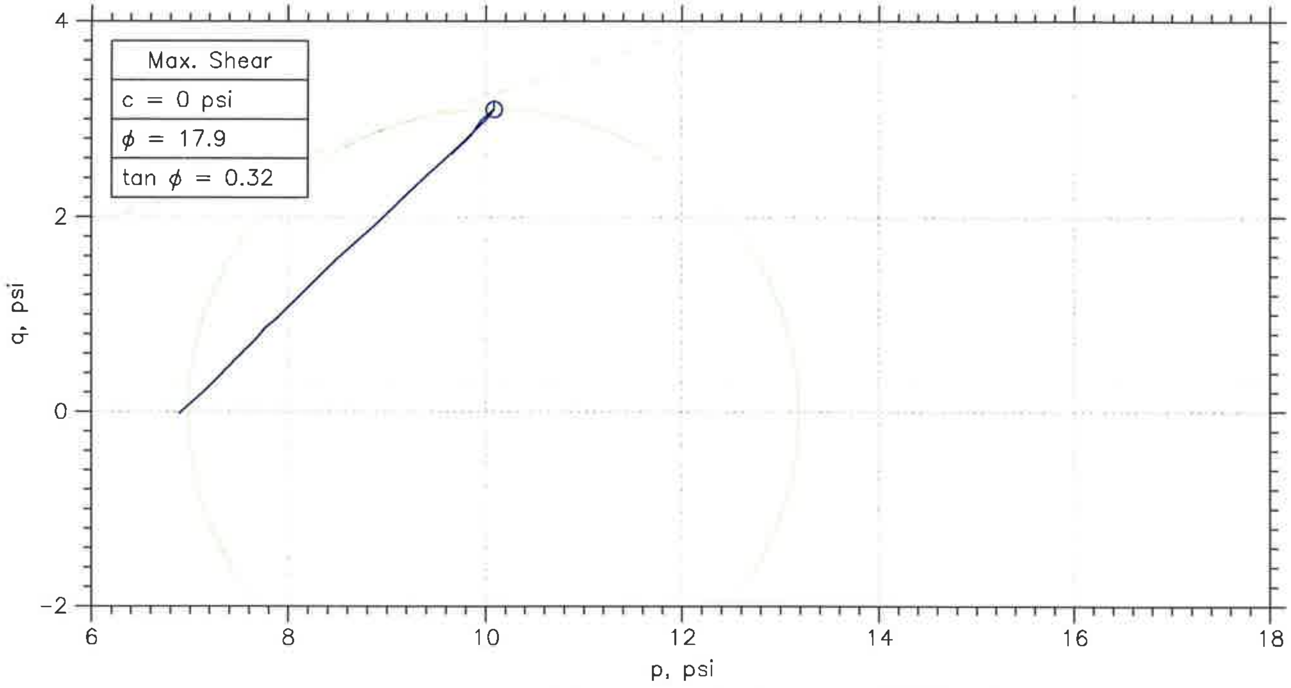


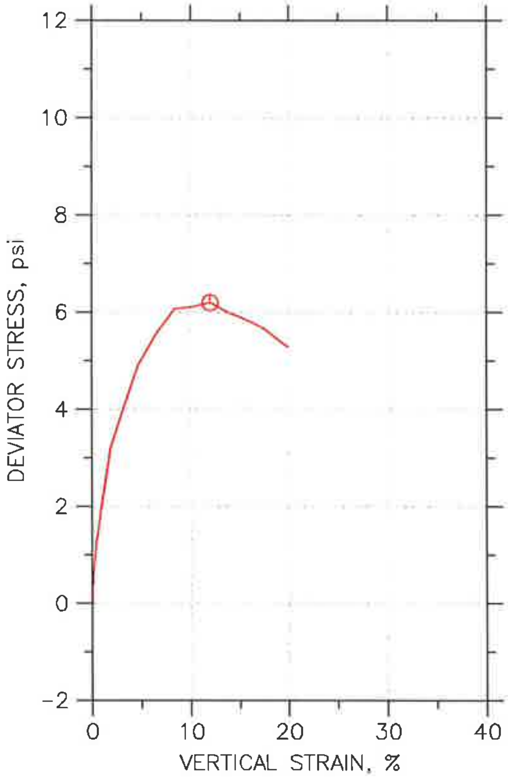
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB4@3.2	13-233	3.2-3.7	JMA	4/10/13			13-233 MSE.dat

Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB4@3.2-3.7	Sample Type: 2.5"calbrl	
Description: SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:24:32

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





Symbol		○			
Sample No.		HB4@8.6'			
Test No.		13-235			
Depth		8.6-9.1			
Initial	Diameter, in	2.38			
	Height, in	5.65			
	Water Content, %	38.3			
	Dry Density, pcf	79.71			
	Saturation, %	95.3			
	Void Ratio	1.05			
Before Shear	Water Content, %	38.3			
	Dry Density, pcf	80.15			
	Saturation*, %	96.4			
	Void Ratio	1.04			
	Back Press., psi	.E-17			
Ver. Eff. Cons. Stress, psi		6.924			
Shear Strength, psi		3.103			
Strain at Failure, %		12			
Strain Rate, %/min		1			
B-Value		---			
Estimated Specific Gravity		2.62			
Liquid Limit		---			
Plastic Limit		---			

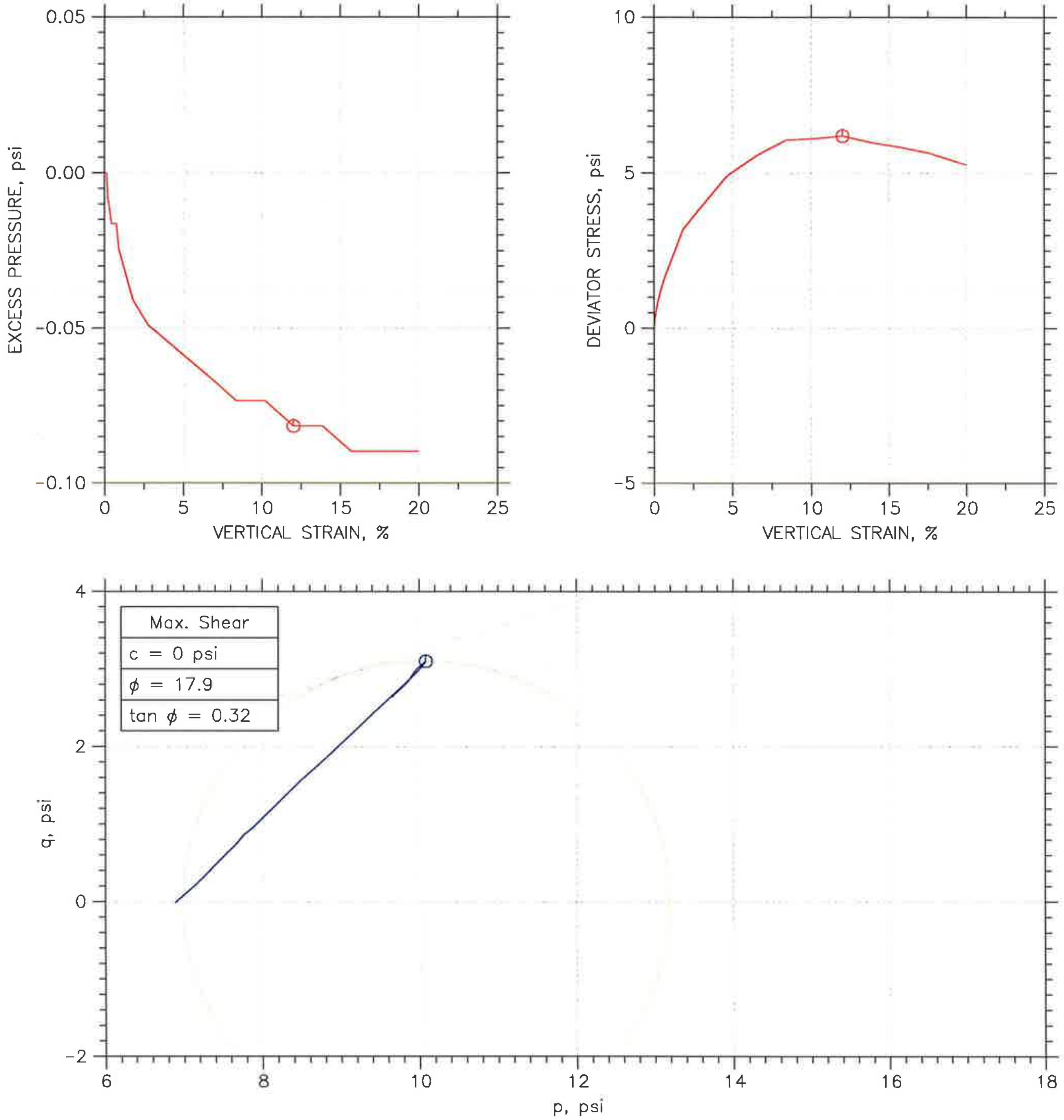
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB3
Sample Type: 2.5"calbrl
Description: Blue Gray SILT
Remarks: Unconsolidated Undrained

Thu, 18-APR-2013 12:25:52

Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

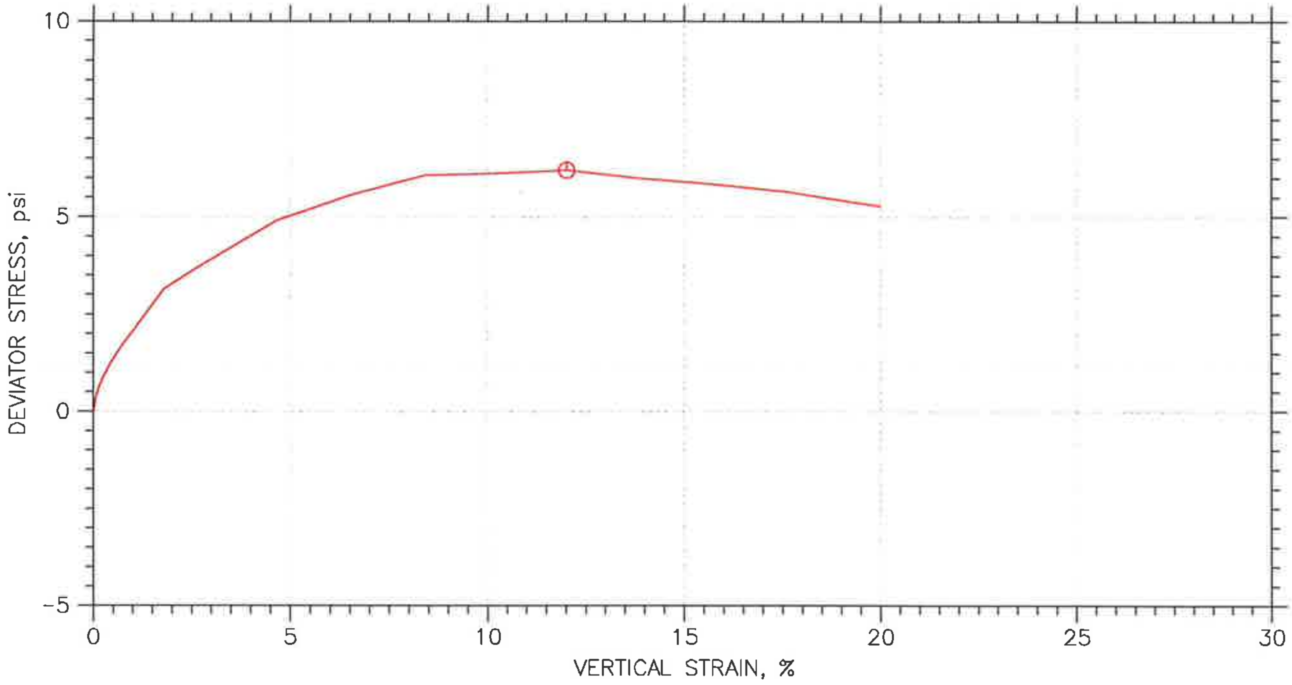


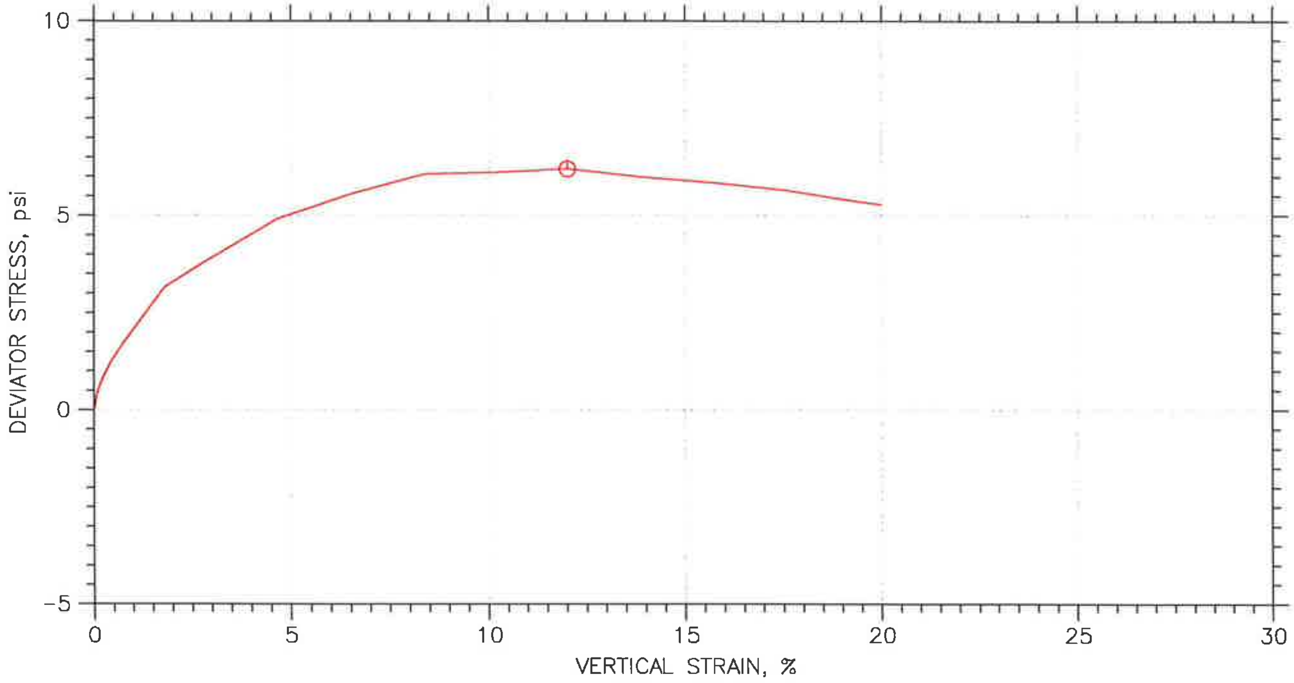
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB4@8.6'	13-235	8.6-9.1	JMA	4/10/13			13-235 MSE.dat

Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB3	Sample Type: 2.5"calbrl	
Description: Blue Gray SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:25:53

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST



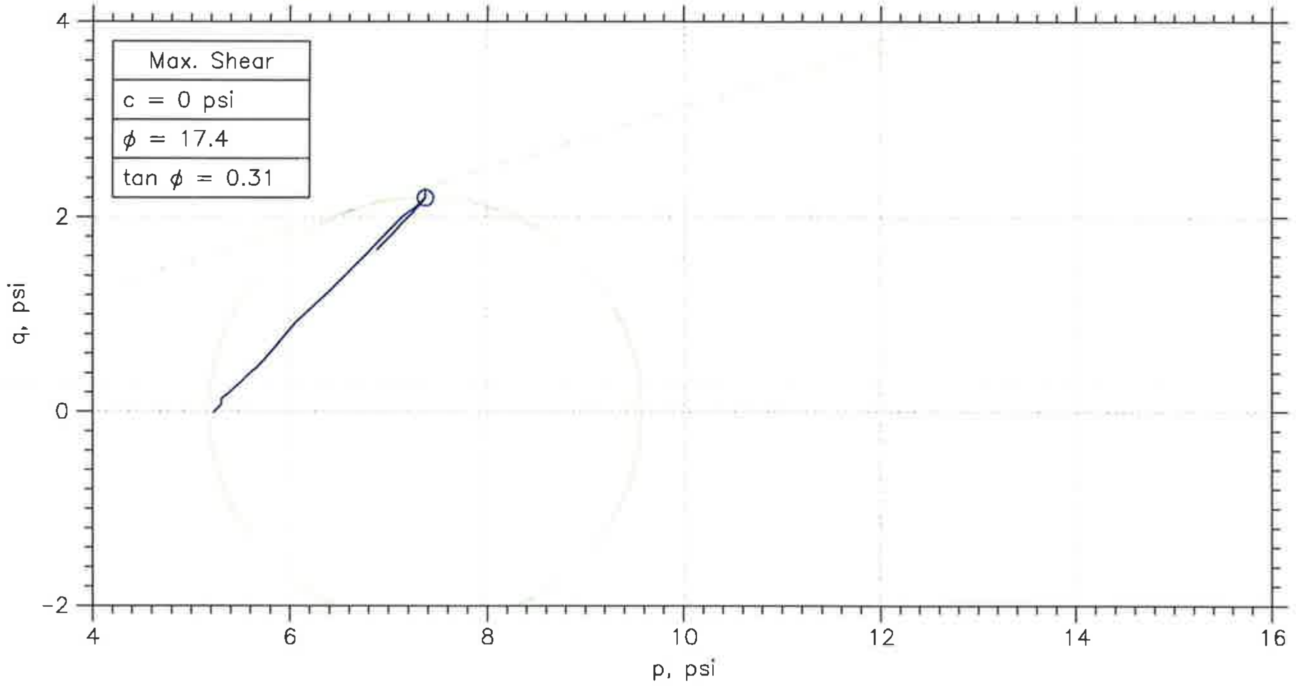


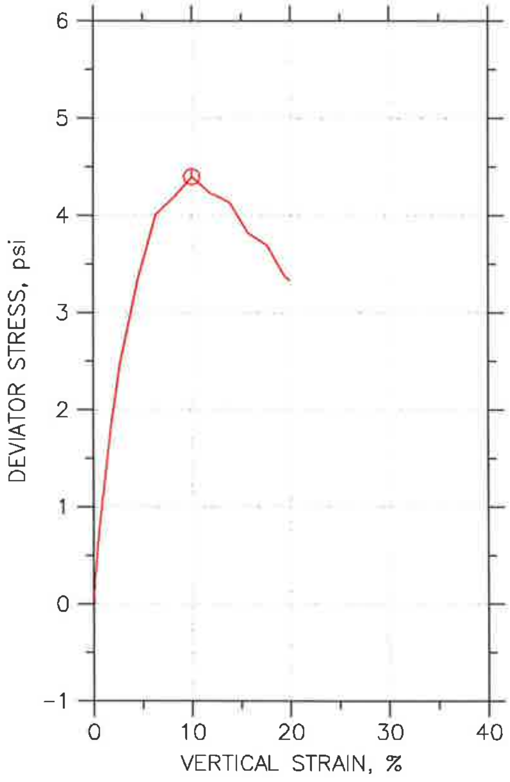
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⏀	HB4@8.6'	13-235	8.6-9.1	JMA	4/10/13			13-235 MSE.dat

Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB3	Sample Type: 2.5"calbrl	
Description: Blue Gray SILT		
Remarks: Unconsolidated Undrained		

Thu, 18-APR-2013 12:25:53

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





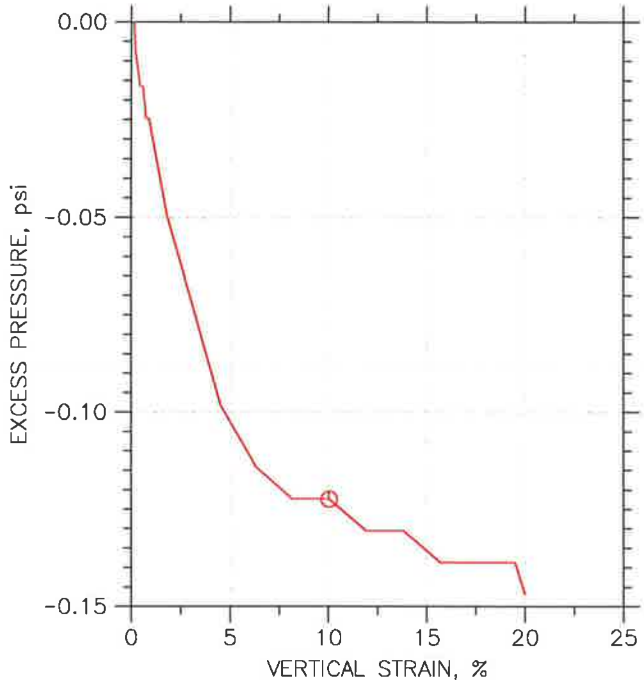
Symbol		○			
Sample No.		HB12@5'			
Test No.		13-243			
Depth		5-5.5			
Initial	Diameter, in	2.38			
	Height, in	5.07			
	Water Content, %	45.3			
	Dry Density, pcf	73.01			
	Saturation, %	95.6			
	Void Ratio	1.24			
Before Shear	Water Content, %	45.3			
	Dry Density, pcf	73.17			
	Saturation*, %	96.1			
	Void Ratio	1.24			
	Back Press., psi	.E-17			
Ver. Eff. Cons. Stress, psi		5.207			
Shear Strength, psi		2.201			
Strain at Failure, %		10.1			
Strain Rate, %/min		1			
B-Value		---			
Estimated Specific Gravity		2.62			
Liquid Limit		---			
Plastic Limit		---			

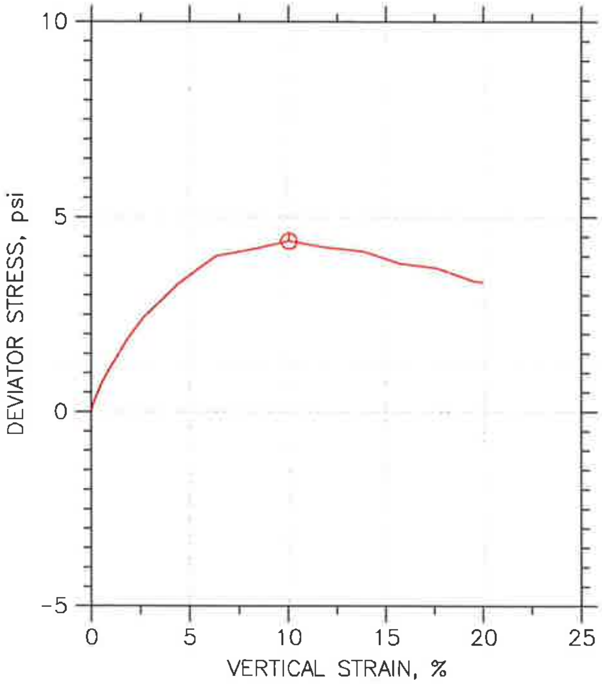
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB12@5-5.5
Sample Type: 2.5"calbrl
Description: Brown SILT
Remarks: Unconsolidated undrained

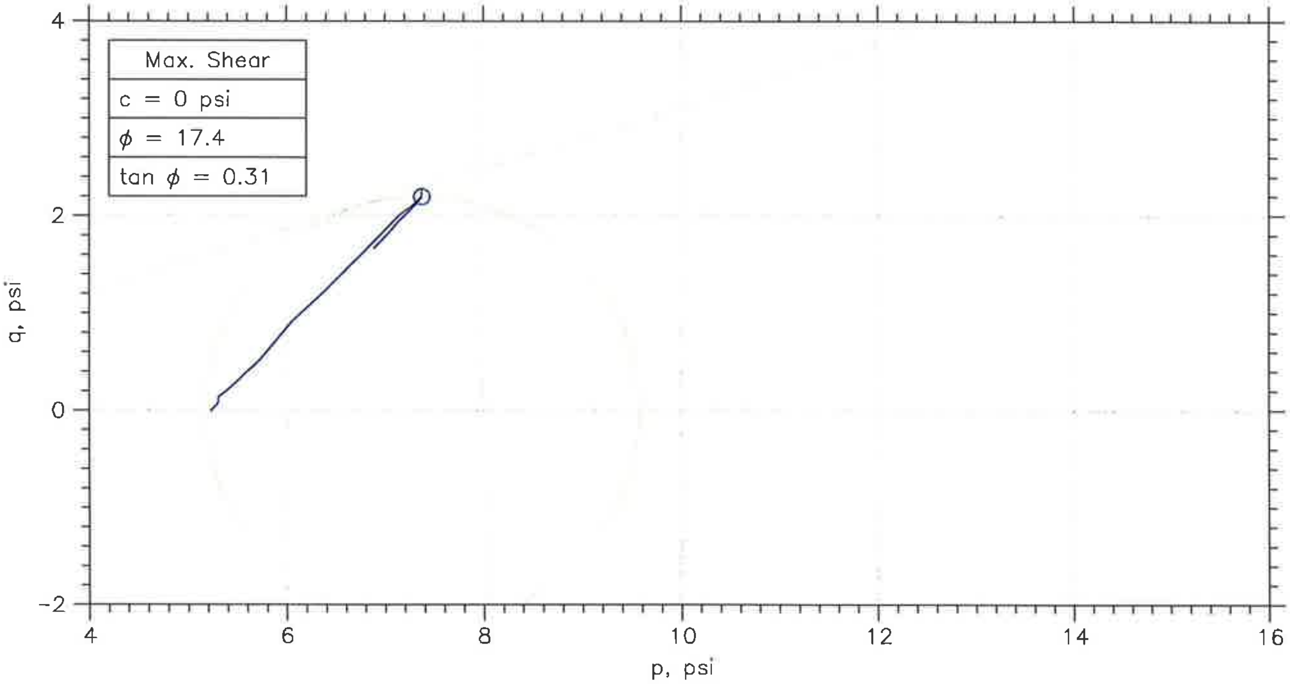
Phase calculations based on start and end of test.
* Saturation is set to 100% for phase calculations.

Thu, 18-APR-2013 12:27:22

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





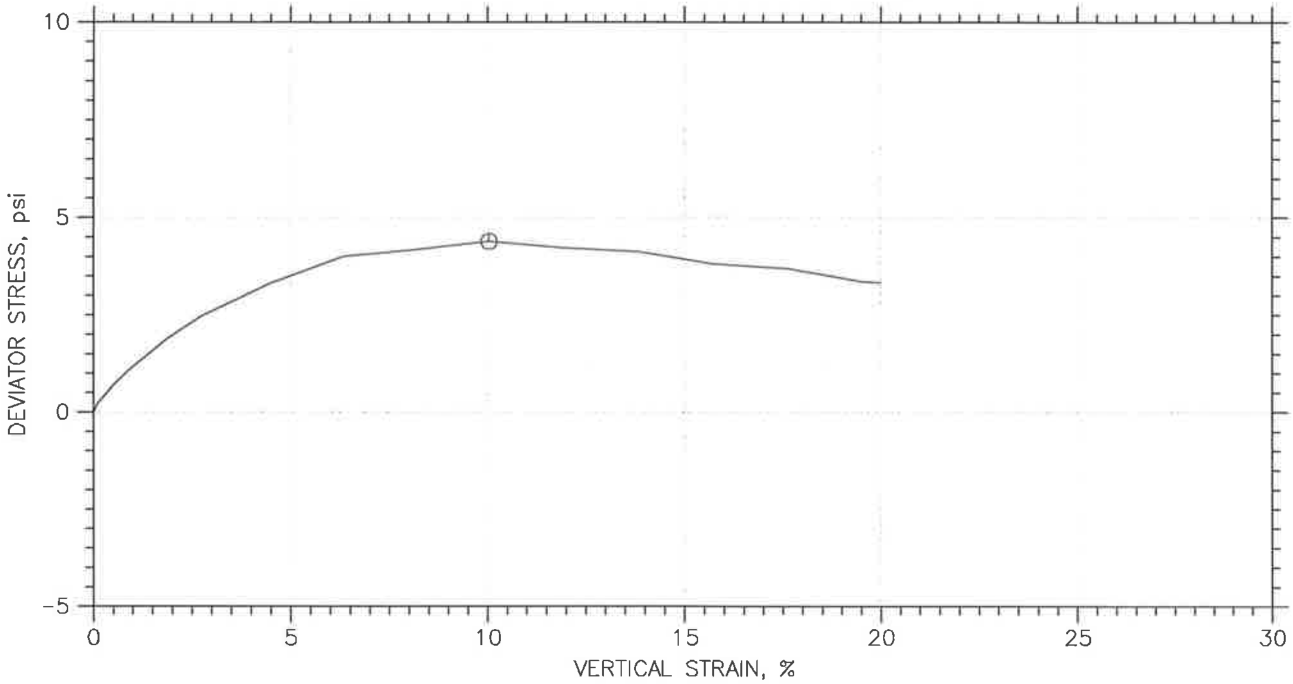


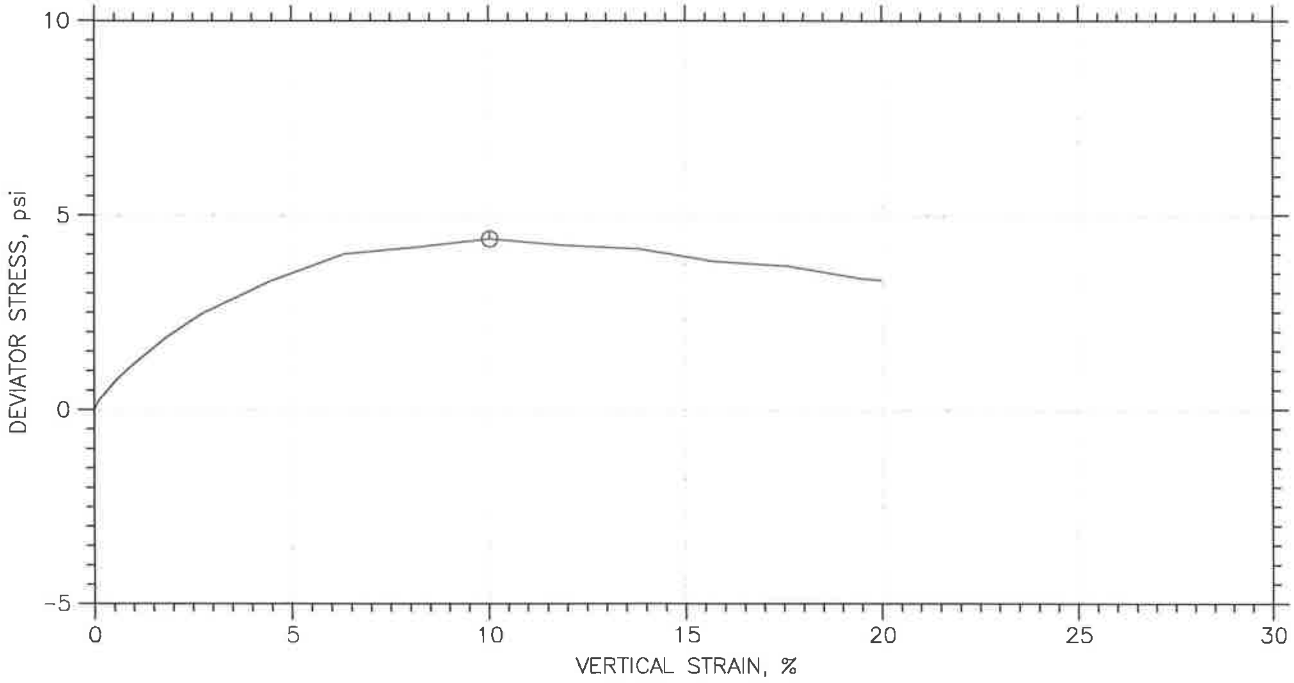
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	HB12@5'	13-243	5-5.5	JMA	4/11/13			13-243 MSE.dat

Project: Martin Slough Enhancem	Location: Eureka	Project No.: 013035
Boring No.: HB12@5-5.5	Sample Type: 2.5"calbrl	
Description: Brown SILT		
Remarks: Unconsolidated undrained		

Thu, 18-APR-2013 12:27:22

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB12@5'	13-243	5-5.5	JMA	4/11/13			13-243 MSE.dat

Project: Martin Slough Enhancem	Location: Eureka	Project No.: 013035
Boring No.: HB12@5-5.5	Sample Type: 2.5"calbrl	

Description: Brown SILT

Remarks: Unconsolidated undrained

Thu, 18-APR-2013 12:27:23

Appendix C

USDA Laboratory Test Results

A & L WESTERN AGRICULTURAL LABORATORIES

1311 WOODLAND AVE #1 • MODESTO, CALIFORNIA 95351 • (209) 529-4080 • FAX (209) 529-4736

REPORT NUMBER: 13-101-050

CLIENT NO: 2946-D

SEND TO: SHN CONSULTING ENGINEERS
812 W. WABASH
EUREKA, CA 95501-

SUBMITTED BY: CINDY WILCOX

GROWER: RC4A-013035

DATE OF REPORT: 04/17/13

SOIL ANALYSIS REPORT

PAGE: 1

SAMPLE ID	LAB NUMBER	Organic Matter		Phosphorus		Potassium	Magnesium	Calcium	Sodium	pH		Hydrogen	Cation Exchange Capacity	PERCENT CATION SATURATION (COMPUTED)				
		* % Rating	** ENR lbs/A	P1 (Weak Bray) **** * ppm	$NaHCO_3$-P (OlsenMethod) **** * ppm	K ***** * ppm	Mg *** * ppm	Ca *** * ppm	Na *** * ppm	Soil pH	Buffer Index	H meq/100g	C.E.C. meq/100g	K %	Mg %	Ca %	H %	Na %
HB-5B	54356	1.3L	57	5VL	9**	129M	355M	276VL	503VH	4.6	6.3	8.0	14.8	2.2	19.7	9.3	54.0	14.8
HB-6	54357	2.6M	82	3VL	19**	42L	276M	300VL	154H	4.4	6.3	7.4	12.0	0.9	19.0	12.5	62.0	5.6
HB-8A	54358	3.9H	108	2VL	11**	45L	495VH	836VL	104M	5.4	6.6	3.5	12.3	0.9	33.0	33.9	28.5	3.7
HB-8B	54359	3.3M	96	1VL	6**	52L	579VH	685VL	177H	5.7	6.7	2.4	11.5	1.1	41.4	29.7	21.0	6.7
HB-11	54360	3.8H	107	1VL	8**	53L	637VH	643VL	128M	5.2	6.5	4.8	13.9	1.0	37.6	23.0	34.5	4.0

** NaHCO3-P unreliable at this soil pH

SAMPLE NUMBER	Nitrogen NO_3-N ppm	Sulfur SO_4-S ppm	Zinc Zn ppm	Manganese Mn ppm	Iron Fe ppm	Copper Cu ppm	Boron B ppm	Excess Lime Rating	Soluble Salts mmhos/cm	Chloride Cl ppm		PARTICLE SIZE ANALYSIS SAND %	SILT %	CLAY %	SOIL TEXTURE
HB-5B	1VL	67VH						L	1.8M						
HB-6	5L	64VH						L	0.6L						
HB-8A	8L	13M						L	0.3L						
HB-8B	3VL	19M						L	0.4L						
HB-11	4VL	33H						L	0.3L						

* CODE TO RATING: VERY LOW (VL), LOW (L), MEDIUM (M), HIGH (H), AND VERY HIGH (VH).
** ENR - ESTIMATED NITROGEN RELEASE
*** MULTIPLY THE RESULTS IN ppm BY 2 TO CONVERT TO LBS. PER ACRE OF THE ELEMENTAL FORM
**** MULTIPLY THE RESULTS IN ppm BY 4.6 TO CONVERT TO LBS. PER ACRE P_2O_5
***** MULTIPLY THE RESULTS IN ppm BY 2.4 TO CONVERT TO LBS. PER ACRE K_2O
MOST SOILS WEIGH TWO (2) MILLION POUNDS (DRY WEIGHT) FOR AN ACRE OF SOIL 6-2/3 INCHES DEEP

This report applies only to the sample(s) tested. Samples are retained a maximum of thirty days after testing.

M Buttress
Mike Buttress, CPAg
A & L WESTERN LABORATORIES, INC.

A & L WESTERN AGRICULTURAL LABORATORIES
1311 WOODLAND AVE #1 • MODESTO, CALIFORNIA 95351 • (209) 529-4080 • FAX (209) 529-4736

REPORT NUMBER: 13-101-050

CLIENT NO: 2946-D

SEND TO: SHN CONSULTING ENGINEERS
812 W. WABASH
EUREKA, CA 95501-

SUBMITTED BY: CINDY WILCOX

GROWER: RC4A-013035

DATE OF REPORT: 04/17/13

SOIL ANALYSIS REPORT

PAGE: 2

SAMPLE ID	LAB NUMBER	Organic Matter		Phosphorus		Potassium	Magnesium	Calcium	Sodium	pH		Hydrogen	Cation Exchange Capacity	PERCENT CATION SATURATION (COMPUTED)				
		* % Rating	** ENR lbs/A	P1 (Weak Bray) **** * ppm	$NaHCO_3$-P (OlsenMethod) **** * ppm	K ***** * ppm	Mg *** * ppm	Ca *** * ppm	Na *** * ppm	Soil pH	Buffer Index	H meq/100g	C.E.C. meq/100g	K %	Mg %	Ca %	H %	Na %
HB-13	54361	1.9L	69	1VL	8**	111L	480H	395VL	315H	4.5	6.1	10.5	18.0	1.6	21.9	10.9	58.0	7.6
HB14A	54362	3.0M	90	1VL	10**	47L	282VH	343VL	160H	4.8	6.6	4.3	9.1	1.3	25.4	18.7	47.0	7.6
HB14B	54363	1.1L	51	1VL	16**	58M	270VH	223VL	321VH	5.4	6.7	1.9	6.8	2.2	32.5	16.3	28.5	20.5
HB-15	54364	3.5M	100	2VL	17**	40L	220H	298VL	73M	4.5	6.5	5.1	8.9	1.1	20.5	16.8	58.0	3.6

** NaHCO3-P unreliable at this soil pH

SAMPLE NUMBER	Nitrogen NO_3-N ppm	Sulfur SO_4-S ppm	Zinc Zn ppm	Manganese Mn ppm	Iron Fe ppm	Copper Cu ppm	Boron B ppm	Excess Lime Rating	Soluble Salts mmhos/cm	Chloride Cl ppm		PARTICLE SIZE ANALYSIS SAND %	SILT %	CLAY %	SOIL TEXTURE
HB-13	1VL	228VH						L	1.8M						
HB14A	5L	58VH						L	0.4L						
HB14B	1VL	34H						L	0.5L						
HB-15	3VL	44VH						L	0.2VL						

* CODE TO RATING: VERY LOW (VL), LOW (L), MEDIUM (M), HIGH (H), AND VERY HIGH (VH).
** ENR - ESTIMATED NITROGEN RELEASE
*** MULTIPLY THE RESULTS IN ppm BY 2 TO CONVERT TO LBS. PER ACRE OF THE ELEMENTAL FORM
**** MULTIPLY THE RESULTS IN ppm BY 4.6 TO CONVERT TO LBS. PER ACRE P_2O_5
***** MULTIPLY THE RESULTS IN ppm BY 2.4 TO CONVERT TO LBS. PER ACRE K_2O
MOST SOILS WEIGH TWO (2) MILLION POUNDS (DRY WEIGHT) FOR AN ACRE OF SOIL 6-2/3 INCHES DEEP

This report applies only to the sample(s) tested. Samples are retained a maximum of thirty days after testing.

M Buttress
Mike Buttress, CPAg
A & L WESTERN LABORATORIES, INC.

A & L WESTERN AGRICULTURAL LABORATORIES

1311 WOODLAND AVE #1 • MODESTO, CALIFORNIA 95351 • (209) 529-4080 • FAX (209) 529-4736

REPORT NUMBER: 13-101-049

CLIENT NO: 2946-D

SEND TO: SHN CONSULTING ENGINEERS
812 W. WABASH
EUREKA, CA 95501-

SUBMITTED BY: CINDY WILCOX

GROWER: RC4A-013035

DATE OF REPORT: 04/17/13

SOIL ANALYSIS REPORT

PAGE: 1

SAMPLE ID	LAB NUMBER	Organic Matter		Phosphorus		Potassium	Magnesium	Calcium	Sodium	pH		Hydrogen	Cation Exchange Capacity	PERCENT CATION SATURATION (COMPUTED)				
		* % Rating	** ENR lbs/A	P1 (Weak Bray) **** * ppm	$NaHCO_3$-P (OlsenMethod) **** * ppm	K ***** * ppm	Mg *** * ppm	Ca *** * ppm	Na *** * ppm	Soil pH	Buffer Index	H meq/100g	C.E.C. meq/100g	K %	Mg %	Ca %	H %	Na %
HB-2A	54352	3.4M	99	13L	20**	145M	246H	259VL	258VH	4.9	6.6	3.8	8.6	4.3	23.6	15.0	44.0	13.1
HB-2B	54353	2.7M	83	8L	17**	142M	361H	312VL	521VH	4.9	6.5	5.6	12.8	2.8	23.3	12.2	44.0	17.7
HB-5A	54354	5.7VH	145	14L	39**	86L	184M	175VL	169H	4.2	6.3	7.9	11.3	2.0	13.4	7.7	70.4	6.5
HB-10	54355	1.1L	51	4VL	15**	170M	349H	323VL	222H	4.7	6.4	6.0	11.9	3.7	24.1	13.6	50.5	8.1

** NaHCO3-P unreliable at this soil pH

SAMPLE NUMBER	Nitrogen NO_3-N ppm	Sulfur SO_4-S ppm	Zinc Zn ppm	Manganese Mn ppm	Iron Fe ppm	Copper Cu ppm	Boron B ppm	Excess Lime Rating	Soluble Salts mmhos/cm	Chloride Cl ppm		PARTICLE SIZE ANALYSIS			
												SAND %	SILT %	CLAY %	SOIL TEXTURE
HB-2A	7L	33H	1.1M	3M	160VH	1.0M	0.5L	L	0.7M						
HB-2B	5L	45VH	1.1M	2L	158VH	1.8H	0.6M	L	1.4M						
HB-5A	3VL	57VH	0.4VL	2L	141VH	0.9M	0.4L	L	0.9M						
HB-10	1VL	60VH	1.4M	6M	150VH	3.3VH	0.7M	L	1.6M						

* CODE TO RATING: VERY LOW (VL), LOW (L), MEDIUM (M), HIGH (H), AND VERY HIGH (VH).
** ENR - ESTIMATED NITROGEN RELEASE
*** MULTIPLY THE RESULTS IN ppm BY 2 TO CONVERT TO LBS. PER ACRE OF THE ELEMENTAL FORM
**** MULTIPLY THE RESULTS IN ppm BY 4.6 TO CONVERT TO LBS. PER ACRE P_2O_5
***** MULTIPLY THE RESULTS IN ppm BY 2.4 TO CONVERT TO LBS. PER ACRE K_2O
MOST SOILS WEIGH TWO (2) MILLION POUNDS (DRY WEIGHT) FOR AN ACRE OF SOIL 6-2/3 INCHES DEEP

This report applies only to the sample(s) tested. Samples are retained a maximum of thirty days after testing.

Mike Buttress, CPAg
A & L WESTERN LABORATORIES, INC.

A & L WESTERN AGRICULTURAL LABORATORIES

1311 WOODLAND AVE #1 • MODESTO, CALIFORNIA 95351 • (209) 529-4080 • FAX (209) 529-4736

REPORT NUMBER: 13-101-049

CLIENT: 2946

SUBMITTED BY: CINDY WILCOX

SEND TO: SHN CONSULTING ENGINEERS
812 W. WABASH
EUREKA, CA 95501-

GROWER: RC4A-013035

DATE OF REPORT: 04/17/13

SOIL SALINITY ANALYSIS REPORT

PAGE: 1

Sample ID	Lab Number	SAR	ESP	Na meq/L	Ca meq/L	Mg meq/L	pH	CO_3 meq/L	HCO_3 meq/L	E.C. dS/m	Cl meq/L	B ppm	Saturation %
HB-2A	54352	10.4	12.4	6.7	0.5	0.3	4.9	0.0	1.1	0.7	5.0	0.3	54.0
HB-2B	54353	13.8	16.0	13.0	1.0	0.7	4.9	0.0	1.1	1.4	8.0	0.4	52.4
HB-5A	54354	7.0	8.3	6.3	0.8	0.8	4.2	0.0	1.4	0.9	3.6	0.3	59.3
HB-10	54355	6.8	8.1	9.9	1.5	2.7	4.7	0.0	1.1	1.6	7.4	0.7	53.2

NOTES:

M Buttress
Mike Buttress, CPAg
A & L WESTERN LABORATORIES, INC.

Appendix B

HEC-RAS Modeling Results Peak Flow

HEC-RAS   River: Martin   Reach: Mainstem   Profile: 13FEB2003 0920

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mainstem	7500	13FEB2003 0920	10-Year Storm	176.36	4.36	8.84		8.99	0.002330	3.07	57.47	24.01	0.35
Mainstem	7500	13FEB2003 0920	100 year	424.70	4.36	9.56		9.96	0.005258	5.24	104.27	98.32	0.54
Mainstem	7500	13FEB2003 0920	2 year	82.77	4.36	8.14		8.20	0.001207	1.98	41.84	20.68	0.25
Mainstem	7400	13FEB2003 0920	10-Year Storm	176.37	2.63	8.65		8.78	0.002087	2.93	74.44	101.61	0.32
Mainstem	7400	13FEB2003 0920	100 year	424.78	2.63	9.28		9.50	0.003605	4.36	182.79	247.14	0.43
Mainstem	7400	13FEB2003 0920	2 year	82.78	2.63	8.04		8.09	0.000873	1.81	45.72	18.23	0.20
Mainstem	7250			Lat Struct									
Mainstem	7100	13FEB2003 0920	10-Year Storm	255.56	2.24	8.33		8.37	0.000689	2.02	315.22	507.81	0.19
Mainstem	7100	13FEB2003 0920	100 year	422.66	2.24	8.83		8.85	0.000468	1.82	574.69	518.54	0.16
Mainstem	7100	13FEB2003 0920	2 year	121.36	2.24	7.80		7.85	0.000592	1.73	92.48	208.27	0.18
Mainstem	7000	13FEB2003 0920	10-Year Storm	255.50	1.96	8.24		8.32	0.000912	2.49	186.11	209.21	0.22
Mainstem	7000	13FEB2003 0920	100 year	422.52	1.96	8.72		8.81	0.001038	2.87	291.34	227.10	0.24
Mainstem	7000	13FEB2003 0920	2 year	121.44	1.96	7.75		7.79	0.000469	1.66	95.10	138.71	0.16
Mainstem	6900	13FEB2003 0920	10-Year Storm	255.46	1.68	8.20		8.23	0.000424	1.82	333.36	372.03	0.16
Mainstem	6900	13FEB2003 0920	100 year	422.36	1.68	8.67		8.70	0.000439	1.99	517.54	404.11	0.16
Mainstem	6900	13FEB2003 0920	2 year	121.60	1.68	7.73		7.75	0.000268	1.34	171.76	273.93	0.12
Mainstem	6800	13FEB2003 0920	10-Year Storm	255.41	1.40	8.18		8.20	0.000253	1.53	428.60	411.19	0.12
Mainstem	6800	13FEB2003 0920	100 year	422.11	1.40	8.64		8.66	0.000279	1.71	624.10	426.89	0.13
Mainstem	6800	13FEB2003 0920	2 year	121.92	1.40	7.71		7.72	0.000159	1.13	245.82	353.99	0.10
Mainstem	6700	13FEB2003 0920	10-Year Storm	255.37	1.20	8.16		8.17	0.000171	1.29	490.56	378.31	0.10
Mainstem	6700	13FEB2003 0920	100 year	421.84	1.20	8.62		8.64	0.000214	1.53	670.07	400.97	0.11
Mainstem	6700	13FEB2003 0920	2 year	122.35	1.20	7.70		7.71	0.000099	0.92	321.57	352.17	0.08
Mainstem	6600	13FEB2003 0920	10-Year Storm	255.33	1.01	8.15		8.16	0.000102	1.01	604.84	387.27	0.08
Mainstem	6600	13FEB2003 0920	100 year	421.57	1.01	8.61		8.62	0.000139	1.24	785.68	407.73	0.09
Mainstem	6600	13FEB2003 0920	2 year	122.89	1.01	7.69		7.70	0.000053	0.68	433.46	358.94	0.06
Mainstem	6500	13FEB2003 0920	10-Year Storm	255.30	0.81	8.14		8.15	0.000139	1.19	539.60	407.27	0.09
Mainstem	6500	13FEB2003 0920	100 year	421.27	0.81	8.59		8.60	0.000177	1.41	729.79	436.03	0.10
Mainstem	6500	13FEB2003 0920	2 year	123.45	0.81	7.69		7.69	0.000077	0.83	362.58	369.57	0.07
Mainstem	6400	13FEB2003 0920	10-Year Storm	255.27	0.62	8.12		8.14	0.000203	1.34	452.53	426.70	0.11
Mainstem	6400	13FEB2003 0920	100 year	420.95	0.62	8.57		8.58	0.000243	1.56	648.66	454.63	0.12
Mainstem	6400	13FEB2003 0920	2 year	123.91	0.62	7.67		7.68	0.000119	0.96	270.90	380.51	0.08
Mainstem	6300	13FEB2003 0920	10-Year Storm	255.23	0.42	8.10		8.12	0.000174	1.30	429.67	459.37	0.10
Mainstem	6300	13FEB2003 0920	100 year	420.60	0.42	8.54		8.56	0.000222	1.55	638.59	495.32	0.11
Mainstem	6300	13FEB2003 0920	2 year	124.50	0.42	7.66		7.67	0.000083	0.86	257.58	321.41	0.07
Mainstem	6250			Lat Struct									
Mainstem	6200	13FEB2003 0920	10-Year Storm	255.21	0.36	8.09		8.11	0.000130	1.14	467.18	447.93	0.09
Mainstem	6200	13FEB2003 0920	100 year	420.23	0.36	8.52		8.54	0.000176	1.40	664.62	469.75	0.11
Mainstem	6200	13FEB2003 0920	2 year	125.29	0.36	7.66		7.67	0.000061	0.74	295.02	346.86	0.06
Mainstem	6100	13FEB2003 0920	10-Year Storm	255.18	0.31	8.08		8.09	0.000104	1.07	505.63	381.75	0.08
Mainstem	6100	13FEB2003 0920	100 year	419.88	0.31	8.51		8.52	0.000153	1.35	670.21	394.13	0.10
Mainstem	6100	13FEB2003 0920	2 year	126.04	0.31	7.65		7.66	0.000047	0.69	349.15	343.86	0.05
Mainstem	6000	13FEB2003 0920	10-Year Storm	255.16	0.25	8.07		8.09	0.000157	1.19	406.37	368.73	0.10
Mainstem	6000	13FEB2003 0920	100 year	419.55	0.25	8.48		8.51	0.000226	1.49	561.87	379.46	0.12
Mainstem	6000	13FEB2003 0920	2 year	126.64	0.25	7.65		7.65	0.000062	0.75	269.51	281.14	0.06
Mainstem	5900	13FEB2003 0920	10-Year Storm	284.20	0.20	8.02	2.87	8.06	0.000313	1.75	163.72	149.22	0.14
Mainstem	5900	13FEB2003 0920	100 year	419.33	0.20	8.40	3.54	8.48	0.000506	2.32	232.70	216.81	0.18
Mainstem	5900	13FEB2003 0920	2 year	140.50	0.20	7.63	1.97	7.64	0.000095	0.94	149.71	31.30	0.08
Mainstem	5840			Bridge									
Mainstem	5800	13FEB2003 0920	10-Year Storm	284.20	0.14	8.01		8.04	0.000199	1.47	333.24	279.98	0.11
Mainstem	5800	13FEB2003 0920	100 year	419.33	0.14	8.40		8.44	0.000263	1.77	499.22	449.17	0.13
Mainstem	5800	13FEB2003 0920	2 year	140.50	0.14	7.62		7.64	0.000076	0.88	229.52	254.53	0.07
Mainstem	5700	13FEB2003 0920	10-Year Storm	284.18	0.08	7.99		8.02	0.000188	1.44	345.38	281.83	0.11
Mainstem	5700	13FEB2003 0920	100 year	418.92	0.08	8.38		8.41	0.000249	1.73	514.61	450.82	0.13
Mainstem	5700	13FEB2003 0920	2 year	140.88	0.08	7.62		7.63	0.000072	0.86	243.38	262.47	0.07
Mainstem	5600	13FEB2003 0920	10-Year Storm	284.16	0.03	7.98		8.00	0.000172	1.28	420.15	366.73	0.10
Mainstem	5600	13FEB2003 0920	100 year	418.52	0.03	8.36		8.38	0.000212	1.48	585.73	454.57	0.11
Mainstem	5600	13FEB2003 0920	2 year	141.40	0.03	7.61		7.62	0.000066	0.79	299.26	310.23	0.06
Mainstem	5550			Lat Struct									
Mainstem	5500	13FEB2003 0920	10-Year Storm	284.13	-0.03	7.97		7.98	0.000131	1.20	510.43	449.37	0.09
Mainstem	5500	13FEB2003 0920	100 year	418.05	-0.03	8.34		8.36	0.000158	1.38	683.26	464.38	0.10
Mainstem	5500	13FEB2003 0920	2 year	141.98	-0.03	7.61		7.62	0.000057	0.76	357.12	404.02	0.06
Mainstem	5400	13FEB2003 0920	10-Year Storm	284.10	-0.08	7.95		7.97	0.000149	1.29	424.57	337.76	0.10
Mainstem	5400	13FEB2003 0920	100 year	417.60	-0.08	8.32		8.35	0.000195	1.54	551.84	347.66	0.11
Mainstem	5400	13FEB2003 0920	2 year	142.52	-0.08	7.60		7.61	0.000059	0.79	311.35	309.26	0.06
Mainstem	5300	13FEB2003 0920	10-Year Storm	284.07	-0.14	7.94		7.96	0.000111	1.21	414.79	298.55	0.08
Mainstem	5300	13FEB2003 0920	100 year	417.17	-0.14	8.30		8.33	0.000160	1.51	528.63	325.54	0.10

HEC-RAS   River: Martin   Reach: Mainstem   Profile: 13FEB2003 0920 (Continued)

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mainstem	5300	13FEB2003 0920	2 year	142.94	-0.14	7.60		7.61	0.000040	0.70	318.72	263.75	0.05
Mainstem	5200	13FEB2003 0920	10-Year Storm	284.05	-0.17	7.93		7.95	0.000068	0.99	532.84	345.73	0.07
Mainstem	5200	13FEB2003 0920	100 year	416.70	-0.17	8.30		8.31	0.000099	1.23	662.07	366.45	0.08
Mainstem	5200	13FEB2003 0920	2 year	143.35	-0.17	7.60		7.60	0.000025	0.57	422.85	304.74	0.04
Mainstem	5100	13FEB2003 0920	10-Year Storm	284.01	-0.21	7.93		7.94	0.000072	1.01	537.84	444.41	0.07
Mainstem	5100	13FEB2003 0920	100 year	416.07	-0.21	8.28		8.30	0.000101	1.24	727.30	576.21	0.08
Mainstem	5100	13FEB2003 0920	2 year	143.77	-0.21	7.59		7.60	0.000027	0.59	403.95	364.55	0.04
Mainstem	5000.*	13FEB2003 0920	10-Year Storm	283.97	-0.25	7.92		7.93	0.000075	1.02	524.00	475.23	0.07
Mainstem	5000.*	13FEB2003 0920	100 year	415.25	-0.25	8.28		8.29	0.000106	1.26	728.60	643.55	0.09
Mainstem	5000.*	13FEB2003 0920	2 year	144.24	-0.25	7.59		7.60	0.000027	0.59	384.66	363.87	0.04
Mainstem	4900	13FEB2003 0920	10-Year Storm	283.91	-0.28	7.91		7.92	0.000066	0.94	623.18	594.68	0.07
Mainstem	4900	13FEB2003 0920	100 year	414.34	-0.28	8.27		8.28	0.000090	1.14	861.74	763.96	0.08
Mainstem	4900	13FEB2003 0920	2 year	144.85	-0.28	7.59		7.59	0.000026	0.57	447.91	485.28	0.04
Mainstem	4850			Lat Struct									
Mainstem	4800	13FEB2003 0920	10-Year Storm	303.18	-0.30	7.90		7.92	0.000074	1.03	579.67	557.46	0.07
Mainstem	4800	13FEB2003 0920	100 year	421.79	-0.30	8.26		8.27	0.000092	1.19	810.28	694.60	0.08
Mainstem	4800	13FEB2003 0920	2 year	153.83	-0.30	7.59		7.59	0.000026	0.59	428.27	366.29	0.04
Mainstem	4700	13FEB2003 0920	10-Year Storm	303.13	-0.32	7.89		7.91	0.000098	1.17	382.30	411.08	0.08
Mainstem	4700	13FEB2003 0920	100 year	420.91	-0.32	8.24		8.27	0.000131	1.40	561.06	559.84	0.10
Mainstem	4700	13FEB2003 0920	2 year	154.28	-0.32	7.58		7.59	0.000032	0.65	296.04	225.39	0.05
Mainstem	4600	13FEB2003 0920	10-Year Storm	303.10	-0.34	7.88		7.90	0.000104	1.18	324.31	295.74	0.08
Mainstem	4600	13FEB2003 0920	100 year	420.10	-0.34	8.23		8.26	0.000150	1.47	475.71	508.96	0.10
Mainstem	4600	13FEB2003 0920	2 year	154.58	-0.34	7.58		7.58	0.000033	0.65	271.61	111.01	0.05
Mainstem	4500	13FEB2003 0920	10-Year Storm	303.07	-0.36	7.87		7.89	0.000162	1.20	265.21	254.74	0.10
Mainstem	4500	13FEB2003 0920	100 year	419.50	-0.36	8.21		8.24	0.000243	1.46	375.01	345.29	0.12
Mainstem	4500	13FEB2003 0920	2 year	154.71	-0.36	7.57		7.58	0.000046	0.65	236.65	52.00	0.05
Mainstem	4400	13FEB2003 0920	10-Year Storm	303.02	-0.38	7.85		7.88	0.000117	1.22	289.68	303.07	0.09
Mainstem	4400	13FEB2003 0920	100 year	419.00	-0.38	8.19		8.22	0.000168	1.50	419.83	424.68	0.11
Mainstem	4400	13FEB2003 0920	2 year	154.78	-0.38	7.57		7.58	0.000036	0.66	233.97	42.11	0.05
Mainstem	4300	13FEB2003 0920	10-Year Storm	302.96	-0.40	7.85		7.87	0.000099	1.15	392.31	386.20	0.08
Mainstem	4300	13FEB2003 0920	100 year	418.35	-0.40	8.18		8.20	0.000135	1.39	527.73	439.02	0.10
Mainstem	4300	13FEB2003 0920	2 year	154.98	-0.40	7.57		7.57	0.000033	0.65	292.21	295.11	0.05
Mainstem	4200	13FEB2003 0920	10-Year Storm	302.89	-0.42	7.84		7.85	0.000068	0.97	549.35	429.57	0.07
Mainstem	4200	13FEB2003 0920	100 year	417.65	-0.42	8.17		8.19	0.000091	1.16	696.16	460.30	0.08
Mainstem	4200	13FEB2003 0920	2 year	155.39	-0.42	7.57		7.57	0.000024	0.56	436.36	386.87	0.04
Mainstem	4100	13FEB2003 0920	10-Year Storm	302.80	-0.44	7.83		7.85	0.000074	1.01	503.30	433.21	0.07
Mainstem	4100	13FEB2003 0920	100 year	416.93	-0.44	8.16		8.18	0.000097	1.21	650.19	466.19	0.08
Mainstem	4100	13FEB2003 0920	2 year	155.83	-0.44	7.56		7.57	0.000026	0.58	391.89	389.94	0.04
Mainstem	4000	13FEB2003 0920	10-Year Storm	302.71	-0.46	7.83		7.84	0.000077	1.02	473.79	365.83	0.07
Mainstem	4000	13FEB2003 0920	100 year	416.12	-0.46	8.15		8.17	0.000109	1.23	601.16	416.64	0.09
Mainstem	4000	13FEB2003 0920	2 year	156.21	-0.46	7.56		7.57	0.000025	0.58	384.50	306.56	0.04
Mainstem	3900	13FEB2003 0920	10-Year Storm	302.62	-0.47	7.82		7.83	0.000079	1.05	426.99	319.79	0.07
Mainstem	3900	13FEB2003 0920	100 year	415.25	-0.47	8.14		8.16	0.000111	1.29	558.62	484.20	0.09
Mainstem	3900	13FEB2003 0920	2 year	156.50	-0.47	7.56		7.56	0.000026	0.59	353.31	259.24	0.04
Mainstem	3800	13FEB2003 0920	10-Year Storm	302.49	-0.49	7.81		7.83	0.000066	0.99	526.84	481.57	0.07
Mainstem	3800	13FEB2003 0920	100 year	414.40	-0.49	8.13		8.14	0.000089	1.19	700.08	587.65	0.08
Mainstem	3800	13FEB2003 0920	2 year	156.83	-0.49	7.56		7.56	0.000023	0.57	419.55	318.97	0.04
Mainstem	3700	13FEB2003 0920	10-Year Storm	302.31	-0.51	7.81		7.82	0.000067	1.00	554.10	597.97	0.07
Mainstem	3700	13FEB2003 0920	100 year	413.44	-0.51	8.12		8.14	0.000085	1.16	746.71	626.24	0.08
Mainstem	3700	13FEB2003 0920	2 year	157.21	-0.51	7.55		7.56	0.000023	0.57	431.10	382.32	0.04
Mainstem	3600	13FEB2003 0920	10-Year Storm	302.12	-0.52	7.80		7.81	0.000074	1.05	459.86	433.89	0.07
Mainstem	3600	13FEB2003 0920	100 year	412.19	-0.52	8.11		8.13	0.000101	1.26	643.65	680.19	0.08
Mainstem	3600	13FEB2003 0920	2 year	157.52	-0.52	7.55		7.56	0.000025	0.59	368.33	318.11	0.04
Mainstem	3250			Lat Struct									
Mainstem	3200	13FEB2003 0920	10-Year Storm	322.87	-0.59	7.78		7.79	0.000048	0.92	439.25	101.95	0.06
Mainstem	3200	13FEB2003 0920	100 year	409.11	-0.59	8.08		8.09	0.000065	1.11	470.12	102.96	0.07
Mainstem	3200	13FEB2003 0920	2 year	166.25	-0.59	7.55		7.55	0.000015	0.50	415.98	100.92	0.03
Mainstem	3100	13FEB2003 0920	10-Year Storm	322.78	-0.60	7.77		7.78	0.000048	0.92	439.40	101.98	0.06
Mainstem	3100	13FEB2003 0920	100 year	408.87	-0.60	8.07		8.09	0.000065	1.10	470.09	103.01	0.07
Mainstem	3100	13FEB2003 0920	2 year	166.29	-0.60	7.54		7.55	0.000014	0.50	416.47	100.96	0.03
Mainstem	3050			Lat Struct									
Mainstem	3000	13FEB2003 0920	10-Year Storm	310.25	-0.62	7.77		7.78	0.000044	0.89	440.04	102.01	0.06
Mainstem	3000	13FEB2003 0920	100 year	432.88	-0.62	8.06		8.08	0.000073	1.17	469.99	103.06	0.07
Mainstem	3000	13FEB2003 0920	2 year	165.45	-0.62	7.54		7.55	0.000014	0.49	417.31	101.01	0.03
Mainstem	2900	13FEB2003 0920	10-Year Storm	284.76	-0.63	7.77		7.77	0.000037	0.81	440.46	102.05	0.05
Mainstem	2900	13FEB2003 0920	100 year	393.91	-0.63	8.06		8.07	0.000060	1.06	470.38	103.10	0.07

HEC-RAS River: Martin Reach: Mainstem Profile: 13FEB2003 0920 (Continued)

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mainstem	2900	13FEB2003 0920	2 year	156.65	-0.63	7.54		7.55	0.000013	0.47	417.80	101.04	0.03
Mainstem	2800	13FEB2003 0920	10-Year Storm	262.71	-0.65	7.76		7.77	0.000031	0.75	441.28	102.09	0.05
Mainstem	2800	13FEB2003 0920	100 year	360.25	-0.65	8.06		8.07	0.000050	0.97	471.17	103.15	0.06
Mainstem	2800	13FEB2003 0920	2 year	148.81	-0.65	7.54		7.54	0.000011	0.44	418.69	101.09	0.03
Mainstem	2700	13FEB2003 0920	10-Year Storm	243.68	-0.67	7.76		7.77	0.000027	0.69	442.12	102.13	0.04
Mainstem	2700	13FEB2003 0920	100 year	331.07	-0.67	8.05		8.06	0.000042	0.89	471.97	103.20	0.06
Mainstem	2700	13FEB2003 0920	2 year	141.80	-0.67	7.54		7.54	0.000010	0.42	419.59	101.14	0.03
Mainstem	2600	13FEB2003 0920	10-Year Storm	227.23	-0.68	7.76		7.77	0.000023	0.64	442.56	102.17	0.04
Mainstem	2600	13FEB2003 0920	100 year	305.87	-0.68	8.05		8.06	0.000036	0.82	472.38	103.24	0.05
Mainstem	2600	13FEB2003 0920	2 year	135.54	-0.68	7.54		7.54	0.000009	0.40	420.09	101.17	0.03
Mainstem	2500	13FEB2003 0920	10-Year Storm	212.94	-0.70	7.76		7.77	0.000020	0.60	443.42	102.21	0.04
Mainstem	2500	13FEB2003 0920	100 year	283.97	-0.70	8.05		8.06	0.000031	0.76	473.22	103.29	0.05
Mainstem	2500	13FEB2003 0920	2 year	129.94	-0.70	7.54		7.54	0.000009	0.38	420.99	101.22	0.03
Mainstem	2400	13FEB2003 0920	10-Year Storm	200.49	-0.71	7.76		7.76	0.000018	0.57	443.94	102.26	0.04
Mainstem	2400	13FEB2003 0920	100 year	264.67	-0.71	8.05		8.06	0.000026	0.71	473.72	103.34	0.04
Mainstem	2400	13FEB2003 0920	2 year	124.92	-0.71	7.54		7.54	0.000008	0.37	421.55	101.27	0.02
Mainstem	2300	13FEB2003 0920	10-Year Storm	189.62	-0.73	7.76		7.76	0.000016	0.53	444.75	102.29	0.03
Mainstem	2300	13FEB2003 0920	100 year	247.86	-0.73	8.05		8.05	0.000023	0.66	474.50	103.38	0.04
Mainstem	2300	13FEB2003 0920	2 year	120.42	-0.73	7.54		7.54	0.000007	0.35	422.41	101.31	0.02
Mainstem	2200	13FEB2003 0920	10-Year Storm	180.12	-0.74	7.76		7.76	0.000014	0.51	445.29	102.34	0.03
Mainstem	2200	13FEB2003 0920	100 year	233.05	-0.74	8.05		8.05	0.000020	0.62	475.03	103.43	0.04
Mainstem	2200	13FEB2003 0920	2 year	116.37	-0.74	7.54		7.54	0.000007	0.34	422.98	101.35	0.02
Mainstem	2100	13FEB2003 0920	10-Year Storm	171.80	-0.76	7.76		7.76	0.000013	0.48	446.15	102.37	0.03
Mainstem	2100	13FEB2003 0920	100 year	219.92	-0.76	8.05		8.05	0.000018	0.58	475.87	103.45	0.04
Mainstem	2100	13FEB2003 0920	2 year	112.73	-0.76	7.54		7.54	0.000006	0.33	423.88	101.39	0.02
Mainstem	2000	13FEB2003 0920	10-Year Storm	164.51	-0.77	7.76		7.76	0.000012	0.46	446.65	102.42	0.03
Mainstem	2000	13FEB2003 0920	100 year	208.30	-0.77	8.04		8.05	0.000016	0.55	476.37	103.52	0.04
Mainstem	2000	13FEB2003 0920	2 year	109.46	-0.77	7.54		7.54	0.000006	0.32	424.42	101.44	0.02
Mainstem	1900	13FEB2003 0920	10-Year Storm	158.10	-0.79	7.76		7.76	0.000011	0.44	447.55	102.47	0.03
Mainstem	1900	13FEB2003 0920	100 year	197.92	-0.79	8.04		8.05	0.000014	0.52	477.27	103.57	0.03
Mainstem	1900	13FEB2003 0920	2 year	107.61	-0.79	7.54		7.54	0.000006	0.31	425.34	101.49	0.02
Mainstem	1800	13FEB2003 0920	10-Year Storm	152.61	-0.81	7.75		7.76	0.000010	0.43	448.44	102.51	0.03
Mainstem	1800	13FEB2003 0920	100 year	188.69	-0.81	8.04		8.05	0.000013	0.50	478.15	103.61	0.03
Mainstem	1800	13FEB2003 0920	2 year	106.75	-0.81	7.54		7.54	0.000005	0.31	426.24	101.53	0.02
Mainstem	1700	13FEB2003 0920	10-Year Storm	149.87	-0.82	7.75		7.76	0.000010	0.42	448.94	102.55	0.03
Mainstem	1700	13FEB2003 0920	100 year	183.41	-0.82	8.04		8.04	0.000012	0.48	478.65	103.66	0.03
Mainstem	1700	13FEB2003 0920	2 year	106.50	-0.82	7.54		7.54	0.000005	0.31	426.77	101.57	0.02
Mainstem	1600	13FEB2003 0920	10-Year Storm	148.87	-0.84	7.75		7.76	0.000009	0.41	449.83	102.60	0.03
Mainstem	1600	13FEB2003 0920	100 year	181.00	-0.84	8.04		8.04	0.000012	0.48	479.53	103.71	0.03
Mainstem	1600	13FEB2003 0920	2 year	106.54	-0.84	7.54		7.54	0.000005	0.31	427.68	101.62	0.02
Mainstem	1500	13FEB2003 0920	10-Year Storm	148.68	-0.85	7.75		7.75	0.000009	0.41	450.36	102.64	0.03
Mainstem	1500	13FEB2003 0920	100 year	180.22	-0.85	8.04		8.04	0.000012	0.47	480.06	103.76	0.03
Mainstem	1500	13FEB2003 0920	2 year	106.68	-0.85	7.54		7.54	0.000005	0.31	428.26	101.67	0.02
Mainstem	1400	13FEB2003 0920	10-Year Storm	148.68	-0.87	7.75		7.75	0.000010	0.44	358.51	54.28	0.03
Mainstem	1400	13FEB2003 0920	100 year	180.13	-0.87	8.04		8.04	0.000013	0.51	374.17	54.85	0.03
Mainstem	1400	13FEB2003 0920	2 year	106.82	-0.87	7.53		7.54	0.000006	0.32	346.86	53.78	0.02
Mainstem	1300	13FEB2003 0920	10-Year Storm	148.80	-0.88	7.75		7.75	0.000007	0.37	490.55	107.23	0.02
Mainstem	1300	13FEB2003 0920	100 year	180.08	-0.88	8.04		8.04	0.000009	0.43	521.50	108.36	0.03
Mainstem	1300	13FEB2003 0920	2 year	107.07	-0.88	7.53		7.54	0.000004	0.28	467.55	106.26	0.02
Mainstem	1200	13FEB2003 0920	10-Year Storm	149.19	-0.90	7.75		7.75	0.000007	0.37	491.55	107.27	0.02
Mainstem	1200	13FEB2003 0920	100 year	180.12	-0.90	8.04		8.04	0.000009	0.43	522.49	108.40	0.03
Mainstem	1200	13FEB2003 0920	2 year	107.49	-0.90	7.53		7.54	0.000004	0.28	468.57	106.30	0.02
Mainstem	1100	13FEB2003 0920	10-Year Storm	150.50	-0.91	7.75		7.75	0.000007	0.37	492.09	107.31	0.02
Mainstem	1100	13FEB2003 0920	100 year	180.84	-0.91	8.04		8.04	0.000009	0.43	523.02	108.44	0.03
Mainstem	1100	13FEB2003 0920	2 year	108.35	-0.91	7.53		7.53	0.000004	0.28	469.13	106.34	0.02
Mainstem	1000	13FEB2003 0920	10-Year Storm	153.87	-0.92	7.75		7.75	0.000008	0.38	492.50	107.32	0.02
Mainstem	1000	13FEB2003 0920	100 year	183.19	-0.92	8.03		8.04	0.000009	0.43	523.42	108.46	0.03
Mainstem	1000	13FEB2003 0920	2 year	110.05	-0.92	7.53		7.53	0.000004	0.28	469.59	106.36	0.02
Mainstem	900	13FEB2003 0920	10-Year Storm	159.82	-0.93	7.75		7.75	0.000008	0.40	492.88	107.34	0.03
Mainstem	900	13FEB2003 0920	100 year	189.37	-0.93	8.03		8.04	0.000010	0.45	523.78	108.48	0.03
Mainstem	900	13FEB2003 0920	2 year	113.31	-0.93	7.53		7.53	0.000005	0.29	470.03	106.38	0.02
Mainstem	800	13FEB2003 0920	10-Year Storm	167.03	-0.94	7.75		7.75	0.000009	0.41	493.32	107.36	0.03
Mainstem	800	13FEB2003 0920	100 year	199.28	-0.94	8.03		8.03	0.000011	0.47	524.18	108.53	0.03
Mainstem	800	13FEB2003 0920	2 year	117.10	-0.94	7.53		7.53	0.000005	0.30	470.52	106.40	0.02
Mainstem	700	13FEB2003 0920	10-Year Storm	175.80	-0.94	7.74		7.75	0.000010	0.44	493.28	107.38	0.03
Mainstem	700	13FEB2003 0920	100 year	211.37	-0.94	8.03		8.03	0.000012	0.50	524.11	108.53	0.03
Mainstem	700	13FEB2003 0920	2 year	121.43	-0.94	7.53		7.53	0.000005	0.31	470.57	106.42	0.02

HEC-RAS River: Martin Reach: Mainstem Profile: 13FEB2003 0920 (Continued)

Reach	River Sta	Profile	Plan	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mainstem	600	13FEB2003 0920	10-Year Storm	186.68	-0.95	7.74		7.75	0.000011	0.46	493.67	107.40	0.03
Mainstem	600	13FEB2003 0920	100 year	226.46	-0.95	8.03		8.03	0.000014	0.53	524.44	108.56	0.03
Mainstem	600	13FEB2003 0920	2 year	126.40	-0.95	7.53		7.53	0.000006	0.33	471.05	106.45	0.02
Mainstem	500	13FEB2003 0920	10-Year Storm	200.49	-0.96	7.74		7.74	0.000013	0.50	493.97	107.41	0.03
Mainstem	500	13FEB2003 0920	100 year	245.93	-0.96	8.02		8.03	0.000017	0.58	524.65	108.58	0.04
Mainstem	500	13FEB2003 0920	2 year	132.17	-0.96	7.53		7.53	0.000006	0.34	471.47	106.46	0.02
Mainstem	400	13FEB2003 0920	10-Year Storm	218.74	-0.97	7.74		7.74	0.000015	0.54	494.19	107.42	0.03
Mainstem	400	13FEB2003 0920	100 year	272.32	-0.97	8.02		8.03	0.000021	0.64	524.74	108.60	0.04
Mainstem	400	13FEB2003 0920	2 year	138.92	-0.97	7.53		7.53	0.000007	0.36	471.88	106.48	0.02
Mainstem	300	13FEB2003 0920	10-Year Storm	243.95	-0.98	7.73		7.74	0.000019	0.60	494.33	107.43	0.04
Mainstem	300	13FEB2003 0920	100 year	312.34	-0.98	8.01		8.02	0.000027	0.73	524.61	108.63	0.05
Mainstem	300	13FEB2003 0920	2 year	147.05	-0.98	7.53		7.53	0.000008	0.38	472.31	106.51	0.02
Mainstem	200	13FEB2003 0920	10-Year Storm	274.86	-0.98	7.73		7.74	0.000024	0.68	493.96	107.43	0.04
Mainstem	200	13FEB2003 0920	100 year	363.98	-0.98	8.01		8.02	0.000037	0.86	523.86	108.65	0.05
Mainstem	200	13FEB2003 0920	2 year	156.98	-0.98	7.53		7.53	0.000009	0.40	472.30	106.52	0.03
Mainstem	100	13FEB2003 0920	10-Year Storm	314.78	-0.99	7.72		7.73	0.000032	0.78	493.80	107.43	0.05
Mainstem	100	13FEB2003 0920	100 year	436.87	-0.99	7.99		8.01	0.000053	1.03	522.96	108.64	0.06
Mainstem	100	13FEB2003 0920	2 year	169.25	-0.99	7.53		7.53	0.000010	0.43	472.71	106.54	0.03
Mainstem	50			Lat Struct									
Mainstem	25			Lat Struct									
Mainstem	10	13FEB2003 0920	10-Year Storm	0.11	-0.99	7.73		7.73	0.000000	0.00	495.00	107.48	0.00
Mainstem	10	13FEB2003 0920	100 year	0.12	-0.99	8.01		8.01	0.000000	0.00	525.07	108.68	0.00
Mainstem	10	13FEB2003 0920	2 year	0.10	-0.99	7.53		7.53	0.000000	0.00	473.09	106.56	0.00
Mainstem	0	13FEB2003 0920	10-Year Storm	0.10	-0.99	7.73	-0.96	7.73	0.000000	0.00	495.00	107.48	0.00
Mainstem	0	13FEB2003 0920	100 year	0.10	-0.99	8.01	-0.96	8.01	0.000000	0.00	525.07	108.68	0.00
Mainstem	0	13FEB2003 0920	2 year	0.10	-0.99	7.53	-0.96	7.53	0.000000	0.00	473.09	106.56	0.00

Appendix B

HEC-RAS Peak Water Surface Elevations

Appendix B - HEC-RAS Peak Water Surface Elevations

HEC-RAS Plan: Prop 2-Year Profile: Max WS

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Downstream Meand	0	Max WS	0.01	-0.98	7.492	-0.959	7.492	0.000000	0.00	337.79	159.00	0.00
Downstream Meand	10	Max WS	0.15	-0.98	7.492		7.492	0.000000	0.00	337.79	159.00	0.00
Downstream Meand	20	Max WS	0.31	-0.98	7.492		7.492	0.000000	0.00	337.79	159.00	0.00
Downstream Meand	25		Lat Struct									
Downstream Meand	50		Lat Struct									
Downstream Meand	100	Max WS	3.08	-0.98	7.492		7.492	0.000000	0.01	337.79	159.00	0.00
Downstream Meand	200	Max WS	11.57	-0.96	7.493		7.493	0.000000	0.04	370.94	162.86	0.00
Downstream Meand	300	Max WS	11.76	-0.95	7.493		7.493	0.000000	0.03	442.57	186.25	0.00
Downstream Meand	400	Max WS	13.54	-0.93	7.493		7.493	0.000000	0.04	352.84	97.24	0.00
Downstream Meand	500	Max WS	15.91	-0.91	7.493		7.493	0.000000	0.04	375.16	104.78	0.00
Downstream Meand	600	Max WS	18.58	-0.89	7.493		7.494	0.000000	0.05	356.39	100.88	0.00
Downstream Meand	700	Max WS	21.44	-0.87	7.494		7.494	0.000000	0.06	360.23	100.86	0.01
Downstream Meand	800	Max WS	24.51	-0.85	7.494		7.494	0.000001	0.07	356.37	102.00	0.01
Downstream Meand	900	Max WS	22.60	-0.84	7.494		7.494	0.000001	0.09	297.27	108.65	0.01
Downstream Meand	950		Lat Struct									
Downstream Meand	969.37	Max WS	27.08	-0.82	7.494		7.495	0.000001	0.11	250.24	49.03	0.01
Downstream Meand	1033.73	Max WS	27.11	-0.81	7.495		7.495	0.000001	0.11	251.11	45.63	0.01
Downstream Meand	1100	Max WS	27.16	-0.80	7.496		7.496	0.000001	0.11	266.79	68.47	0.01
Mainstem Meander	1200	Max WS	29.39	-0.78	7.496		7.496	0.000002	0.11	261.71	64.66	0.01
Mainstem Meander	1249.11	Max WS	29.44	-0.78	7.496		7.496	0.000001	0.12	242.99	48.87	0.01
Mainstem Meander	1339.63	Max WS	34.29	-0.76	7.497		7.497	0.000003	0.18	189.91	25.00	0.01
Mainstem Meander	1425.2	Max WS	34.31	-0.75	7.498		7.499	0.000003	0.18	188.09	25.00	0.01
Mainstem Meander	1476.69	Max WS	40.71	-0.74	7.499		7.500	0.000002	0.17	284.20	109.18	0.01
Mainstem Meander	1527.55	Max WS	40.72	-0.73	7.500		7.500	0.000002	0.17	277.44	97.38	0.01
Mainstem Meander	1600	Max WS	40.78	-0.72	7.501		7.501	0.000002	0.17	287.49	134.53	0.01
Mainstem Meander	1630		Lat Struct									
Mainstem Meander	1703.33	Max WS	48.15	-0.70	7.502		7.502	0.000002	0.16	370.77	167.64	0.01
Mainstem Meander	1770.97	Max WS	48.18	-0.69	7.503		7.503	0.000004	0.17	321.94	175.62	0.02
Mainstem Meander	1839.35	Max WS	115.05	-0.68	7.504		7.507	0.000023	0.41	327.47	197.04	0.04
Mainstem Meander	1885.46	Max WS	115.10	-0.67	7.505		7.508	0.000022	0.40	330.63	184.99	0.04
Mainstem	1948.75	Max WS	113.87	-0.66	7.505		7.509	0.000019	0.49	333.91	193.04	0.04
Mainstem	1997.13	Max WS	113.95	-0.66	7.506		7.510	0.000019	0.50	340.08	229.73	0.04
Mainstem	2045.07	Max WS	115.46	-0.65	7.507		7.511	0.000020	0.52	336.30	228.22	0.04
Mainstem	2100	Max WS	115.49	-0.64	7.508		7.512	0.000025	0.53	316.06	295.49	0.04
Mainstem	2200	Max WS	115.61	-0.63	7.511		7.515	0.000024	0.55	306.18	227.82	0.04
Mainstem	2300	Max WS	117.27	-0.61	7.514		7.517	0.000023	0.53	359.01	367.90	0.04
Mainstem	2400	Max WS	117.36	-0.60	7.516		7.520	0.000023	0.54	360.46	421.21	0.04
Mainstem	2500	Max WS	118.98	-0.59	7.518		7.522	0.000023	0.55	356.50	352.36	0.04
Mainstem	2600	Max WS	119.01	-0.57	7.520		7.525	0.000025	0.57	315.45	312.50	0.04
Mainstem	2700	Max WS	119.11	-0.56	7.523		7.528	0.000026	0.56	327.01	287.94	0.04
Mainstem	2800	Max WS	120.52	-0.54	7.526		7.530	0.000025	0.56	333.93	283.83	0.04
Mainstem	2900	Max WS	120.60	-0.53	7.529		7.533	0.000024	0.53	367.42	314.61	0.04
Mainstem	3000	Max WS	121.99	-0.51	7.531		7.535	0.000027	0.56	371.42	391.65	0.04
Mainstem	3100	Max WS	122.05	-0.50	7.533		7.539	0.000032	0.62	238.19	256.82	0.05
Mainstem	3200	Max WS	123.34	-0.47	7.536		7.543	0.000040	0.67	186.16	39.99	0.05
Mainstem	3273.64	Max WS	123.36	-0.45	7.539		7.546	0.000040	0.67	192.01	52.63	0.05
Mainstem	3298.06	Max WS	123.37	-0.44	7.540		7.547	0.000039	0.67	191.50	52.61	0.05
Mainstem	3352.67	Max WS	124.72	-0.42	7.542		7.549	0.000039	0.68	196.33	63.58	0.05
Mainstem	3406.58	Max WS	124.76	-0.41	7.546		7.551	0.000031	0.60	355.03	358.10	0.04
Mainstem	3498.87	Max WS	126.19	-0.38	7.549		7.554	0.000030	0.59	388.81	376.13	0.04
Mainstem	3563.55	Max WS	112.73	-0.36	7.552		7.555	0.000022	0.51	427.54	423.44	0.04
Mainstem	3610		Lat Struct									
Mainstem	3640.9	Max WS	114.23	-0.33	7.553		7.557	0.000022	0.51	381.15	368.73	0.04
Mainstem	3706.91	Max WS	114.25	-0.31	7.554		7.559	0.000029	0.58	309.64	297.15	0.04
Mainstem	3788.76	Max WS	114.28	-0.28	7.557		7.561	0.000025	0.55	348.57	272.33	0.04
Mainstem	3895.44	Max WS	115.48	-0.24	7.559		7.564	0.000027	0.57	339.62	311.76	0.04
Mainstem	4000	Max WS	115.53	-0.20	7.562		7.567	0.000028	0.56	327.22	304.95	0.04
Mainstem	4050		Lat Struct									
Mainstem	4100	Max WS	116.63	-0.16	7.565		7.571	0.000038	0.64	283.39	269.51	0.05
Mainstem	4200	Max WS	116.67	-0.12	7.569		7.574	0.000035	0.61	321.72	324.15	0.05
Mainstem	4300	Max WS	117.68	-0.08	7.573		7.576	0.000024	0.50	491.80	482.93	0.04
Mainstem	4400	Max WS	117.72	-0.04	7.575		7.580	0.000036	0.61	327.88	328.29	0.05
Mainstem	4500	Max WS	117.79	0.00	7.578		7.584	0.000037	0.63	287.33	268.46	0.05
Mainstem	4549.41	Max WS	117.82	0.03	7.578		7.588	0.000069	0.82	169.25	71.75	0.06
Mainstem	4650		Bridge									
Mainstem	4703.6	Max WS	118.50	0.13	7.586	2.157	7.601	0.000118	0.98	121.01	27.43	0.08
Mainstem	4786.07	Max WS	111.47	0.18	7.600		7.612	0.000097	0.89	167.77	231.76	0.07
Mainstem	4897.84	Max WS	112.85	0.24	7.611		7.618	0.000061	0.74	304.70	320.93	0.06
Mainstem	4982.74	Max WS	112.88	0.30	7.616		7.622	0.000053	0.70	339.33	404.54	0.06
Mainstem	5040		Lat Struct									
Mainstem	5100	Max WS	113.53	0.37	7.622		7.630	0.000075	0.78	268.12	336.31	0.07
Mainstem	5198.35	Max WS	114.13	0.42	7.629		7.635	0.000072	0.74	336.58	398.55	0.06
Mainstem	5300	Max WS	114.69	0.47	7.636		7.641	0.000053	0.69	402.83	395.16	0.06
Mainstem	5400	Max WS	114.72	0.51	7.641		7.644	0.000041	0.57	465.36	378.76	0.05
Mainstem	5500	Max WS	115.20	0.56	7.645		7.650	0.000065	0.72	372.84	371.07	0.06
Mainstem	5600	Max WS	115.23	0.60	7.651		7.661	0.000105	0.95	260.98	358.23	0.08
Mainstem	5700	Max WS	115.62	0.65	7.659		7.675	0.000154	1.08	182.77	262.52	0.09
Mainstem	5800	Max WS	116.00	0.70	7.672		7.694	0.000206	1.20	119.97	153.52	0.11
Mainstem	5877.68	Max WS	116.36	0.73	7.686		7.710	0.000226	1.25	93.44	23.87	0.11
Mainstem	5946	Max WS	109.88	0.76	7.704		7.725	0.000201	1.17	95.79	48.81	0.10
Mainstem	6010		Lat Struct									
Mainstem	6031.23	Max WS	81.40	1.88	7.735		7.750	0.000142	0.98	84.86	34.31	0.09
Mainstem	6102.93	Max WS	81.41	2.51	7.744		7.763	0.000224	1.12	74.60	54.12	0.11

HEC-RAS Plan: Prop 2-Year Profile: Max WS (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mainstem	6200	Max WS	81.42	3.08	7.764		7.788	0.000301	1.26	64.63	20.84	0.13
Mainstem	6300	Max WS	81.42	4.24	7.794		7.827	0.000495	1.48	55.11	20.92	0.16
Mainstem	6500	Max WS	82.90	8.00	9.236		9.560	0.016943	4.57	18.15	19.43	0.83

Appendix B - HEC-RAS Peak Water Surface Elevations

HEC-RAS Plan: Prop10-Year   Profile: Max WS

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Downstream Meand	0	Max WS	0.01	-0.98	7.693	-0.959	7.693	0.000000	0.00	371.51	176.69	0.00
Downstream Meand	10	Max WS	0.01	-0.98	7.693		7.693	0.000000	0.00	371.51	176.69	0.00
Downstream Meand	20	Max WS	-0.07	-0.98	7.693		7.693	0.000000	0.00	371.51	176.69	0.00
Downstream Meand	25		Lat Struct									
Downstream Meand	50		Lat Struct									
Downstream Meand	100	Max WS	72.05	-0.98	7.689		7.690	0.000005	0.26	370.88	176.38	0.02
Downstream Meand	200	Max WS	98.63	-0.96	7.692		7.693	0.000009	0.30	405.32	181.80	0.02
Downstream Meand	300	Max WS	173.01	-0.95	7.696		7.699	0.000022	0.43	480.69	188.19	0.04
Downstream Meand	400	Max WS	171.63	-0.93	7.698		7.702	0.000028	0.46	372.93	98.79	0.04
Downstream Meand	500	Max WS	171.06	-0.91	7.702		7.705	0.000024	0.44	397.15	106.05	0.04
Downstream Meand	600	Max WS	177.44	-0.89	7.705		7.708	0.000029	0.48	377.85	102.18	0.04
Downstream Meand	700	Max WS	189.11	-0.87	7.708		7.711	0.000032	0.50	381.98	102.38	0.04
Downstream Meand	800	Max WS	204.57	-0.85	7.710		7.715	0.000038	0.56	378.65	103.74	0.05
Downstream Meand	900	Max WS	207.07	-0.84	7.712		7.720	0.000042	0.76	322.11	119.68	0.05
Downstream Meand	950		Lat Struct									
Downstream Meand	969.37	Max WS	220.73	-0.82	7.711		7.723	0.000053	0.86	261.30	52.90	0.06
Downstream Meand	1033.73	Max WS	225.78	-0.81	7.716		7.728	0.000056	0.87	261.32	46.85	0.06
Downstream Meand	1100	Max WS	225.80	-0.80	7.720		7.732	0.000051	0.87	282.68	73.05	0.06
Mainstem Meander	1200	Max WS	216.41	-0.78	7.720		7.730	0.000072	0.78	276.34	65.64	0.07
Mainstem Meander	1249.11	Max WS	220.54	-0.78	7.722		7.734	0.000070	0.87	254.14	49.65	0.07
Mainstem Meander	1339.63	Max WS	220.54	-0.76	7.723		7.743	0.000101	1.13	195.57	25.00	0.07
Mainstem Meander	1425.2	Max WS	220.55	-0.75	7.733		7.753	0.000104	1.14	193.95	25.00	0.07
Mainstem Meander	1476.69	Max WS	277.02	-0.74	7.748		7.766	0.000089	1.10	313.39	125.22	0.08
Mainstem Meander	1527.55	Max WS	277.03	-0.73	7.752		7.770	0.000093	1.12	304.01	119.80	0.08
Mainstem Meander	1600	Max WS	277.83	-0.72	7.759		7.777	0.000088	1.12	324.98	155.44	0.08
Mainstem Meander	1630		Lat Struct									
Mainstem Meander	1703.33	Max WS	278.69	-0.70	7.772		7.783	0.000061	0.88	419.35	191.76	0.07
Mainstem Meander	1770.97	Max WS	278.70	-0.69	7.777		7.789	0.000094	0.90	376.62	224.58	0.08
Mainstem Meander	1839.35	Max WS	279.60	-0.68	7.784		7.795	0.000103	0.89	384.91	213.96	0.08
Mainstem Meander	1885.46	Max WS	280.52	-0.67	7.789		7.800	0.000101	0.88	383.61	189.04	0.08
Mainstem	1948.75	Max WS	279.20	-0.66	7.789		7.804	0.000090	1.10	392.83	228.73	0.08
Mainstem	1997.13	Max WS	279.21	-0.66	7.793		7.809	0.000087	1.11	407.86	242.81	0.08
Mainstem	2045.07	Max WS	279.22	-0.65	7.797		7.813	0.000090	1.14	404.89	246.26	0.08
Mainstem	2100	Max WS	280.08	-0.64	7.802		7.819	0.000106	1.12	412.04	388.34	0.08
Mainstem	2200	Max WS	280.95	-0.63	7.811		7.830	0.000104	1.19	391.58	348.52	0.08
Mainstem	2300	Max WS	281.80	-0.61	7.824		7.839	0.000092	1.10	485.65	450.60	0.08
Mainstem	2400	Max WS	282.65	-0.60	7.833		7.848	0.000091	1.11	515.22	547.98	0.08
Mainstem	2500	Max WS	283.45	-0.59	7.842		7.858	0.000095	1.14	505.64	575.76	0.08
Mainstem	2600	Max WS	284.21	-0.57	7.849		7.867	0.000101	1.17	450.38	495.20	0.08
Mainstem	2700	Max WS	284.91	-0.56	7.858		7.876	0.000106	1.15	432.69	340.44	0.08
Mainstem	2800	Max WS	285.61	-0.54	7.869		7.887	0.000100	1.15	447.38	377.10	0.08
Mainstem	2900	Max WS	285.63	-0.53	7.880		7.895	0.000094	1.08	490.77	388.40	0.08
Mainstem	3000	Max WS	286.33	-0.51	7.890		7.904	0.000094	1.09	522.67	449.71	0.08
Mainstem	3100	Max WS	286.99	-0.50	7.895		7.918	0.000126	1.28	361.60	380.05	0.09
Mainstem	3200	Max WS	287.00	-0.47	7.902		7.932	0.000172	1.43	253.05	327.36	0.11
Mainstem	3273.64	Max WS	287.64	-0.45	7.916		7.947	0.000169	1.44	239.90	303.67	0.10
Mainstem	3298.06	Max WS	287.64	-0.44	7.919		7.951	0.000165	1.45	224.78	166.31	0.10
Mainstem	3352.67	Max WS	288.29	-0.42	7.927		7.959	0.000164	1.45	237.58	250.49	0.10
Mainstem	3406.58	Max WS	288.98	-0.41	7.945		7.960	0.000102	1.14	556.55	591.85	0.08
Mainstem	3498.87	Max WS	289.03	-0.38	7.954		7.967	0.000094	1.10	565.98	500.25	0.08
Mainstem	3563.55	Max WS	258.35	-0.36	7.962		7.971	0.000068	0.92	617.37	506.57	0.07
Mainstem	3610		Lat Struct									
Mainstem	3640.9	Max WS	259.31	-0.33	7.966		7.977	0.000068	0.95	559.17	480.84	0.07
Mainstem	3706.91	Max WS	259.34	-0.31	7.969		7.985	0.000094	1.10	459.65	442.37	0.08
Mainstem	3788.76	Max WS	259.96	-0.28	7.976		7.990	0.000084	1.05	482.03	368.92	0.08
Mainstem	3895.44	Max WS	260.00	-0.24	7.985		7.998	0.000084	1.06	494.17	409.57	0.08
Mainstem	4000	Max WS	260.57	-0.20	7.993		8.006	0.000085	1.04	469.77	356.85	0.08
Mainstem	4050		Lat Struct									
Mainstem	4100	Max WS	261.10	-0.16	8.001		8.018	0.000113	1.16	411.02	315.79	0.08
Mainstem	4200	Max WS	261.12	-0.12	8.014		8.027	0.000100	1.06	473.81	357.45	0.08
Mainstem	4300	Max WS	261.61	-0.08	8.025		8.032	0.000060	0.84	725.23	543.19	0.06
Mainstem	4400	Max WS	261.64	-0.04	8.029		8.042	0.000099	1.06	485.95	399.84	0.08
Mainstem	4500	Max WS	261.68	0.00	8.037		8.054	0.000114	1.16	435.13	461.44	0.09
Mainstem	4549.41	Max WS	261.70	0.03	8.035		8.069	0.000230	1.56	272.13	362.50	0.12
Mainstem	4650		Bridge									
Mainstem	4703.6	Max WS	262.37	0.13	8.065	3.270	8.123	0.000430	1.94	144.39	190.77	0.16
Mainstem	4786.07	Max WS	245.49	0.18	8.113		8.139	0.000247	1.46	319.15	349.60	0.12
Mainstem	4897.84	Max WS	246.32	0.24	8.142		8.153	0.000127	1.11	505.74	419.49	0.09
Mainstem	4982.74	Max WS	246.34	0.30	8.151		8.161	0.000098	1.03	577.30	470.75	0.08
Mainstem	5040		Lat Struct									
Mainstem	5100	Max WS	246.71	0.37	8.163		8.176	0.000146	1.16	490.26	467.62	0.09
Mainstem	5198.35	Max WS	247.03	0.42	8.178		8.186	0.000114	1.01	566.36	427.33	0.08
Mainstem	5300	Max WS	247.06	0.47	8.188		8.195	0.000087	0.96	623.16	411.25	0.07
Mainstem	5400	Max WS	247.31	0.51	8.197		8.201	0.000072	0.80	677.06	387.21	0.06
Mainstem	5500	Max WS	247.32	0.56	8.204		8.211	0.000104	0.96	581.93	381.32	0.08
Mainstem	5600	Max WS	247.34	0.60	8.214		8.228	0.000160	1.27	480.51	409.11	0.10
Mainstem	5700	Max WS	247.51	0.65	8.231		8.252	0.000266	1.45	379.82	378.38	0.12
Mainstem	5800	Max WS	247.52	0.70	8.250		8.292	0.000406	1.83	253.67	278.85	0.15
Mainstem	5877.68	Max WS	247.64	0.73	8.273		8.335	0.000565	2.09	183.83	247.38	0.18
Mainstem	5946	Max WS	229.29	0.76	8.325		8.375	0.000450	1.89	193.61	242.65	0.16
Mainstem	6010		Lat Struct									
Mainstem	6031.23	Max WS	171.19	1.88	8.369		8.406	0.000311	1.61	169.29	225.45	0.13
Mainstem	6102.93	Max WS	171.28	2.51	8.397		8.430	0.000393	1.61	185.02	229.24	0.15

HEC-RAS Plan: Prop10-Year Profile: Max WS (Continued)

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mainstem	6200	Max WS	171.39	3.08	8.422		8.494	0.000727	2.16	86.62	83.82	0.20
Mainstem	6300	Max WS	171.71	4.24	8.490		8.579	0.001030	2.42	76.62	50.51	0.24
Mainstem	6500	Max WS	206.33	8.00	9.878	10.060	10.365	0.017511	5.85	52.55	170.58	0.90

HEC-RAS Plan: Prop100-Year   Profile: Max WS

Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Downstream Meand	0	Max WS	0.01	-0.98	8.021	-0.959	8.021	0.000000	0.00	432.07	187.02	0.00
Downstream Meand	10	Max WS	0.03	-0.98	8.021		8.021	0.000000	0.00	432.07	187.02	0.00
Downstream Meand	20	Max WS	-0.09	-0.98	8.021		8.021	0.000000	0.00	432.07	187.02	0.00
Downstream Meand	25		Lat Struct									
Downstream Meand	50		Lat Struct									
Downstream Meand	100	Max WS	434.32	-0.98	7.985		8.013	0.000134	1.43	425.36	186.50	0.10
Downstream Meand	200	Max WS	360.70	-0.96	8.013		8.028	0.000089	1.00	466.55	193.04	0.08
Downstream Meand	300	Max WS	339.28	-0.95	8.027		8.035	0.000061	0.76	543.11	189.70	0.06
Downstream Meand	400	Max WS	343.45	-0.93	8.032		8.043	0.000083	0.85	407.71	112.22	0.07
Downstream Meand	500	Max WS	344.49	-0.91	8.041		8.051	0.000075	0.81	433.41	108.20	0.07
Downstream Meand	600	Max WS	363.20	-0.89	8.046		8.058	0.000093	0.89	413.06	104.35	0.08
Downstream Meand	700	Max WS	392.89	-0.87	8.052		8.066	0.000103	0.96	422.84	136.79	0.08
Downstream Meand	800	Max WS	435.26	-0.85	8.057		8.075	0.000130	1.09	421.59	168.31	0.09
Downstream Meand	900	Max WS	416.95	-0.84	8.063		8.093	0.000137	1.43	374.68	205.85	0.10
Downstream Meand	950		Lat Struct									
Downstream Meand	969.37	Max WS	455.71	-0.82	8.057		8.101	0.000187	1.69	290.42	144.74	0.11
Downstream Meand	1033.73	Max WS	456.82	-0.81	8.069		8.112	0.000190	1.67	279.42	64.46	0.11
Downstream Meand	1100	Max WS	457.94	-0.80	8.082		8.124	0.000173	1.67	311.14	90.30	0.11
Mainstem Meander	1200	Max WS	417.83	-0.78	8.082		8.112	0.000211	1.39	300.37	67.37	0.12
Mainstem Meander	1249.11	Max WS	417.83	-0.78	8.086		8.122	0.000208	1.53	272.41	51.06	0.12
Mainstem Meander	1339.63	Max WS	417.10	-0.76	8.087		8.152	0.000320	2.04	204.66	25.00	0.13
Mainstem Meander	1425.2	Max WS	418.66	-0.75	8.129		8.194	0.000726	2.04	205.35	78.84	0.19
Mainstem Meander	1476.69	Max WS	419.64	-0.74	8.176		8.209	0.000156	1.52	374.00	166.31	0.10
Mainstem Meander	1527.55	Max WS	419.66	-0.73	8.182		8.216	0.000162	1.54	369.42	188.22	0.11
Mainstem Meander	1600	Max WS	420.76	-0.72	8.196		8.229	0.000150	1.53	402.72	212.44	0.10
Mainstem Meander	1630		Lat Struct									
Mainstem Meander	1703.33	Max WS	421.96	-0.70	8.218		8.237	0.000100	1.18	519.25	258.05	0.08
Mainstem Meander	1770.97	Max WS	423.21	-0.69	8.225		8.244	0.000140	1.17	481.58	237.42	0.10
Mainstem Meander	1839.35	Max WS	423.23	-0.68	8.236		8.254	0.000150	1.14	488.84	240.99	0.10
Mainstem Meander	1885.46	Max WS	424.53	-0.67	8.243		8.261	0.000149	1.14	486.72	252.74	0.10
Mainstem	1948.75	Max WS	470.64	-0.66	8.243		8.275	0.000183	1.64	571.14	550.87	0.11
Mainstem	1997.13	Max WS	470.66	-0.66	8.254		8.285	0.000171	1.62	574.55	523.17	0.11
Mainstem	2045.07	Max WS	471.82	-0.65	8.261		8.294	0.000175	1.67	637.67	693.14	0.11
Mainstem	2100	Max WS	471.85	-0.64	8.272		8.300	0.000179	1.54	686.67	758.97	0.11
Mainstem	2200	Max WS	472.97	-0.63	8.286		8.317	0.000177	1.62	649.68	708.08	0.11
Mainstem	2300	Max WS	474.01	-0.61	8.305		8.328	0.000143	1.44	744.42	607.86	0.10
Mainstem	2400	Max WS	474.98	-0.60	8.319		8.339	0.000130	1.40	822.34	658.98	0.09
Mainstem	2500	Max WS	475.04	-0.59	8.331		8.352	0.000130	1.41	802.71	616.72	0.10
Mainstem	2600	Max WS	475.88	-0.57	8.343		8.368	0.000147	1.49	731.69	591.84	0.10
Mainstem	2700	Max WS	476.63	-0.56	8.358		8.383	0.000166	1.51	634.28	439.45	0.11
Mainstem	2800	Max WS	476.67	-0.54	8.374		8.399	0.000153	1.50	660.99	458.62	0.10
Mainstem	2900	Max WS	477.35	-0.53	8.390		8.411	0.000142	1.41	710.67	461.82	0.10
Mainstem	3000	Max WS	477.99	-0.51	8.404		8.423	0.000133	1.36	764.44	482.89	0.09
Mainstem	3100	Max WS	478.01	-0.50	8.412		8.446	0.000191	1.66	576.99	455.00	0.11
Mainstem	3200	Max WS	478.58	-0.47	8.428		8.473	0.000270	1.87	471.43	435.50	0.13
Mainstem	3273.64	Max WS	478.60	-0.45	8.442		8.493	0.000274	1.94	426.32	380.41	0.14
Mainstem	3298.06	Max WS	479.10	-0.44	8.447		8.501	0.000277	1.99	421.33	434.11	0.14
Mainstem	3352.67	Max WS	479.12	-0.42	8.469		8.515	0.000246	1.88	521.62	603.80	0.13
Mainstem	3406.58	Max WS	479.59	-0.41	8.489		8.505	0.000121	1.31	896.68	645.03	0.09
Mainstem	3498.87	Max WS	479.63	-0.38	8.500		8.516	0.000120	1.31	873.20	589.10	0.09
Mainstem	3563.55	Max WS	479.10	-0.36	8.508		8.521	0.000107	1.22	906.54	545.06	0.09
Mainstem	3610		Lat Struct									
Mainstem	3640.9	Max WS	480.04	-0.33	8.514		8.531	0.000112	1.29	832.50	510.52	0.09
Mainstem	3706.91	Max WS	480.06	-0.31	8.520		8.542	0.000153	1.49	714.47	478.36	0.10
Mainstem	3788.76	Max WS	480.09	-0.28	8.532		8.554	0.000149	1.49	705.42	437.75	0.10
Mainstem	3895.44	Max WS	480.38	-0.24	8.548		8.568	0.000139	1.44	736.62	448.34	0.10
Mainstem	4000	Max WS	480.40	-0.20	8.561		8.583	0.000146	1.44	680.63	385.66	0.10
Mainstem	4050		Lat Struct									
Mainstem	4100	Max WS	480.62	-0.16	8.576		8.602	0.000189	1.59	604.66	367.37	0.11
Mainstem	4200	Max WS	480.63	-0.12	8.596		8.616	0.000160	1.43	686.24	372.60	0.10
Mainstem	4300	Max WS	480.66	-0.08	8.614		8.623	0.000085	1.06	1048.97	556.02	0.07
Mainstem	4400	Max WS	480.69	-0.04	8.621		8.639	0.000150	1.39	750.73	467.65	0.10
Mainstem	4500	Max WS	480.80	0.00	8.634		8.656	0.000162	1.48	730.14	507.60	0.10
Mainstem	4549.41	Max WS	480.80	0.03	8.635		8.680	0.000330	2.00	511.81	410.15	0.15
Mainstem	4650		Bridge									
Mainstem	4703.6	Max WS	480.80	0.13	8.722	4.455	8.817	0.000711	2.67	316.77	351.46	0.21
Mainstem	4786.07	Max WS	413.14	0.18	8.796		8.818	0.000236	1.53	565.08	370.15	0.12
Mainstem	4897.84	Max WS	413.15	0.24	8.822		8.832	0.000119	1.17	797.54	437.66	0.09
Mainstem	4982.74	Max WS	413.22	0.30	8.832		8.840	0.000095	1.09	907.93	500.32	0.08
Mainstem	5040		Lat Struct									
Mainstem	5100	Max WS	413.22	0.37	8.843		8.854	0.000129	1.19	827.37	525.77	0.09
Mainstem	5198.35	Max WS	413.23	0.42	8.856		8.864	0.000109	1.08	869.41	466.71	0.08
Mainstem	5300	Max WS	413.23	0.47	8.866		8.873	0.000090	1.05	916.64	454.15	0.08
Mainstem	5400	Max WS	413.24	0.51	8.876		8.881	0.000079	0.91	950.57	421.10	0.07
Mainstem	5500	Max WS	413.24	0.56	8.884		8.891	0.000106	1.06	851.44	413.51	0.08
Mainstem	5600	Max WS	413.24	0.60	8.894		8.905	0.000143	1.30	766.62	432.53	0.09
Mainstem	5700	Max WS	413.35	0.65	8.911		8.924	0.000242	1.30	651.14	419.25	0.12
Mainstem	5800	Max WS	413.36	0.70	8.929		8.963	0.000370	1.92	448.17	297.75	0.15
Mainstem	5877.68	Max WS	413.50	0.73	8.952		9.006	0.000528	2.23	374.88	298.39	0.18
Mainstem	5946	Max WS	406.52	0.76	8.988		9.040	0.000503	2.19	357.43	251.33	0.17
Mainstem	6010		Lat Struct									
Mainstem	6031.23	Max WS	414.22	1.88	9.023		9.095	0.000657	2.56	319.95	235.34	0.20
Mainstem	6102.93	Max WS	415.97	2.51	9.082		9.134	0.000658	2.34	387.45	329.21	0.20

HEC-RAS Plan: Prop100-Year Profile: Max WS (Continued)

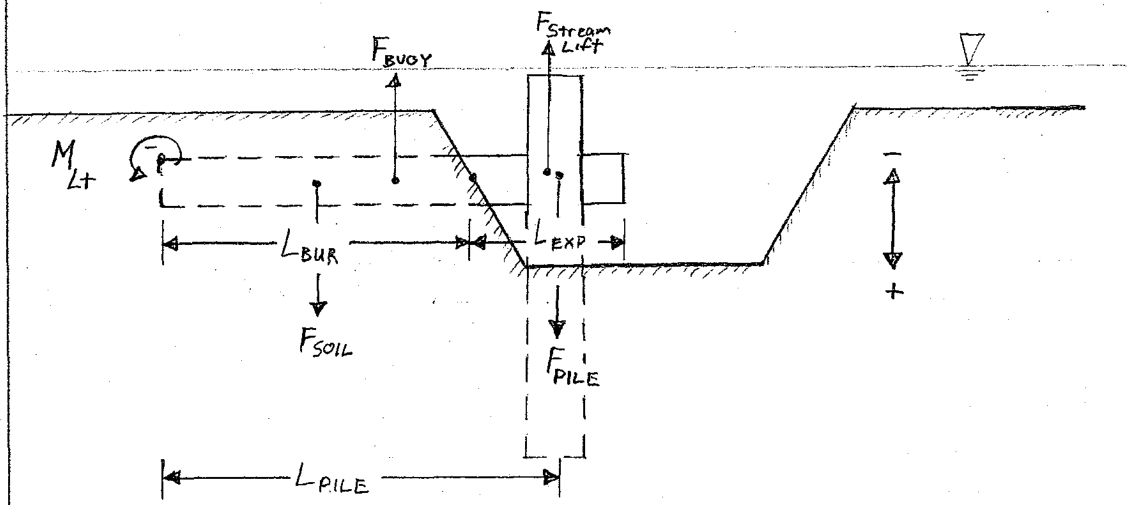
Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Flow Area	Top Width	Froude # Chl
			(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(sq ft)	(ft)	
Mainstem	6200	Max WS	417.78	3.08	9.126		9.302	0.001670	3.68	200.27	220.70	0.31
Mainstem	6300	Max WS	426.78	4.24	9.274		9.540	0.002501	4.36	138.85	108.56	0.39
Mainstem	6500	Max WS	498.57	8.00	10.483		10.725	0.009026	5.30	193.49	267.43	0.68

Appendix C

Large Wood Structures Anchoring Calculations

STABILITY ANALYSIS FOR LARGE WOODY STRUCTURES

ADAPTED FROM NRCS, 2007, TECHNICAL SUPPLEMENT J USE OF LARGE WOODY MATERIAL FOR HABITAT AND BANK PROTECTION, PART 654 NATIONAL ENGINEERING MANUAL



$$M_{Lt} = \frac{L_{BUR}+L_{EXP}}{2} * F_{BUOY} + \left(L_{BUR}+\frac{L_{EXP}}{2}\right) * F_{\substack{Stream\\Lift}} - \frac{L_{BUR}}{2} * F_{SOIL} - L_{PILE} * F_{PILE}$$

ASSUMPTIONS:

STRUCTURE AND SOIL ARE FULLY SUBMERGED

ABBREVIATIONS:

BUOY	BUOYANT
BUR	BURIED
EXP	EXPOSED
F	FORCE
L	LENGTH
Lt	LEFT
M	MOMENT

MLA 8/4/2015 MARTIN SLOUGH NICKERSON

Stability Computations for Large Woody Structures - Martin Slough

Adapted from NRCS, 2007, Techincal Supplement J Use of Large Woody Material for Habitat and Bank Protection, Part 654 National Engineering Manual

Soil Type: silty clay based on Geotechnical Report (SHN, 2013)

Constants and Parameters Applied to Calculations

Douglas Fir Specific Gravity[A]	**Soil Angle of Internal Friction**[B]	**Coefficient of Passive Earth Pressure**[C]	**Specific Gravity Moist Soil**[D]	**Specific Gravity Water**	**Density Water**	**Lift Coefficient**[E]	**Gravity**
SGlog	degrees	--	--	--	lb/ft3	--	ft/sec2
0.51	20.00	2.04	1.63	1.0	62.4	0.45	32.2

[A] Average value for "seasoned" Douglass Fir (D'oust 1998, USFS 1999)

[B] Average minimum internal angle of friction for organic silts and silty clay (http://www.geotechdata.info/parameter/angle-of-friction.html)

[C] $tan^2\left(45+\frac{\emptyset}{2}\right)$ Where $\emptyset$ is the angle of internal friction (NRCS, 2007)

[D] based on a soil dry density of 130 lb/cf for silty clay (NAVFAC 7.01, 1986), porosity of 0.35 (http://web.ead.anl.gov/resrad/datacoll/porosity.htm), and 80% moisture level.

[E] Lift Coefficient ~0.45 when cylinder is perpendicular to flow (NRCS, 2007)

Stability Computations for Large Woody Structures - Martin Slough

Adapted from NRCS, 2007, Techincal Supplement J Use of Large Woody Material for Habitat and Bank Protection, Part 654 National Engineering Manual

Resulting Factor of Safety from Moment Analysis of Applied Forces

Engineered Log Structure Type and Log Members	Factor of Safety (Moment Analysis)		Uplift Forces Forces						
			Buoyancy			Lift			
	Left	Right	Log Length ft	Log Diameter ft	Buoyancy lbs	Exposed Log Length ft	Water Surface Slope ft/ft	Velocity ft/sec	Lift lbs
N-CH Weir Log	**2.0**	**2.0**	20.0	2	**-1,921**	10	0.02	5.5	**-264**
N-CH Footer Log	**2.0**	**2.0**	20.0	2	**-1,921**	10	0.02	5.5	**-264**
N-CH Training Log	**2.8**	**3.4**	15.0	1.5	**-810**	7	0.02	5.5	**-138**
Pond C Weir Log	**2.4**	**2.4**	15.0	1.5	**-810**	5	0.02	5.5	**-99**
Pond C Footer Log	**2.4**	**2.4**	15.0	1.5	**-810**	5	0.02	5.5	**-99**
Pond C Training Log	**2.6**	**3.1**	15.0	1.5	**0**	7	0.02	5.5	**-138**
Pond D Weir Log	**2.5**	**2.5**	20.0	1.5	**-1,081**	10	0.02	5.5	**-198**
Pond D Footer Log	**2.5**	**2.5**	20.0	1.5	**-1,081**	10	0.02	5.5	**-198**
Pond D Training Log	**2.8**	**3.4**	15.0	1.5	**-810**	7	0.02	5.5	**-138**
Log Constrictor Top	**5.2**	**2.1**	20.0	1.5	**-1,081**	8	0.0033	3.5	**-64**
Log Constrictor Bottom	**3.3**	**2.3**	20.0	1.5	**-1,081**	8	0.0033	3.5	**-64**
Log cover	**2.8**	**2.8**	15.0	1.5	**-810**	15	0	1	**-10**
Root Wad Deflector	**2.2**	**2.5**	10.0	2.75	**-1,816**	6	0.0033	3.5	**-88**
Root Wad Habitat	**2.2**	**2.2**	10.0	2.75	**-1,816**	10	0	1	**-12**

Resulting Factor of Safety from Moment Analysis of Applied Forces (Continued)

Engineered Log Structure Type and Log Members	Resisting Forces													
	Left end of Log							Right end of Log						
	Soil			Anchor				Soil			Anchor			
	Length Buried ft	Mean Depth Buried ft	Saturated Soil Weight lbs	Anchor Type*	Effective Buried Depth ft	Resistant Force lbs	Distance from Left End ft	Length Buried ft	Mean Depth Buried ft	Saturated Soil Weight lbs	Anchor Type*	Effective Buried Depth ft	Resistant Force lbs	Distance from Right End ft
N-CH Weir Log	5	2.5	**989**	Pile	10	**1150**	2	5	2.5	**989**	Pile	10	**1150**	2
N-CH Footer Log	5	2.5	**989**	Pile	10	**1150**	2	5	2.5	**989**	Pile	10	**1150**	2
N-CH Training Log	0	0	**0**	Pile	a	**2370**	5	8	1.25	**594**	-	-	**-**	-
Pond C Weir Log	5	1.5	**445**	Pile	9	**662**	2	5	1.5	**445**	Pile	9	**662**	2
Pond C Footer Log	5	1.5	**445**	Pile	9	**662**	2	5	1.5	**445**	Pile	9	**662**	2
Pond C Training Log	0	0	**0**	Pile	a	**2091**	5	8	1.25	**594**	S.A.	6	**0**	0
Pond D Weir Log	5	1.5	**445**	Pile	10	**1150**	2	5	1.5	**445**	Pile	10	**1150**	2
Pond D Footer Log	5	1.5	**445**	Pile	10	**1150**	2	5	1.5	**445**	Pile	10	**1150**	2
Pond D Training Log	0	0	**0**	Pile	a	**2370**	5	8	1.25	**594**	-	-	**-**	-
Log Constrictor Top	0	0	**0**	Pile	7	**662**	2	12	2	**1,425**	S.A.	6	**2000**	2
Log Constrictor Bottom	0	0	**0**	Pile	7	**662**	2	12	3.5	**2,493**				
Log cover	0	0	**0**	Pile	8	**2301**	7.5	0	0	**0**	-	-	**-**	-
Root Wad Deflector	4	1	**435**	S.A.	6	**2000**	2	0	0	**0**	S.A.	6	**2000**	2
Root Wad Habitat	0	0	**0**	S.A.	6	**2000**	2	0	0	**0**	S.A.	6	**2000**	2

* S.A. = Soil Anchor, Type MR-2

[a] Log Weirs are composed of 1-Weir Log, 1-Footer Log, and 2-Training Logs. Log Weir structures share 2 piles such that the resistant force imparted by each pile is shared between 1-training log and one end of a Weir Log and Footer logs.

Plie Skin Friction calculations for Martin Slough wood structures

Bearing Capacity of Embedded Piles From Shaft Skin Friction

Derived from: http://www.geotechnicalinfo.com/bearing_capacity_technical_guidance.html#deepfoundations

Wood Pile Properties

Pile Diameter (D)	**1.5**	ft
Pile circumference (perimeter) (p)	4.7	sq ft

Soil Properties

cohesion (c, lb/sf)	0.1	Minimal value, project area contains sands and gravels
adhesion (ca) (lbs/sf)	0.050	Equals 1/2 soil cohesion (Section 38-4 M. Lindberg, Civil Engineering Reference Manual, 2003)
Lateral earth pressure coefficient for piles (k)	**1.0**	Conservatively used 1. USACE recommends 1-1. 5 for piles in sand that are not pre-bored, jetted or vibrated (http://www.geotechnicalinfo.com/lateral_earth_pressure_coefficient.html)
Effective unit weight of soil (lbs/cf)	**62.4**	Effective unit weight, γ, is the unit weight of the soil for soils above the water table and capillary rise. For saturated soils, the effective unit weight is the unit weight of water.
Internal angle of friction (degrees)	**20**	From table 2.9 (NRCS, 2005)
External angle of friction (degrees)	**13.332**	2/3 of internal angle for wood (http://www.geotechnicalinfo.com/external_friction_angle.html) based on Broms Method

Forces

Total Net Upward Force of Pile	**-810**	↑
Factor of safety (piles)	**2**	

Skin Friction Equations for Non-Cohesive Soils

Effective Pile Length (L) (ft)	Skin Friction Capacity Per Pile (lbs)	Available Resistance Force Remaining (lbs)
6	2,109	**488.3**
7	2,946	**1324.7**
8	3,922	**2300.6**
9	5,037	**3415.7**
10	6,291	**4670.3**

↓

Skin (Shaft) Friction Capacity of Pile Foundation

$Q_f = A_f q_f$ for one homogeneous layer of soil

Where:

Q_f = Theoretical bearing capacity due to shaft friction, or adhesion between

A_f = pL; Effective surface area of the pile shaft, m^2 (ft^2)

q_f = c_A + kσ tan δ = Theoretical unit friction capacity for silts, kN/m^2 (lb/ft^2)

D = diameter or width of pile, m (ft)

p = perimeter of pile cross-section, m (ft)

L = Effective length of pile, m (ft)

c_A = adhesion

c = cohesion of soil, kN/m2 (lb/ft2)

d = external friction angle of soil and wall contact (deg)

f = angle of internal friction (deg)

σ = γD = effective overburden pressure, kN/m^2, (lb/ft^2)

k = lateral earth pressure coefficient for piles

γ = effective unit weight of soil, kN/m3 (lb/ft3)

D = Effective depth of pile, m (ft), where **$D < D_c$**

D_c = critical depth for piles in loose silts or sands m (ft).

Dc = 10B, for loose silts and sands

Dc = 15B, for medium dense silts and sands

Dc = 20B, for dense silts and sands

Soil anchor holding capacities for Martin Slough wood structures

Manta Ray Anchor Holding Capacities in Listed Soils

Common Soil Type Description	**Typical Blow Count "N" Per ASTM-D1586**	**MR-68**	**MR-88**	**MR-4**	**MR-3**	**MR-2**	**MR-1**	**MR-SR**	**MK-B**
Peat, Organic Silts; **Inundates Silts, Fly Ash**	0 - 5	N.A.	0.2-0.9 kips (.9-4 kN) (4,6)	0.3-1.5 kips (1.3-7 kN) (4,6)	0.8-3 kips (3.5-13 kN) (4,6)	**2-5 kips** (9-22 kN) (4,6)	3-8 kips (13-37 kN) (4,6)	4-12 kips (18-53 kN) (4,6)	6-16 kips (27-71 kN) (4,6)
Loose fine Sand; Alluvium; Soft-Firm Clays; Varied Clays; Fills	4 - 8	0.4-.8 (1.8-3.5 kN) (4,6)	0.9-1.5 kips (4-7 kN) (4,6)	1.5-2.5 kips (7-11 kN) (4,6)	3-5 kips (13-22 kN) (4,6)	5-8 kips (22-36 kN) (4,6)	8-12 kips (36-53 kN) (4,6)	9-14 kips (40-62 kN) (4,6)	13-20 kips (58-89 kN) (4,6)
Loose to Medium Dense Fine to Coarse Sand; Firm to Stiff Clays and Silts	7 - 14	0.75-1.25 kips (3.5 -6 kN) 4	1.5-2.5 kips (7-11 kN) 4	2.5-4 kips (11-18 kN) 4	5-8 kips (22-36 kN) 4	7-10 kips (31-44 kN) 4	10-15 kips (44-67 kN) 4	14-18 kips (62-80 kN) 4	20 kips (89 kN) 4
Medium Dense Coarse Sand and Sandy Gravel; Stiff to very Stiff Silts and Clays	14 - 25	1-1.5 kips (5-7 kN) 4	2-3 kips (9-13 kN) 4	3.54.5 kips (16-20 kN) 4	7-9 kips (3140 kN) 4	9-12 kips (40-53 kN) 4	15-20 kips (67-89 kN) 4	18-20 kips (80-89 kN) 4	20 kips (89 kN) (2,4)
Medium Dense Sandy Gravel; Very Stiff to Hard Silts and Clays	24 - 40	1.5-2 kips (7-9 kN) 4	34 kips (13-18 kN) 4	4.5-6 kips (20-25 kN) 4	9-10 kips (4045 kN) 4	12-18 kips (53-80 kN) 4	18-20 kips (80-89 kN) (2,4)	20 kips (89 kN) (2,4)	20 kips (89 kN) (2,4)
Dense Clays, Sands and Gravel; Hard Slits and Clays	35 - 50	2-2.5 kips (9-11 kN) 4	4-5 kips (18-22 kN) 4	6-8.5 kips (27-36 kN) 4	10 kips (45 kN) (2,4)	15-20 kips (67-89 kN) (2,4)	20 kips (89 kN) (2,4)	20 kips (89 kN) (2,3,4)	20 kips (89 kN) (1,3)
Dense Fine Sand; Very hard Silts and Clays	45 - 60	2.5 kips (11 kN) (2,3,4)	5 kips (22 kN) (2,3,4)	8.5 kips (36 kN) (2,3,4)	10 kips (45 kN) (2,3,4)	20 kips (89 kN) (2,4)	20 kips (89 kN) (1,3,4)	20 kips (89 kN) (1,3)	20 kips (89 kN) (1,3,5)
Very Dense and/or Cemented Sands; Coarse Gravel and Cobbles	60 - 100+	2.5 kips (1 kN) (1,3)	5 kips (22 kN) (1,3)	8.5 kips (36 kN) (1,3)	10 kips (45 kN) (1,3)	20 kips (89 kN) (1,3,4)	20 kips (89 kN) (1,3)	20 kips (89 kN) (1,3,5)	20 kips (89 kN) (1,3,5)

1 - Drilled hole required to install.
2 - Installation may be difficult. Pilot hole may be required.
3 - Holding capacity limited by working load of anchors
4 - Holding capacity limited by soil failure.
5 - Not recommended in these soils.
6 - Wide variation in soil properties reduces prediction accuracy.
Pre-constructed field test recommended.
Minimum 2:1 Safety Factor Recommended
* Use this chart for estimation only.
* True capacity must be tested with anchor locker.

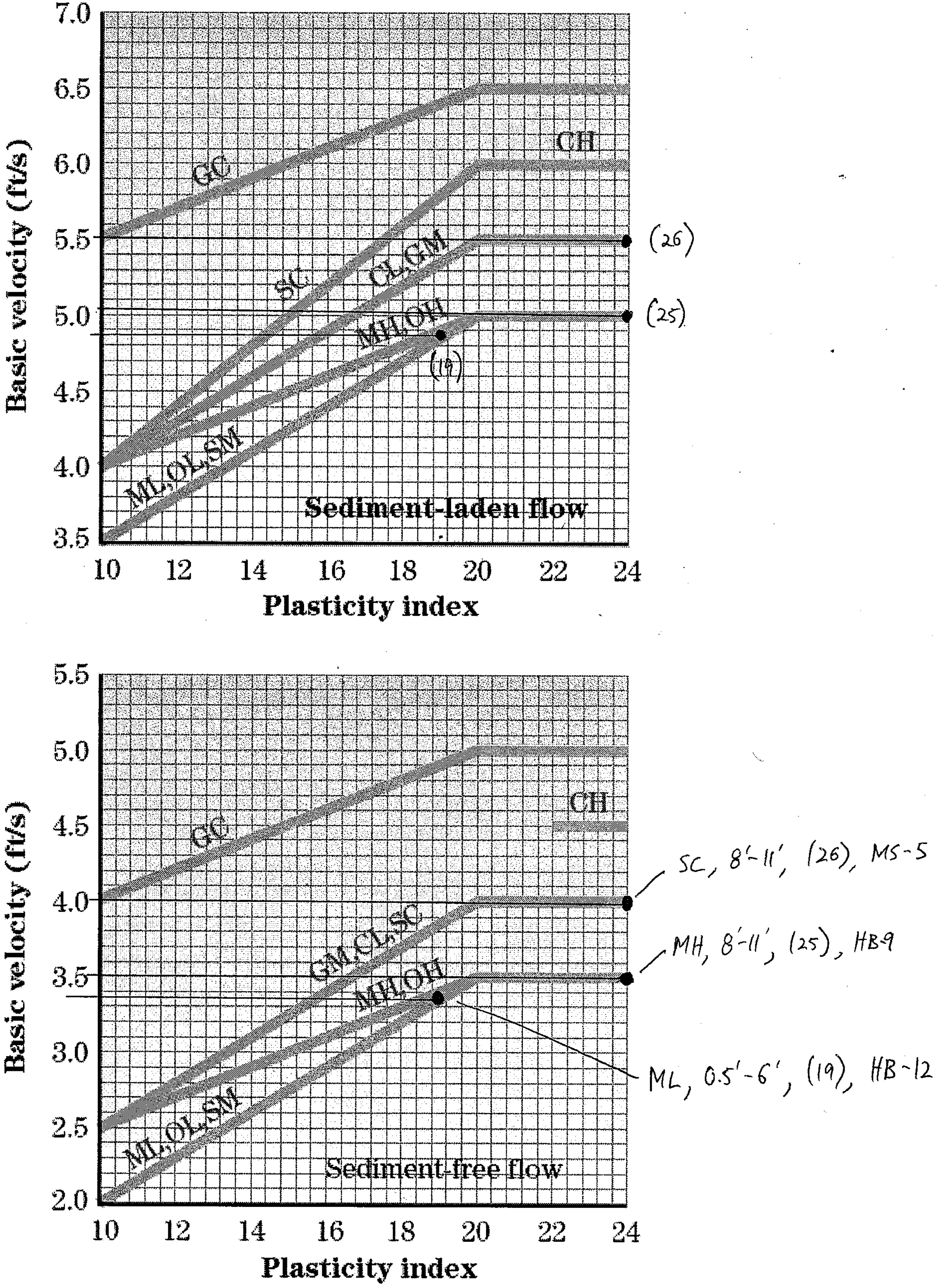
Selected Anchor
MR-2
2000 lbs

http://www.williamsform.com/Ground_Anchors/Manta_Ray_Soil_Anchors/manta_ray_soil_anchors.html

Appendix D

Allowable Water Velocity for Cohesive Soils

Basic velocities for coherent earth materials, v_b



SC, 8'-11', (26), MS-5

MH, 8'-11', (25), HB-9

ML, 0.5'-6', (19), HB-12

USDA. 2007. Chapter 8: Threshold Channel Design. Part 654 Stream Restoration Handbook, National Engineering Handbook.

Appendix E

Geologic Setting for CEQA

Reference: 013035

Geologic Setting

Martin Slough Enhancement Project

Prepared for:

Redwood Community Action Agency

Jason P. Buck, PG

Prepared by:

SHN

Consulting Engineers & Geologists, Inc.
812 W. Wabash Ave.
Eureka, CA 95501-2138

May 2013

Table of Contents

List of Illustrations

1.0 Introduction

This report provides a discussion of the Martin Slough Enhancement project's geologic setting intended to be used in support of CEQA compliance documentation. A geotechnical report focused on providing recommendations for the specific project elements has been provided under separate cover.

2.0 Project Elements

The Martin Slough Enhancement project consists of recontouring the drainage network within the axis of the valley including the development of a series of ponds, and as proposed will include a substantial amount of earthwork. The project also includes infrastructural improvements such as the replacement of the tide gate at the Swain Slough junction and the construction of new bridges for agricultural and golf cart access.

3.0 Site Conditions

The proposed project is located in the floor of the Martin Slough valley, and as such is generally within valley fill sediments. Colluvial deposits near the valley margins would be anticipated to consist of moderately consolidated silty sands, sandy silts, and clayey sands. Valley fill sediments, as discussed below are unconsolidated and uniformly soft and wet. Subsurface investigations indicate that the valley fill sediments tend to contain higher percentages of organics (peats and woody materials) farther up-valley, and increasing amounts of sand toward the valley mouth as materials grade to marine estuarine deposits.

3.1 Groundwater Conditions

Subsurface investigations conducted in the Martin Slough valley bottom and other low-lying areas encountered a uniformly high groundwater table. Groundwater levels adjacent to the mainstem in the lower part of the Martin Slough valley are influenced by tidal fluctuations, such that the water table rises during high tides. During the rainy season, water frequently ponds at the ground surface throughout the Martin Slough valley.

Groundwater will likely be encountered within most of the proposed excavations for this project. It should be noted, however, that although groundwater levels are generally shallow, the permeability of the fine-grained soils are typically low. Because of this, groundwater generally seeps into excavations at a relatively low rate. In past excavations associated with the Interceptor project, for instance, rapid infiltration of groundwater was generally only observed when lenses of sandy or woody material were encountered. Groundwater infiltration into active excavations should be easily managed with sump pumps.

3.2 Soils

In the study area, site soils consist of sediment carried within the Martin Slough channel (and its tributaries), as well as floodplain deposits that encompass the remainder of the valley floor. Deposits within stream channels tend to be coarser, and would likely contain most sand transported from the adjacent uplands. Floodplain deposits are carried by floodwaters during high flows that extend beyond the stream channel. These deposits are typically fine grained; in this case primarily silt.

Previous subsurface investigations indicate that the majority of alluvium in the Martin Slough valley is fine-grained, therefore, the valley is mostly filled with floodplain deposits. Alluvial deposits grade to estuarine deposits at the mouth of the Martin Slough valley, near Swain Slough.

Alluvial textures encountered during subsurface investigations include clayey silt (ML), silty clay (CL), silty sand (SM), clayey sand (SC), and sand (SP). The alluvial materials are locally organic, particularly in the upper reaches of the Martin Slough valley. These materials range in consistency from soft to medium stiff for fine-grained soils or loose to medium dense for granular soils. Blow counts obtained during past subsurface sampling of alluvium were generally less than 10 blows per foot, although sandier zones were sometimes associated with higher values (CPT estimates up to 50 blows per foot for short intervals).

4.0 Project Geologic Setting

4.1 Regional Setting

The project is located within Martin Slough, a coastal valley that opens into the eastern shore of Humboldt Bay at the southern margin of the City of Eureka. The Humboldt Bay region occupies a complex geologic environment characterized by very high rates of active tectonic deformation and seismicity. The area lies just north of the Mendocino Triple Junction, the intersection of three crustal plates (the North American, Pacific, and Gorda plates). North of Cape Mendocino, the Gorda plate is being actively subducted beneath North America, forming what is commonly referred to as the Cascadia subduction zone. In the Humboldt Bay region, the subduction zone is manifested on-land as a series of northwest-trending, southwest-vergent thrust faults, and intervening folds (“fold and thrust belt”). The geomorphic landscape of the Humboldt Bay region is largely a manifestation of the active tectonic processes in this dynamic coastal environment.

Basement rock beneath Humboldt Bay is the Paleocene-Eocene Yager terrane, a part of the Coastal belt of the Franciscan Complex (Blake et al., 1985; Clarke, 1992). The Franciscan Complex is a regional bedrock unit that consists of a series of "terranes,” which are discrete blocks of highly deformed oceanic crust that have been welded to the western margin of the North American plate over the past 140 million years. The Yager terrane consists of as much as 9,800 feet of well-indurated marine mudstone and thin-bedded siltstone. Yager terrane bedrock is in excess of 1,000 feet below the ground surface in the vicinity of Humboldt Bay, based on a deep exploratory well south of Eureka (Woodward-Clyde Consultants, 1980). The Blackwood Nichols No. 1 well encountered Yager terrane bedrock at a depth of about 1,400 feet.

Basement rock in the Humboldt Bay region is unconformably overlain by a late Miocene to middle Pleistocene age sequence of marine and terrestrial deposits referred to as the Wildcat Group (Ogle, 1953). The marine portion of the Wildcat Group includes some 6,000 to 8,000 feet of mudstone and lesser amounts of sandstone that were deposited in a deep coastal basin (for example, the Eel River basin). Gradationally overlying the marine portion of the Wildcat Group are 2,500 to 3,250 feet of nonmarine sandstone and conglomerate, which represent the uppermost part of the Wildcat depositional sequence. The Wildcat Group is truncated at its top by an unconformity of middle Pleistocene age, and is overlain by coastal plain and fluvial deposits of middle to late Pleistocene age. In the Eureka area, these middle and late Pleistocene age deposits are referred to as the Hookton Formation (Ogle, 1953). Hookton Formation sediments are described as gravel, sand, silt, and clay which have a characteristically yellow-orange color (Ogle, 1953).

Along the coast of northern California between Cape Mendocino on the south and Big Lagoon, about 60 miles (100 kilometers [km]) to the north, a sequence of uplifted late Pleistocene age marine terraces is preserved   The terraces are preserved as erosional remnants of raised shore platforms and associated cover sediments. Sea level has fluctuated throughout the late Pleistocene in response to the advance and retreat of large continental ice sheets. Marine terraces preserved along the coast represent surfaces eroded during the highest levels of these sea level fluctuations, superimposed on a coastline being uplifted by regional tectonics. Marine terraces in the region range in age from about 64,000 years old, to as much as 240,000 years old.

The City of Eureka occupies a series of northward-dipping terrace surfaces eroded onto the Hookton Formation. Mapping presented in Carver and Burke (1992) states that the project area spans marine terraces that are assigned ages of 83,000, 96,000, and 103,000 years. These terrace surfaces are differentiated based on subtle elevation changes, as well as increases in soil profile development within the terrace sediments of older terraces. For simplicity, individual marine terrace surfaces underlying Eureka are not distinguished herein, but rather are referred to as the “Eureka terrace.” Marine terraces in the study area are associated with 10 to 20 feet of predominantly silty sand covering the abrasion platform (for example, “marine terrace deposits” in this report).

Beneath Humboldt Bay, and along its margins, the Hookton Formation and marine terrace deposits are overlain by late Holocene age (younger than about 5-6,000 years old) bay muds and associated littoral and estuarine deposits. Near alluvial sources at the fringes of the bay, bay muds are intermixed with terrestrial alluvial deposits. These youthful, unconsolidated deposits vary in thickness and composition around the bay and in the adjacent coastal valleys, often exhibiting large amounts of lateral variation over very small distances. Bay deposits typically consist of silty clays or clayey silts (bay muds) interbedded with clean sand lenses and beds. During the latter part of the 1800s and early part of the 1900s, extensive areas of natural marshlands along the eastern margin of Humboldt Bay were "re-claimed" by placement of uncontrolled fill. Natural estuarine channels and pre-existing marsh surfaces were buried by fill (often including significant amounts of timber slash and/or mill waste) and subsequently developed. Because the natural "pre-fill" surface had significant relief, fill thickness varies considerably along the bay margin.

Martin Slough and other coastal valleys around Humboldt Bay represent sediment-filled estuaries that reflect the late Quaternary history of sea level changes and tectonic deformation. Formation of these coastal valleys likely post-dates the Formation of the adjacent marine terrace platforms, the youngest of which in the Martin Slough area is thought to be some 83,000 years old. Because of its coastal setting, Martin Slough is sensitive to base level fluctuations associated with the rise and fall of sea level. During most of the late Quaternary, sea level was lower than its present position, resulting in a shoreline located farther to the west, and a lower fluvial base level to which all coastal streams would be graded. During these low sea levels, streams within the coastal valleys around Humboldt Bay would be incised. Subsequent sea level fluctuations would result in cycles of filling and incision in these coastal valleys, depending on the relative base level (the ocean shoreline). Sea level apparently reached its current high level in the mid-Holocene, about 6,000 years ago. As such, at least the uppermost part of the sediment filling the Martin Slough valley would be anticipated to be mid-Holocene in age, or younger.

Sediment filling Martin Slough is generally fine-grained (silt, with lesser amounts of clay). The material is derived from alluvial sources (overbank/floodplain deposits) in the upper part of the canyon, and estuarine sources (tidal marine deposits, etc.) in the lower reaches of the valley nearest the bay. Evidence of marine influence (deposits with marine shells for example) does not appear to extend very far up the Martin Slough valley (no evidence upstream of the pump station site), based on subsurface investigations for this study, indicating that most of the sediment in the valley is derived from alluvial sources. Valley fill sediments are uniformly soft, unconsolidated materials that locally contain a high amount of organic materials. Sandy deposits are present locally, particularly near alluvial sources and approaching the bay margins.

4.2 Geohazards

4.2.1 Faults and Seismicity

4.2.1.1 Nomenclature

The State of California designates faults as active, potentially active, and inactive depending on the recency of movement that can be substantiated for a fault. Fault activity is rated based upon the age criteria noted in Table 1.

Table 1
Fault Activity Ratings

Fault Activity Rating	Geologic Period of Last Rupture	Timing of Last Rupture (Years)
Active	Holocene	Within last 11,000 Years
Potentially Active	Quaternary	>11,000 to 1.6 Million Years
Inactive	Pre-Quaternary	Greater than 1.6 Million Years

The California Geologic Survey (CGS) evaluates the activity rating of a fault in fault evaluation reports (FER). FERs compile available geologic and seismologic data, and evaluate if a fault should be zoned as active, potentially active, or inactive. If an FER determines that a fault is active, then the fault is typically incorporated into an Earthquake Fault Zone in accordance with the Alquist-Priolo Earthquakes Hazards Act (A.P.), in order to mitigate surface fault rupture potential. A.P. Earthquake Fault Zones require site-specific evaluation of fault location and require a structure setback if the fault is found traversing a project site.

4.2.1.2 Seismic Setting

The project site is located in a region of high seismicity. Over sixty earthquakes have produced discernible damage in the region since the mid-1800s (Dengler et al., 1992). Historic seismicity and paleoseismic studies in the area suggest there are six distinct sources of damaging earthquakes in the Eureka region (Figures 3 and 4): (1) the Gorda Plate; (2) the Mendocino fault; (3) the Mendocino Triple Junction; (4) the northern end of the San Andreas fault; (5) faults within the North American Plate (including the Mad River fault zone); and (6) the Cascadia Subduction Zone (Dengler et al., 1992).

Earthquakes originating within the Gorda Plate account for the majority of historic seismicity. These earthquakes occur primarily offshore along left-lateral faults, and are generated by the internal deformation within the plate as it moves toward the subduction zone. Significant historic Gorda Plate earthquakes have ranged from magnitude 5 to 7.5. The November 8, 1980, earthquake (magnitude 7.2) was generated 30 miles (48 km) off the coast of Trinidad on a left-lateral fault within the Gorda Plate.

The Mendocino fault is the second most frequent source of earthquakes in the region. The fault represents the plate boundary between the Gorda and Pacific plates, and typically generates right lateral strike-slip displacement. Significant historic Mendocino fault earthquakes have ranged from magnitude 5 to magnitude 7.5. The September 1, 1994, magnitude 7.2 event originating west of Petrolia was generated along the Mendocino fault. The Mendocino triple junction was identified as a separate seismic source only after the magnitude 6.0 August 17, 1991, earthquake. Significant seismic events associated with the triple junction are shallow onshore earthquakes that appear to range from magnitude 5 to 6. Raised Holocene age marine terraces near Cape Mendocino suggest larger events are possible in this region.

Earthquakes originating on the northern San Andreas fault are extremely rare, but can be very large. The northern San Andreas fault is a right lateral strike-slip fault that represents the plate boundary between the Pacific and North American plates. The fault extends through the Point Delgada region and terminates at the Mendocino triple junction. The 1906 San Francisco earthquake (magnitude 8.3) caused the most significant damage in the north coast region, with the possible exception of the April 1992 Petrolia earthquake (Dengler et. al., 1992).

Earthquakes originating within the North American plate can be anticipated from a number of intraplate sources, including the Mad River fault zone and Little Salmon fault. There have been no large magnitude earthquakes associated with faults within the North American plate, although the December 21, 1954, magnitude 6.5 event may have occurred in the Mad River fault zone. Damaging North American plate earthquakes are expected to range from magnitude 6.5 to 8. The Little Salmon fault appears to be the most active fault in the Humboldt Bay region, and is capable of generating very large earthquakes.

4.2.1.3 Regional Faults

As noted above, the project area is located in a region that has numerous onshore and offshore faults. There are no known active faults passing through the project area. The nearest known active fault is the Little Salmon fault, just over 2 miles to the southwest. Other significant faults in the project area include thrust faults within the Mad River fault zone, and the Cascadia Subduction Zone. The North Spit fault has been imaged offshore of the North Spit, and projects toward the project area, but its existence on-land has never been demonstrated. We observed no evidence to suggest the presence of this fault within the project area. Table 2 presents fault location and information data collected from the CGS database (Blake, 1999a).

Table 2
Fault Information

Fault Name	Fault Activity Rating[1]	Distance From Site		Upper Bound Earthquake (M_w)
		Miles	Kilometers	
Little Salmon (onshore)	A	2.1	3.3	7.0
Table Bluff	A	4.4	7.1	7.0
Little Salmon (offshore)	A	4.5	7.3	7.1
Cascadia Subduction Zone	A	11.6	18.7	9.0
Mad River	A	11.6	18.7	7.1
Fickle Hill	A	12.0	19.3	6.9
McKinleyville	A	13.9	22.4	7.0
Trinidad	A	18.0	28.9	7.3
Big Lagoon – Bald Mountain	A	29.0	46.6	7.3
San Andreas	A	37.0	59.5	7.9

1. 1 A: active, PA: potentially active, per Peterson et al. (1996).

Little Salmon fault. The Little Salmon fault is the closest known active fault to the project area (Wills, 1990). The Little Salmon fault is a northwest-trending, southwest-vergent reverse fault (the northeast side of the fault slides up and over the southwest side of the fault along a northeast-dipping fault plane). Offset relations within the upper Wildcat Group suggest vertical separation exceeds 5,900 feet (1,800 meters), representing about 4.4 miles (7 km) of dip-slip motion on the Little Salmon fault since the Quaternary (in the past 700,000 to 1 million years). Paleoseismic studies of the Little Salmon fault indicate that the fault deforms late Holocene sediments at the southern end of Humboldt Bay (Clarke and Carver, 1992). Estimates of the amount of fault slip for individual earthquakes along the fault range from 15 to 23 feet (4.5 to 7 meters). Radiocarbon dating suggests that earthquakes have occurred on the Little Salmon fault about 300, 800, and 1,600 years ago. Average slip rate for the Little Salmon fault for the past 6,000 years is between 6 and 10 mm/yr. Based on currently available fault parameters, the maximum magnitude earthquake for the Little Salmon fault is thought to be between 7.0 (CDMG/USGS, 1996) and 7.3 (Geomatrix Consultants, 1994).

Cascadia Subduction Zone. The Cascadia Subduction Zone (CSZ) represents the most significant potential earthquake source in the north coast region. The CSZ is the location where the oceanic crust of the Gorda and Juan de Fuca plates are being subducted beneath continental crust of the North American Plate. A great subduction event may rupture along 200 km or more of the coast from Cape Mendocino to British Columbia, may be up to magnitude 9.5, and could result in extensive tsunami inundation in low-lying coastal areas. The April 25, 1992, Petrolia earthquake (magnitude 7.1) appears to be the only historic earthquake involving slip along the subduction zone, but this event was confined to the southernmost portion of the fault. It is estimated that there have been 6 significant subduction zone events along the CSZ in the last 3,000 years (Darienzo and Peterson, 1995). Paleoseismic studies along the subduction zone suggest that great earthquakes are generated along the zone every 300 to 500 years. Historic records from Japan describing a tsunami thought to have originated along the Cascadia Subduction Zone suggest the most recent great subduction event occurred on January 27, 1700. A great subduction earthquake would generate long duration, very strong ground shaking throughout the north coast region.

The CSZ is located offshore, west of the north coast region. Available mapping indicates that the surface expression of the subduction zone is located some 30 to 35 miles west of the project site (Clarke, 1992; McLaughlin et al., 2000). Seismic profiles suggest that the subduction interface dips landward at an angle of about 11 degrees (McPherson, 1992), which would place it at a depth of about 6 miles beneath the project area (using right angle projection). The CGS fault database shown in Table 4 suggests the fault is only 12 miles west of the site, although we can find no corroborative evidence to substantiate that estimate.

North Spit fault. The North Spit fault was identified in seismic profiles offshore of the North Spit, west of Humboldt Bay. The fault's existence or extent is uncertain, however, because it was not imaged in seismic profiles farther offshore (McCulloch and others, 1977), and it has never been identified on-land. Despite its uncertainty, the fault is relevant to this project because it projects toward the project area. The fault is not recognized or zoned by the State as an active or potentially active fault.

4.2.1.4 Historical Strong Ground Motion

Northern California is a seismically active area that has been subjected to numerous historical earthquakes. Between 1949 and 1985, a total of 927 earthquakes with local magnitudes (M_L) equal or greater than 3.0 occurred (Uhrhammer, 1991). Approximately two-thirds of those earthquakes occurred in the seismically active region along the Cascadia Subduction zone (Gorda Escarpment) or within the Gorda Plate itself (intraplate events).

A search of historical earthquakes occurring between 1800 and 1999, listed in the CGS catalog, was performed for a 100-mile radius around the project site (Blake, 1999b). That search found that 492 earthquakes have occurred within that area. Of those earthquakes, 104 with moment magnitudes (M_W) of 5 or greater, 26 with M_W 6 or greater, and 5 with M_W 7 or greater have occurred. The largest earthquake to affect the area was a M_W 7.3 that occurred on January 31, 1922, approximately 71 miles from the site. The closest earthquakes to affect the site were all located approximately 3.4 miles (5.5 km) from the site, occurred in 1853, 1860, 1903, and 1907, and ranged in M_W from 4.6 to 5.7. The November 13, 1860 earthquake generated an estimated horizontal site ground acceleration of 0.55g, which is the largest acceleration estimated from the database. The most recent significant earthquake to affect the project area was a M_W 5.5 earthquake that occurred on December 26, 1994, approximately 6.9 miles (11.1 km) from the site, generating an estimated horizontal ground acceleration of about 0.28g. The April 25, 1992 Petrolia earthquake generated measured accelerations in excess of 1.0 g at several locations in southern and central Humboldt County. Historic seismic events have generated large accelerations locally within Humboldt County, and should be accounted for in any seismic modeling.

4.2.1.5 Seismic Design Parameters

Where applicable, the project elements should be designed and built to withstand strong seismic shaking. As in all of Humboldt County, the site is subject to strong ground motion from seismic sources.

The 2010 California Building Code requires the following information for seismic design. Based on our knowledge of subsurface and geologic conditions, we estimate a Site Class E (soft soil profile) for the project. Based on the Site Class and the latitude and longitude, we calculated the design spectral response acceleration parameters S_S, S_1, F_a, F_v, S_{MS}, S_{M1}, S_{DS} and S_{D1} using the United States Geological Survey (USGS) seismic calculator program, “Seismic Hazard Curves, Response Parameters, Design Parameters: Seismic Hazard Curves, and Uniform Hazard Response Spectra”, v. 5.1.0, dated February 10, 2011. Calculated values are presented in the following Table 3, Seismic Design Criteria.

Table 3
Seismic Design Criteria

Latitude	40.752144
Longitude	-124.178327
Site Class	E
S_S	2.57
S_1	1.00
F_a	0.9
F_v	2.40
S_{MS}	2.31
S_{M1}	2.40
S_{DS}	1.54
S_{D1}	1.60
Occupancy Category	II
Seismic Design Category	E

4.2.2 Liquefaction

4.2.2.1 Definitions and Historical Perspectives

Liquefaction is described as the sudden loss of soil shear strength due to a rapid increase of soil pore water pressures caused by cyclic loading from a seismic event. In simple terms, it means that a liquefied soil acts more like a fluid than a solid when shaken during an earthquake. In order for liquefaction to occur, the following are needed:

- granular soils (sand, silty sand, sandy silt, and some gravels);
- a high groundwater table; and
- a low density of the granular soils (usually associated with young geologic age).

The adverse effects of liquefaction include local and regional ground settlement, ground cracking and expulsion of water and sand, the partial or complete loss of bearing and confining forces used to support loads, amplification of seismic shaking, and lateral spreading.

Lateral spreading is defined as lateral earth movement of liquefied soils, or competent strata riding on a liquefied soil layer, downslope toward an unsupported slope face, such as a creek bank, or an inclined slope face. In general, lateral spreading has been observed on low to moderate gradient slopes, but has been noted on slopes inclined as flat as one degree.

Liquefaction has been documented on numerous occasions in the project vicinity following historic moderate to large magnitude earthquakes. Specific accounts of historic ground failures are presented in an excellent compilation prepared by Youd and Hoose (1978).

These occurrences are inferred to have occurred in similar geologic environments as those in much of the project area. As such, the historic record would indicate a high probability of liquefaction and potential impacts to the project during future strong seismic events.

4.2.2.2 Project-Specific Liquefaction Hazards

Low-lying bottomland areas, such as the Martin Slough valley, are subject to liquefaction. In these areas, loose, youthful alluvial sediments are subject to high groundwater conditions, and are susceptible to liquefaction when exposed to strong seismic ground motion. In general, the effects of liquefaction on the project could be: deformation associated with differential settlement; loss of strength of the soils within channel side walls, and settlement of structures (bridges, tide gate, etc.).

Lateral spreading is a potential hazard particularly adjacent to an unsupported free face, in this case the channel banks of Martin Slough mainstem and the pond margins. Lateral spreading would potentially affect the infrastructure immediately adjacent to the channels and ponds (settlement of bridge abutments, displacement of pipelines, etc.) and could disrupt the drainage.

There is no technology currently available to cost-effectively mitigate liquefaction potential on a regional basis as would be required for a project of this type. Available means of liquefaction mitigation (compaction grouting, deep dynamic compaction, chemical grouting, vibrocompaction, vibroreplacement, or permanent lowering of the water table) are appropriate for site-specific cases, but are neither economically nor environmentally feasible at the scale required for this project.

4.2.3 Tectonically-Induced Uplift/Subsidence

Large-scale land level changes are possible during large seismic events on regional faults in the project vicinity. The most recent example of this coseismic phenomenon occurred during the April 25, 1992 Petrolia earthquake. During that event, a large area of coastal reef near Cape Mendocino was uplifted up to 4.5 feet (Jayko and others, 1992). That event is thought to have occurred on the southern end of the Cascadia Subduction Zone. Similar impacts are inferred during paleoseismic studies in marshes around Humboldt Bay. At these sites, stratigraphic evidence suggests large-scale rapid subsidence associated with large earthquakes, most likely associated with the Cascadia Subduction Zone. These studies indicate at least eight rapid subsidence events in the past 3,500 years (Valentine et al., 1992). Regional faults most likely to result in large-scale land-level changes are the Little Salmon fault and the Cascadia Subduction Zone. In general, the project area is subject to subsidence due to its location in a broad syncline between the Little Salmon fault and Mad River fault zones.

There are no means to mitigate the potential for large-scale land level changes and the associated impacts to the project. Such a rare catastrophic event may require replacement of the tide gate.

4.2.4 Landslides and Mass Wasting

Landsliding and mass wasting are most likely to occur in the project area on the valley sidewalls above Martin Slough, and in the adjacent tributary canyons. Most of the failures observed within the vicinity are shallow debris slide type failures, which is consistent with the anticipated failure mode for granular sediments of the Hookton Formation. These landslides typically do not affect areas far from the slopes. The project elements associated with the Martin Slough Enhancement Project are all within the valley floor, and are not anticipated to impact or be impacted by the stability conditions of the adjacent slopes.

4.2.5 Soil Settlement

Static or seismically induced settlement can occur in soils that are loose, soft, or excessively organic-rich. As described above, most soils in the low-lying portions of the study area are loose, soft, and/or organic-rich. As such, there is a potential that static or dynamically induced settlement may occur along the project area. The settlement potential generally applies to the bridges that will be placed on shallow foundations. However, provided the settlement potential is acceptable, and accommodated in the design (approach ramps, etc.), the risks are generally low. Recommendations for minimizing the settlement potential have been provided in our Geotechnical Report for the project.

4.2.6 Soil Erosion

The proposed project will become a source of erosion as a consequence of removing the vegetation cover and widening the channels and excavating the ponds. Erosion potential associated with freshly excavated stream banks will be the highest where soils are granular (sandy), and within areas of relatively high flow velocities. Until vegetative cover is adequately restored, there will be potential erosion associated with rainfall and surface flows.

4.2.7 Tsunami Inundation

Tsunamis are long-period sea waves caused by sea floor deformation associated with submarine fault rupture or submarine landslides, sometimes from sources hundreds or thousands of miles away. Because the project is located in a low-lying coastal area in a seismically active region, the portions of it nearest the margins of Humboldt Bay are subject to tsunami inundation. The hazard associated with tsunami inundation is increased in the Humboldt County area due to the proximity of the Cascadia Subduction Zone and other active offshore seismic sources that are capable of generating very large earthquakes.

Tsunamis have been observed along the northern California coastline following large earthquakes in the recent past. The most significant historical tsunami inundation in the region occurred in Crescent City in 1964 following a magnitude 9.2 earthquake in Alaska. Inundation associated with this tsunami generated over $7 million damage in Crescent City and resulted in ten fatalities. Over 1,000 automobiles were destroyed. The 1964 tsunami resulted in run-up of 6 feet (about 2 meters) in Humboldt Bay (Lander and Lockridge, 1989), but caused no significant damage. The tsunami resulted in fourteen knot currents near the bay entrance, and the bay was filled with logs and other debris. A ten-foot-high sea wall was breached at the Eureka Boat Basin (Lander et al., 1993). More recently, on

April 25, 1992, a series of strong earthquakes occurred near Cape Mendocino. The main shock was magnitude 7.1, and was followed by strong aftershocks with magnitudes of 6.6 and 6.7. The magnitude 7.1 main shock generated a small tsunami that was recorded by tide gauges from Oregon to southern California (Bernard et al., 1994), including at Humboldt Bay. The wave was 0.7 to 1 foot (20 to 30 centimeters [cm]) high at the Humboldt Bay entrance, and caused no damage.

Based on a 33 foot (10 meter [m]) incident tsunami wave, inundation models were developed for the Humboldt Bay region (Bernard et al., 1994). The dynamics of tsunami run-up inside Humboldt Bay are not well understood, but the inundation model suggests that run-up may locally reach 10 feet (3 m). This 10-foot run-up would presumably be added to the tidal height at the time of the tsunami. Therefore, if a tsunami inundation event occurred during a significant high tide, the bayside run-up may be quite extensive. Tsunami inundation would have its most significant impact at sites near the bay margin, and may extend into the Martin Slough valley.

Tsunami inundation hazard associated with the proposed project is highest within the southern portions of the project, and within the Elk River valley. Tsunami inundation hazard decreases rapidly toward the upstream portions of the project. The risk associated with tsunami inundation is primarily focused at above-ground improvements, and should be relatively minor for the project elements as a whole. Wave scour may occur locally during tsunami run-up, potentially damaging the berm along Swain Slough, the tide gate, and bridges. Entrained sediment within a tsunami wave could infill ponds and channels, disrupting the drainage system. This scenario seems unlikely, however, because the velocity of any waves that might make it into the project area will likely be low.

5.0 References

Bernard, E., C. Mader, G. Curtis, and K. Satake. (1994). *Tsunami Inundation Model Study for Eureka and Crescent City, California. National Oceanic and Atmospheric Administration Technical Memorandum ERL PMEL-103*. 80 p. with maps.

Blake, M.C., A.S. Jayko, and R.J. McLaughlin. (1985). "Tectonostratigraphic Terranes of the Northern Coast Ranges," California, *in* D.G. Howell (ed), *Tectonostratigraphic Terranes of the Circum-Pacific Region: Circum-Pacific Council for Energy and Mineral Resources Earth Science Series 1*. P. 159-171.

Blake, T.F. (1999a). *EQFAULT, A Computer Program for the Deterministic Prediction of Peak Horizontal Acceleration from Digitized California Faults, User's Manual*. 79 p.

--- (1999b). *EQSEARCH, A Computer Program for the Estimation of Peak Horizontal Acceleration from California Historical Earthquake Catalogs, User's Manual*. 109 p.

Carver, G.A., and R.M. Burke. (1992). "Late Cenozoic Deformation on the Cascadia Subduction Zone in the Region of the Mendocino Triple Junction," *in* Burke, R.M. and Carver, G.A. (eds). *1992 Friends of the Pleistocene Guidebook, Pacific Cell*, p. 31-63. NR: FOP.

Clarke, S.H., Jr. (1992). *Geology of the Eel River Basin and Adjacent Region: Implications for Late Cenozoic Tectonics of the Southern Cascadia Subduction Zone and Mendocino Triple Junction: AAPG Bulletin,* v. 76, no. 2, p. 199-224. NR: AAPG.

Clarke, S.H., Jr. and G.A. Carver. (1992). "Late Holocene Tectonics and Paleoseismicity of the Southern Cascadia Subduction Zone, Northwestern California," Science, v. 255, p. 188-192.

Darienzo, M.E. and C.D. Peterson (1995), Magnitude and frequency of subduction-zone earthquakes along the northern Oregon coast in the past 3,000 years. Oregon Geology, v. 57, p. 3-12.

Dengler, L., R. McPherson, and G.A. Carver. (1992). "Historic Seismicity and Potential Source Areas of Large Earthquakes in North Coast California," *in* Burke, R.M. and Carver, G.A. (eds). *1992 Friends of the Pleistocene Guidebook, Pacific Cell*, p. 112-118. NR: FOP.

Geomatrix Consultants. (1994). *Seismic Ground Motion Study for Humboldt Bay Bridges on Route 255.* Unpublished Consultants Report for the California Department of Transportation. NR: Geomatrix.

Jayko, A.S., G.A. Marshall, and G.A. Carver. (1992). "Elevation Changes," *in Special Issue: The Cape Mendocino Earthquakes of April 25-26, 1992. Earthquakes and Volcanoes*, v. 23, p. 139-143. NR: NR.

McCulloch, D.S., A.T. Long, and P.A. Utter. (1977). *Acoustic Profiles Offshore Humboldt Bay, California. United States Geological Survey Open-File Report 77-667*. NR: USGS.

McLaughlin, R.J., et al. (2000). "Geology of the Cape Mendocino, Eureka, Garberville, and Southwestern Part of the Hayfork 30 x 60 Minute Quadrangles and Adjacent Offshore Area, Northern California," *U.S. Geological Survey Miscellaneous Field Studies MF-2336*. NR: USGS.

McPherson, R.C. (1992). "Style of Faulting at the Southern End of the Cascadia Subduction Zone," *in* Burke, R.M. and Carver, G.A. (eds). *1992 Friends of the Pleistocene Guidebook, Pacific Cell,* p. 97-111. NR: FOP.

Ogle, B.A. (1953). *Geology of the Eel River Valley Area, Humboldt County, California: California Department of Natural Resources, Division of Mines, Bulletin 164*. Sacramento: CDNR.

Petersen, M.D., et al. (1996). *Probabilistic Seismic Hazard Assessment for the 1996 S. Petersen, M.D., et al. Probabilistic Seismic Hazard Assessment for the State of California, California Division of Mines and Geology Open-File Report 96-08, U.S. Geologic Survey Open-File Report 96-706*, 33 p., with appendices. NR: NR.

Uhrhammer, R.A. (1991). "Chapter 7: Northern California Seismicity," in *Neotectonics of Northern California,* Slemmons, D.B., E.R. Engdahl, M.D. Zoback, and D.D. Blackwell, Eds. pp. 99 – 106. NR: Geological Society of America, Decade of North American Geology.

Valentine, D., G. Vick, G. Carver, and C. Shivelle-Manhart. (1992). "Late Holocene Stratigraphy and Paleoseismicity, Humboldt Bay, California," *in* Burke, R.M. and Carver, G.A. (eds). *1992 Friends of the Pleistocene Guidebook,* Pacific Cell, p. 182-187. NR: FOP.

Wills, C. J. (1990). *Little Salmon and Related Faults. California Division of Mines and Geology Fault Evaluation Report FER-215*. Sacramento: CDMG.

Woodward-Clyde Consultants. (1980). *Evaluation of the potential for resolving the geologic and seismic issues at the Humboldt Bay Power Plant Unit No.* 3. Unpublished consultants report for Pacific Gas and Electric. NR W-C.

Youd, T.L., and S.N. Hoose. (1978). *Historic Ground Failures in Northern California Triggered by Earthquakes.* United States Geological Survey Professional Paper 993. 177 pages, with maps. NR: USGS.