Martin Slough Enhancement Project
Eureka, California
Basis of Design Report

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GHD

May 2013

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Basis of Design Report

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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 has been severely degraded by the installation of tidegates 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. Increase the amount of riparian corridor and riparian canopy,
3. Reduce flood impacts to current land use,
4. Improve sediment transport,
5. Improve water quality (decrease nutrient impacts, decrease sedimentation, salinity)
6. Improve and increase the diversity and amount of freshwater habitat, especially off-channel/ backwater habitats that coho salmon need for over-wintering, and saltwater wetland habitat.

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.

This report covers the current scope of work, which is to develop the selected alternative to the 30% design level. The design development team included GHD and MLA, under direction from RCAA. Input was received from the TAC and other stakeholders, including the new owners of Gene Senestraro's property, the North Coast Regional Land Trust. With feedback, additional modeling, and further design, the project elements remain essentially the same and the project was developed to the 30% design level.

1.1 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 Figure 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 tidegates. 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, a natural gas line, an existing sewer line, a planned and partially constructed sewage interceptor line, 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 tidegates 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 Enhancement Feasibility Study area consists of the general flood plain between Swain Slough and the upper (second) Fairway Drive stream crossing in the lower Martin Slough watershed (Figure 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 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.

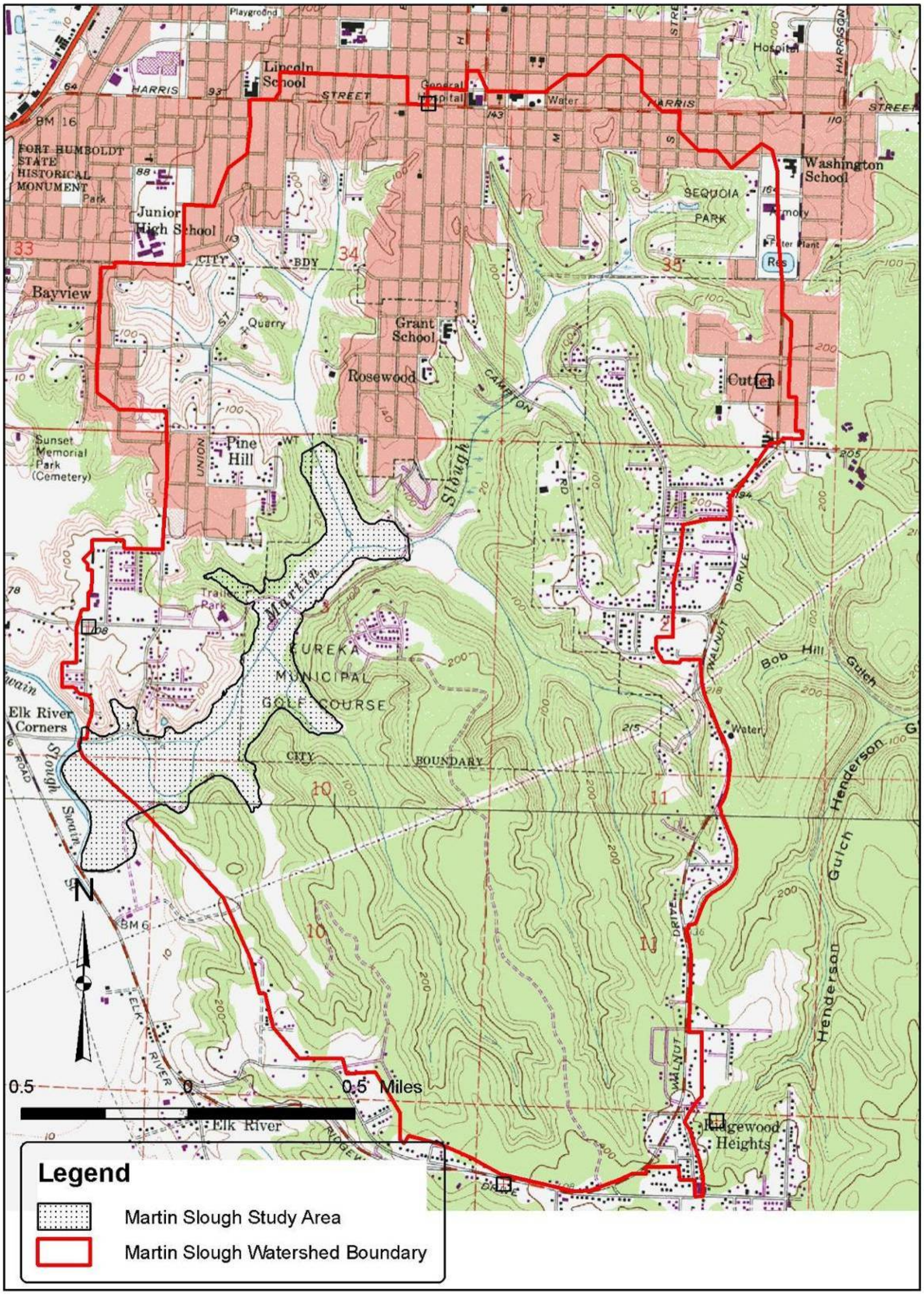


Figure 1-1. Martin Slough Project Area and Watershed Boundary.

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* (Feasibility Study) prepared for Redwood Community Action Agency (W-K et al., 2006). Alternative 4 was selected by the TAC as the preferred alternative.

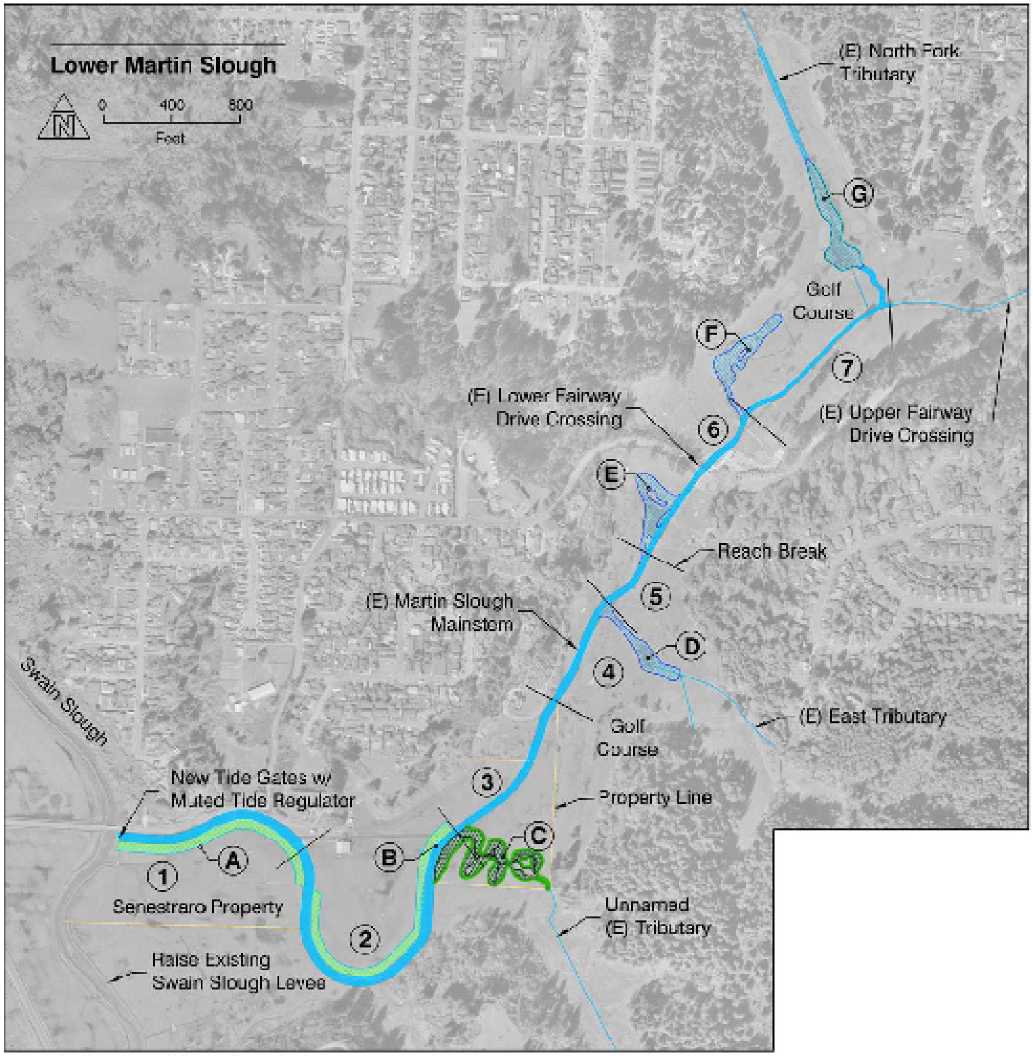
This report presents the conceptual design development of Alternative 4. Alternative 4 proposes reintroducing limited tidal influence into lower Martin Slough through new tidegates, enlarging the channel to accommodate the daily tidal flux, and constructing numerous off-channel ponds and wetlands. These components would create a self-sustaining tidal system while providing increased aquatic habitat and improved routing of floodwaters and sediment.

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

- Replacement of the existing tidegates
- Construction of seven tidal wetlands
- Reoccupy and enlarge the existing slough channel
- Installation of large wood for fisheries habitat throughout the project
- Repair a section of the existing berm between Martin Slough and Swain Slough
- Improve drainage in the areas of play within the golf course
- Replacement of two agricultural culverts with bridges and eight golf course bridges that span the channel
- 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 (1.4 acres & 2.1 acres) - salt marsh plain 50ft wide paralleling slough channel.

C (2.4 acres) - Salt marsh with low and high elevation ponds connected to historic channel network and freshwater tributary.

D & E (0.8 acres & 1.2 acres) - Expanded fresh/brackish wetlands, containing deep open water, littoral benches and elevated outlet sill that minimizes salinity intrusion during wet season.

F (1.2 acres) - Backwater slough with island. Predominately freshwater with deep open water and littoral bench on inside of bend.

G (0.8 acres) - On channel predominately freshwater pond. Deep open water with emergent vegetation along banks.

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	57	1,125
2	52	2,125
3	45	1,025
4	41	625
5	39	400
6	33	1,000
7	23	1,000

MARTIN SLOUGH ENHANCEMENT PROJECT

Figure 2-1. Summary of Martin Slough Enhancement Project Components.

Figure 2-1. Summary of the Martin Slough Enhancement Project components.

2.1 Tide Gate Replacement

2.1.1 Design Objectives

A new tidegate structure will replace the existing undersized and failing tide gates where Martin Slough drains into Swain Slough. The replacement structure will improve discharge capacity, improve aquatic organism passage, and introduce estuarine conditions into Martin Slough. The replacement tide gate structure 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

A new tide gate structure will replace the existing undersized tide gate structure, increasing outflow capacity by nearly three times and reintroducing limited tidal influence into Martin Slough. The proposed replacement structure will consist of three 6-foot by 6-foot gates installed into a new triple bay concrete box culvert (Figure 2-2).

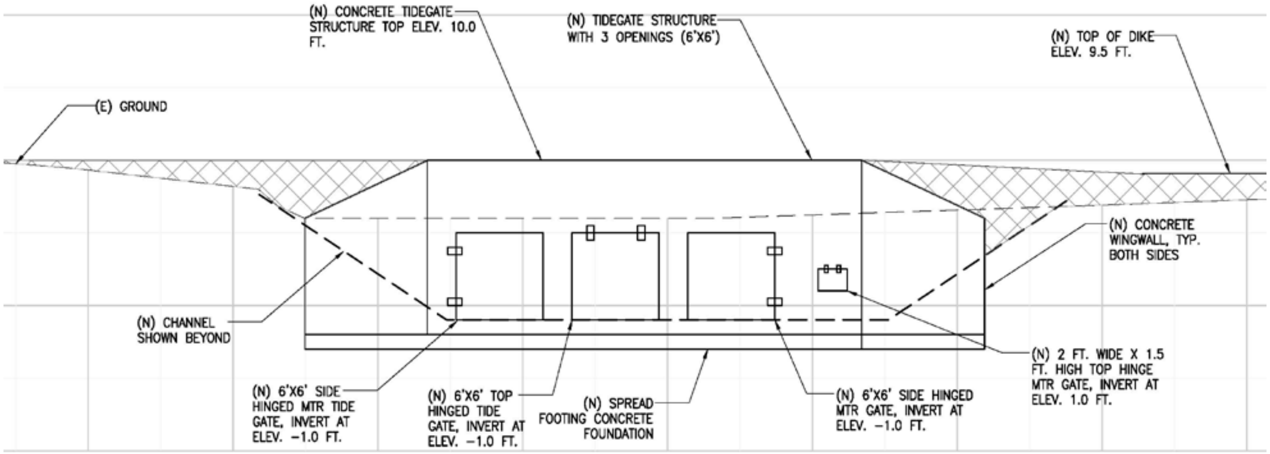


Figure 2-2. Cross section on view from Swain Slough of proposed replacement tide gate structure.

The three main tide gates will open when water levels in Martin Slough are higher than in Swain Slough. The two outer gates will be side-hinged and the middle gate will be 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, will help center the outflow velocities to reduce risk of scouring the adjacent berm and Pine Hill Road Bridge supports.

The invert elevations of the three main tide gate doors were set at an elevation of -1.0 feet, which is slightly deeper than the current elevation of Swain Slough. This elevation will be 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 will be equipped with an adjustable muted tide regulator (MTR).

An auxiliary gate with an adjustable MTR will be constructed either into the southern tide gate door or as a separate opening in the structure. The Auxiliary MTR gate Door will be top-hinged with 1.5- foot high by 2-foot wide gate set at an invert elevation of 1.0 feet.

2.1.3 Muted Tide Regulator (MTR) Gates

Within the new tide gate structure, the southern 6-foot by 6-foot side hinged tidegate and the 1.5-foot high by 2-foot wide Auxiliary Gate will be equipped with MTR systems. The two MTR-controlled gates will 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 will allow for tidal water to flow through the gate and upstream into Martin Slough. The MTR system for each gate will contain an adjustable mechanical lever attached to a float that will close 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 tidegate doors allowing outflow on an ebb tide will create muted tidal conditions inside of Martin Slough.

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 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.

2.1.4 Muted Tide and Design Tidal Prism in Martin Slough

Tidal prism is defined 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 28 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 old Senestraro 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.

2.2 Constructed Tidal Wetlands

2.2.1 Tidal Wetland Descriptions

As part of this project, seven tidal wetlands will be constructed. They are denoted by letters A through G (Figure 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.

Three of the seven constructed tidal wetlands (Ponds D, E, and G) involve enlargement of existing wetlands on the golf course that are currently influenced by leaking of the existing failing tidegates. 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 Senestraro Property.

Location and configuration 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 Senestraro Property were located to minimize fragmentation of the pasture and incorporate the freshwater tributary entering from the southeast corner of the property.

The proposed tidal wetlands will be spatially dispersed to create a continuum of estuarine environments, as found in naturally functioning tidal estuaries. The wetlands will transition from marine salinities at the downstream end of the project area to predominately fresh water at the upstream end. Three of the tidal wetlands will be located on freshwater tributaries to Martin Slough, and will receive freshwater inputs. Salinity concentrations are expected to fluctuate from summer to winter, being higher in the summer when less freshwater is entering the project area.

Table 2-1 summarizes the locations, sizes, and depths of the proposed tidal wetlands.

Table 2-1. Summary of the Five Proposed Tidal Wetlands within the Martin Slough Project Area.

Pond	Area	Outfall Station on Martin Slough Mainstem	Residual Depth
A	1.4 acres	Adjacent to Stations 1+50 to 13+50	NA
B	2.1 acres	Stations 14+00 to 32+00	NA
C	2.4 acres	Station 32+50	NA
D	0.8 acres	Station 48+50	2.0 feet
E	1.2 acres	Station 56+20 (Upstream)	2.0 feet
		Station 52+50 (Downstream)	
F	1.2 acres	Station 62+50	3.0 feet
G	0.8 acres	Station 73+00	2.8 feet

2.2.2 Tidal Wetland Layouts

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. For most of the ponds, this involved the use of flow-through tributaries or multiple inlets.

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. Proposed marsh plains were designed to support salt marsh vegetation.

Marsh Plains A and B

Marsh Plains A and B are approximately 50 feet in width and will parallel the Martin Slough channel for a total of approximately 3,000 feet. The marsh plain surface will be undulating to encourage zonation of marsh vegetation and formation of first and second order tidal channels by concentrating runoff during ebb tides.

Large anchored wood structures will be placed onto the marsh plains to promote local scour and increase topographic complexity, provide cover for fish, and provide perches for birds.

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 an area with several remnant historical tidal channels and will convey flow from a freshwater tributary of Martin Slough that enters from the south. It will contain a main tidal slough channel and several tributary slough channels with in-channel ponds. The channels and ponds will be surrounded by approximately 2.4 acres of salt marsh plain. Several "fingers" of higher ground will project into the marsh plain.

The freshwater tributary that will be incorporated into Pond C currently flows along the property line and through a culvert before discharging into Martin Slough. This channel will be abandoned for the new tidal channel. The new tidal channel will have a top width of 9 feet and depths ranging from 1.8 to 3.4 feet at MHHW. The tidal channel bottom will be at elevation 1.5 feet where it joins Martin Slough. At the property line the channel bottom elevation will meet the existing tributary channel. A large in-channel pond and flow-through side channel will be constructed at the upstream end of the new channel.

Several small side channels will connect to the main slough channel to provide off-channel habitat for aquatic organisms. Wide spots at the confluences provide refuge areas for goby and goby holes are provided at the upstream extends of some of the side channels. Large wood structures placed within the slough channels will sustain scour pools and provide hydraulic controls during ebb and flood tides.

The tidal marsh plain was designed with a range of elevations that is expected to support a diverse range of tidal marsh species.

Brackish to Freshwater Wetlands (Ponds D – G)

In the middle and upper portions of the project, the constructed tidal wetlands Ponds D through G were configured to support a combination of brackish and freshwater emergent wetland vegetation and open waters. Pond D is an in-line pond on the East Tributary that will be enlarged. Pond E is an existing pond connected to the Martin Slough mainstem just downstream of the Lower Fairway Drive bridge crossing that will also be enlarged and converted to a flow-through pond with two entrances. Pond F will be a new pond located just upstream of Lower Fairway Drive that will be connected to Martin Slough via an approximate 150-foot long slough channel. Pond G is an in-line pond on the North Fork Tributary that will be enlarged. The outfall of Pond G will be realigned out of the existing stream channel into a sinuous 300-foot long channel that connects to Martin Slough Mainstream.

The open water areas in each of the ponds generally will have a bottom set near elevation 0 feet. This depth will prevent colonization by emergent wetland vegetation. 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.

Ponds D through G will have littoral benches that gently slope between elevations 4 feet and 5 feet. The benches are 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 between one and two feet during high tides, making the benches suitable for supporting wetland vegetation. At and above the high tide water line, zones of wetland vegetation will change to more upland vegetation.

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

2.2.3 Slough Channel Restoration

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 7,300 feet of the existing Martin Slough channel will be enlarged within the project area to increase conveyance for both flood flows and tidal exchange.

Channel Alignment

The proposed channel alignment will vary from the existing straightened channel alignment in several locations. The most significant change in alignment will be the reoccupation of the 2,100-foot long historical meander on the Senestraro Property. The proposed realignment into the historical meander will lengthen the slough channel by approximately 1,300 feet. The realignment will move the slough channel away from the existing barn and will eliminate an existing undersized culvert crossing. This will allow for the "ditch" adjacent to the barn to be converted from open channel into a marsh that will provide a buffer between the barn and the channel to filter barn area runoff before it enters the channel.

There will be minor changes to the channel alignment along the Mainstem of Martin Slough in the golf course. These changes will add a small amount of sinuosity to the existing linear channel within the constraints of the existing golf course. The North Fork Tributary, between Pond G and the confluence of the Mainstem will be substantially realigned, with three small meanders added.

Slough Channel Cross Section

The channel within the project area was divided into seven reaches, numbered from downstream to upstream (Figure 2-1). Reaches were generally segmented at the confluences of the proposed tidal ponds because they contribute significantly to the tidal prism of a reach.

Table 2-2 presents channel dimensions and contributing tidal prism for each reach. The Martin Slough tidal channel will be constructed with a trapezoidal shape having side-slopes of 1.5H:1V. The shape of these channels is expected to evolve into a more parabolic shape, typical of tidal channels. The resulting stable channel geometries will have top widths ranging from 57 feet wide in Reach 1, along the lower portions of the Senestraro Property near the tide gates, to 23 feet wide in Reach 7, which extends to the confluence with the North Fork of Martin Slough. The constructed channel depths, as measured from the top of bank to bottom of channel, will range between 6.3 feet and 3.9 feet.

Slough Channel Profile

The design bottom elevation of Martin Slough at the downstream end of the project will be set at -1.0 feet This is equal to the invert elevations of the lower two 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.

The channel slopes up at 0.02% to 0.06% for 6,300 feet, then transitions over 1,000 feet at slopes ranging from 0.18% to 0.3% to tie into the existing channel at elevation 2.8 feet just upstream of the realigned confluence with the North Fork Tributary.

Table 2-2. Summary of proposed tidal channel cross section dimensions and contributing tidal prism for each reach of Martin Slough. [1]

Reach	Station (Length)	Bottom Width	Typical Top of Bank Width	Typical Channel Depth	Contributing Tidal Prism (MHHW-MLLW)[1]
1	0+00 to 11+25 (1,125 feet)	30.5 feet	57 feet + Marsh Plain	6.3 feet	4.2 AF Channel 0.5 AF Marsh Plain A
2	11+25 to 32+50 (2,125 feet)	26.0 feet	52 feet+ Marsh Plain	6.1 feet	7.0 AF Channel 0.5 AF Marsh Plain B 1.0 AF Pond C
3	32+50 to 42+75 (1,025 feet)	19.5 feet	45 feet	5.8 feet	2.7 AF Channel
4	42+75 to 49+00 (625 Feet)	17.5 feet	41 feet	5.6 feet	1.5 AF Channel 1.3 AF Pond D
5	49+00 to 53+00 (400 feet)	16.0 feet	39 feet	5.5 feet	0.9 AF Channel 2.0 AF Pond E
6	53+00 to 63+00 (1,000 feet)	9.0 feet	33 Feet	5.3 Feet	1.6 AF Channel 2.0 Pond F
7	63+00 to 73+00 (1,000 Feet)	3.5 Feet	23 Feet	3.9 Feet	0.8 AF Channel 1.6 Pond G
				Total Tidal Prism for Design Conditions	**27.6 AF**

[1] Measured at downstream end of reach

2.2.4 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. Additionally, the existing tidegate has limited outflow capacity that increases the amount of time necessary for storm events to drain out of Martin Slough. 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 tidegate has a much larger outflow capacity, reducing the amount of time for flood flows to drain from Martin Slough.

2.2.5 Replacement Bridge Crossings

The old Senestraro property, now owned by the North Coast Regional Land Trust, is still being managed for cattle grazing. The restoration of Martin Slough channel will affect access to the southern pasture which was resolved by replacing existing culvert crossings with bridge crossings. Two new agricultural bridges are part of the project, one adjacent to and west of the existing barn, and one to the east of the existing barn. The bridge crossings will allow access to the pastures on the property, without impacting the stream function of the restored Martin Slough channel. The bridges will span the active channel.

2.2.6 Revegetation

The 30% 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 preliminary 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.

3.2 Geotechincal 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. 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 (Figure 3-1). Field methods and findings are described in Appendix D of the 2006 Martin Slough Enhancement Feasibility Study.

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) (Figure 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.

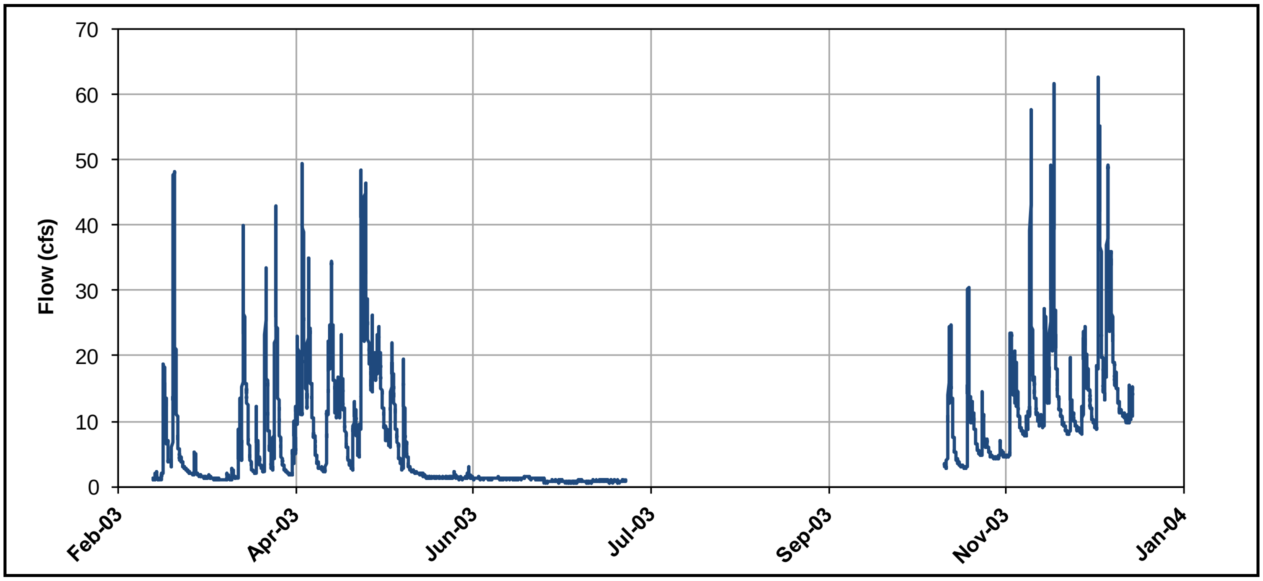


Figure 3-1. Gaged flows in Martin Slough at the Upper Fairway Drive crossing. Low-flows were not gaged during the summer dry season.

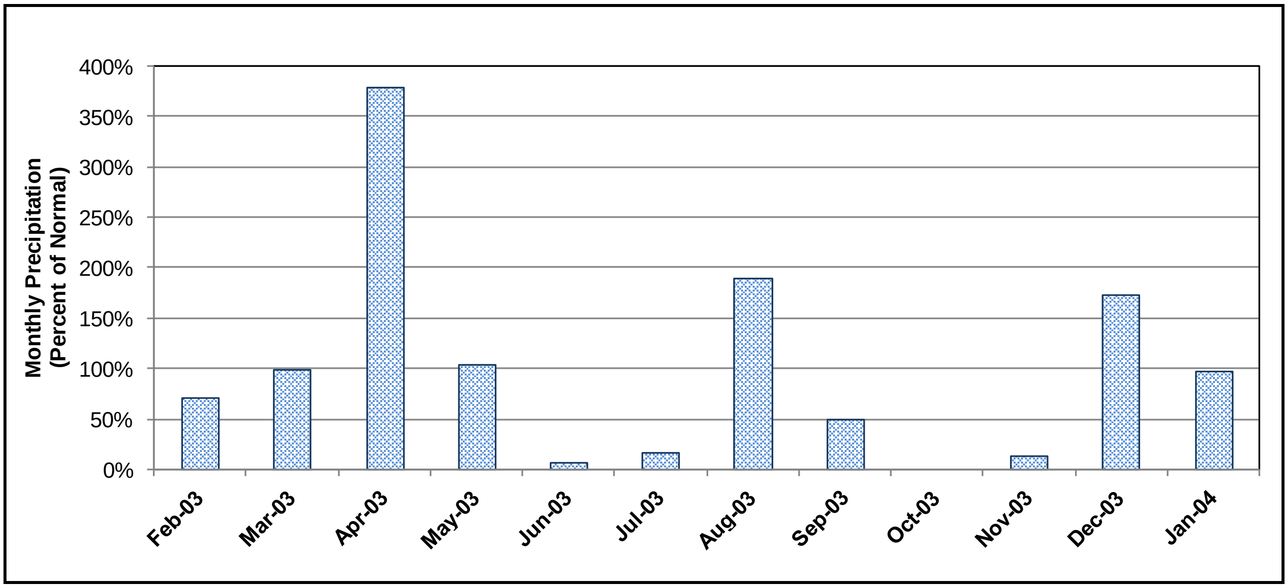


Figure 3-2. Recorded monthly precipitation in Eureka (Woodley Island) as a percent of normal for that month. Months plotted span the period-of-record for the Martin Slough flow gaging station at the Upper Fairway Drive crossing.

Summer Baseflows

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 3-1 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 3-1. Summer baseflow measured in Martin Slough and the North Tributary from August 4 through October 15, 2008.

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 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 variability.

3.3.2 Fish Passage Design Flows

Fish passage design flows were needed to evaluate fish passage through the replacement tide gates. These flows were computed as part of the 2006 Feasibility Study in accordance with CDFG (2002) and NMFS (2001) guidelines for adult salmon and steelhead, adult resident rainbow and cutthroat trout, and juvenile salmonids (W-K et al., 2006). Fish passage flows were computed using a flow duration curve based on daily average flows from the Little River near Trinidad gage (USGS Station No. 11481200) scaled to the Martin Slough drainage area at the tide gates of 5.51 square miles (Table 3-2).

Table 3-2. Fish Passage Design Flows for Martin Slough At The Tide Gates. Flows Computed Using CDFG (2002) and NMFS (2001) Guidelines.

Species and Lifestage	Low Passage Flow	High Passage Flow
Adult Salmon and Steelhead	3.6 cfs	89 cfs
Adult Resident Rainbow and Cutthroat Trout	2.0 cfs	41 cfs
Juvenile Salmonids	1.0 cfs	27 cfs

3.3.3 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 Figure 3-3. Contributing sub-basins to each inflow location are color-coded in Figure 3-4. 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 3-3. 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.

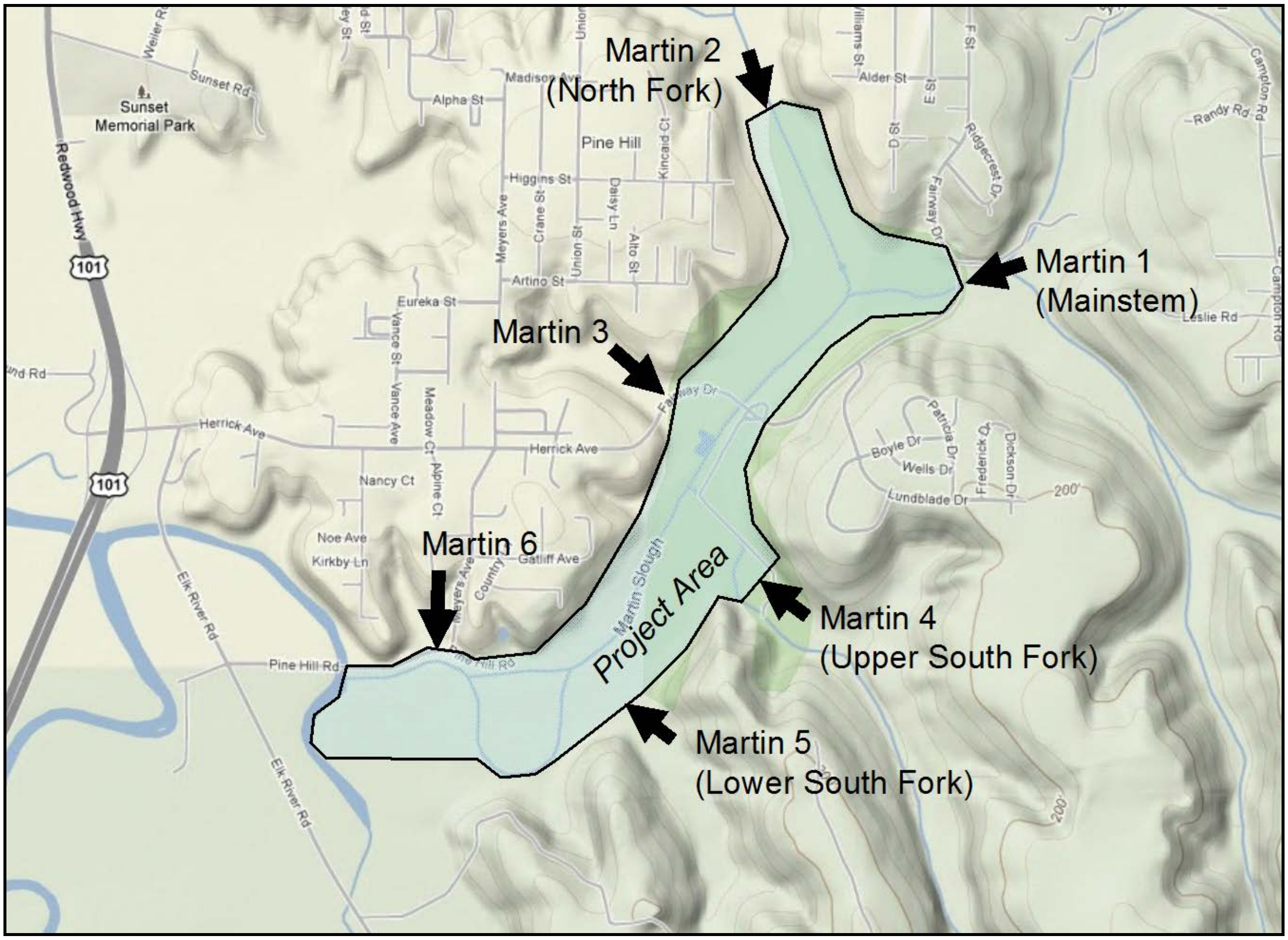


Figure 3-3. General location of flow inputs to the project area. Flow inputs obtained from the HEC-HMS rainfall-runoff simulations prepared in W-K et al. (2006).

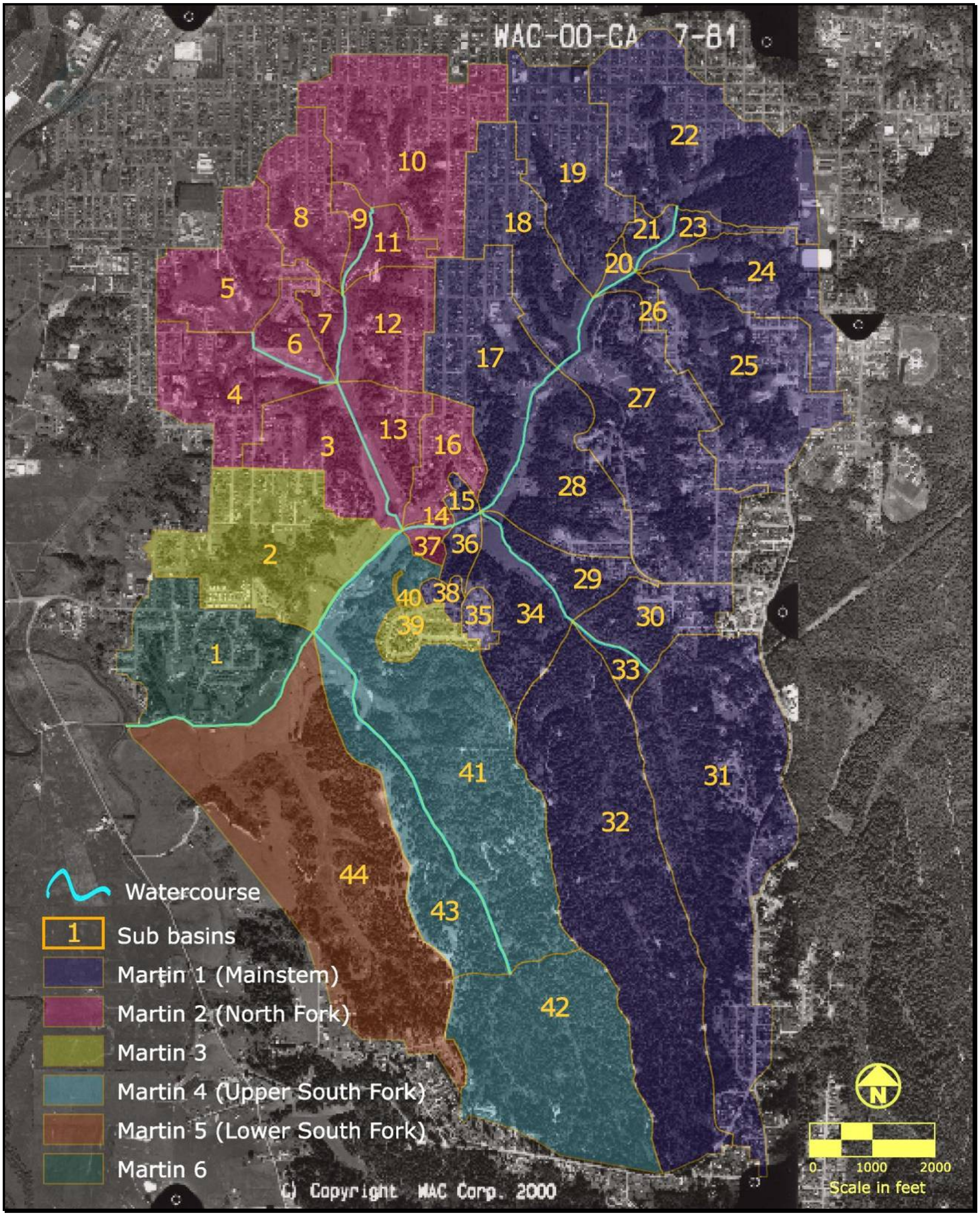


Figure 3-4. Martin Slough HEC-HMS sub-basins grouped by color based on location of flow inputs to the project area. See W-K et al. (2006) for hydrologic model development.

Table 3-3. 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	

3.3.4 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 and Swain Sloughs between February 12, 2003 and February 20, 2003 (Figure 3-5). 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 fell much slower and did not go below approximately 1.5 feet. This is attributed to a sill at the mouth of Elk River; most likely a persistent sandbar (Eicher, 1987).

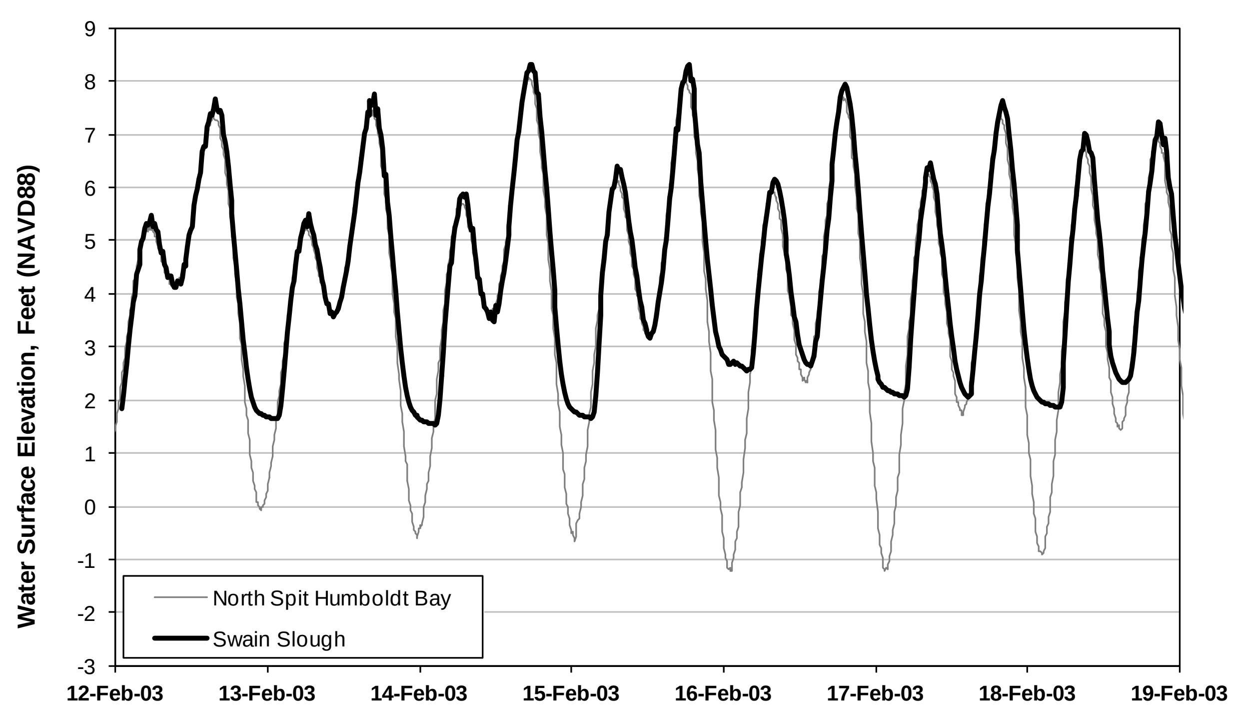


Figure 3-5. Recorded tidal elevations in Swain Slough (Black) and corresponding tidal elevations recorded at North Spit, Humboldt Bay (Grey) (Station No. 9418767). A tidal sill, likely a sand bar at the mouth of Elk River, prevents water levels from dropping below approximately 1.5 feet in Swain Slough.

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 useful 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 3-4). 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 3-4. 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 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 would provide the same 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 (Figure 3-6). 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 (Figure 3-6). 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 (Figure 3-7). 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 (Figure 3-6), except for MLLW, which is equal to the presumed tidal sill elevation of 1.5 feet.

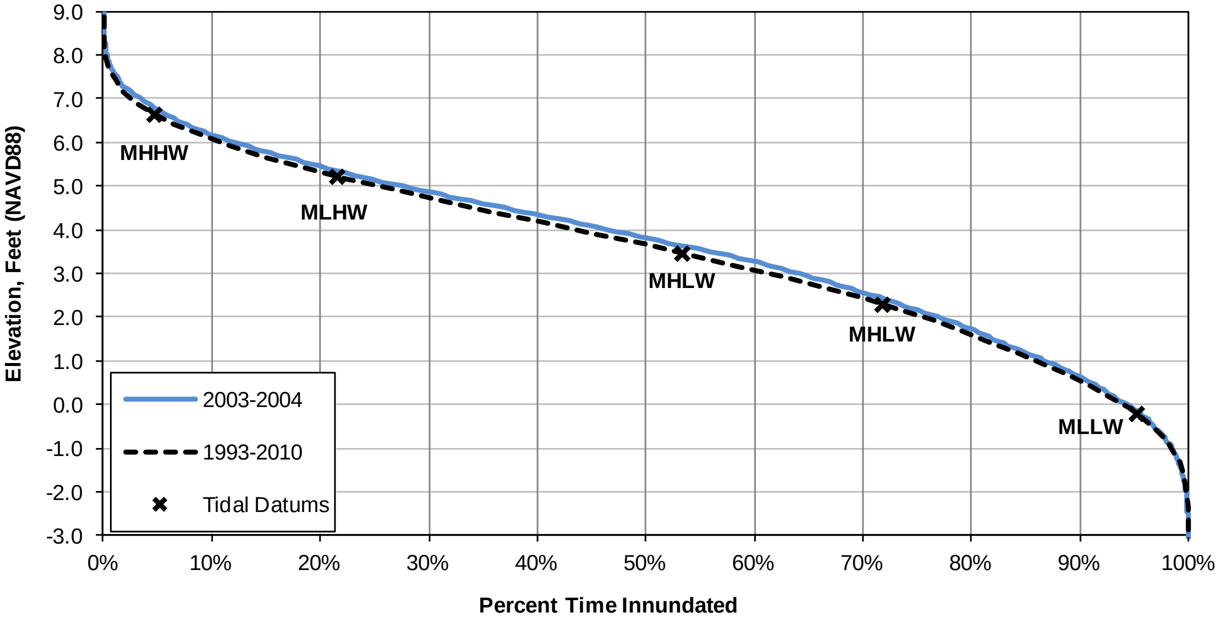


Figure 3-6. Comparison of frequency of tidal inundation for North Spit, Humboldt Bay (Station 9418767) using different record lengths. Tidal datums calculated using the 1993 to 2010 record.

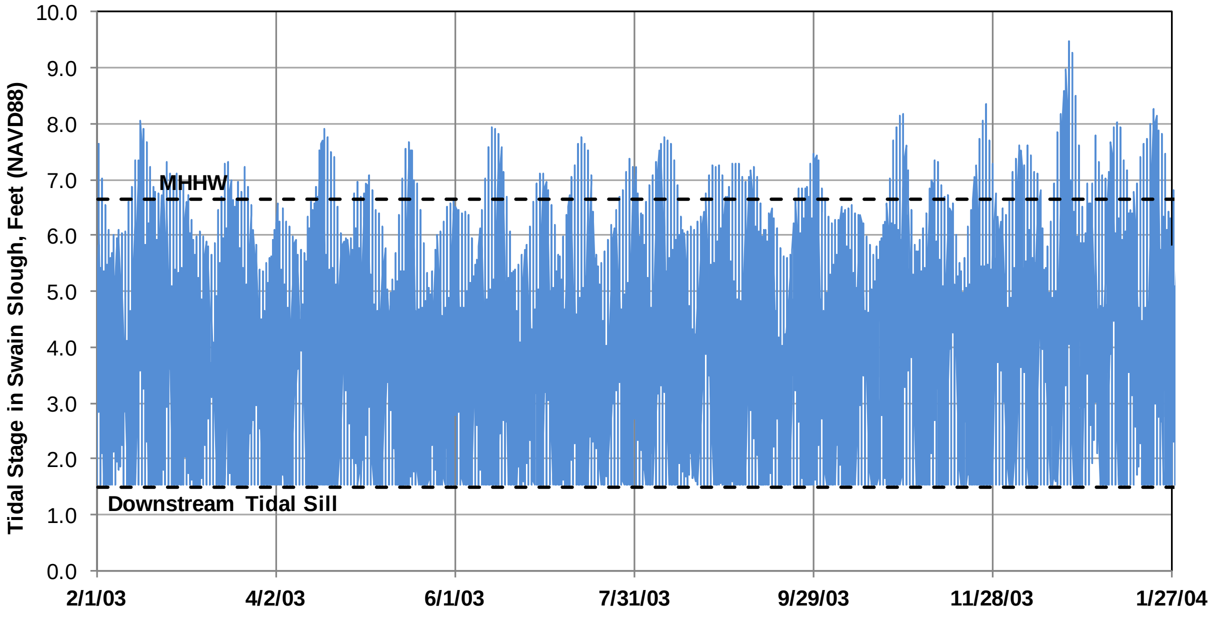


Figure 3-7. One-year of tidal record for Swain Slough constructed using North Spit, Humboldt Bay tidal records (Station No. 9418767), with tide truncated at elevation 1.5 feet to account for downstream tidal sill at the mouth of the Elk River.

4.0 Design Development

Development of individual project elements was an iterative process involving (1) determining initial tide gate, 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). To further evaluate and refine the proposed project, additional slough channel sizing and hydraulic modeling for the project were necessary.

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 a presents the results of the numerical modeling for specific project elements.

4.1 Tidal Slough Channel Design

4.1.1 Martin Slough

The project area of Martin Slough will be wholly 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 was assumed to be governed by the daily tidal flux 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.

The dimensions of the tidal slough channel in Martin Slough was 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 in 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 (Figure 2-1). Reaches were generally segmented at the confluences of proposed off-channel and in-channel wetlands and ponds because they contribute significantly to the tidal prism of a reach.

Contributing tidal prism and stable channel geometry was calculated for each reach (Table 2-2). The contributing tidal prism in the channel ranges from nearly zero at the upstream end of the project area to approximately 28 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 relationships for small tidal marshes in Humboldt County developed by Jeff Anderson and Associates (2009).

4.2 Hydraulic Modeling

The conceptual designs for the project area were 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 the HEC-RAS were used to route flows 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:

- Establishing the sizing of the tide gates for outflow
- Establishing the closing elevation of the MTR gate and Auxiliary Door in the new tidegate structure,
- Verification of the design tidal prism and MHHW elevation,
- Identifying the elevations of design tidal marsh plains to obtain the desired vegetation diversity,
- Assessing fish passage through the replacement tide gate structure,
- Comparing existing and design condition duration and frequency of storm flow flooding,
- Assessing sediment mobility through the project area, and
- Evaluating seasonal salinity within the Martin Slough mainstream and tidal wetlands.

4.2.1 HEC-RAS Model Geometries

Design Condition Geometry

Cross Sections

Cross sections were used, where feasible, to reflect proposed channel and overbank topography within the project area. The cross sectional geometry and spacing is used by HEC-RAS to route flows 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 required larger spacing (Figure 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 lower areas on 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 flowing into Pond G was modeled using surveyed channel cross sections. To maintain model stability, streams flowing into Ponds C and Pond D were modeled as lateral inflows and not as reaches flowing into the ponds.

A one-foot wide, one-foot deep pilot channel was incorporated into the model between Stations 65+00 and 75+00, where the channel slope steepens to meet existing grade at the upstream end of the project area. The purpose of the pilot channel was to maintain model stability when this portion of the channel is not tidally inundated.

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

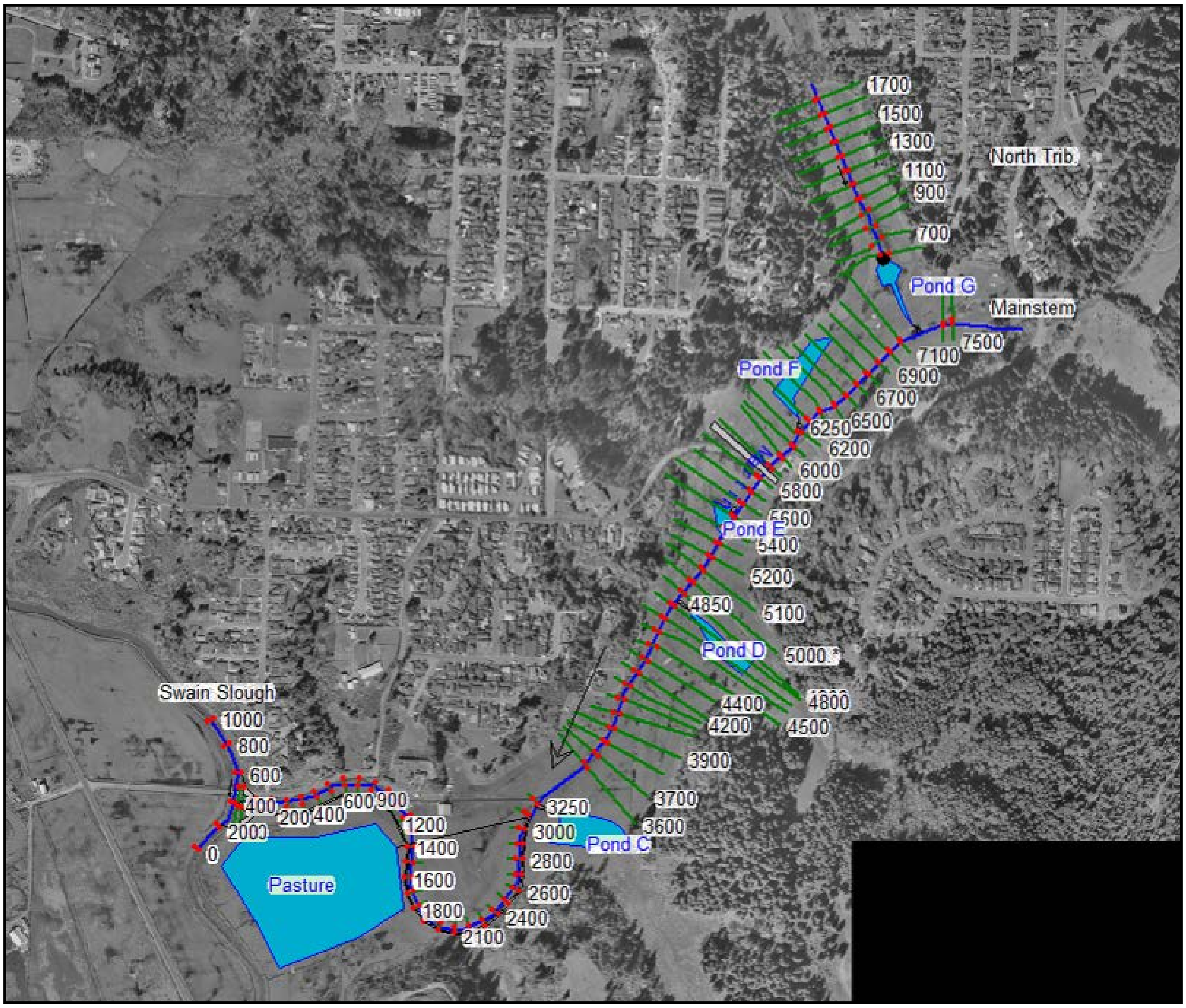


Figure 4-1. Schematic of HEC-RAS cross section locations (green) and storage areas (blue).

Storage Areas and Lateral Structures

Ponds C through G were modeled as Storage Areas connected to Martin Slough using Lateral Structures (Figure 4-1). Storage-elevation relationships were computed at 0.5-foot increments from the digital terrain model of design conditions. The connecting channels between the ponds and the main channel were modeled as 20-foot wide and 20-foot long broad-crested weirs. Storage volumes for the channels connecting the ponds to Martin Slough were incorporated into the pond storage.

On the Senestraro property downstream of the golf course, the large channel meander and low adjacent pasture areas necessitated modeling the overbank areas as a storage area with a lateral structure extending from Stations 0+00 to 31+00. Therefore, the cross sections on the Senestraro property only include the tidal channel and new marsh plain, with the lateral structure allowing overbank flow to enter the storage area at an elevation of 7 feet, the typical elevation of the pasture.

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 (1976) were used for pier dimensions and locations. The pier depths are not specified in the as-built drawings and are unknown. The internal cross sections of the bridge were modified to reflect surveyed conditions under the bridge, including the sloping abutments and golf cart path.

The proposed bridge crossings on the Senestraro Property and on the golf course were not included in the hydraulic modeling. It was assumed that these bridges will 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.

Replacement Tide Gates

The replacement tide gates provide bi-directional hydraulic connectivity between Martin Slough and Swain Slough. These new gates were included in the HEC-RAS modeling as 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 MTR gate and auxiliary door closing elevation, the gate fully closes in one time-step.

Existing Condition Geometry

Existing condition cross sections were located at the same locations as design condition cross sections. Because the existing Ponds E and F are very small, they were not modeled as storage areas. The existing fairway drive bridge was included in the model. The existing tide gate was modeled as a Lateral Structure containing three 48-inch diameter corrugated metal pipes with flaps that allow flow to leave Martin Slough, but prevent tidal inflow. The dimensions and elevations of the existing tide gates were based on field-surveyed information.

Salinity Model Geometry

The water quality module of the HEC-RAS (ACOE 2010a) was used to model design-condition salinity in Martin Slough mainstem and Ponds C, D, E, F and G during low-flow conditions. The HEC-RAS water quality module is not compatible with lateral structures or storage areas, which were used extensively in the design-condition HEC-RAS model. Therefore, it was necessary to adapt the design condition HEC-RAS model geometry into a more simplified geometry for the salinity model.

The simplified geometry included the same main channel cross sections as the model used for the project design. Cross sections spaced at 40-foot intervals replaced storage areas for Ponds C, D, E, F, and G. The cross sections extended the full width of the pond up to the top of bank. Each pond outfall was simulated with two cross sections with invert elevations set to the design outfall elevation.

The lateral structures representing the tidegate at the downstream end of the model were replaced with a stage/flow hydrograph at the location of the tidegate. The stage/flow hydrograph was obtained from the results of the design-condition HEC-RAS model at the cross section just upstream of the tidegate. This hydrograph reflects tidal stage and flow into and out of the project area from the new tidegates.

To maintain channel stability, pilot channels were necessary at each pond weir location. Additionally, to maintain channel stability, an increased theta value (Finite difference solution factor) of 0.75 was necessary rather than the 0.6 that was used for design conditions (ACOE, 2010b). Model solutions for low-flow conditions were compared to those from the design-condition HEC-RAS model, and differences were found to be negligible.

4.2.2 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 (Figure 4-2).

The recorded tidal elevations in Swain Slough between February 13 and 19, 2003 (Section 3.3.4) were used for the corresponding tidal boundary condition.

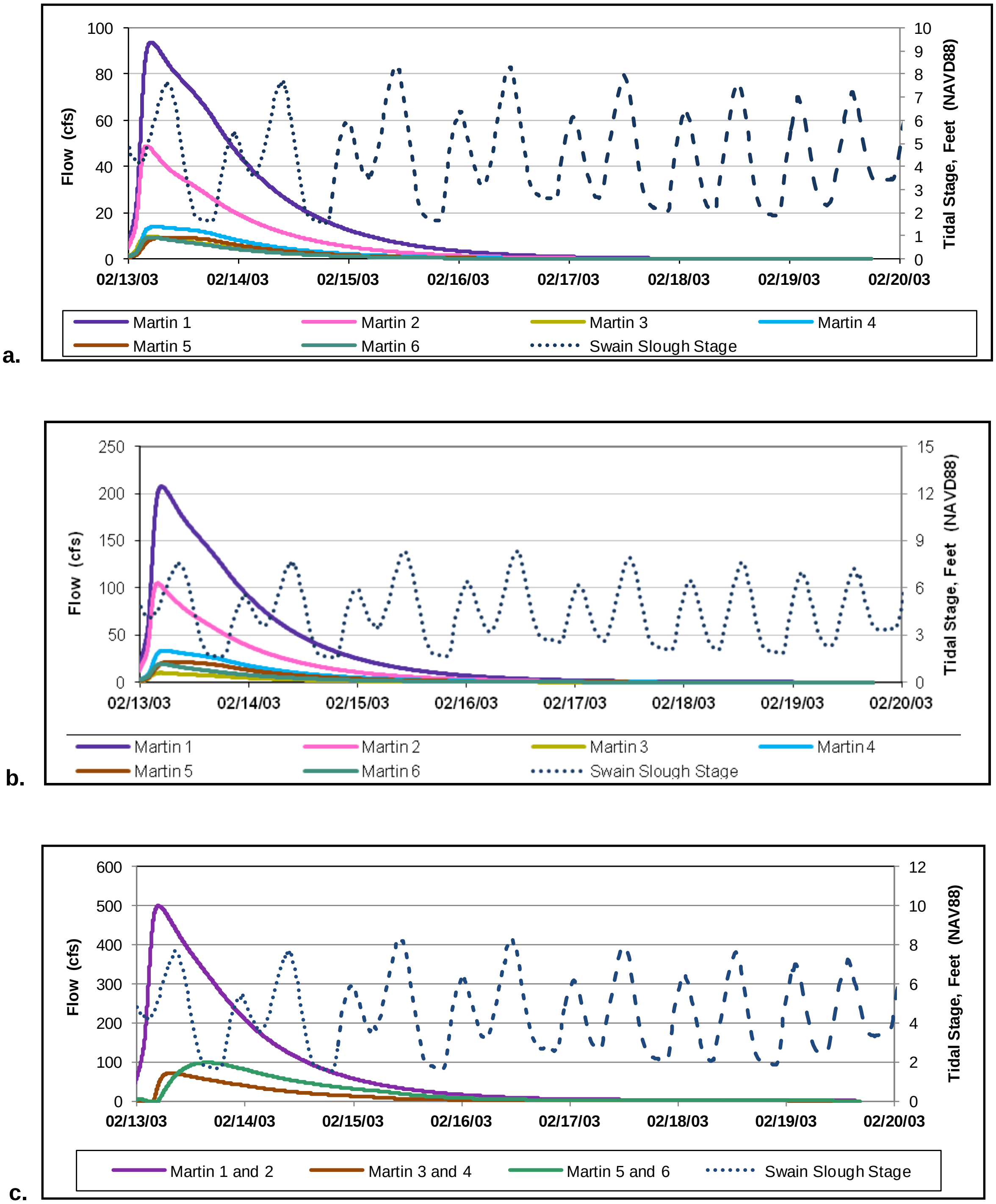


Figure 4-2. HEC-HMS predicted runoff hydrographs for sub basins draining into the project area resulting from a 24-hour rainfall events with return periods of (a) 2 -years, (b) 10 years, and (c) 100 years. The dotted line shows the measured tidal stage in Swain Slough.

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 tidegates 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 3.3.4) was used as the corresponding tidal conditions for the Annual Hydrograph.

Fish Passage Flows and Tides

To evaluate fish passage conditions at the tide gates, each of the four fish passage design flows were modeled individually. The fish passage design flow was kept constant over the entire year and inputted as a single flow at the upstream mainstream cross section. The Annual Tide was used as the corresponding Swain Slough tidal boundary condition.

4.2.3 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 run for the months of February through July and between November through early January. The results of this model reflect salinity conditions in Martin Slough during the rainy season and as the rains end and baseflow recedes in the early summer.

A separate salinity model was prepared for summer dry conditions between July and early September. This model was prepared using a constant baseflow totaling 1 cfs, with flows proportional to the contributing watershed size. Modeling instabilities due to the low flows limited modeling salinities past early September.

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, even though it becomes brackish during runoff events. 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.

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 to Martin Slough.

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

Dispersion coefficients were computed in HEC-RAS using Fischer et al. (1979), with an allowable range limitation of 0.1 to 500 feet^2/second specified. Computed values in the Mainstem ranged from a minimum of 0.1 foot^2/second to 517 feet^2/second, with an average of 4.8 feet^2/second and a median value of 2.5 feet^2/second. Computed dispersion values in the ponds were much smaller, ranging from a minimum of 0.1 foot^2/second to 367 feet^2/second, with an average of 1.9 feet^2/second and a median value of 0.7 feet^2/second. Computed dispersion coefficient values increased during time steps with increased freshwater inflow and with proximity to the tidal effects of Swain Slough. Though some of the higher 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 remainder of the modeling. Salinity results where anomalously high dispersion coefficients computed were not used.

A constant water temperature of 15^{o}C was used in the salinity modeling.

4.2.4 Modeled Scenarios

HEC-RAS model simulations were performed for a variety of boundary conditions dependent on the modeling purpose. Table 4-1 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. Due to file size limitations in HEC-RAS, results for the year-long modeling events were reported at 20-minute intervals. Modeling for short-term stormflow events was reported at 10-minute intervals.

Table 4-1. Scenarios for which HEC-RAS modeling was performed. The results of the modeling were used to design various project elements as noted.

Scenario	Purpose
Scenarios 1-3: Existing Condition Stormflow Geometry: Existing Conditions 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 Project 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 elevations
Scenario 7: **Annual Variation** Geometry: Proposed Project Freshwater Inflow: Gaged Annual Inflow Hydrographs Swain Slough Tidal: Annual Tide Duration of Simulation: 331 days	• Design of MTR tide gates • 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
Scenarios 8-11: Fish Passage Conditions Geometry: Proposed Project Freshwater Inflow: Constant Fish Passage Design Flows Swain Slough Tidal: Annual Tide Duration of Simulations: 365 days	• Evaluate upstream and downstream fish passage conditions through the replacement tide gates at low and high passage flows for adult anadromous and juvenile salmonids
Scenario 12: Low Flow Salinity Transport Geometry: Simplified Proposed Project Freshwater Inflow: Gaged Annual Inflow Hydrographs (Spring, Fall and Winter) and Constant Baseflow (Summer) Swain Slough Tidal: Results of Design-Condition Modeling at Tide Gate Duration of Simulations: Approximately 10 months	• Evaluate extent and concentration of salinity into project area during persistent summer baseflow • Predict salinity for aquatic habitat and vegetation communities

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 Design: Tidegate MTR OPERATION and Marsh plain Design

The sizing and operation of the MTR gate and auxiliary door was developed using results from the HEC-RAS Scenario 7 (Section 4.2.4), which modeled one year of Swain Slough tides and gaged streamflow. The results were used to verify that the design tidal prism is conveyed into and out of the project area, that the maximum design tidal elevation within Martin Slough does not exceed an elevation of 6 feet. The results also allowed for fine-tuning design elevations for the proposed tidal marsh plains and pond outfalls.

5.1.1 Tidegate Operation and Muted tide Characteristics

Figure 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.

Figure 5-1 indicates that the proposed 6-foot by 6-foot 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 MTR gate closes but inflow continues through the auxiliary door. The auxiliary door was sized to restrict inflow, slowing the rate that tidal waters 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 5.1.2). During a spring high tide, the auxiliary door regulates the rate of the incoming tide such that there is little to no time that the tide elevation within Martin Slough is at a constant elevation, which is undesirable for marsh plain vegetation.

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 (assumption to be verified), the maximum elevation for saltwater was targeted at approximate 6 feet in elevation.

If the auxiliary door is left open during large spring tides, the tide 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 (Figure 5-1). Following construction, the elevations at which both the MTR gate and auxiliary door shut can be fine-tuned and the float-switch adjusted as needed.

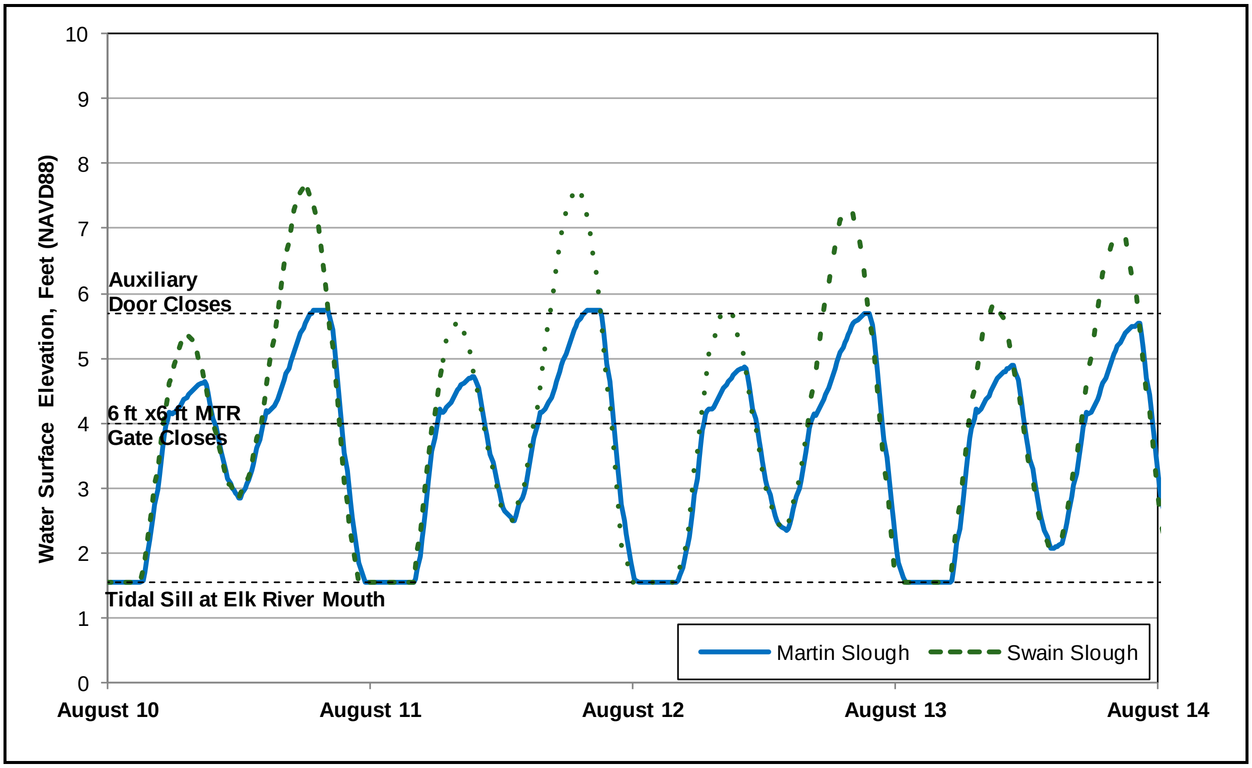


Figure 5-1. Simulated muted tide in Martin Slough during a period of low streamflows and spring tides in Swain Slough. During an incoming tide the 6-ft by 6-ft MTR gate closes when Martin Slough rises to elevation 4.0 ft. The auxiliary door remains open until Martin Slough reaches elevation 5.7 feet.

5.1.2 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.

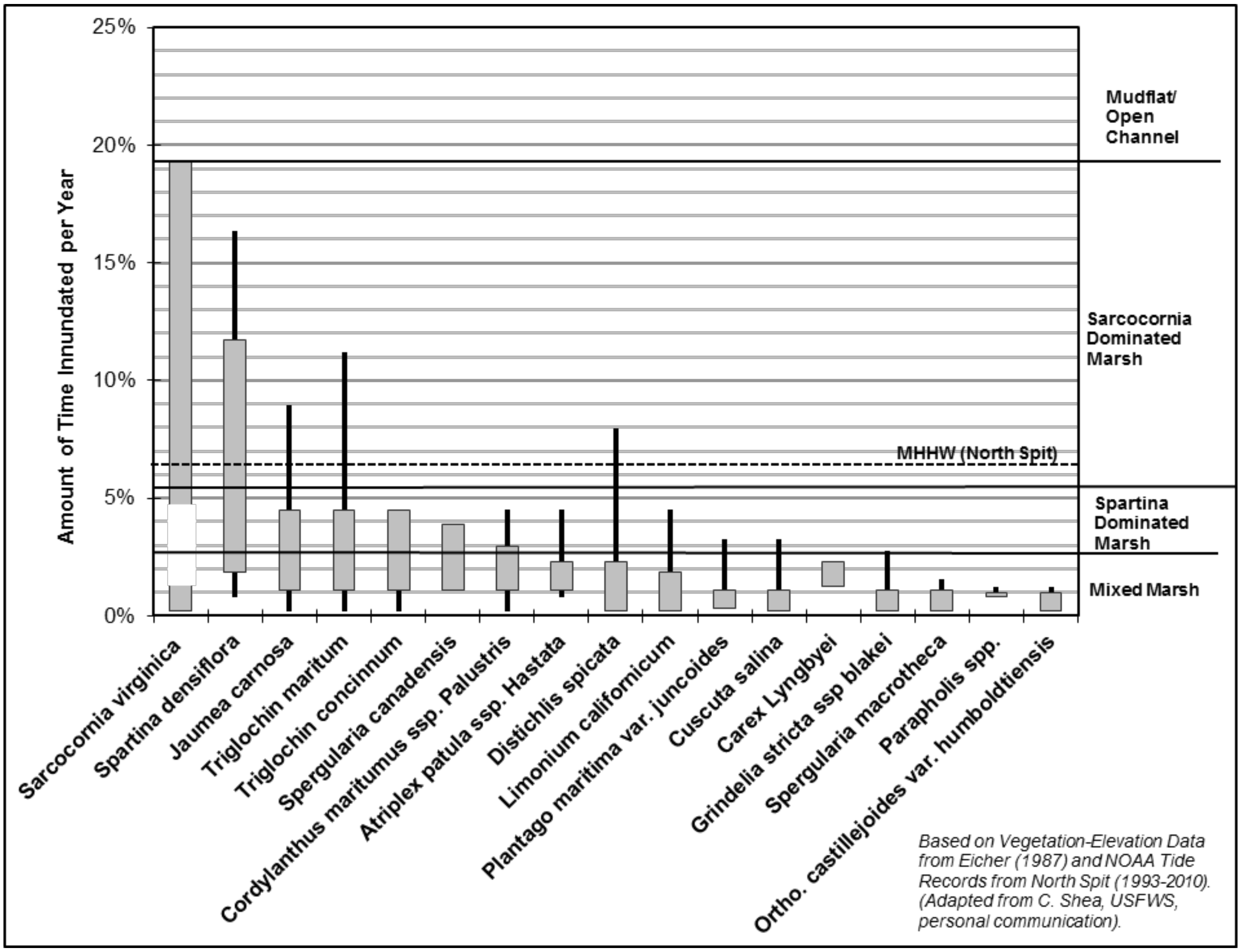
Salt Marsh Plant Community Distribution by Elevation

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.*

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 (Figure 5-2).

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.

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. Figure 5-3 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.



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

Figure 5-2. 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 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. Figure 5-3 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.

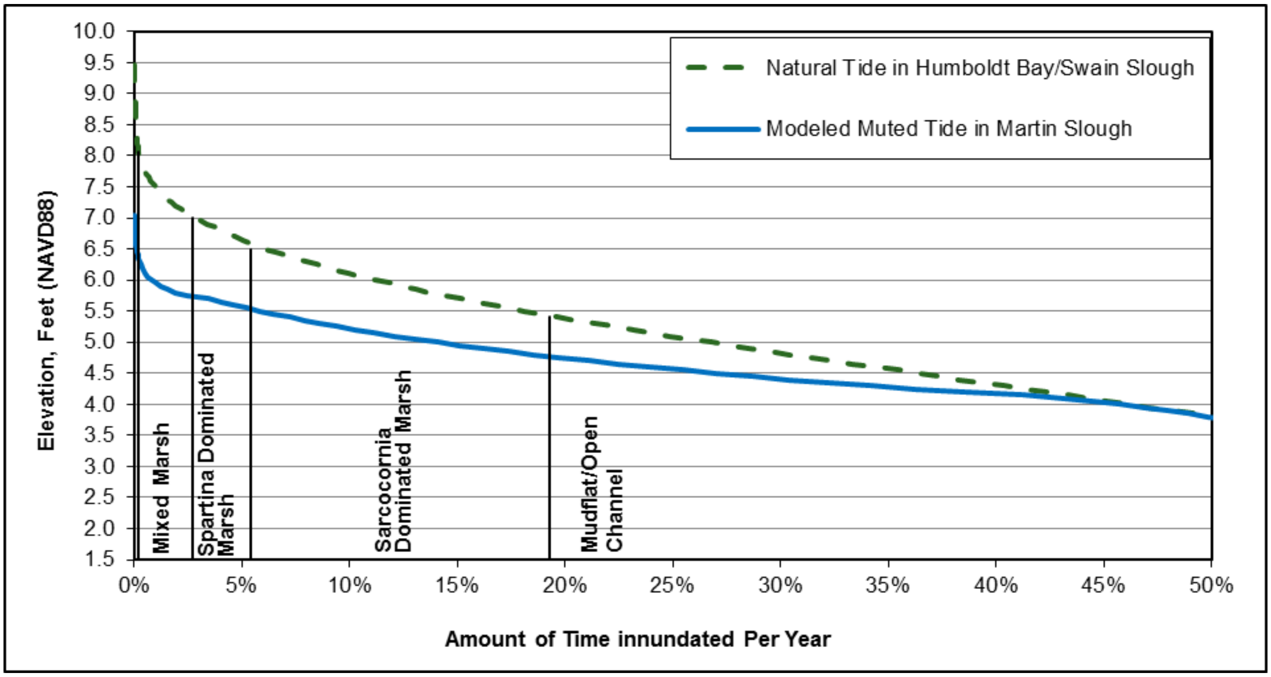


Figure 5-3. Inundation frequency and elevation of specific salt marsh plant species and marsh types identified by Eicher (1987) that occur under natural tidal conditions of Humboldt Bay and can be expected to occur under the muted tidal conditions of Martin Slough.

Tidal Marsh Plains A and B

Approximately 3,000 feet of tidal marsh plain will be constructed along alternating sides of the tidal channel reach on the Senestraro property. The marsh plain will have a width of 50 feet with gentle side slopes of 3H:1V transitioning to existing ground. To minimize the span of the two proposed bridge crossings in this reach, the width of the marsh plain will gently taper to the channel width at the crossings. Similarly, to facilitate flow into the new tide gates, the marsh plain width will taper to the channel width upstream of the tide gate structure.

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. This range in elevations is expected to support a range of salt marsh plant species. As shown in Figure 5-3, 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 a range of marsh communities including *Sarcocornia* dominated marsh and mixed marsh communities. It is expected that mixed marsh will extend a portion of the way up the 3H:1V side slopes, which will be partially inundated by higher tides. Brackish and freshwater vegetation is expected to grow above the salt marsh elevations.

Tidal Marsh Complex C (Pond C)

Approximately 2.4 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 transitioning at

3H:1V to meet existing ground. Several “fingers” of higher ground will project into the marsh plain, where freshwater species will grow.

The marsh plain will be constructed with 1% to 2% slopes to allow drainage towards the channel and minimize salt panne formation (Zedler, 1984; Eicher, 1987). The range of marsh plain and upland elevations was designed to support a full suite of low to high salt marsh vegetation with freshwater vegetation on the higher elevations (Figure 5-3).

Tidal Pond Outfall Design

Ponds C, D, E, and F will be connected to Martin Slough through an elevated pond inlet/outlet channel, referred to as the pond outfall. The pond outfalls were designed as broad crested earthen weirs that will be at a higher elevation than the adjacent pond and channel. Pond G is a flow-through wetland on the North Tributary, which carries a substantial fine sediment load. No outfall weir will be constructed in Pond G to maximize tidal exchange and flow-through, which are expected to reduce sedimentation potential. Marsh Plains A and B will be constructed adjacent to the stream channel, and are expected to be inundated along their full length with an incoming tide and do not have weir structures.

Pond outfall elevations in Ponds D, E, and F were established with the objective of limiting saltwater intrusion while keeping the pond hydraulically connected to the channel under most tidal conditions. Pond C, at the downstream end of Martin Slough is expected to receive saline waters throughout the year; thus excluding saline water at the outfall was not a design objective.

Pond outfall elevations (Table 5-1) were also established to ensure the ponds are flooded twice daily by the tidal cycle. This 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 elevation 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 large volume of the water in the ponds will be flushed twice daily by tidal action, decreasing the amount of time for settlement of smaller particles.

Each of the pond outfalls will be a minimum of 20 feet wide. HEC-RAS modeling indicates peak velocities across the weirs do not exceed 0.5 ft/s. Therefore, grade controls on the pond outfalls are not proposed, but they should be composed of relatively resistant material, such as clays. Large wood may be incorporated into them if native soils are erodible.

Table 5-1.   Summary of pond outfall elevations for the tidal wetlands within the Martin Slough project area.

Pond	Outfall Weir Elevation (NAVD 88)
A	NA
B	NA
C	1.5 feet
D	2.0 feet
E	2.0 feet (Upstream)
	3.0 feet (Downstream)
F	3.0 feet
G	2.8 feet

5.2 Flood Elevations and Durations

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 bridges.

5.2.1 Flood Elevations

Figure 5-4 presents existing (E) and design condition (N) peak 2, 10 and 100-year water surface elevations plotted along the (N) channel alignment. The dotted line in the (E) water surface profile between stations 11+00 and 31+00 connects upstream and downstream locations that the (N) channel diverges from the existing to reoccupy its historical meander.

The channel profile for design conditions is substantially lower than the existing conditions, which results in lower design condition 2, 10 and 100-year water surface elevations. The drop in design condition water surface elevations is a combination of the lower design channel bottom, larger channel cross sectional area and increased outflow capacity of the new tide gates. The drop in flood water surface elevations between (E) and (N) conditions reduces the slight backwater 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.

5.2.2 100-Year Flood Velocities

Peak water velocities during a 100-year event are expected to occur when flows are receding and returning from the floodplain back into the main channel. HEC-RAS modeling indicated that peak channel velocities of approximately 2 ft/s are expected to occur in the Martin Slough Main channel, except at the upstream limit of the project (Figure 5-5). At the upstream end of the project, where the new tidal slough channel transitions to the existing channel, in-channel water velocities of up to 5 ft/s can be expected. Peak floodplain velocities of less than 0.5 feet per second are expected to occur throughout the project area.

Appendix C provides a summary of HEC-RAS modeling results of 100-year peak velocities for the channel and overbank areas.

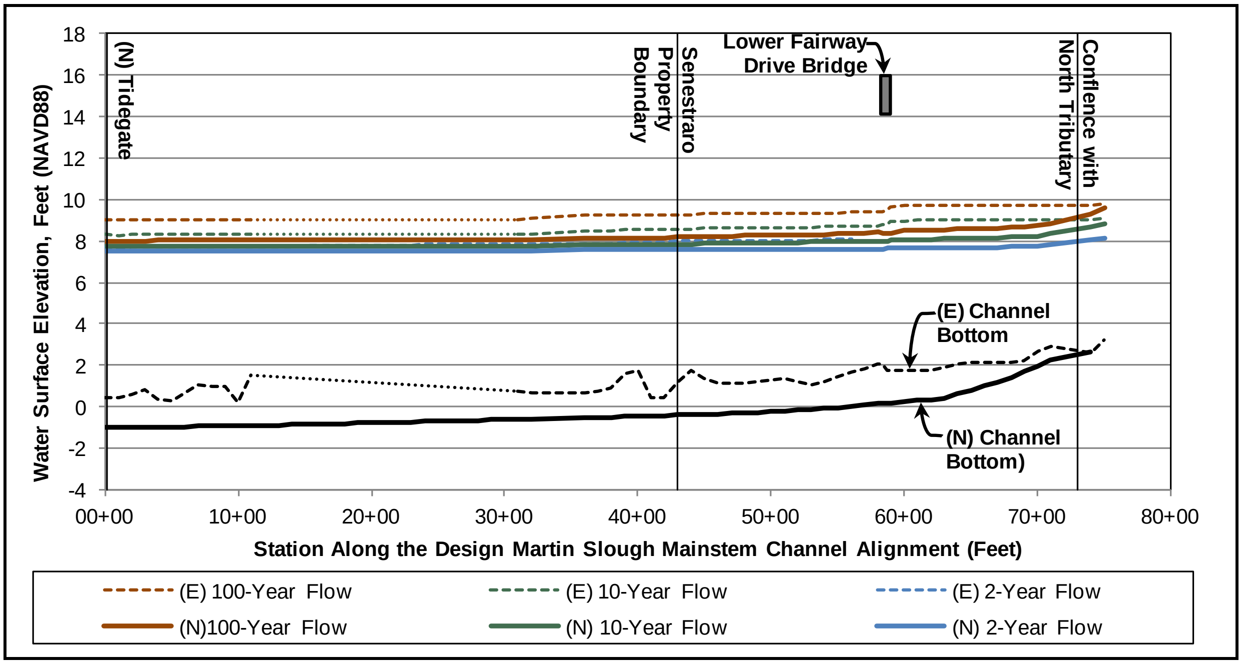


Figure 5-4. The 2, 10, and 100-year water surface profiles and channel profiles in Martin Slough for existing (E) and new (N) design conditions. Stationing is along the (N) channel alignment, which diverges from the existing between 11+00 and 31+00 (shown with dashed lines).

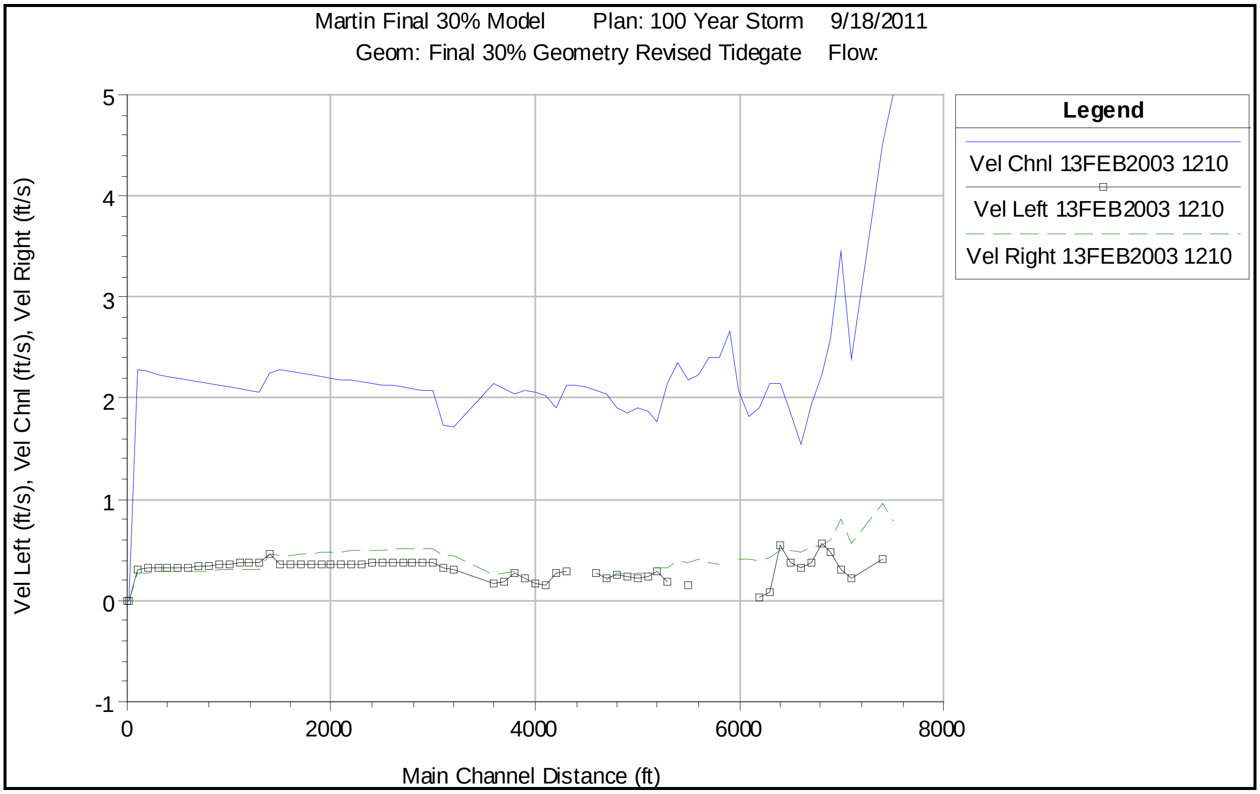


Figure 5-5. Peak channel and floodplain water velocities expected under Design Conditions.

5.2.3 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 existing tide gates have very limited outflow capacity that increases the amount of time necessary for floodwaters to drain out of Martin Slough, resulting in longer 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.

Figure 5-6 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.

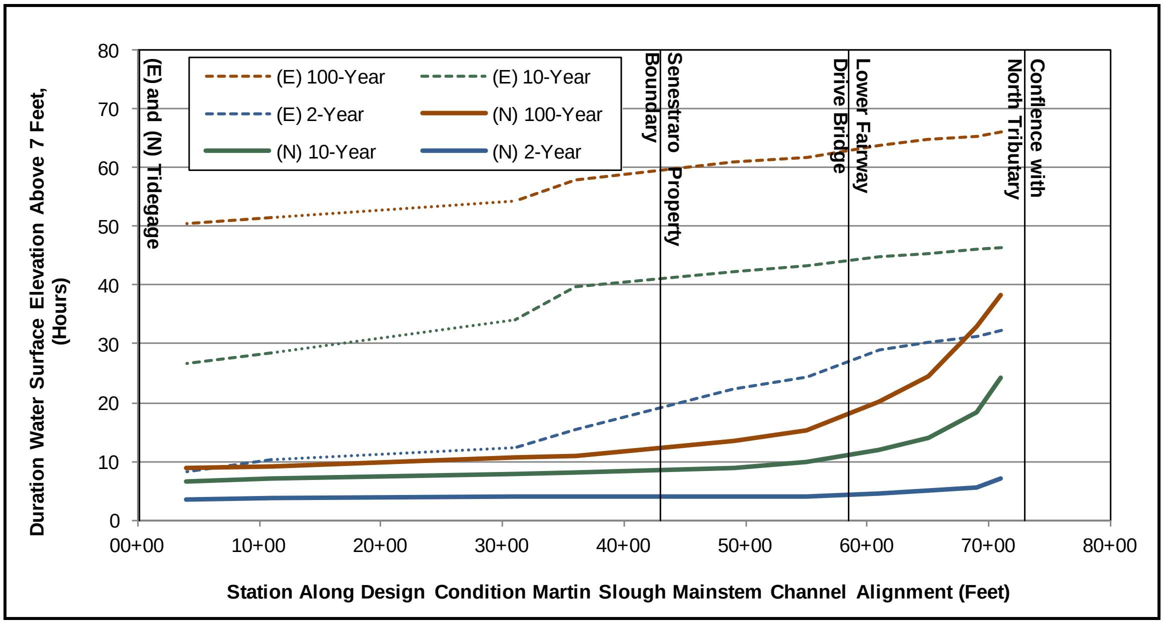


Figure 5-6. Duration that flood flow are above elevation 7.0 feet (NAVD88) for existing (E) and design conditions (N)along the mainstem Martin Slough. Stationing is along the (N) channel alignment, which diverges from the existing between 11+00 and 31+00 (shown with dashed lines).

5.3 Fish Passage through the Replacement Tide Gates

Fish passage was assessed using results from the HEC-RAS Scenarios 8 through 11 (Section 4.2.4), which modeled one year of Swain Slough tides and a constant freshwater inflow set equal to the analyzed fish passage design flow. The model results allow an assessment of passage conditions at each design flow across a range of tidal conditions.

There are no specific fish passage design requirements for tide gates. However, the CDFG (2002) and NOAA Fisheries (2001) fish passage guidelines prescribe minimum water depths and maximum average water velocities for passage of salmonids at road-stream crossings (Table 5-2), which can be applied to tide gates. These criteria should be satisfied between the low and high fish passage design flows. CDFG and NMFS guidelines recognize the criteria cannot always be satisfied, and suggest the criteria be applied as a guide for design.

Upstream and downstream fish passage was assessed through the tide gates for juvenile and adult salmon and steelhead at the low and high fish passage design flows (Section 3.3.2). 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 5-2. CDFG and NMFS Fisheries fish passage depth and velocity criteria applied to the passage analysis of the Martin Slough replacement tidegates.

Lifestage	Minimum Water Depth	Maximum Water Velocity
Adult Salmon and Steelhead Trout	1.0 feet	6 feet per second
Juvenile Salmon and Steelhead Trout	0.5 feet	2 feet per second [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 5-3 presents the results of upstream and downstream fish passage assessment for the replacement tide gate. A detailed summary of the fish passage analysis is presented in Appendix D.

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.

Table 5-3. Computed upstream and downstream fish passage at the Martin Slough replacement tide gates for adult and juvenile salmonids.

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%

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.

5.4 Existing and Design Condition 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 4.2.4). Peak shear stresses occur on outgoing flows. Transport competence during summer baseflow was also assessed 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 water surface slope, 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.

Critical dimensionless shear stress is a function of particle size, shape, arrangement, grain, Reynolds number and fluid properties. 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 5-4).

Table 5-4. 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 Mainstem

In the Martin Slough Mainstem upstream of the Senestraro Property, peak channel shear stresses during a 2-year 24-hour flow event are increased substantially from existing conditions (Figure 5-7). This is a result of the increased channel capacity and improved flood conveyance throughout the project area. Although peak channel shear stresses are lower under design conditions than existing within reaches on the Senestraro Property, the 2-year peak shear stresses continue to have the competence to transport medium sands through the project area to the tidegate. 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.

During summer baseflow conditions, the majority of the Martin Slough mainstem also has the competence to transport particle sizes up to medium sands, except through an approximately 700-foot long reach between the outfalls of Pond E and Pond F. As slightly higher flows, when sands are likely in transport, this reach gains the competency transport medium sand.

Downstream of station 35+00, design condition peak shear stresses are expected to be lower than existing conditions, but still have the competence to transport medium sand. The reason for the drop in shear stresses with design conditions is caused by a decrease in the water surface elevation and depth with the replacement tidegate.

5.4.2 North Fork Tributary

The flow competence of the North Fork Tributary under design conditions is substantially less than the flow competence in the mainstem (Figure 5-8). Though design condition sediment transport competence is low in the North Fork Tributary, competence is improved from existing conditions except through Pond G.

Upstream of Pond G, the largest particle size that can be transported during design condition peak stresses from a 2-year 24-hour storm are medium to coarse silts. Channel shear stresses drop substantially when flows enter the open water area of Pond G. Design condition flow competence is below the entrainment threshold for very fine silts, thus it is expected that sediments delivered to Pond G may accumulate in the pond. As project development moves forward, it may be desirable to make Pond G an off-line pond connected to the North Fork Tributary. This could reduce sedimentation risks.

For simulated storm events, the North Fork Tributary peak flow elevation occurs several hours before the Mainstream flows peak. Therefore, there is a steep drop in the water surface elevation in the downstream reaches of the North Fork Tributary that increases peak channel shear stresses in the outfall channel of Pond G. This may assist in flushing deposited sediment at the confluence.

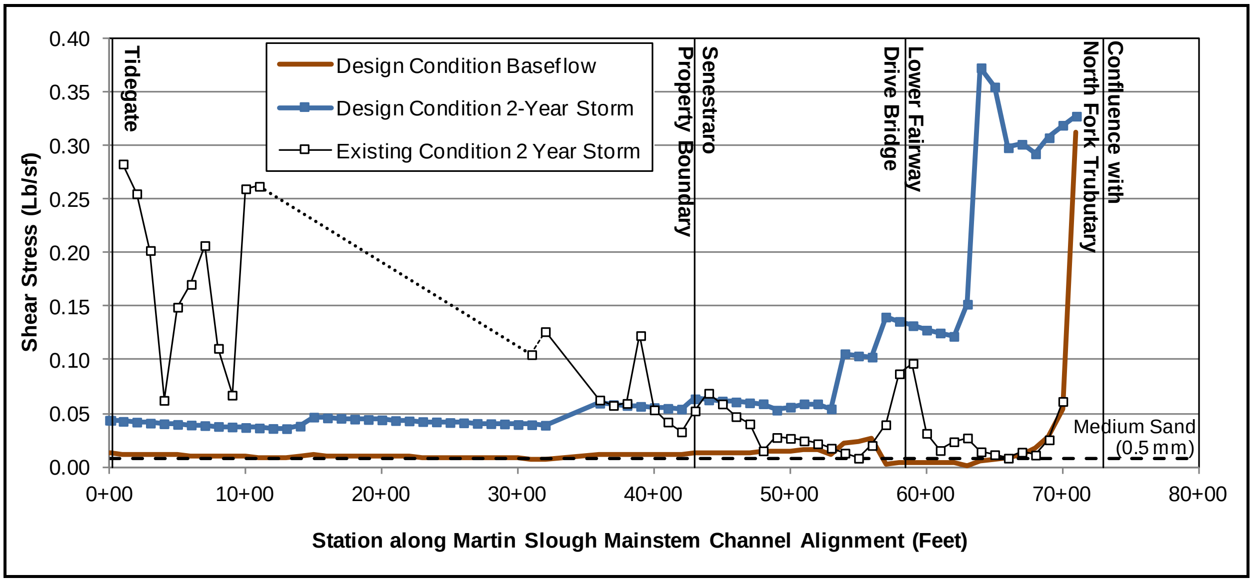


Figure 5-7. Computed peak shear stresses along the Martin Slough Mainstream for proposed and design conditions. Stationing is along the (N) channel alignment, which diverges from the existing between 11+00 and 31+00 (shown with dashed lines).

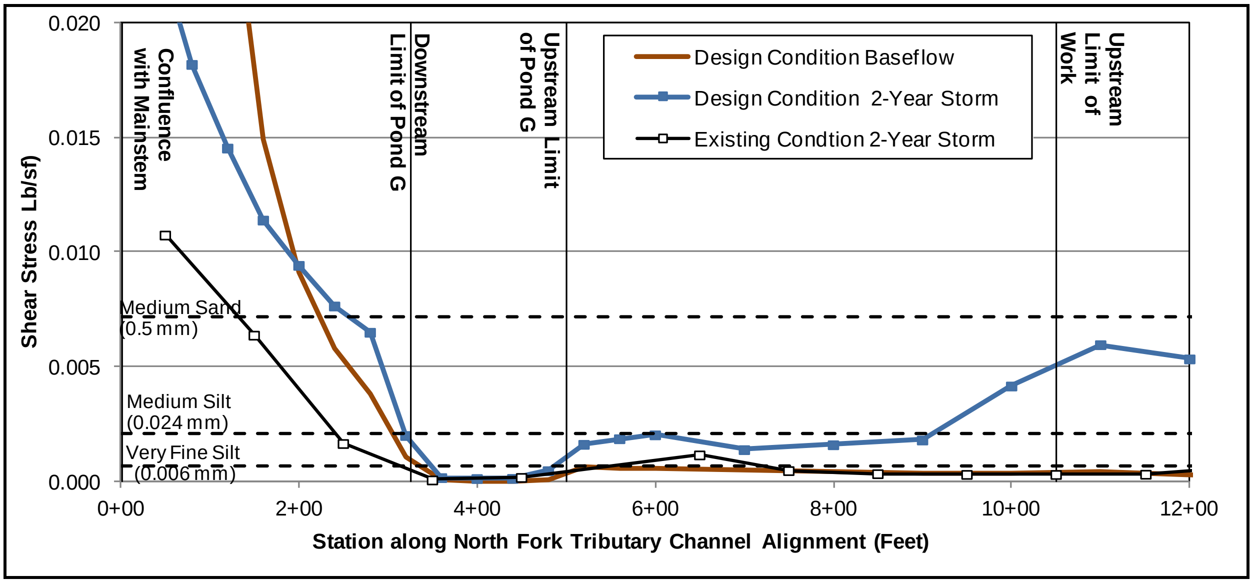


Figure 5-8. Computed peak shear stresses along North Fork Martin Slough for proposed and design conditions. Stationing is along the (N) channel alignment, which diverges from the existing between 11+00 and 31+00.

5.5 Design Condition Salinities

Salinity was modeled using HEC-RAS Scenario 12 (Section 4.2.4). 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 summer. The modeling is based on mass-balance flow mixing, and does not compute horizontal freshwater/saltwater stratification. Stratification is expected during low-flow periods, which results in a lens of freshwater on top of the water column.

Figure 5-9 through Figure 5-12 present the results of the salinity modeling along select mainstem cross sections and in 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.

5.5.1 Wet Season

Wet season salinity modeling was performed for the months of February through July and between November and January, reflecting salinity conditions in Martin Slough during the rainy season and as the rains end and baseflow recedes in the early summer.

Figure 5-9 presents a graphical representation of predicted salinity concentrations within Martin Slough for one modeling timestep reflecting a high tide between rain events in March, which represents average wet season baseflow conditions. Inflow into Martin Slough Mainstem is approximately 8 cfs. This figure represents the predicted extent and concentrations of salinity into Martin Slough between rain events. During rain events, salinity concentrations throughout the project area will decrease substantially (Figure 5-11 and Figure 5-12).

Through the modeled wet season, depth-averaged salinity concentrations greater than 8 ppt are expected to extend upstream in the Martin Slough Mainstem to the upstream entrance to Pond E (Figure 5-9). Fully saline (marine) conditions are expected to occur frequently at Marsh Plains A and B. The downstream end of Tidal Marsh Complex C (Pond C) is expected to be brackish, but the upstream end of the pond where the freshwater tributary enters is expected to maintain depth-averaged salinities less than 6 ppt. Similarly, Pond D will be slightly brackish at the downstream end, but will become fresher upstream in the pond closer to the tributary outfall where salinities will be approximately 1 ppt. Pond E will experience a range of salinities from 2 ppt at its upstream end to 23 ppt at its downstream end. Ponds, F, and G, located in the upper reaches of the Martin Slough Mainstem are expected to have salinities less than 1 ppt.

5.5.2 Dry Season

Summer dry season salinity modeling was performed for the months of July through early September. Figure 5-10 presents a graphical representation of predicted depth-averaged salinities within Martin Slough for one modeling timestep at a high tide at the end of 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 and lower amplitude tides, salinity concentrations throughout the project area will be substantially less (Figure 5-11 and Figure 5-12).

Near the end of the dry season when the stream’s baseflow is at its lowest, salinities up to 30 ppt are expected to extend from Swain Slough to the upstream of Pond E. Fully saline (marine) conditions are expected at Marsh Plains A and B. Pond C may remain relatively saline, depending on the actual amount of freshwater baseflow that enters into the pond during the summer. A similar situation may occur for Pond D. Pond E will likely stay moderately saline. Ponds F and G are expected to have a range of brackish to fresher salinities, dependent on the amount of freshwater inflow from upstream.

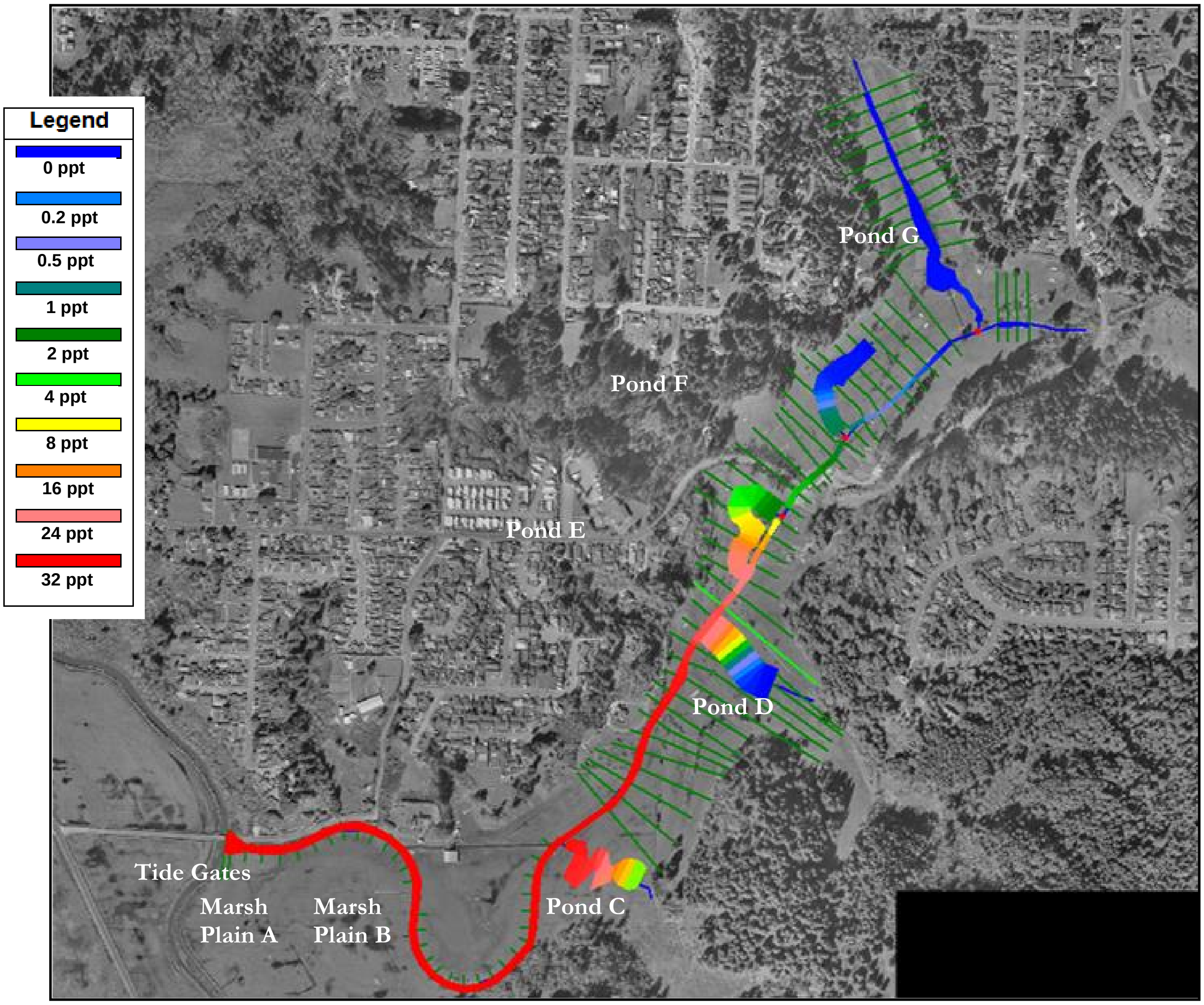


Figure 5-9. Modeled salinity in Martin Slough at a high tide between rain events in March, which represents average low-flow conditions during the wet season. Inflow into Martin Slough Mainstem is approximately 8 cfs.

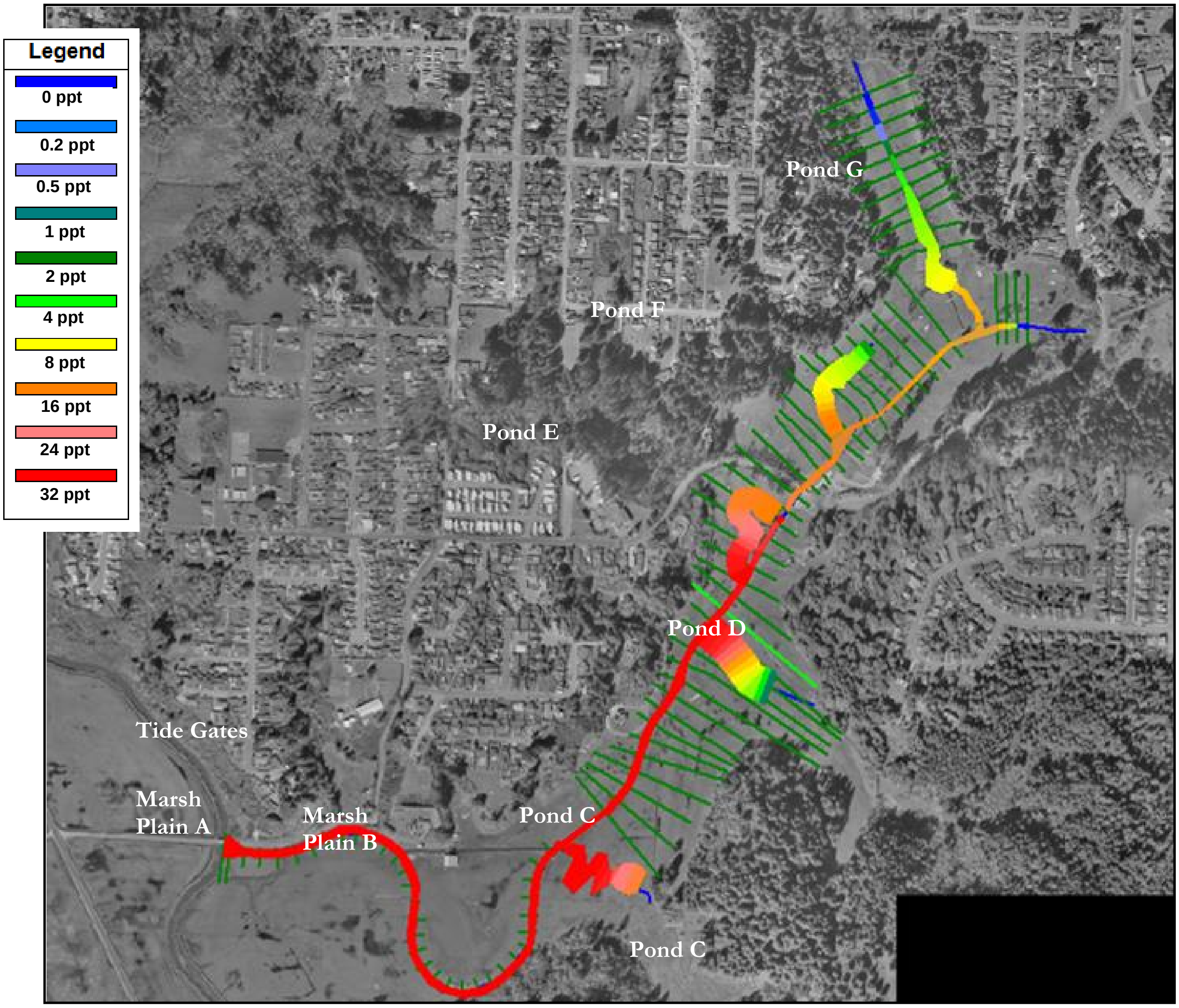


Figure 5-10. Modeled salinity in Martin Slough at a high tide at the end of September, which represents late summer dry-season conditions when baseflows have receded. Inflow into the Martin Slough is a constant 1 cfs.

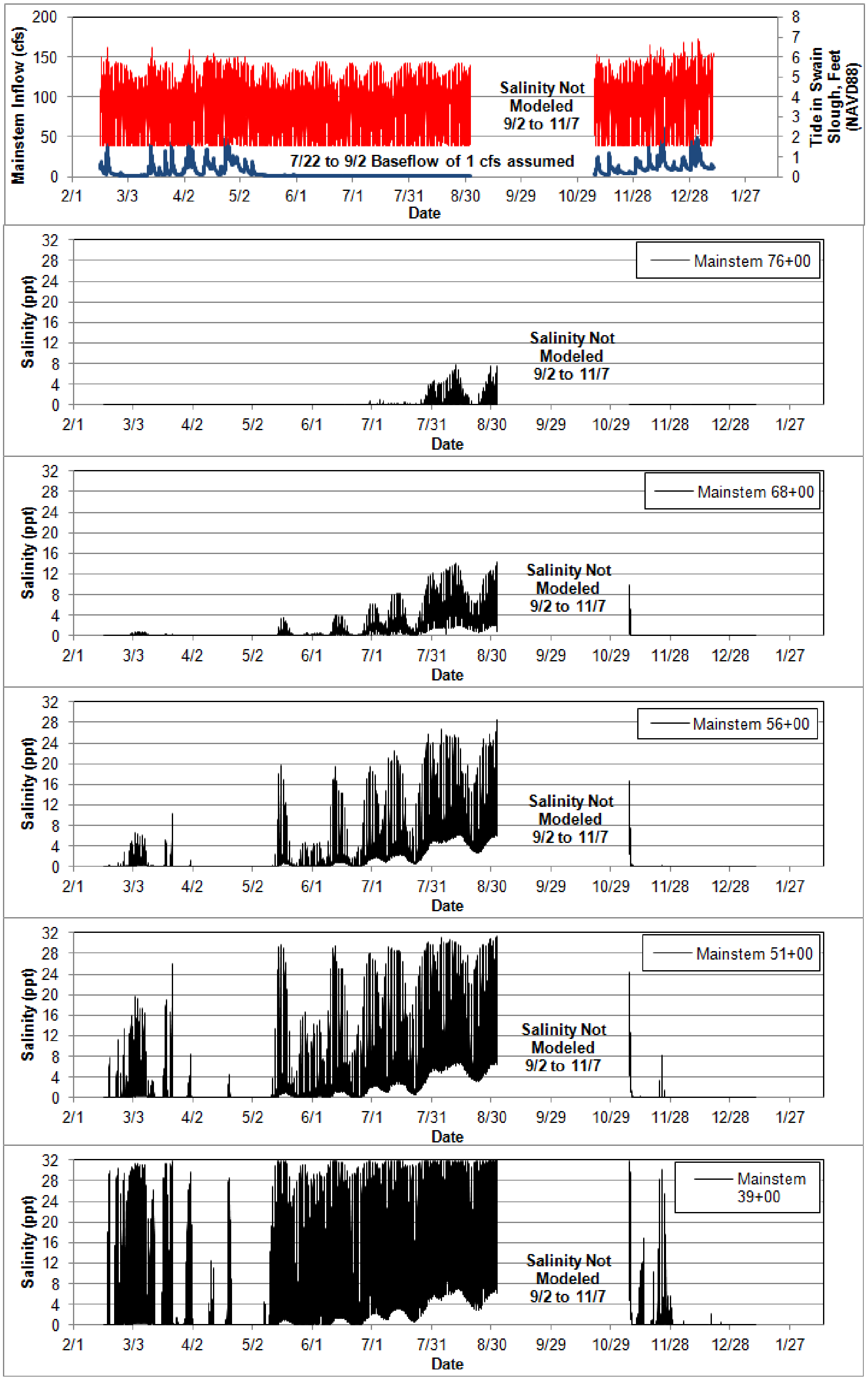


Figure 5-11. Results of salinity modeling at various locations along the Martin Slough mainstem. Tidal elevations and freshwater inflows are shown on the top figure.

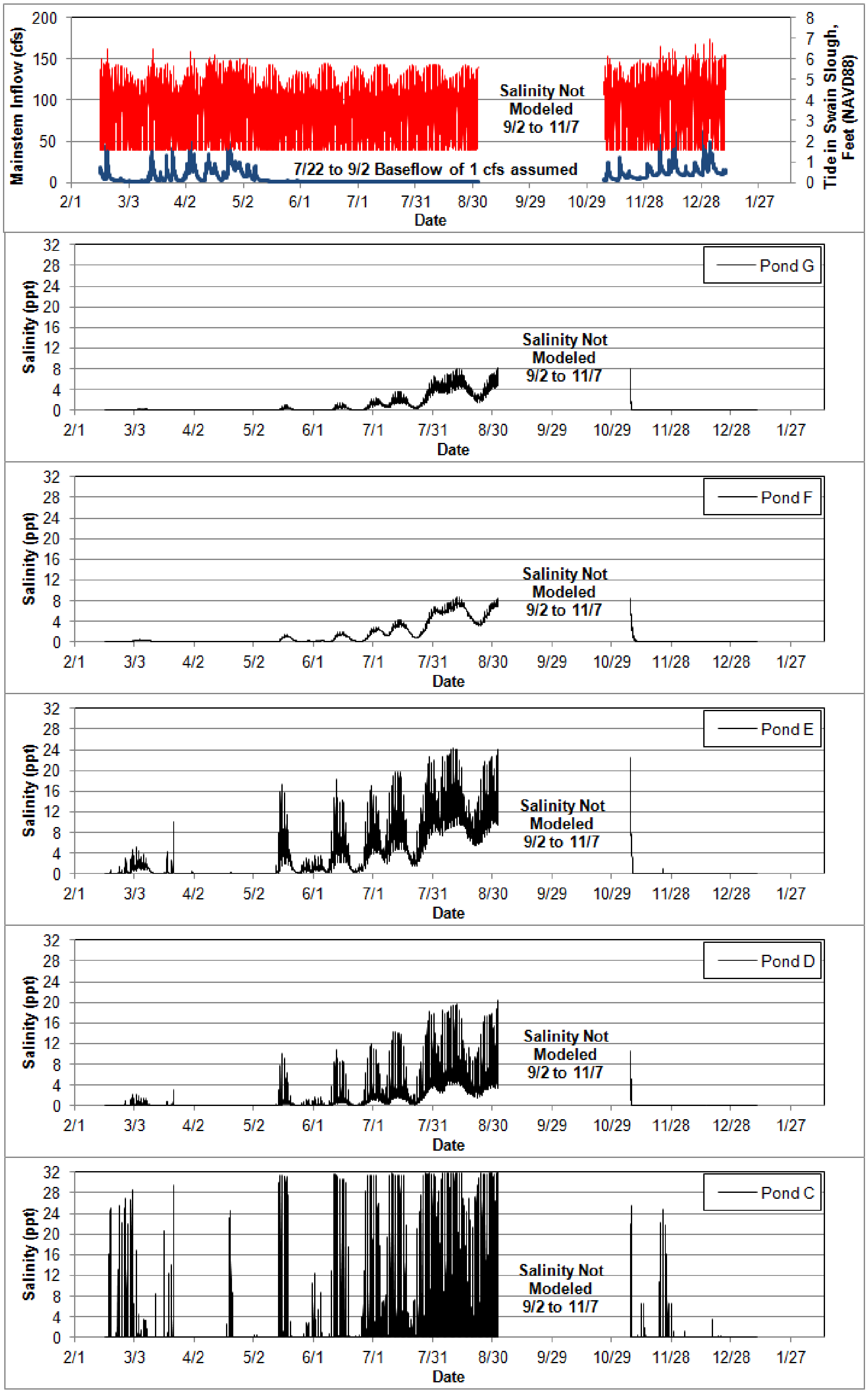


Figure 5-12. Results of salinity modeling in each of the Ponds during the summer dry season. Tidal elevations and freshwater inflows are shown on the top figure.

6.0 Construction BMPs

6.1.1 Sediment Re-use

The construction of new ponds and expanded channels will result in a large excess of excavated material. It is most beneficial to the project and environment to re-use as much sediment on site as feasible. Primary re-use areas would be to rebuild existing berms, and for beneficial use on land adjacent to the channel and ponds. The excavated material will include some topsoil, some silts and sands, and potentially 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.

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. Laboratory analyses 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 and in uplands such as those previously delineated. 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 scraper, belly- 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 each 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, scrapers and bull-dozers. Excavated material will be loaded into either belly- or end-dump trucks and hauled to the reuse areas. 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 a fleet of dump trucks operating continuously during the excavation activities. Table 3 shows the range of project construction equipment estimates for any given construction season.

Table 6.1. Estimates of Equipment Needed for Project Construction

Equipment Type	Estimated Quantity
Excavators	1-5
Scrapers	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

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 30% 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 30% Design Plans. Areas 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.

6.1.4 Temporary Haul Roads

The construction of temporary haul roads will be required to transport excavated materials from the channel corridor to City, County and State Roads depending upon the final re-use 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 30% Design Plans.

6.1.5 Construction Erosion and Sediment Control BMPs

Prior to Project construction, a Storm Water Pollution Prevention Plan (SWPPP) will be developed and approved by 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. Individual SWPPPs may be prepared for various construction components or phases (e.g., demolition of existing site structures, grading of one parcel, dredging channels, etc.). BMPs that will be implemented as part of the SWPPP will 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 flowing streams, after fish removal efforts have been completed.
- Silt fences and or silt curtains will be deployed in the vicinity of the coffer dams and at excavation of sloughs 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 Stream Diversion Sequencing

During excavation within the channel and replacement of the tidegate, 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.

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. The following construction phases and associated dewatering and diversions activities are proposed to occur in the order presented below:

Tidegate Replacement: During the tidegate replacement and instream channel excavation a combination of pumped and/or gravity diversion pipes and or ditches will be used to route flow around the active work areas. Nuisance water (i.e., turbid water seeping into excavated areas from ground water) will be pumped to adjacent fields for infiltration or into settling basins. Clean water (e.g., water from Martin Slough and contributing tributaries) will be diverted using cofferdams that will prevent clean freshwater and clean tidal water from entering the excavation. Cofferdams will be placed in the Martin Slough channel immediately upstream and downstream from the existing tidegates. The cofferdams will preclude freshwater and tidal inflow into the work zone during construction. Diversion of freshwater from the upstream cofferdam will be pumped or gravity piped through a temporary culvert that will discharge into Swain Slough. The culvert will be adequately sized to convey the Martin Slough freshwater baseflow and will be equipped with a flap gate on the outlet end. Adjustments to the temporary flap gate can be made during the construction period to allow a variation in tidal water inflow to the Martin Slough channel during the construction period and to emulate pre-project brackish water conditions in the upstream reaches. The downstream cofferdam will be placed along the edge of Swain Slough at the tidegate outfall and positioned such that tidal exchange will persist in the upper reaches of Swain Slough during construction.

Lower Martin Slough Channel Including Ponds C, and D: 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. 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.

Upper Martin Slough Channel Including Pond E, F, and G: 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 and D. Fish that cannot be corralled to areas outside of the construction limits will be captured and relocated.

6.1.7 Revegetation

The 30% 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 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.

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 (cattle, 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 Hydologic Engineering Center. Davis, California.

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

California Data Excange 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.

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.

Jeff Anderson and Associates. 2009. Tidal Wetland Geometric Relations in Humboldt Bay: Mad River Slough Pilot Study. Prepared for the United States Fish and Wildlife Service.

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.

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.

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.

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

Geotechnical Investigations

Geotechnical Report

Martin Slough Enhancement Project

Prepared for:

Redwood Community Action Agency

Consulting Engineers & Geologists, Inc.

812 W. Wabash Ave.
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May 2013
013035

Reference: 013035

Geotechnical Report

Martin Slough Enhancement Project

Prepared for:

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QA/QC: GDS gds

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Table of Contents

List of Illustrations

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.

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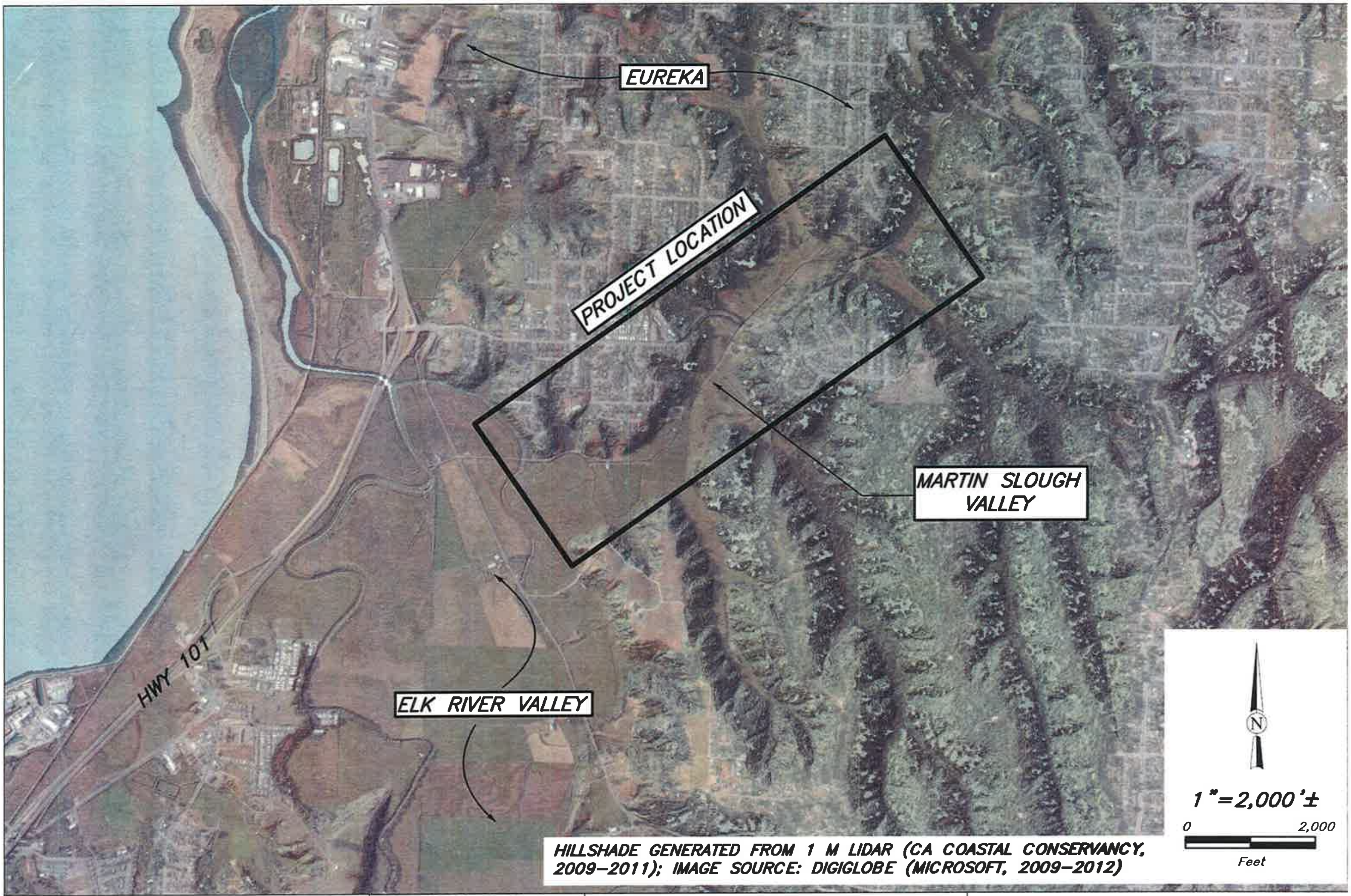


HILLSHADE GENERATED FROM 1 M LIDAR (CA COASTAL CONSERVANCY, 2009–2011); IMAGE SOURCE: DIGIGLOBE (MICROSOFT, 2009–2012)

SHN
Consulting Engineers
& Geologists, Inc.

Redwood Community Action Agency
Martin Slough Enhancement Project
Eureka, California

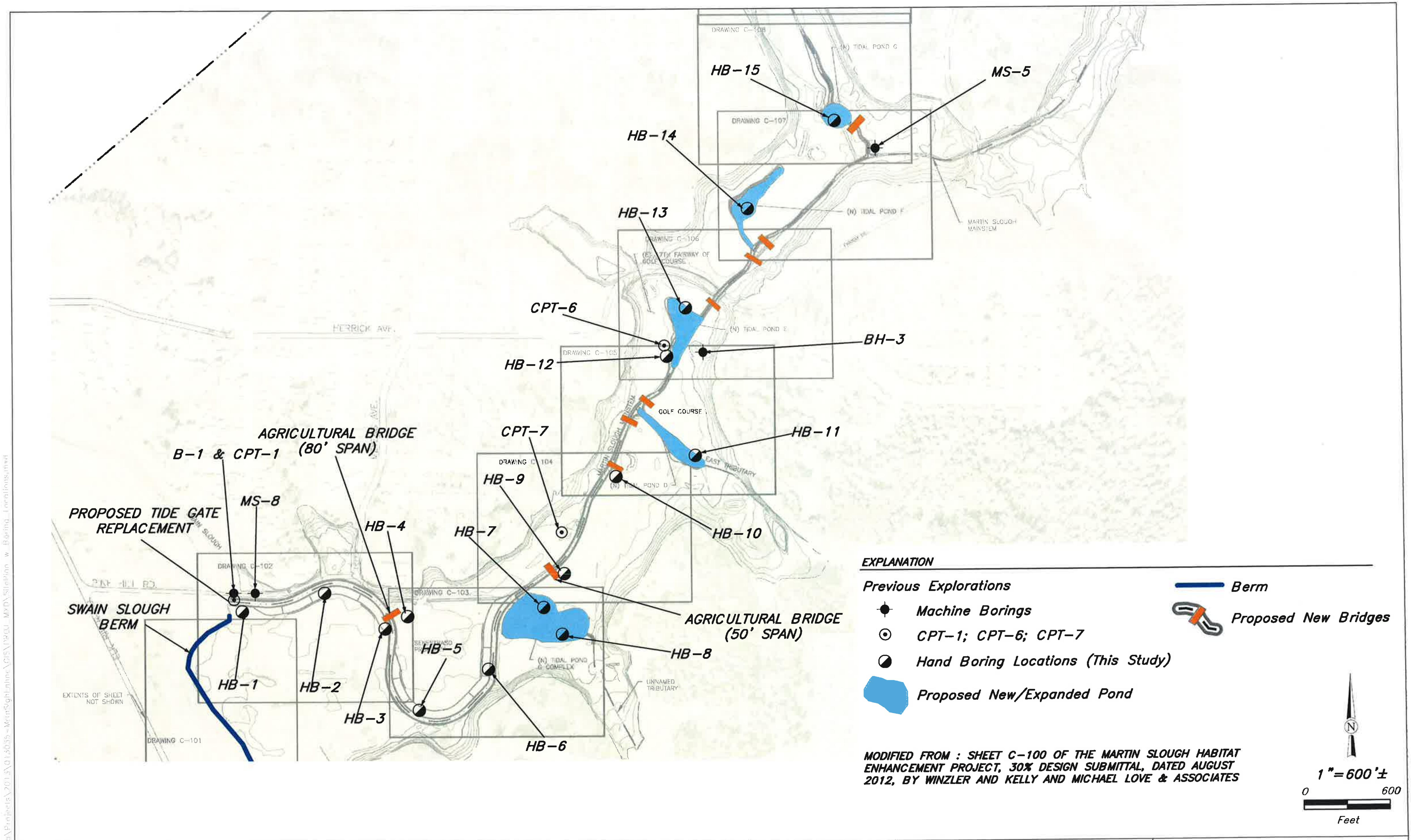
Project Location

SHN 013035

May 2013

Figure1

Figure 1



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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.

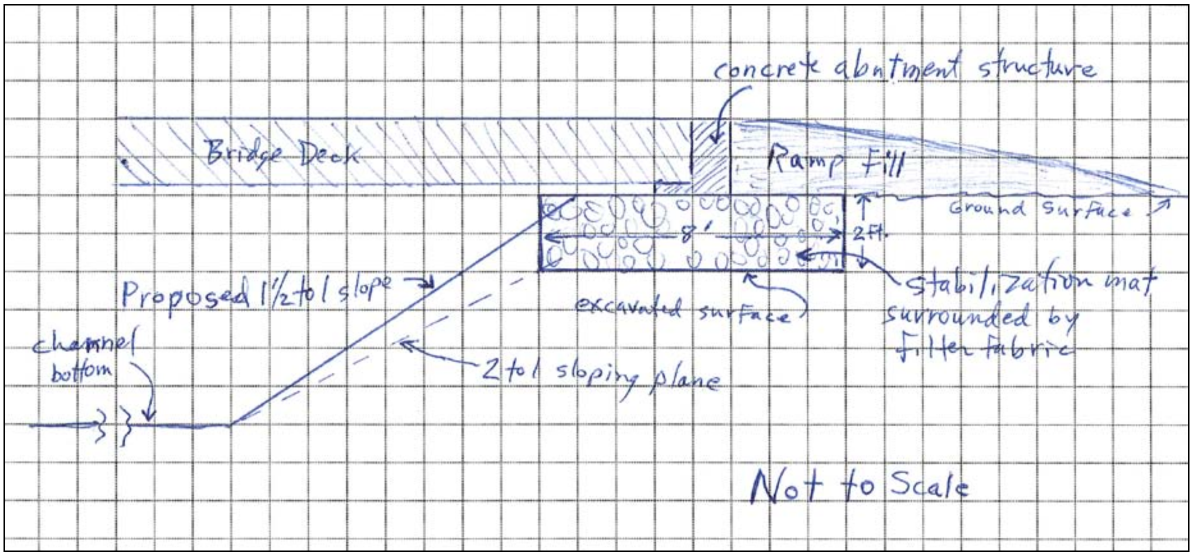


Figure 3. Schematic Drawing of Foundation System for Bridges with Spans Greater Than 30 feet
(actual dimensions will vary)

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.

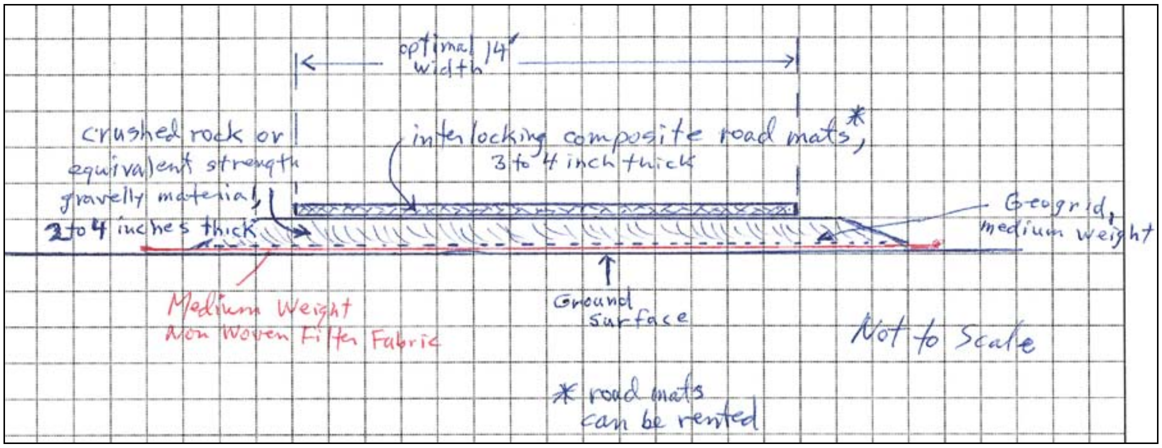


Figure 4. Schematic Drawing of a Temporary Haul Road Using a Mat System
(actual dimensions will vary)

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.

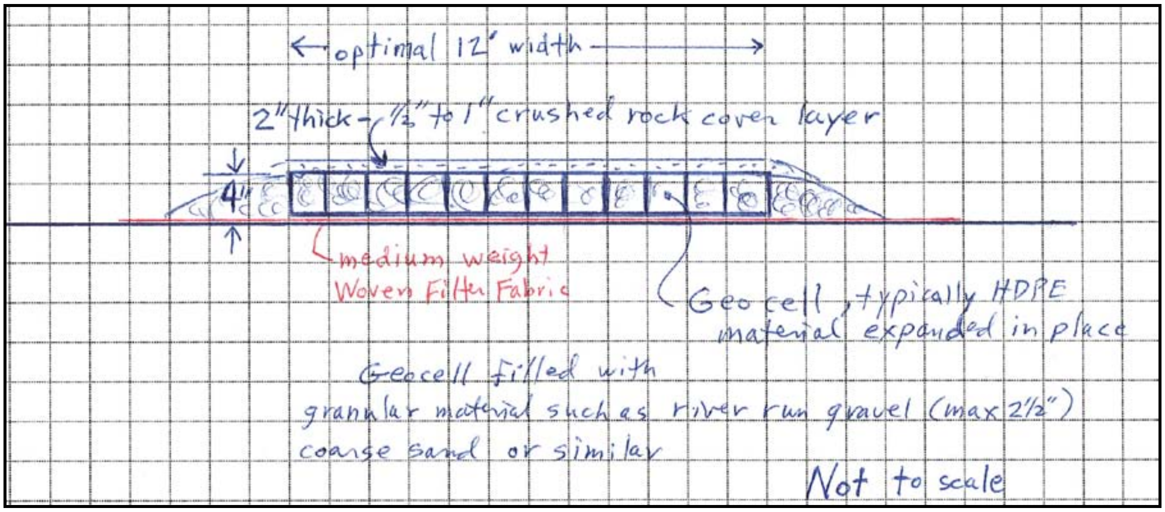


Figure 5. Schematic Drawing of a Temporary Haul Road Using a Geocell System
(actual dimensions will vary)

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

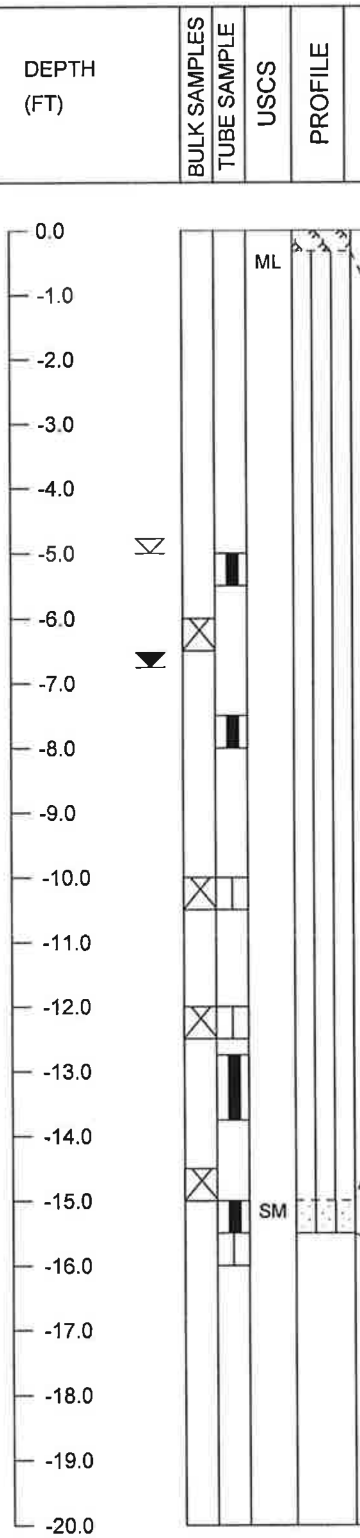
SHN Consulting Engineers & Geologists, Inc.

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

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 Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
0.0					Grass, roots to 4".									
	☒		ML		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

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

BORING NUMBER
HB-3

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

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; 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														

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: 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

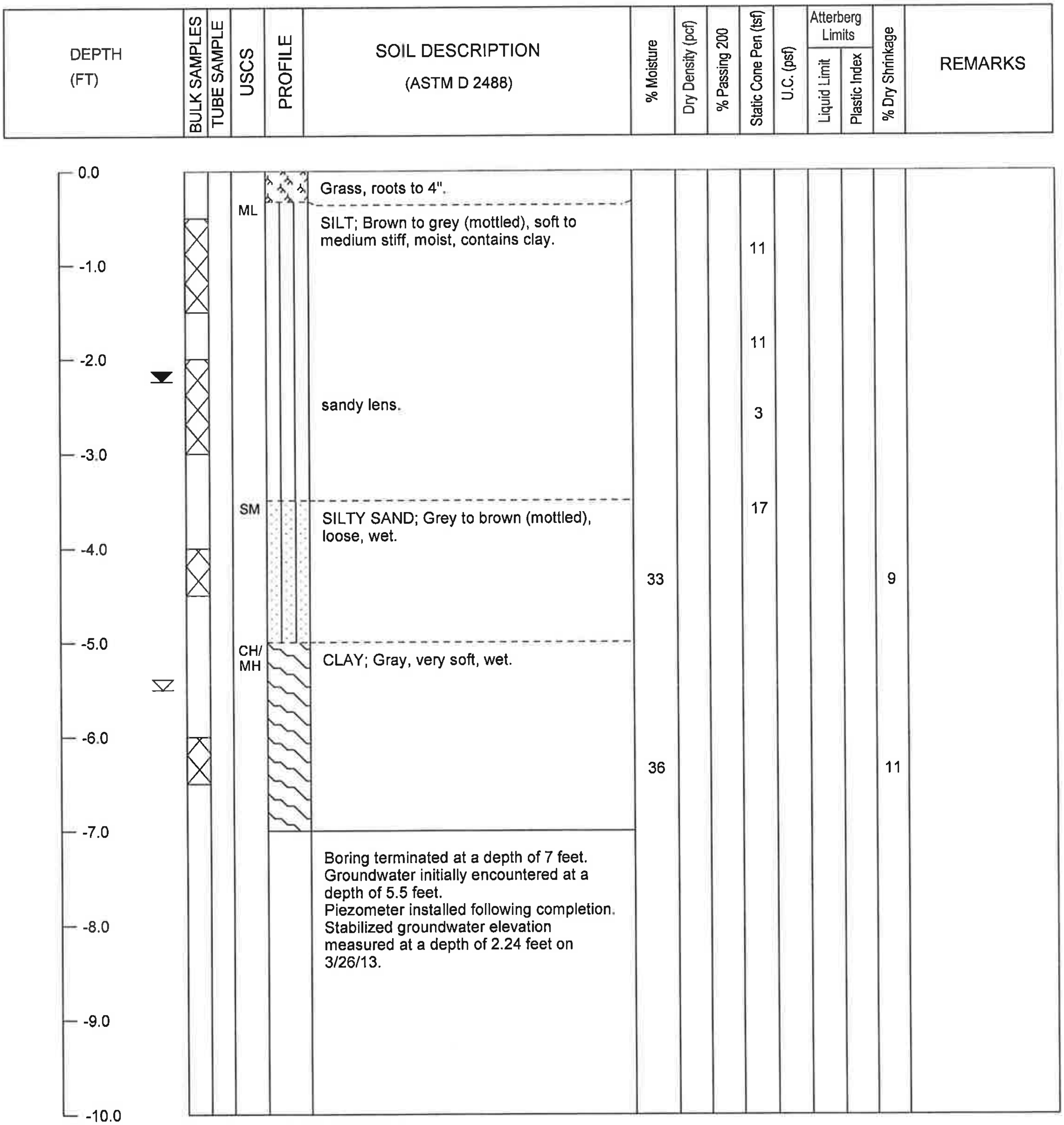
DEPTH (FT)	BULK SAMPLES	TUBE SAMPLE	USCS	PROFILE	SOIL DESCRIPTION (ASTM D 2488)	% Moisture	Dry Density (pcf)	% Passing 200	Static Cone Pen (tsf)	U.C. (psf)	Atterberg Limits		% Dry Shrinkage	REMARKS
											Liquid Limit	Plastic Index		
0.0			ML		Grass, roots to 4".									
-1.0					SILT; Dark brownish-grey 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														

LOG OF BORING

PROJECT: Martin Slough Enhancement Project
LOCATION: East of south end of west 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-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.									
-1.0									11					
-2.0									11					
					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.									

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		% Dry Shrinkage	REMARKS
											Liquid Limit	Plastic Index		
0.0					Grass, roots to 4".									
	☒		ML		SILT; Greyish-brown, medium stiff, moist.									
-1.0	☒								10.5					
-2.0	☒													
-3.0														
									4					
-4.0	☒		SM		SILTY SAND; Grey, loose, wet, fine sand.									
▼	☒		CH/ MH		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							4	
	☒					19		28						
-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														

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: 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														

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: 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
0.0					Grass, roots to 4".									
-1.0	☒		CL/ ML		CLAY; Strong brown to brownish-grey (mottled), medium stiff, moist.				9					
-2.0	☒		SM/ SC		CLAYEY SAND; Strong brown to brownish-grey (mottled), medium dense, wet.				5					
-3.0 -4.0 -5.0	☒		CL/ ML		SILT; Brownish-grey to bluish-grey, medium stiff to stiff, wet, weakly cemented nodules of iron oxide.	18			5 7				9	
-6.0 -7.0 -8.0 -9.0 -10.0					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

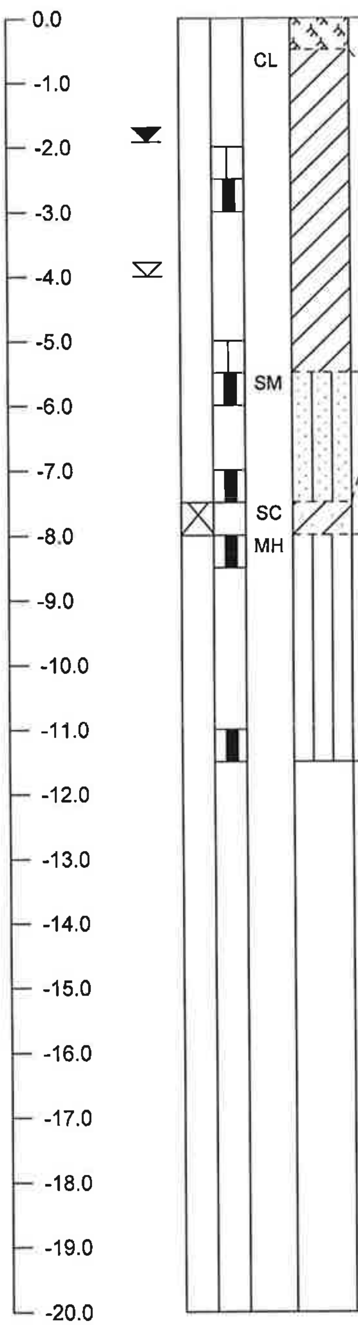
Consulting Engineers & Geologists, Inc.

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: Liquid Limit	Atterberg Limits: Plastic Index	% Dry Shrinkage	REMARKS
	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

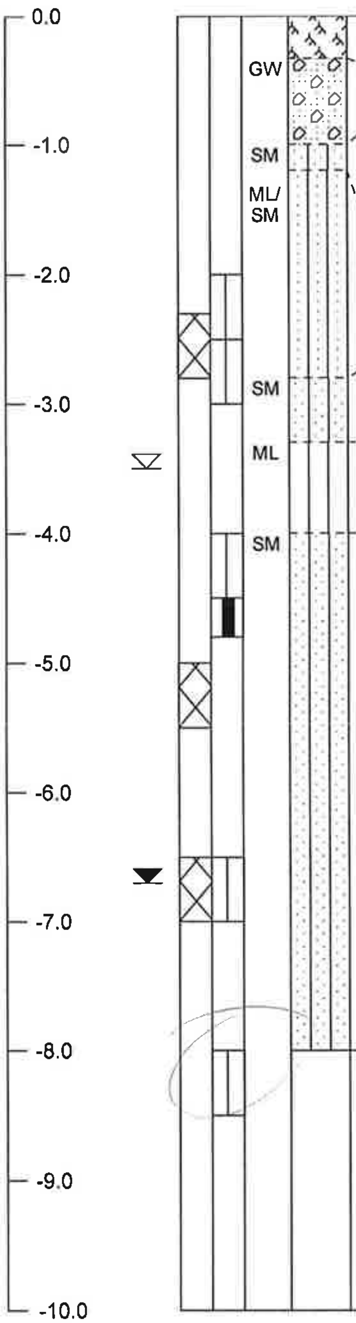
JOB NUMBER: 013035

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						
	Boring terminated at a depth of 8 feet. 8.5'? 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 Liquid Limit	Atterberg Limits Plastic Index	% Dry Shrinkage	REMARKS
0.0					Grass, roots to 4".									
	☒		ML		SANDY SILT; Brown to gray (mottled), soft, damp to moist, contains clay.									
-1.0	☒													
▼									8.5					
-2.0	☒					46							7	
									8					
▽			CL	▨	SILTY CLAY; Grey to olive brown (weakly mottled), soft, wet.									
-3.0				▨										
-4.0				▨										
	☒			▨		42							12	
-5.0				▨										
			ML		SILT; Grey, soft, wet, organics and peat filaments.									
-6.0	☒													
-7.0					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.									
-8.0														
-9.0														
-10.0														

LOG OF BORING

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		% Dry Shrinkage	REMARKS
											Liquid Limit	Plastic Index		
0.0					Grass, roots to 4".									
			ML		SILT; Brown, soft, damp, contains clay.									
-2.0		■				42	75						10	
▼														
-3.0 ▽														
					grades bluish-grey.									
-5.0		■				45	73				46	19		
			SM		SAND; Grey, loose, saturated.									
			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														

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.

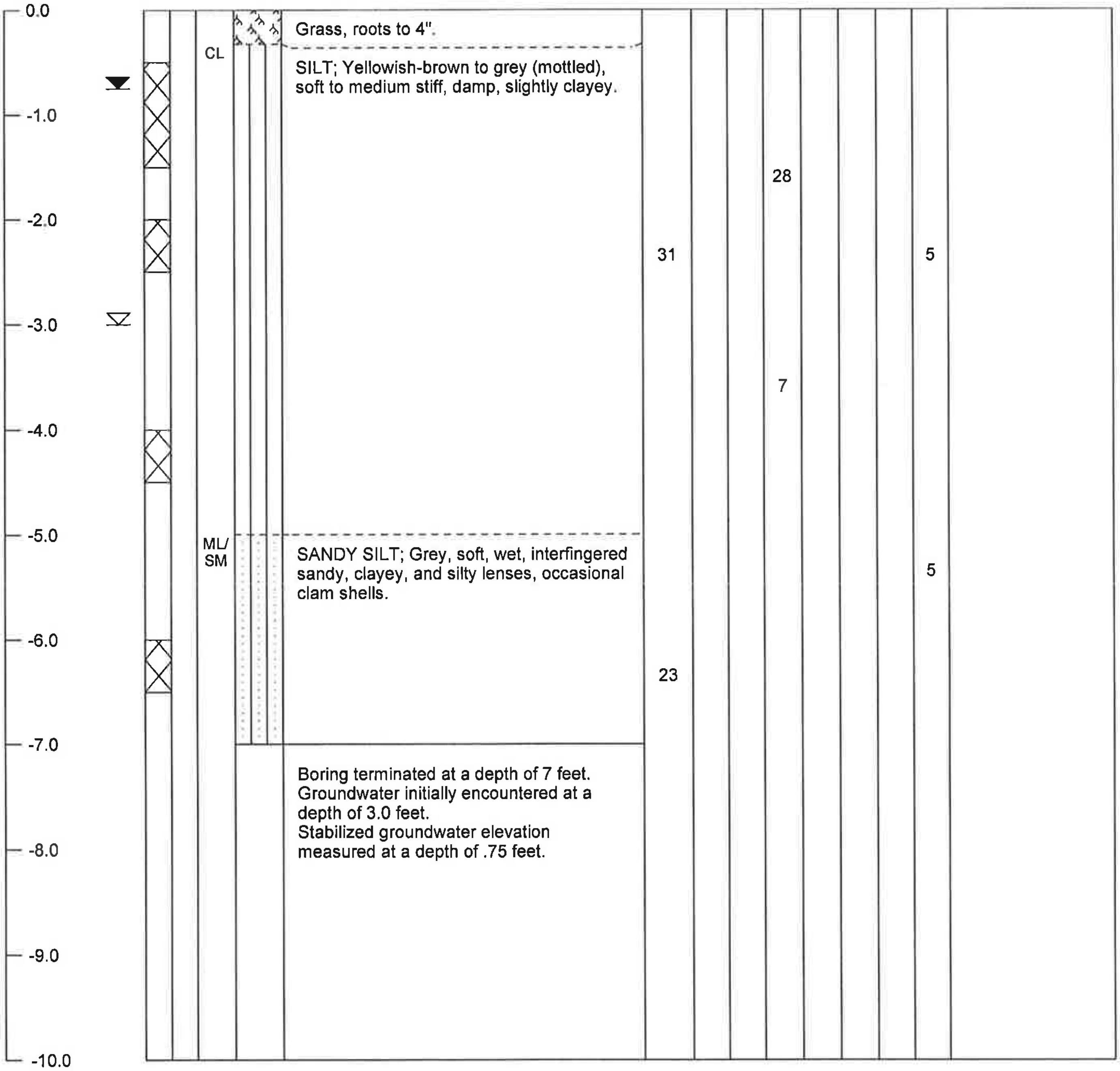
LOG OF BORING

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 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.	31			28 7				5	
-5.0			ML/ SM		SANDY SILT; Grey, soft, wet, interfingered sandy, clayey, and silty lenses, occasional clam shells.	23							5	
-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.									

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
JOB NUMBER: 013035
LOCATION: Tidal Pond F
DATE DRILLED: 03/22/13
GROUND SURFACE ELEVATION: 17 feet
TOTAL DEPTH OF BORING: 7 feet
EXCAVATION METHOD: Hand Auger
SAMPLER TYPE: Bulk
LOGGED BY: AC, JMA

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
0.0					Grass, roots to 4".									
	☒		ML		SILT; Yellowish-brown to gray (mottled), soft, damp to moist.				6					
-1.0 ▼	☒													
	☒								5					
-2.0 ▽	☒													
			SM		SILTY SAND; Brownish-grey, loose, saturated, flowing, fine sand.				5					
-3.0			ML		SILT; Grey, soft, wet, organics, decomposed plant fragments.				14					
-4.0	☒					27							8	
-5.0														
-6.0	☒													
			SM		SILTY SAND; Grey, loose, wet.									
-7.0					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.									
-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.

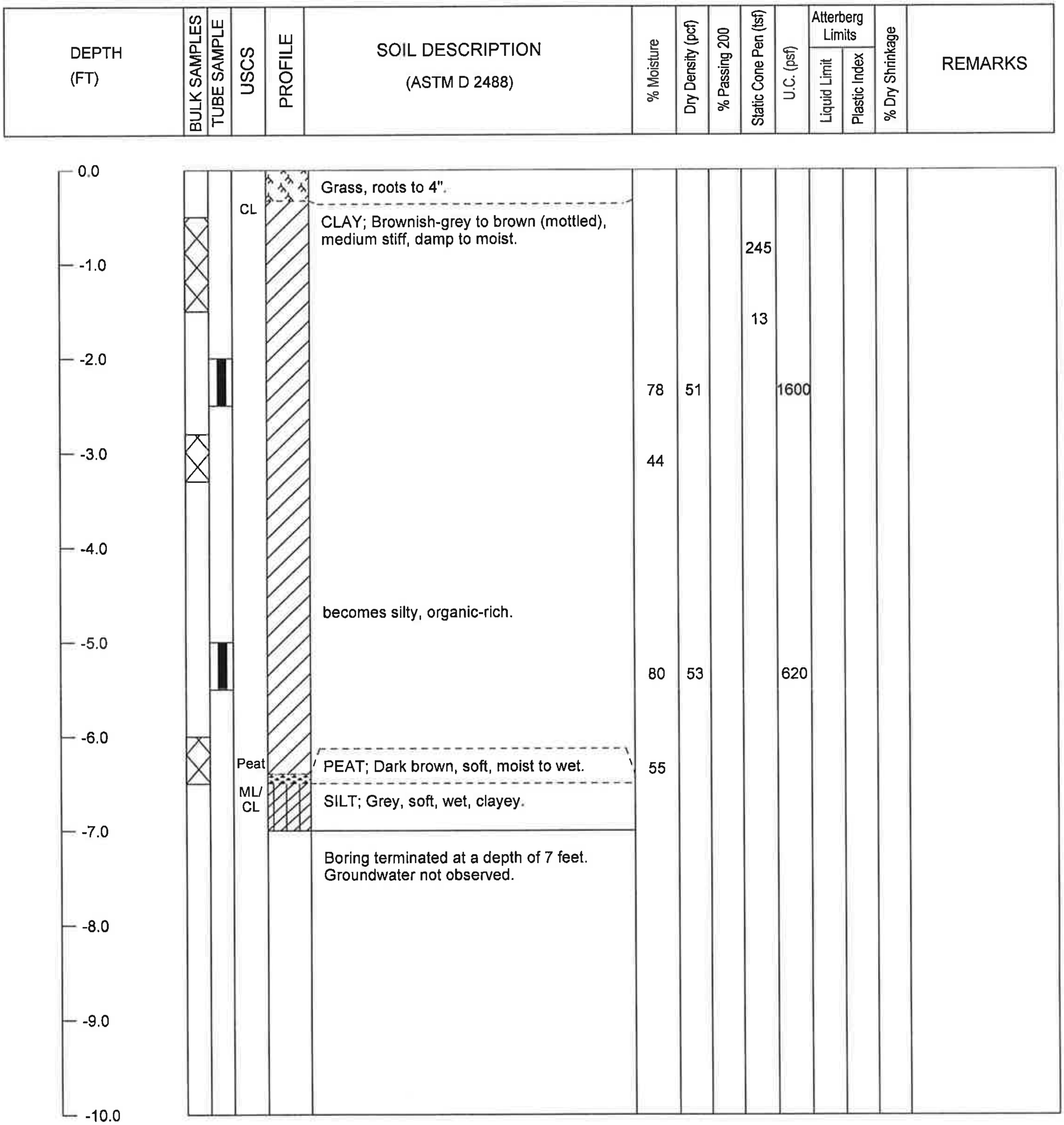
LOG OF BORING

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 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



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".									
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

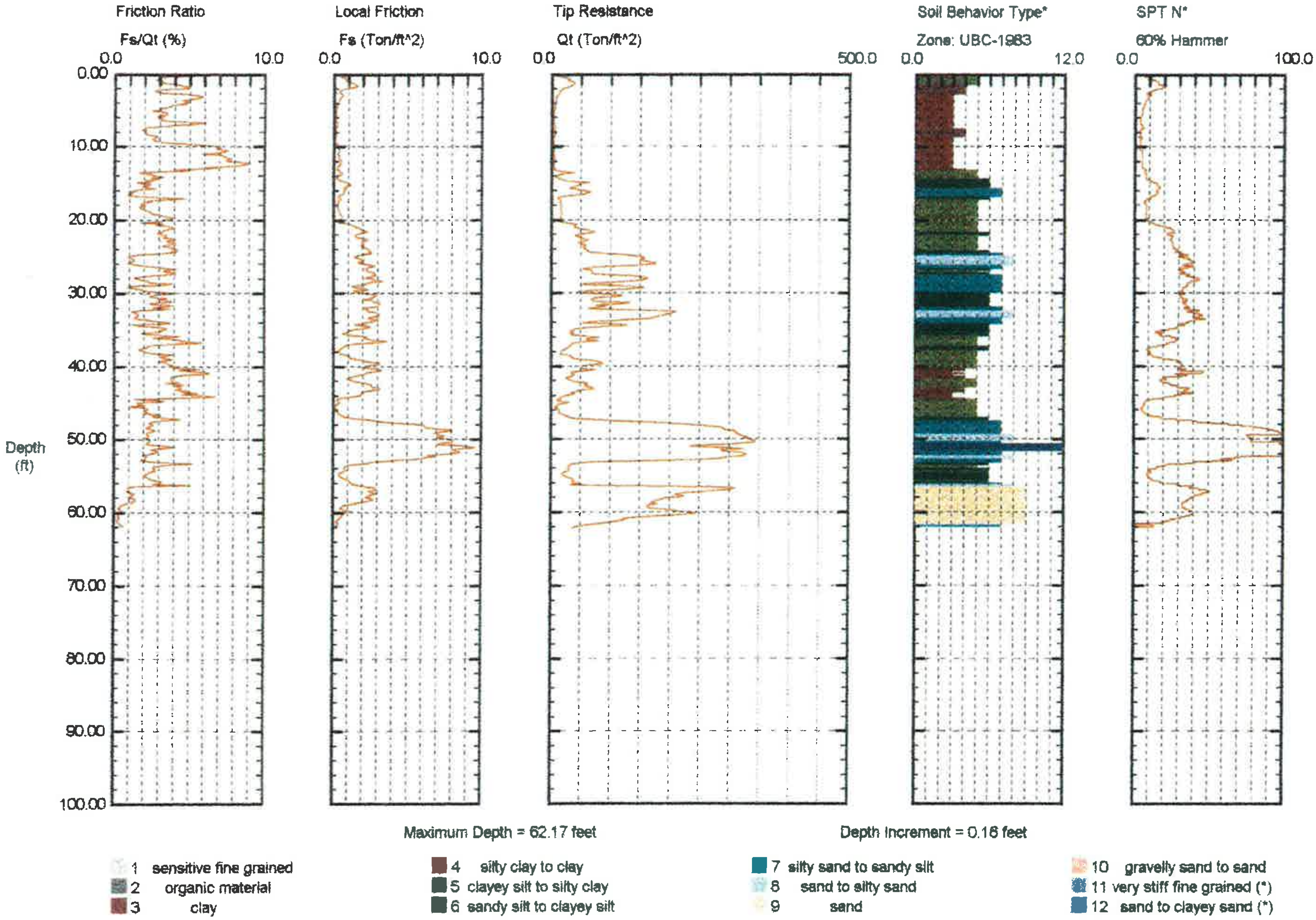
DEPTH (FT)	BULK SAMPLES	SS SAMPLES	SPT BLOWS PER 0.5'	USCS	PROFILE	DESCRIPTION	% Moisture	Dry Density (pcf)	Unc. Comp. (psf)	% Passing 200	Atterberg Limits Liquid Limit	Atterberg Limits Plastic Index	REMARKS
0.0				ML-CL	▨	SILT, clayey, very sandy, fine, with few to moderate organics, soft, dry, very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

behavior type and SPT based on data from UBC-1983

VBI In-Situ Testing

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

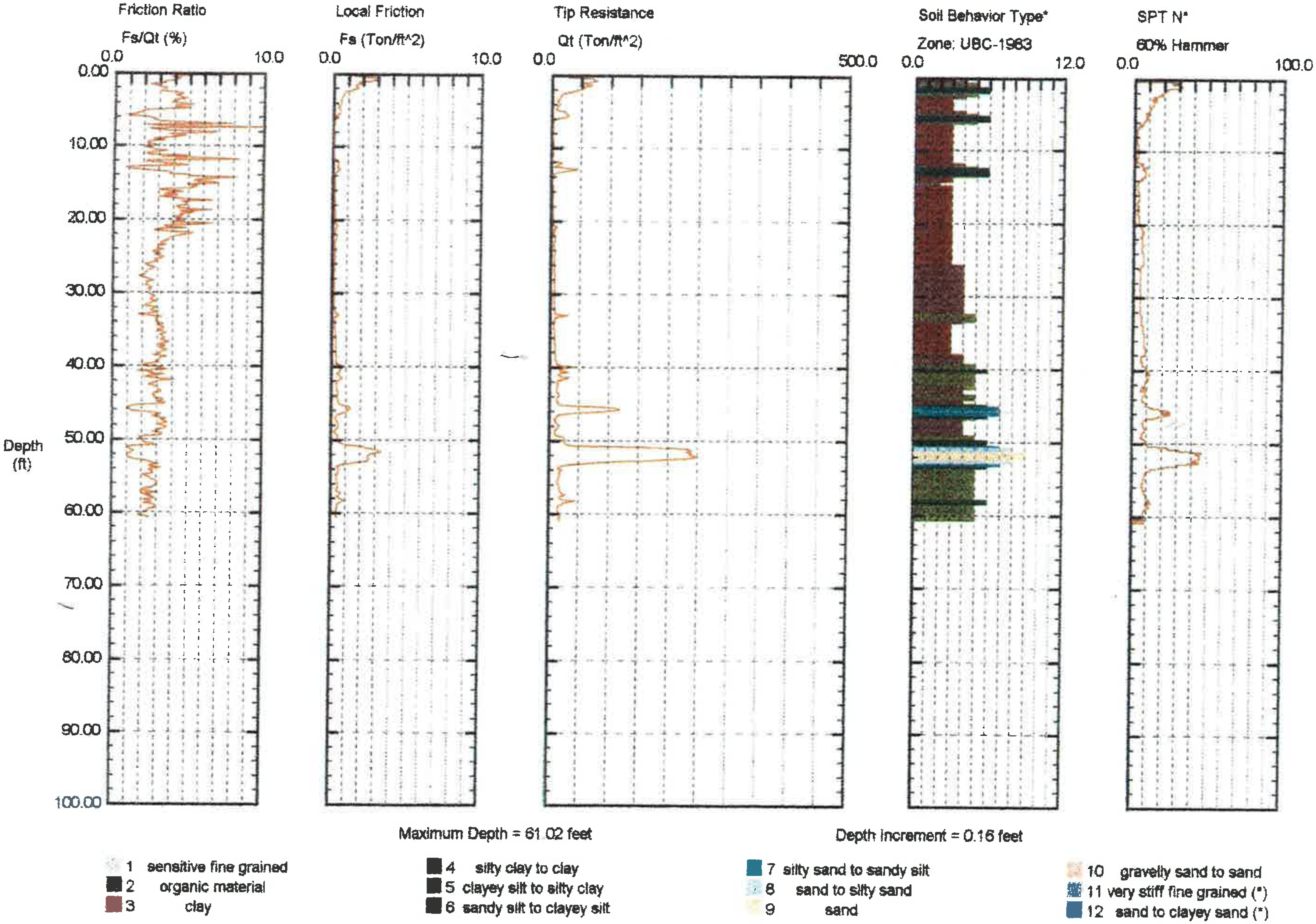
CPT Date/Time: 09-25-02 09:26
Location: CPT-7
Job Number: MARTIN SLOUGH



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



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													

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	
-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 Liquid Limit	Atterberg Limits Plastic Index	REMARKS
-36.0			10		▨								
-37.0													
-38.0													
-39.0													
-40.0			4 8	SM SP	░	FINE SAND, with silt, medium dense, moist, grey.							
-41.0			8		░								
-42.0													
-43.0													
-44.0													
-45.0			4 6	SM SP	░	FINE SAND, with silt, medium dense, moist, grey.							
-46.0			8		░								
-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

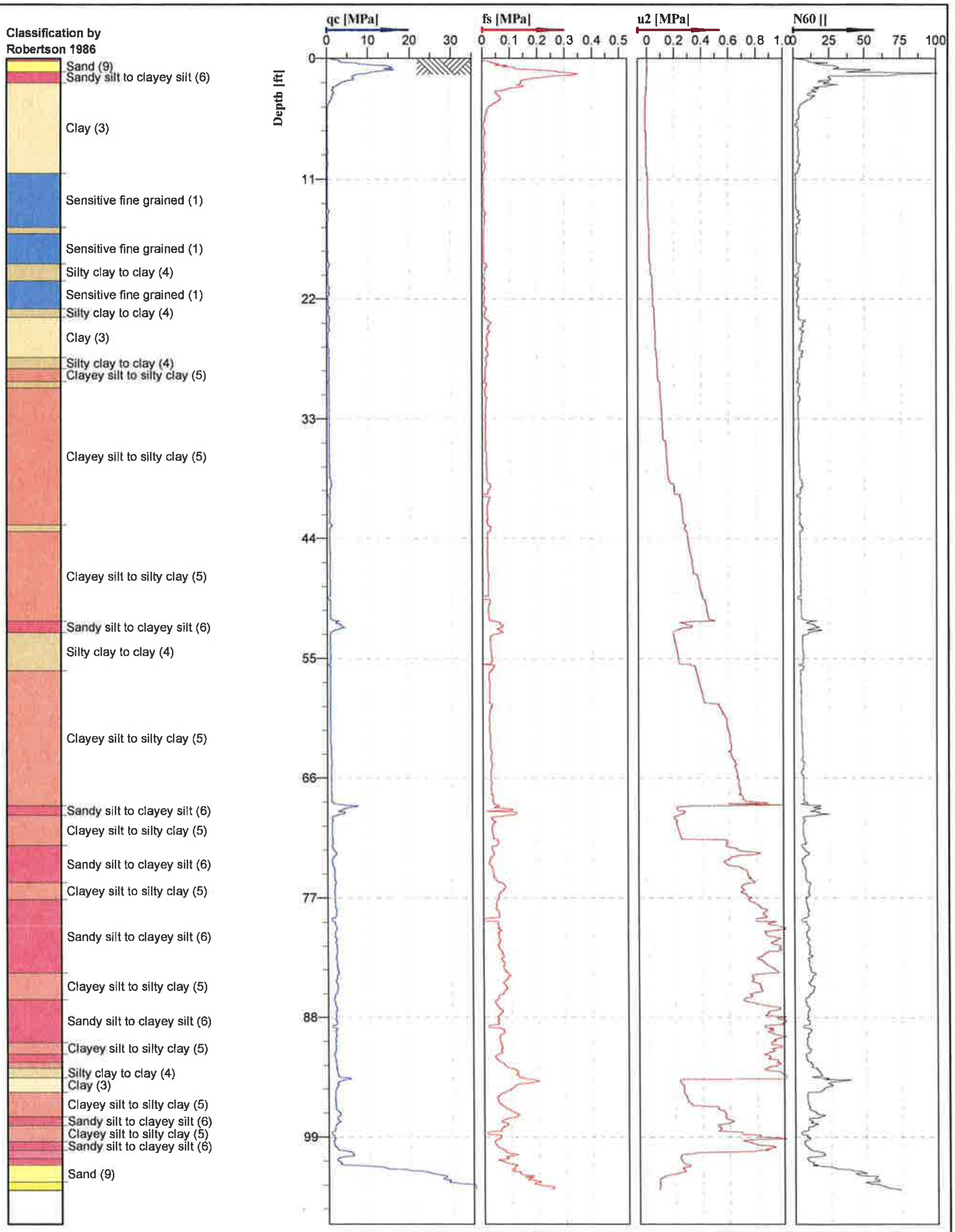
PROJECT: Martin Slough
LOCATION: Golf Course, Eureka, CA
GROUND SURFACE ELEVATION: –
EXCAVATION METHOD: Solid Stem Flight Auger (4")
LOGGED BY: SMB

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		REMARKS
										Liquid Limit	Plastic Index	
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.							
		1 2 4	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.
-10.0		3 7										
		7	SM		SAND, fine to medium, slightly silty, medium dense, wet, dark yellowish brown.	18.8	112					
			SP-SM		SAND, fine to medium, slightly silty, with few organics, medium dense, wet.							SM/SP-SM Contact inferred
-15.0		5 10 6 1 2				122.6	42			86.4	39	Woody debris in sampling shoe
		2	OH		SILT, sl. clayey to clayey, sl.sandy, fine, with many organics, soft, wet, dark yellowish brown.							
			SM-SP		SAND, fine to medium, slightly silty, medium dense, wet, dark yellowish brown.							OH/SM-SP Contact inferred
-20.0		3 3										
		3	ML		SILT, sandy, fine, slightly clayey, with few organics, medium stiff, wet, dark yellowish brown.							
					Bottom of boring at 21.5 feet.							
-25.0												

FIELD BORING LOG



Fisch Drilling

U2

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 Replaccment **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 SHELBY	P				consol TXCU w=31.4% γ_d=89pcf
	15							
	20						minor decomposing organics	TXCU LL=32, PI=12 W=34.6% γ_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 $W = 37,1\%$ $\gamma_d = 82$ pcf
	35							
	40				MH/OH	CL	Grey CLAYEY SILT, soft to medium stiff, saturated, minor sand and organics	TXCU LL=53, PI=24 $W = 39.7\%$ $\gamma_d = 80$ pcf
	45							
					SM		Brownish-gray SILTY SAND medium dense, saturated, few organics and shell fragments	TXCU $W = 42,3\%$ $\gamma_d = 77$ psf
	50				MH/OH		Gray CLAYEY SILT, soft to medium stiff, saturated, few organics and shell fragments	
	55							
					SM		Brownish-gray SILTY SAND medium dense, saturated	
	60			4			layer of decomposing roots w/ Few rounded gravels to 1/2"	

Project:

Project Location: SEE PAGE 1

Project Number: 012163

Log of Boring B-1

Sheet ~~1 of 1~~ 3 of 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
	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							
				35	SP		SAND, very dense, saturated, medium-grained	collected sample using catcher
	90			50/4"			BOH 90½'	-200 = 12.9%

Appendix B

ASTM Laboratory Test Results

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: 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		

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

Performed By: JMA

Checked By: Dh

Project Manager: JPB

Job Number: 013035

Date: 4/12/2013

Date: 4/16/13

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	

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PERCENT PASSING # 200 SIEVE (ASTM - D1140)

Project Name:	**Martin Slough Enhancement**	**Project Number:**	**013035**
Performed By:	**JMA**	**Date:**	**4/15/2013**
Checked By:	Jh	**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**				

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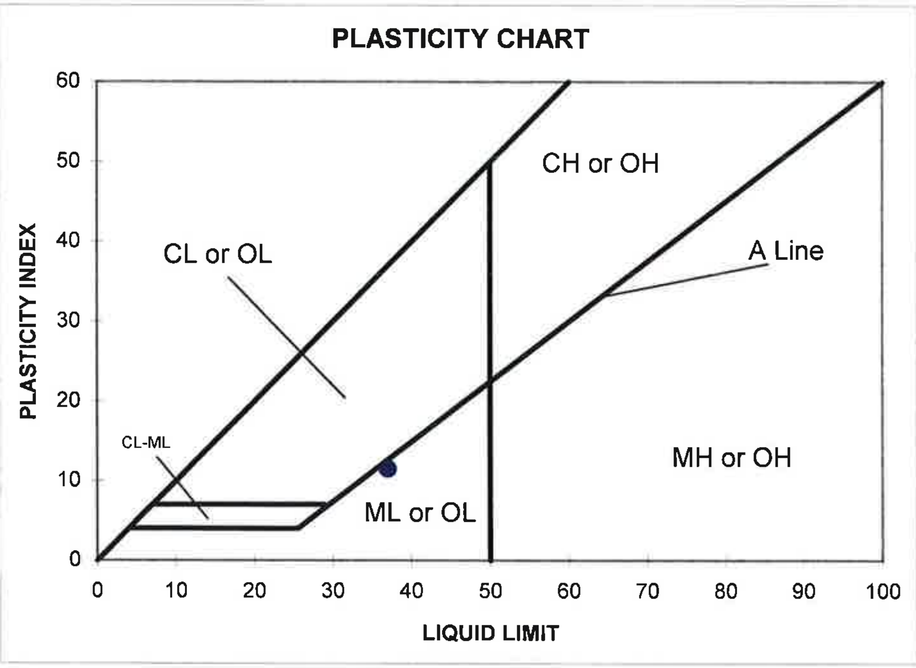
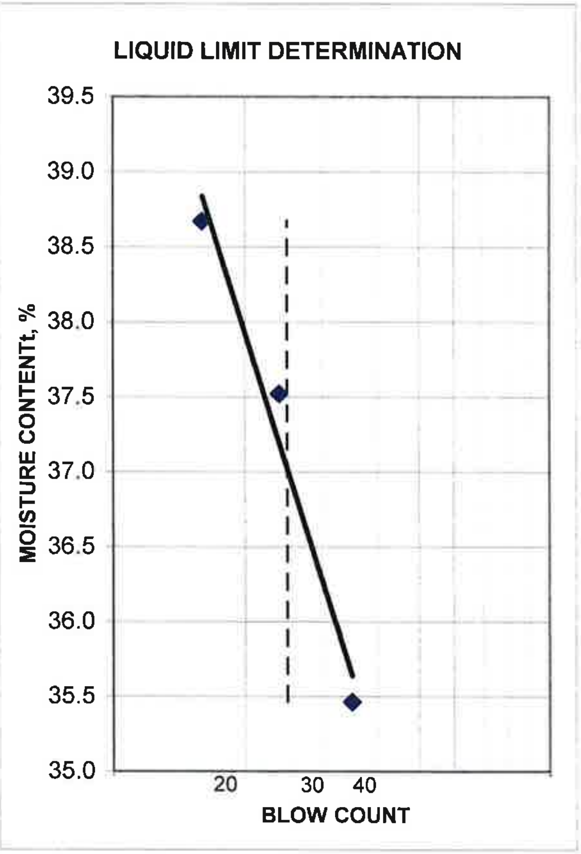
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

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





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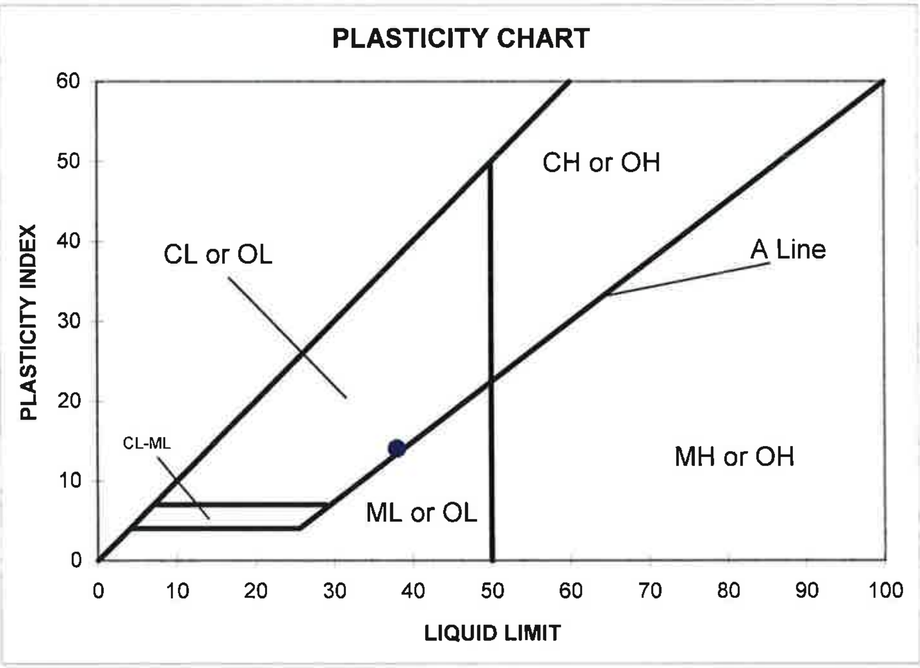
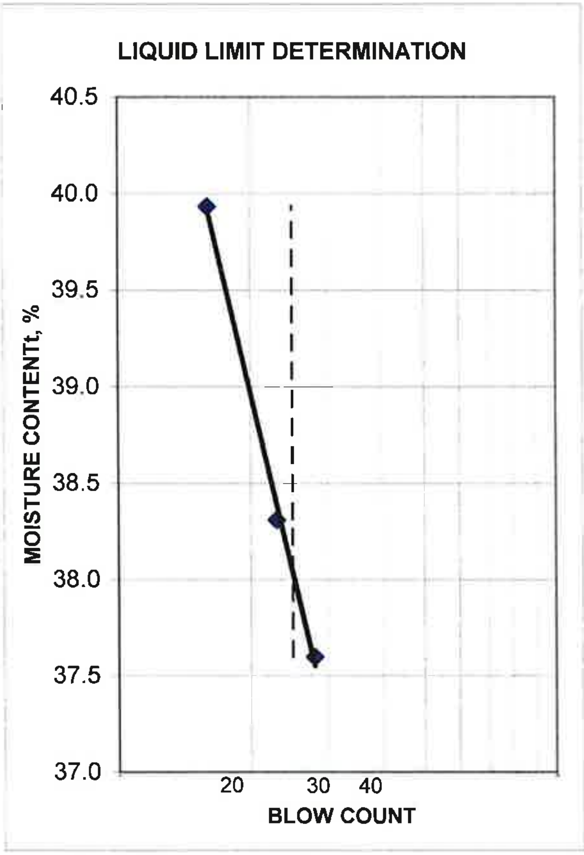
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

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





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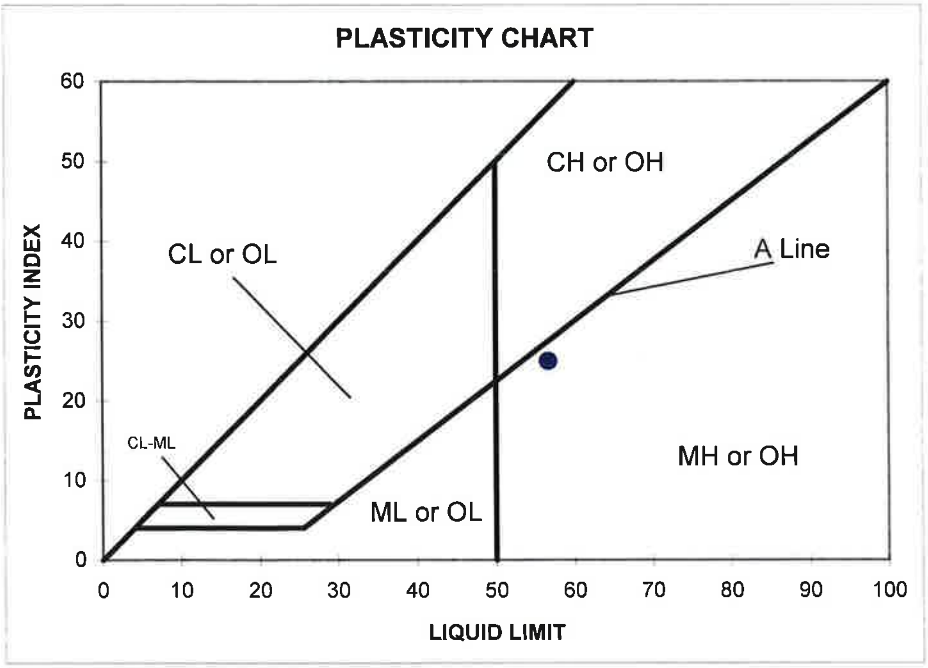
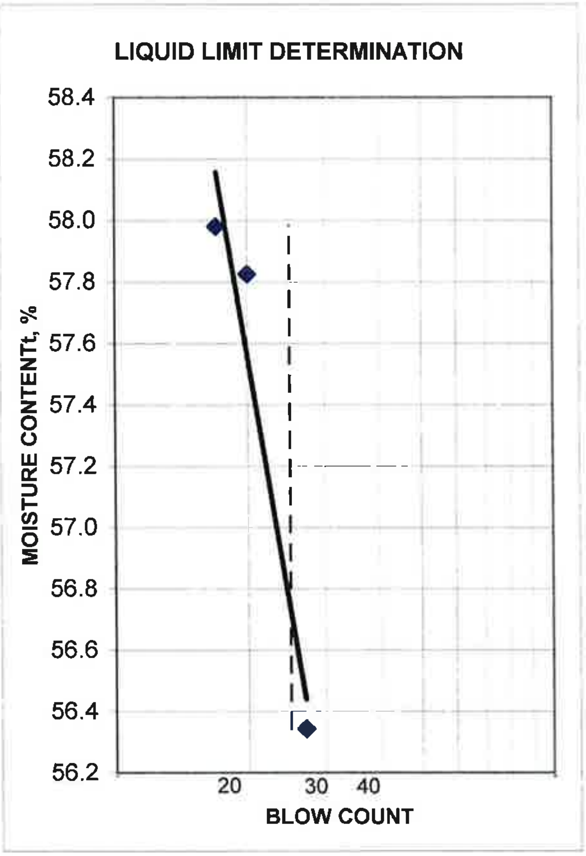
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

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





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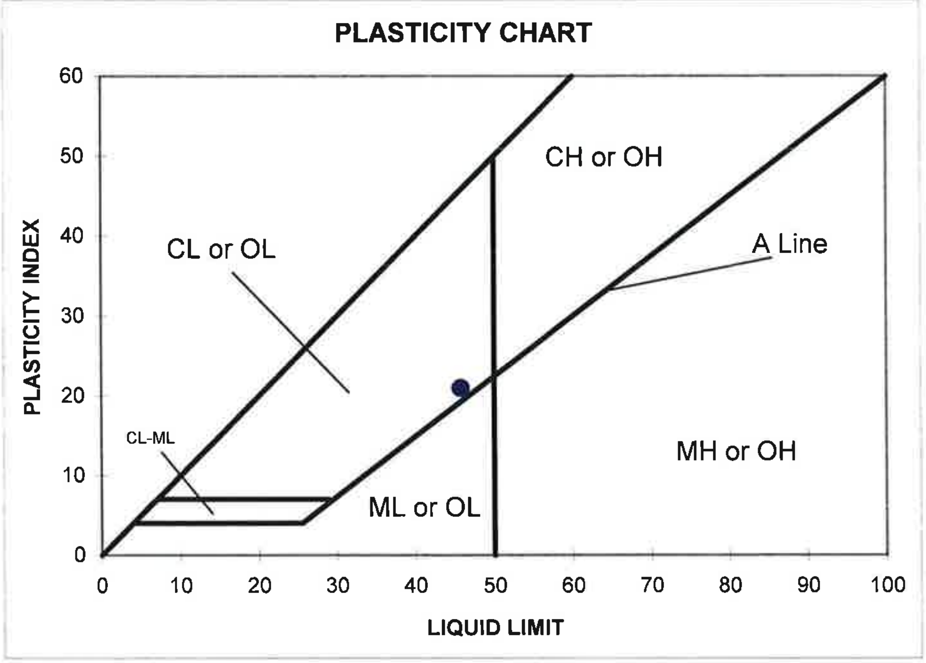
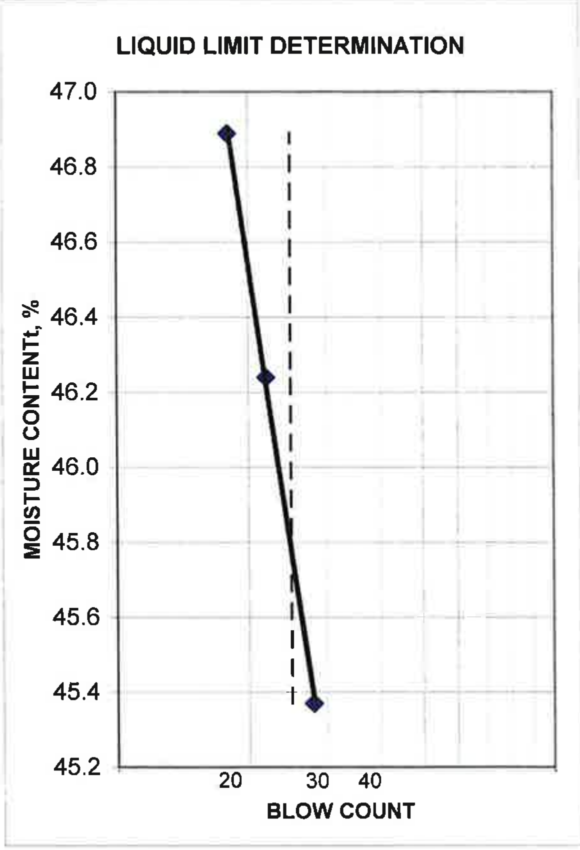
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

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


PLASTICITY CHART


LIQUID LIMIT DETERMINATION

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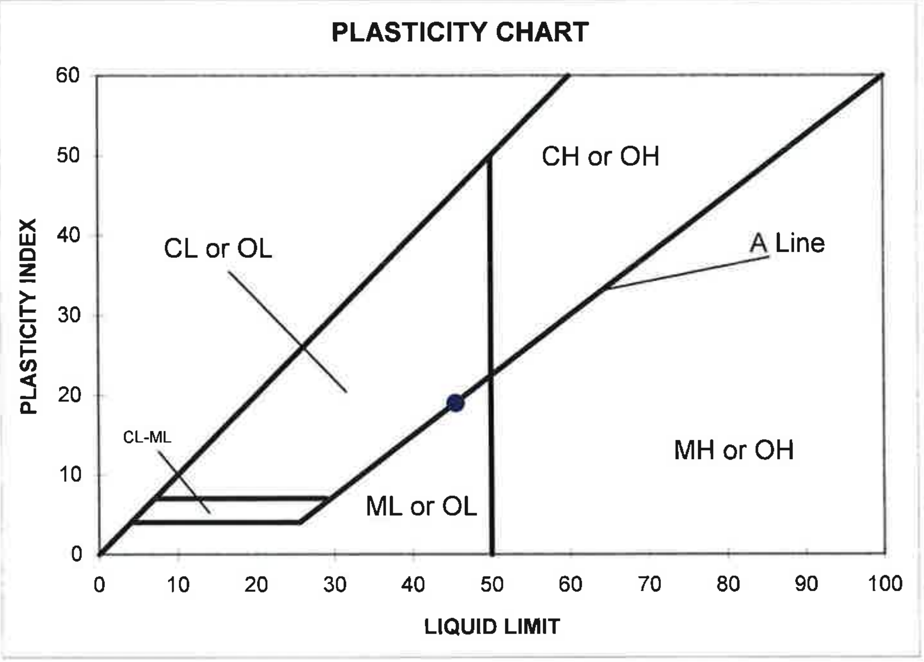
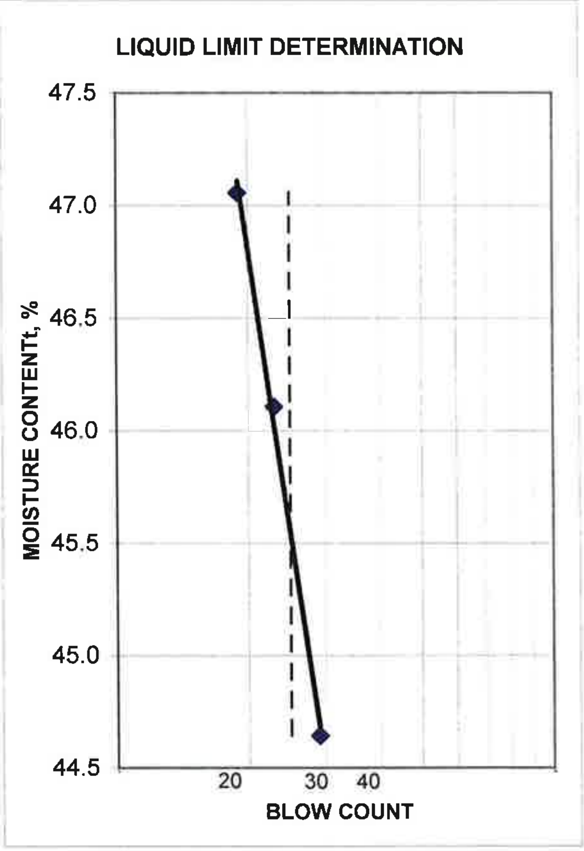
LIQUID LIMIT, PLASTIC LIMIT, and PLASTICITY INDEX (ASTM-D4318)

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

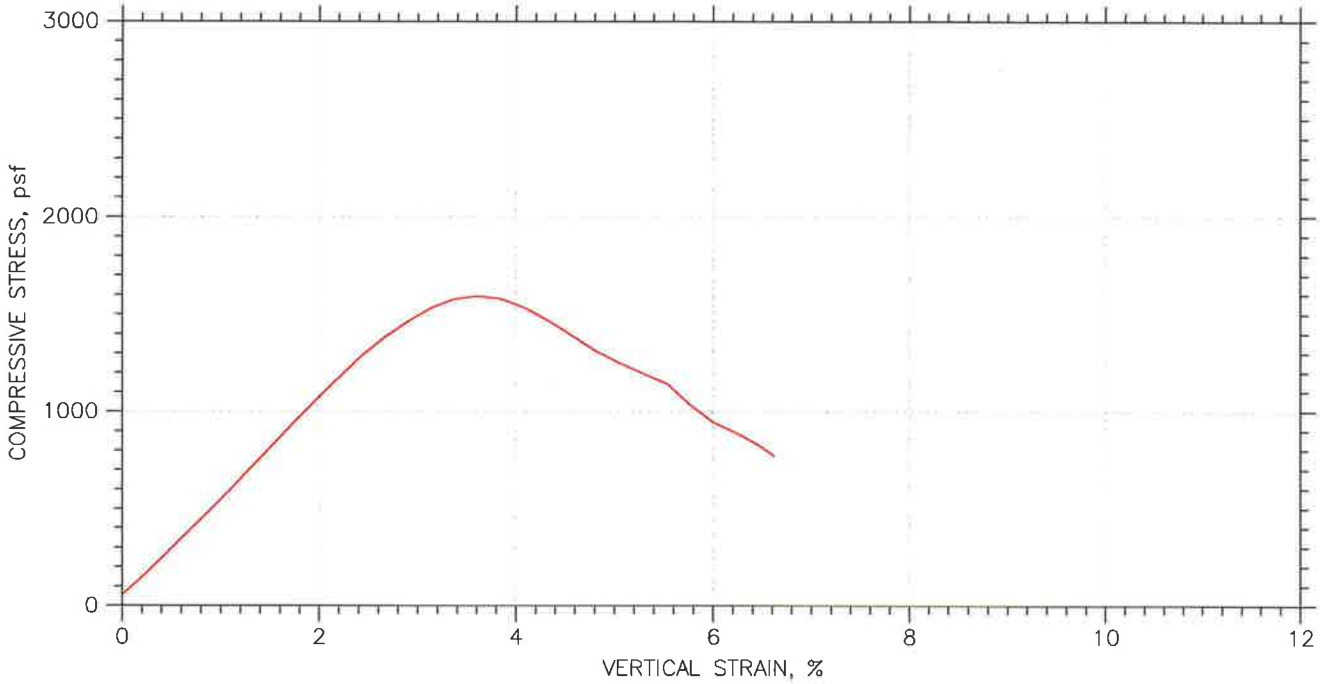
LINE NO.		TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 1	TRIAL NO. 2	TRIAL NO. 3
A	PAN #	13	14	7	8	9
B	PAN WT. (g)	22.170	19.950	29.000	29.140	28.710
C	WT. WET SOIL & PAN (g)	31.390	26.720	36.290	36.460	34.710
D	WT. DRY SOIL & PAN (g)	29.460	25.300	34.040	34.150	32.790
E	WT. WATER (C-D)	1.930	1.420	2.250	2.310	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

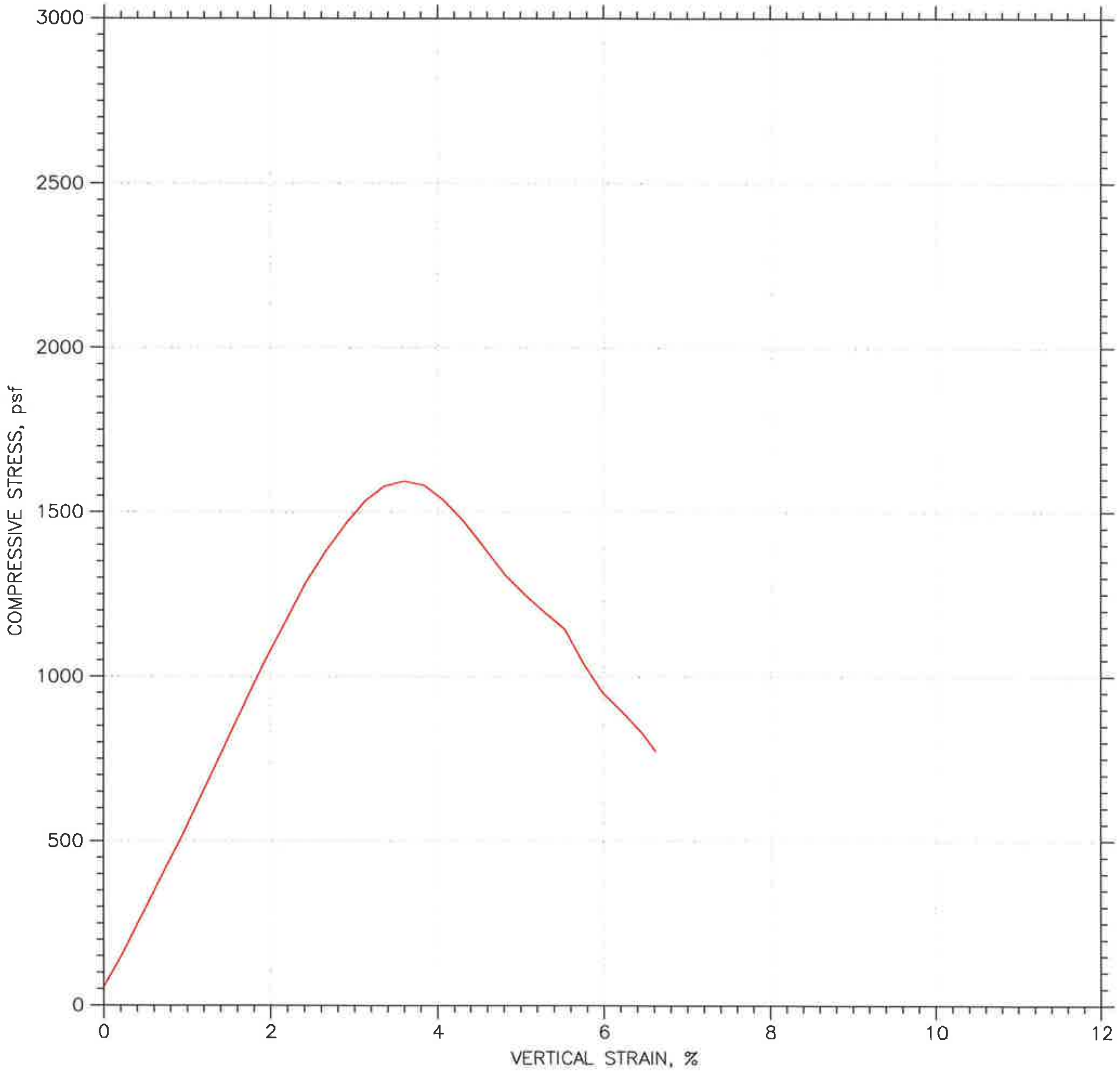


Symbol					
Test No.		13-244			
Initial	Diameter, in	2.38			
	Height, in	5.39			
	Water Content, %	78.04			
	Dry Density, pcf	51.309			
	Saturation, %	92.98			
	Void Ratio	2.2243			
Unconfined Compressive Strength, psf		1595.6			
Undrained Shear Strength, psf		797.81			
Time to Failure, min		3.7539			
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

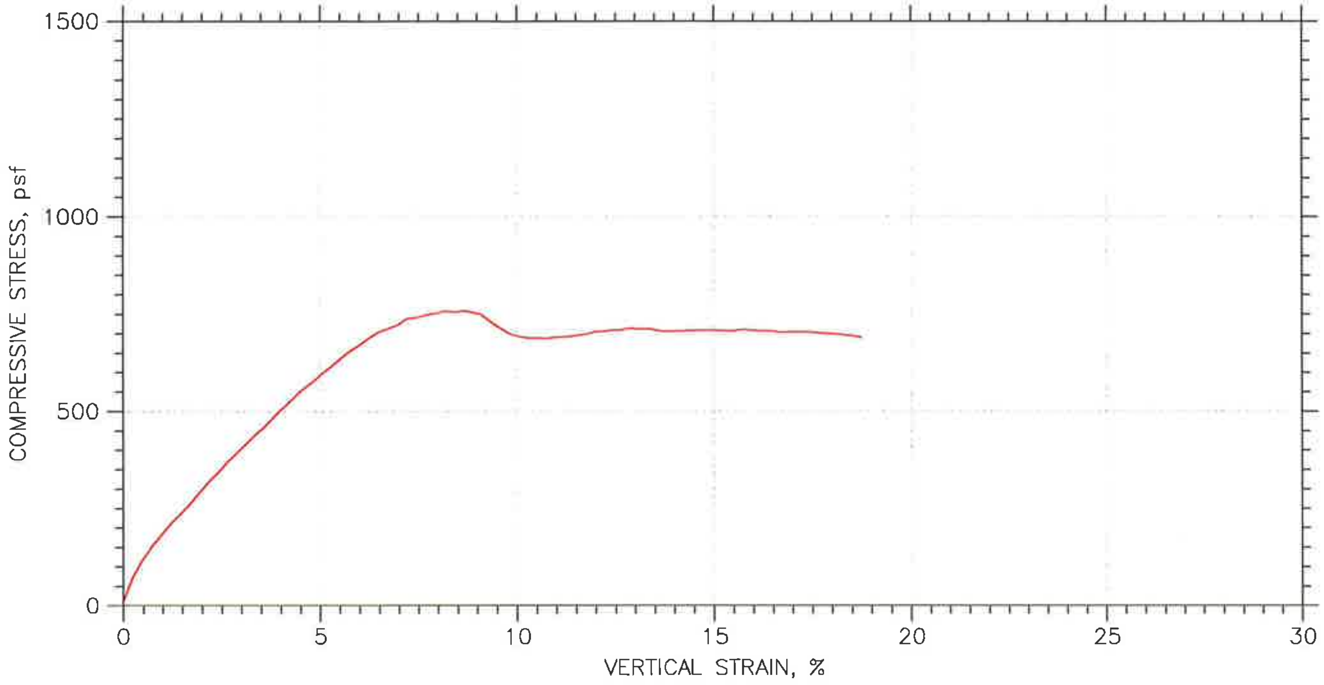
UNCONFINED COMPRESSION TEST REPORT



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

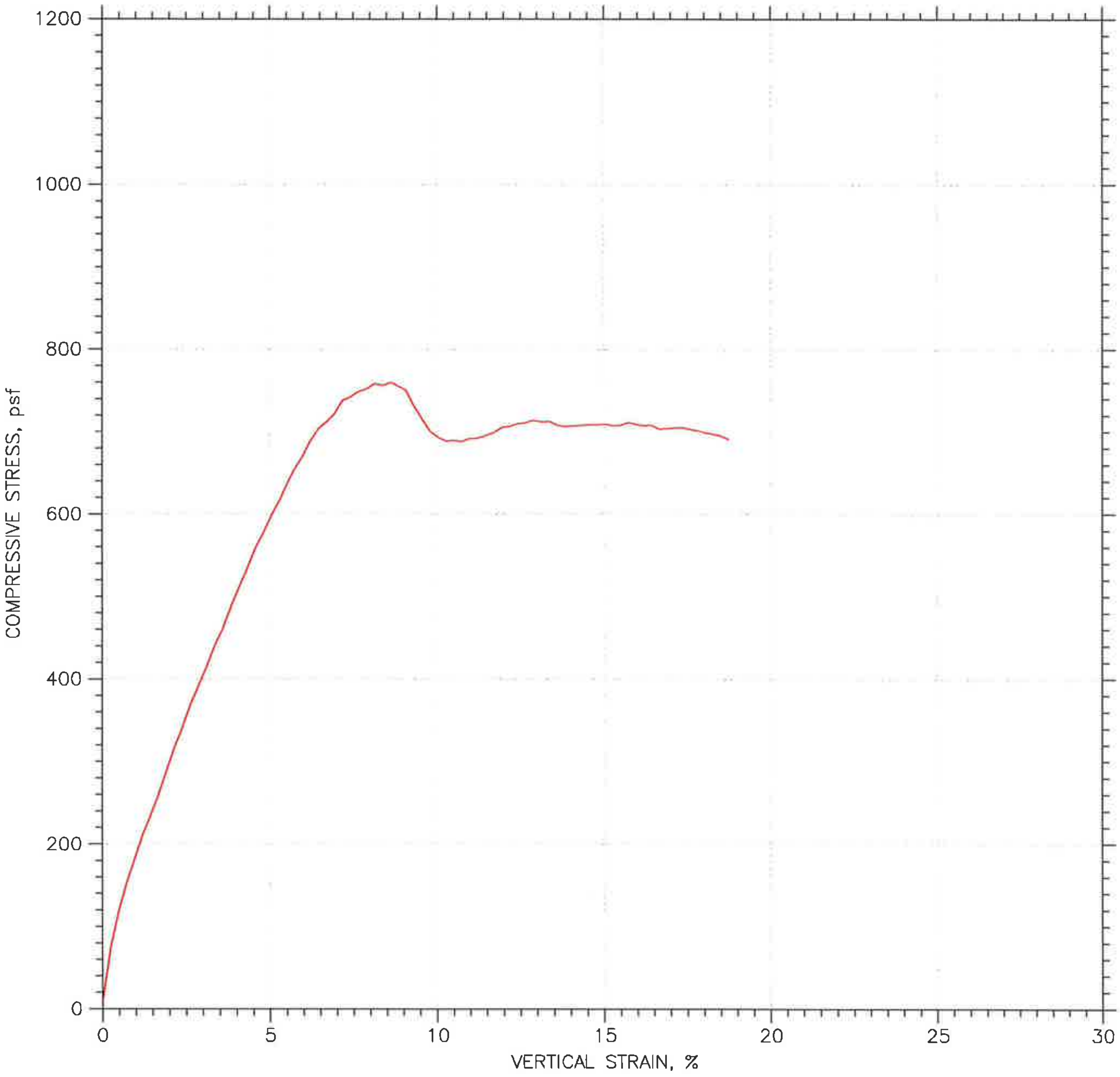


Symbol					
Test No.		13-234			
Initial	Diameter, in	2.38			
	Height, in	5.8			
	Water Content, %	66.67			
	Dry Density, pcf	58.507			
	Saturation, %	96.68			
	Void Ratio	1.8276			
Unconfined Compressive Strength, psf		760.18			
Undrained Shear Strength, psf		380.09			
Time to Failure, min		9.0011			
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

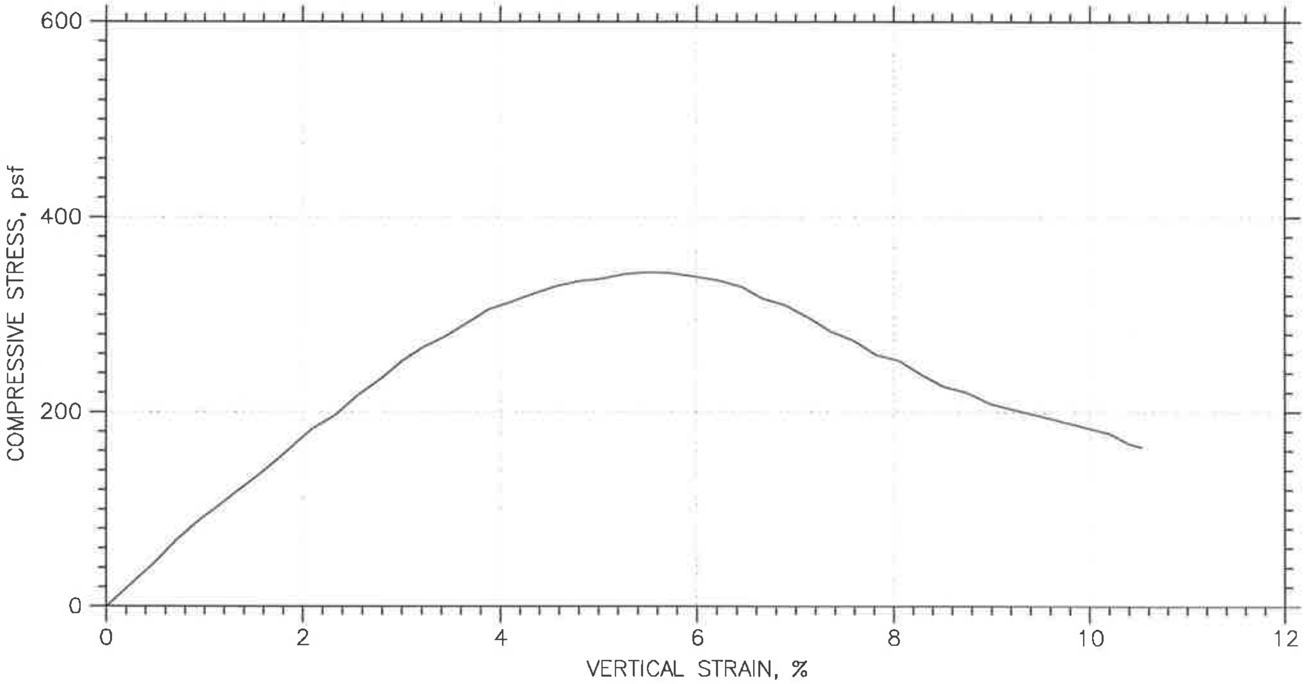
UNCONFINED COMPRESSION TEST REPORT



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

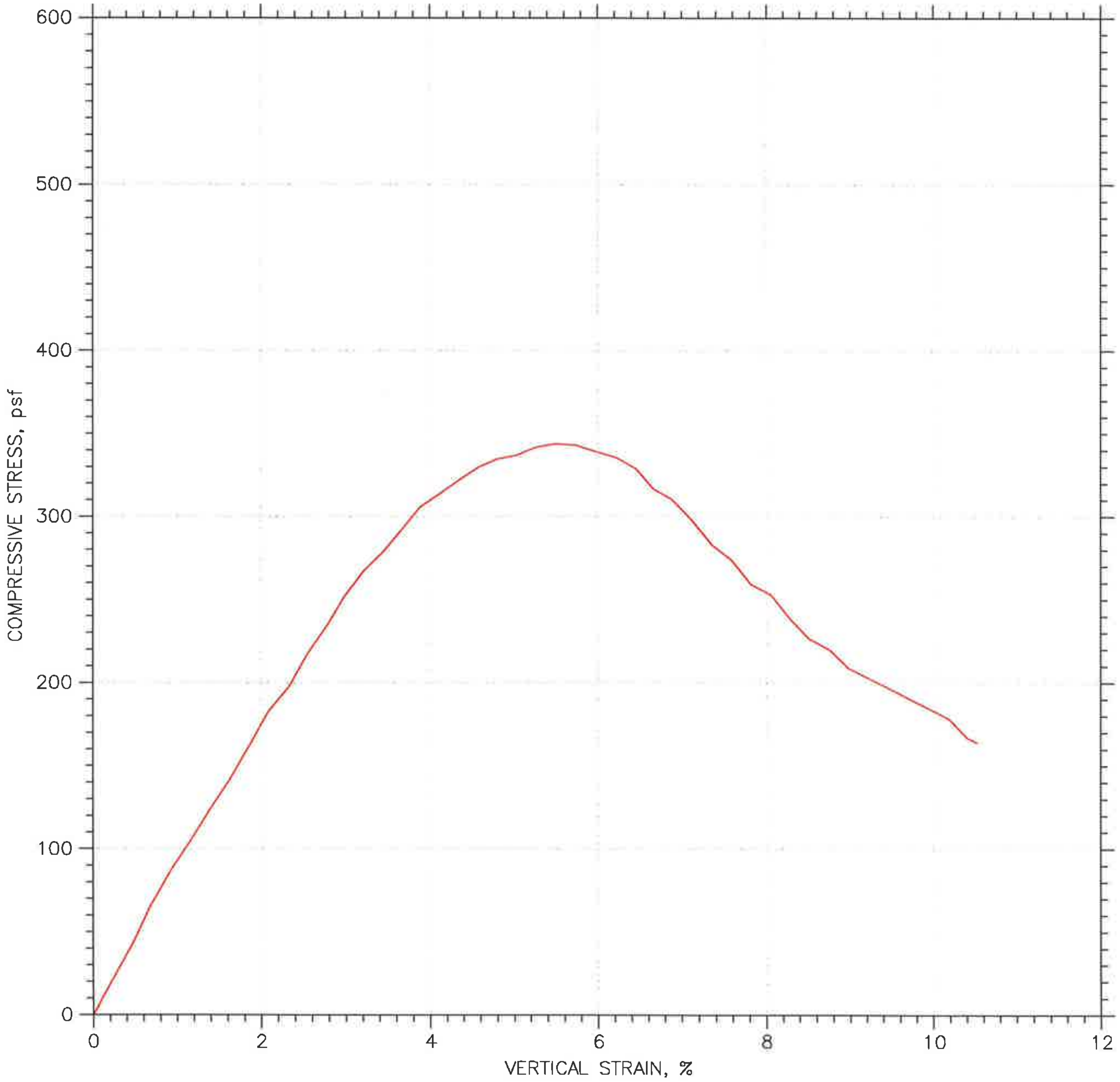


Symbol					
Test No.		13-239			
Initial	Diameter, in	2.38			
	Height, in	5.09			
	Water Content, %	33.63			
	Dry Density, pcf	86.153			
	Saturation, %	96.83			
	Void Ratio	0.92023			
Unconfined Compressive Strength, psf		344.04			
Undrained Shear Strength, psf		172.02			
Time to Failure, min		6.0021			
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

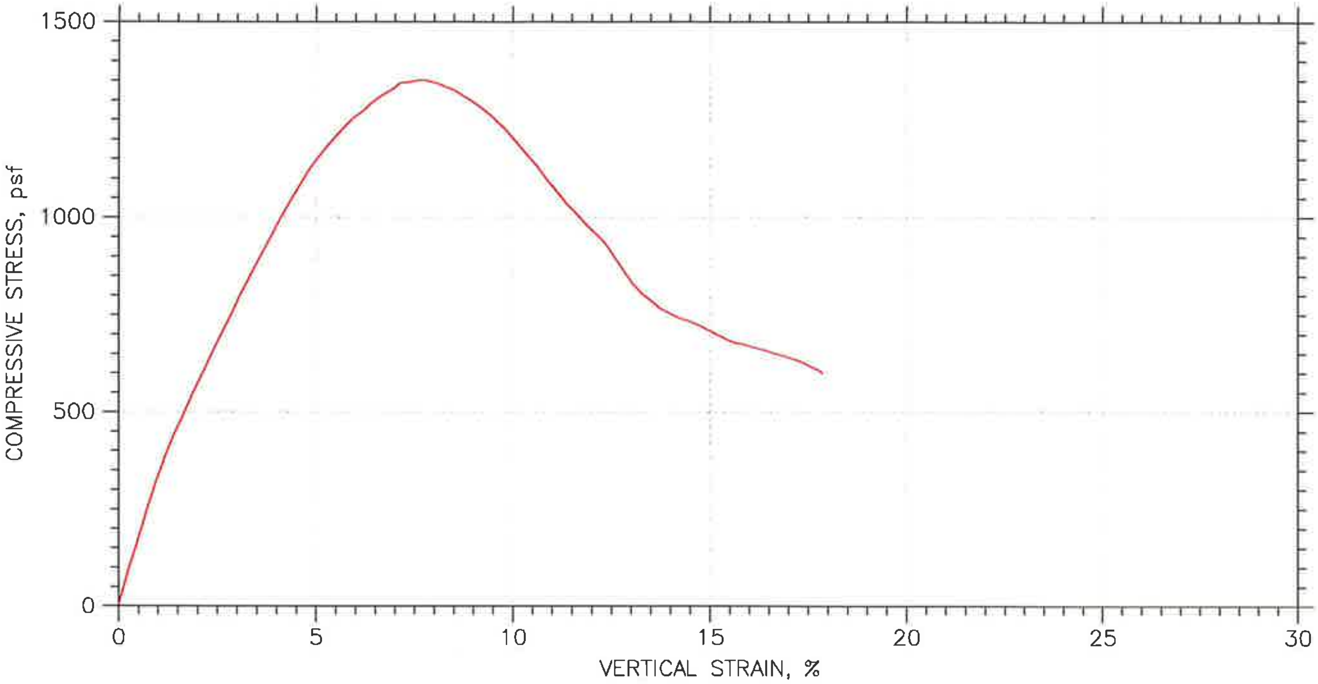
UNCONFINED COMPRESSION TEST REPORT



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

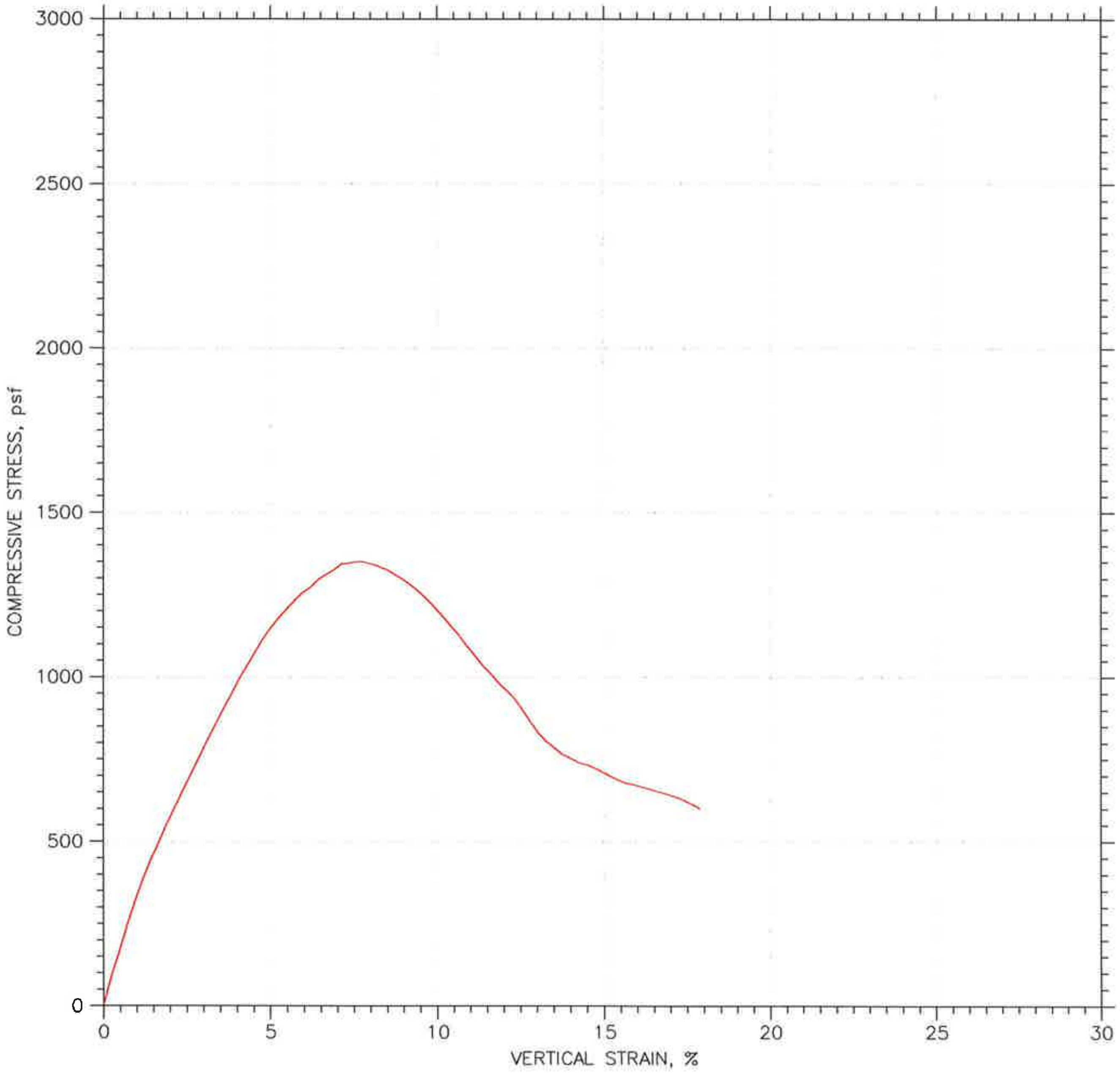
Sample Type: 2.5"shelby

Description: Strong Brown SILT

Remarks:

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

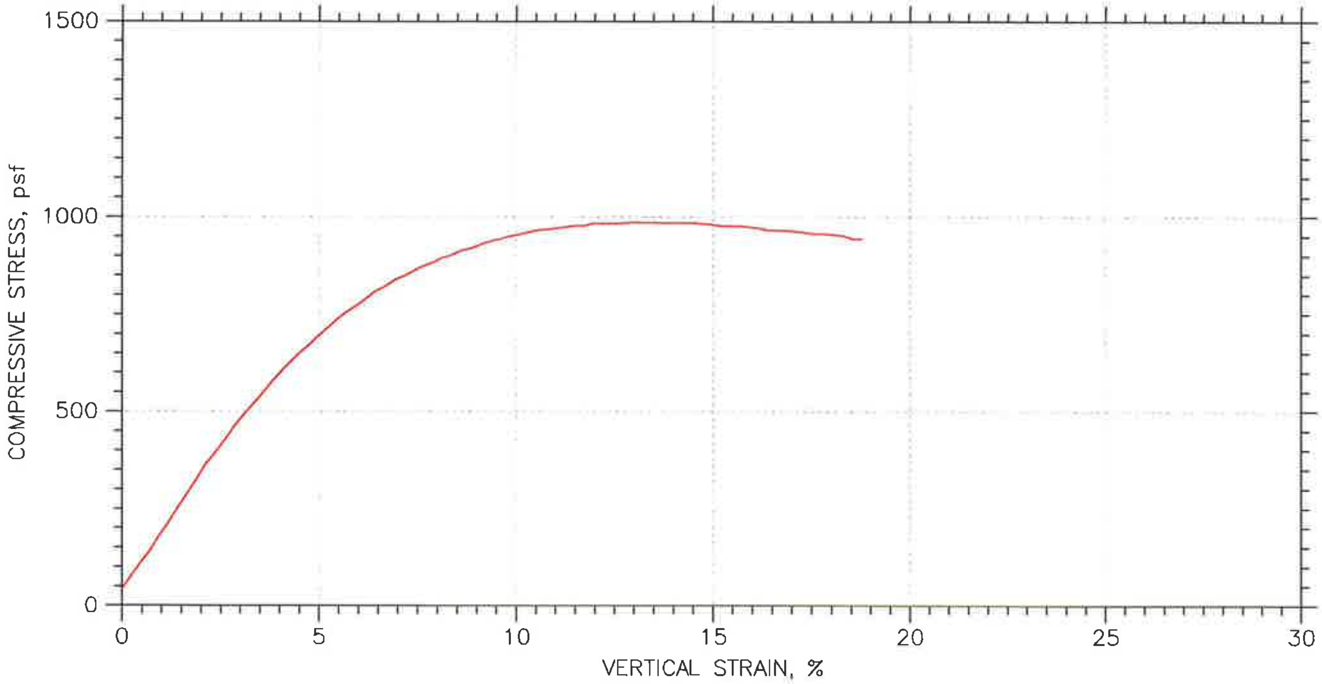
UNCONFINED COMPRESSION TEST REPORT



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

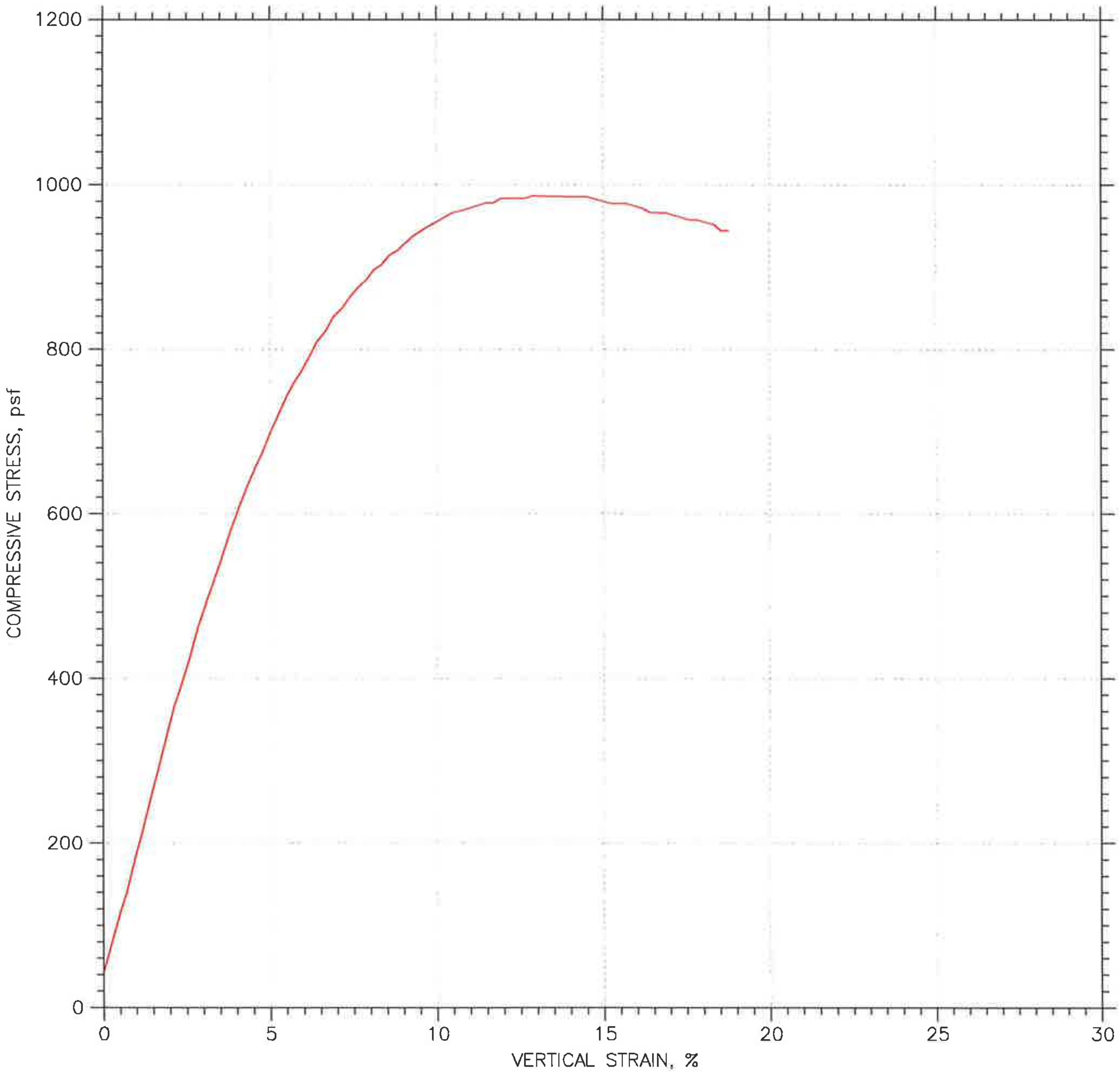


Symbol					
Test No.		13-227			
Initial	Diameter, in	2.38			
	Height, in	5.95			
	Water Content, %	40.10			
	Dry Density, pcf	85.476			
	Saturation, %	113.61			
	Void Ratio	0.93545			
Unconfined Compressive Strength, psf		986.42			
Undrained Shear Strength, psf		493.21			
Time to Failure, min		13.752			
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

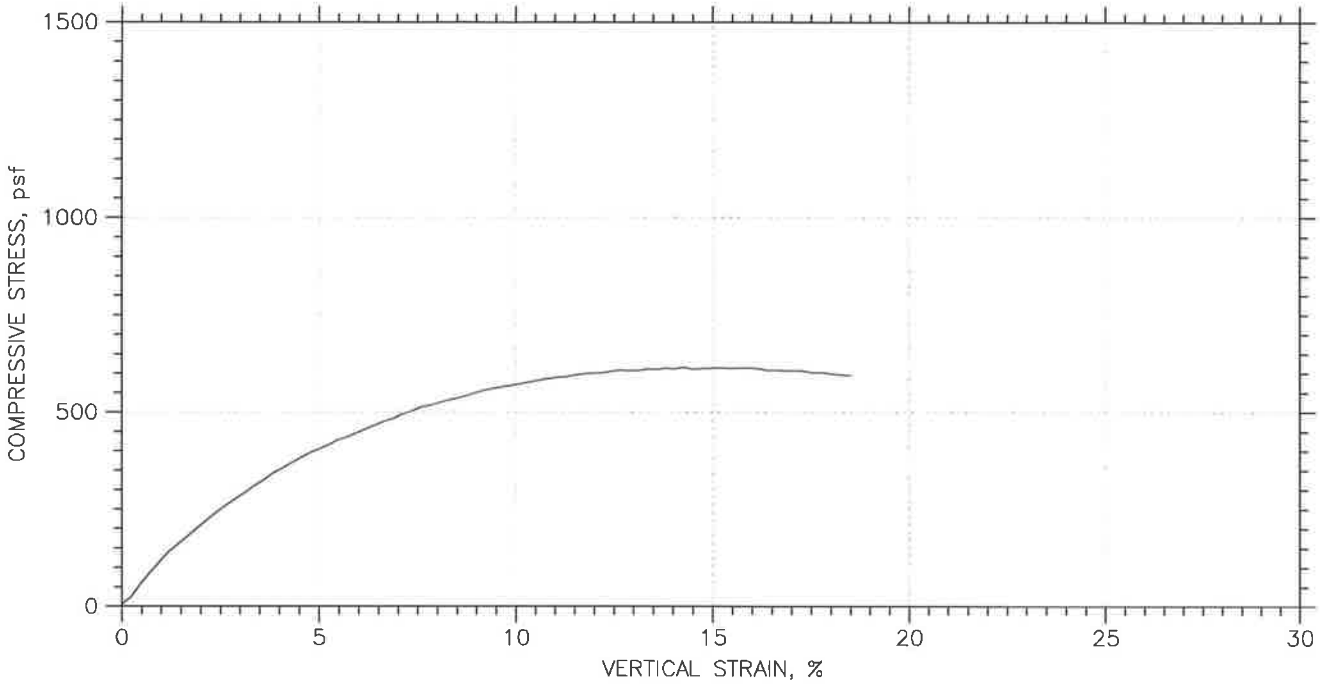
UNCONFINED COMPRESSION TEST REPORT



Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB1@12.75	Tested By: JMA	Checked By: Dh 4/10/13
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'

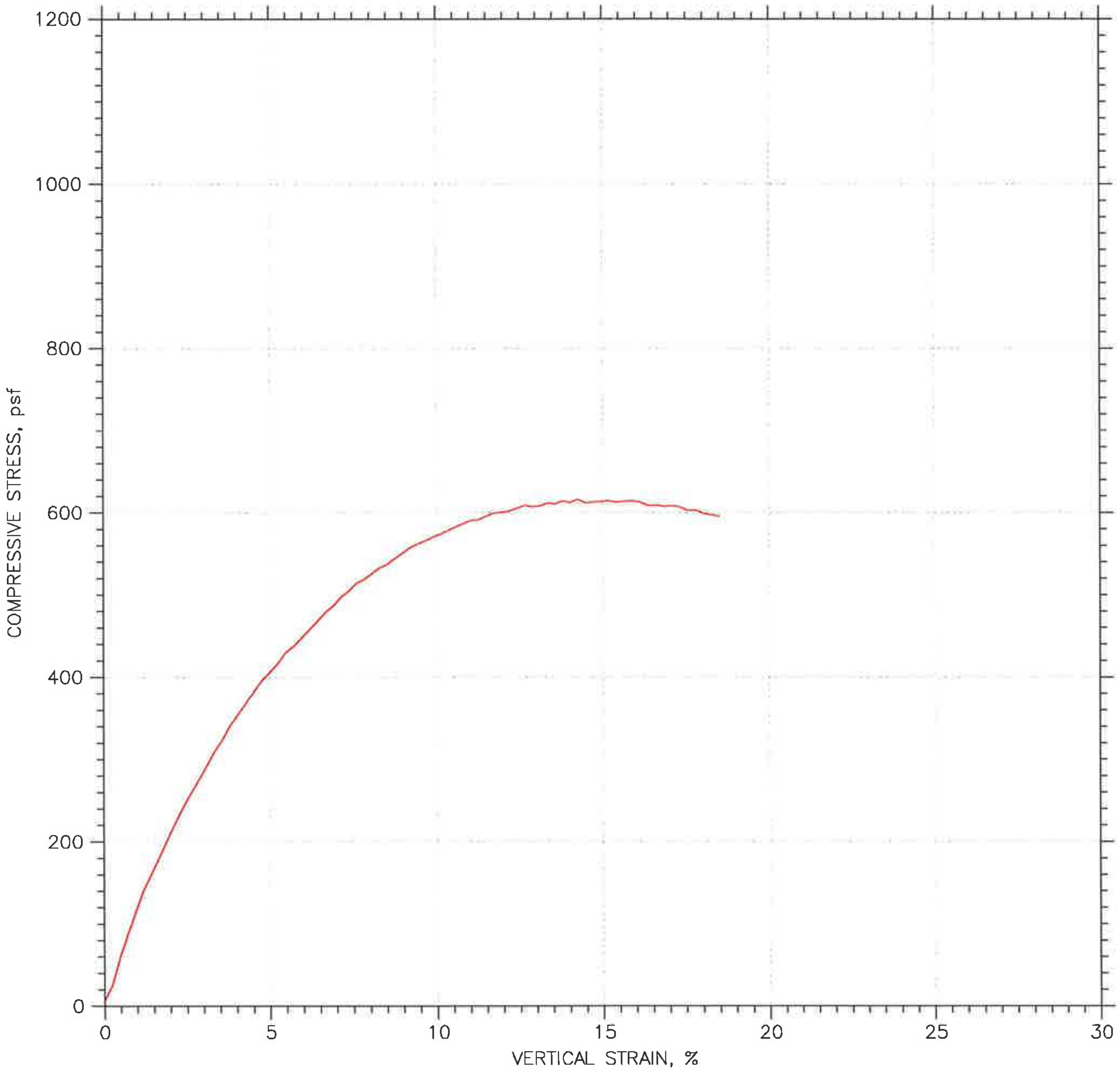
Sample Type: 2.5"shelby

Description: Strong Brown SILT

Remarks: Organics in specimen

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

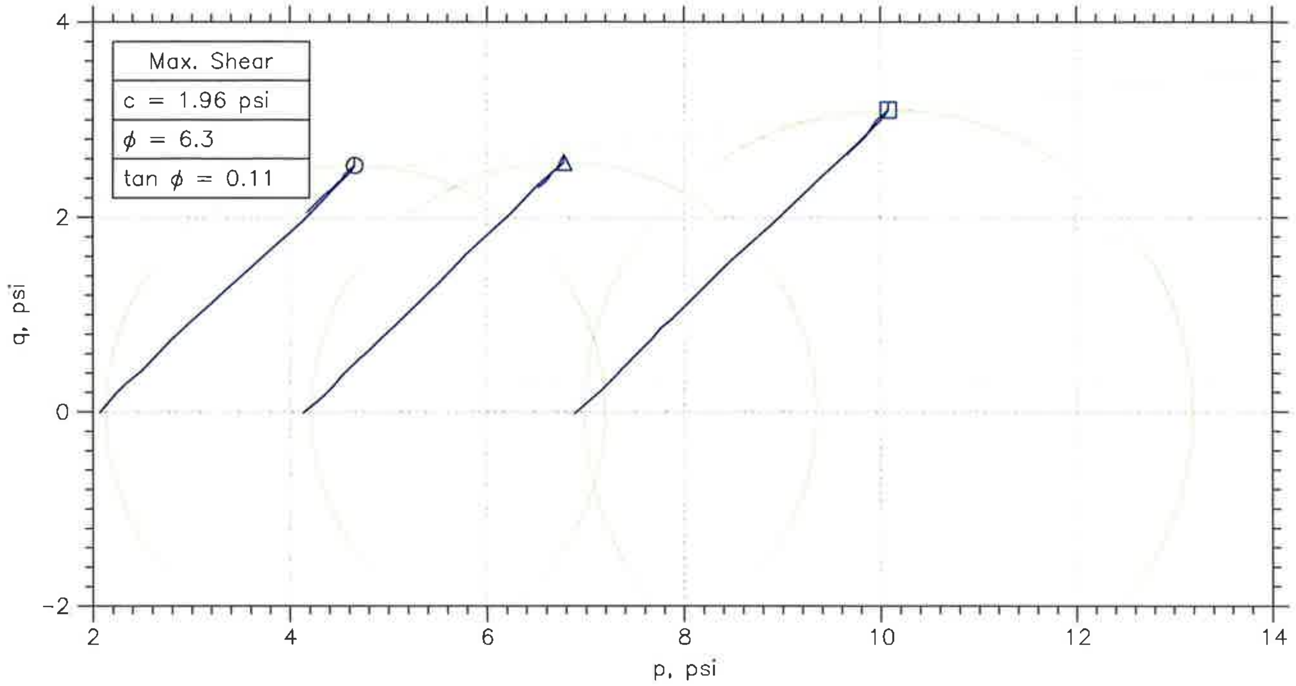
UNCONFINED COMPRESSION TEST REPORT

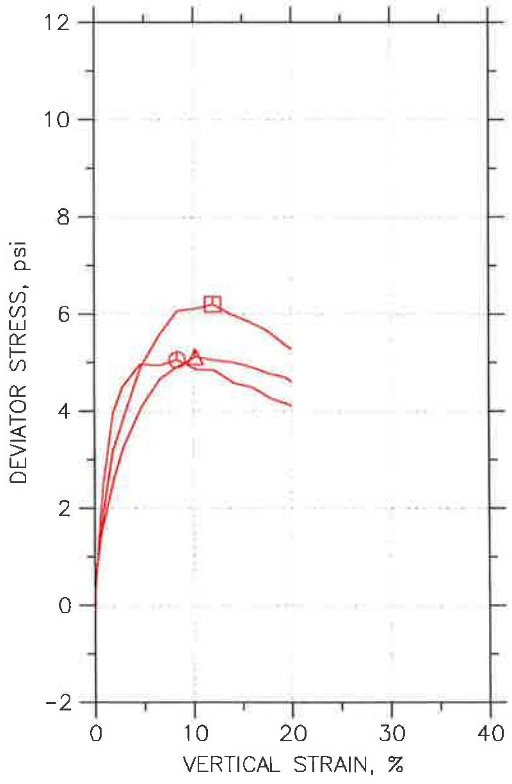


Project: Martin Slough Enhancement	Location: Eureka	Project No.: 013035
Boring No.: HB15@5'	Tested By: JMA	Checked By: Dh 4/18/13
Sample No.: 13-245	Test Date: 4/9/12	Depth: 5-5.5
Test No.: 13-245	Sample Type: 2.5"shelby	Elevation:
Description: Strong Brown SILT		
Remarks: Organics in specimen		

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

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





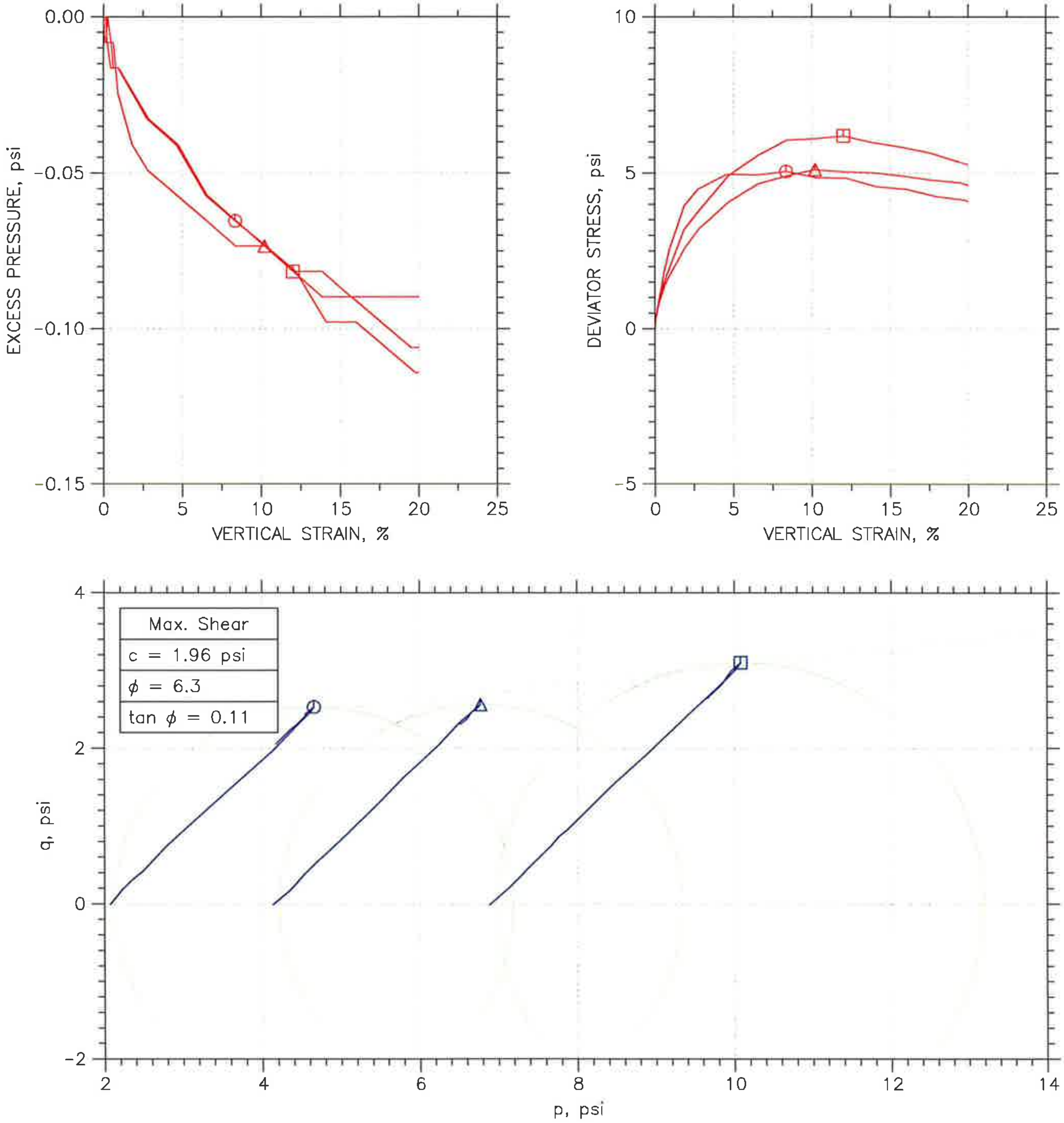
Symbol		○	△	□	
Sample No.		HB3@2.75	HB3@5'	HB4@8.6'	
Test No.		13-229	13-230	13-235	
Depth		2.25-2.75	5-5.5	8.6-9.1	
Initial	Diameter, in	2.38	2.38	2.38	
	Height, in	4.79	5.5	5.65	
	Water Content, %	48.5	43.0	38.3	
	Dry Density, pcf	71.94	79.49	79.71	
	Saturation, %	99.9	106.5	95.3	
	Void Ratio	1.27	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

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

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

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

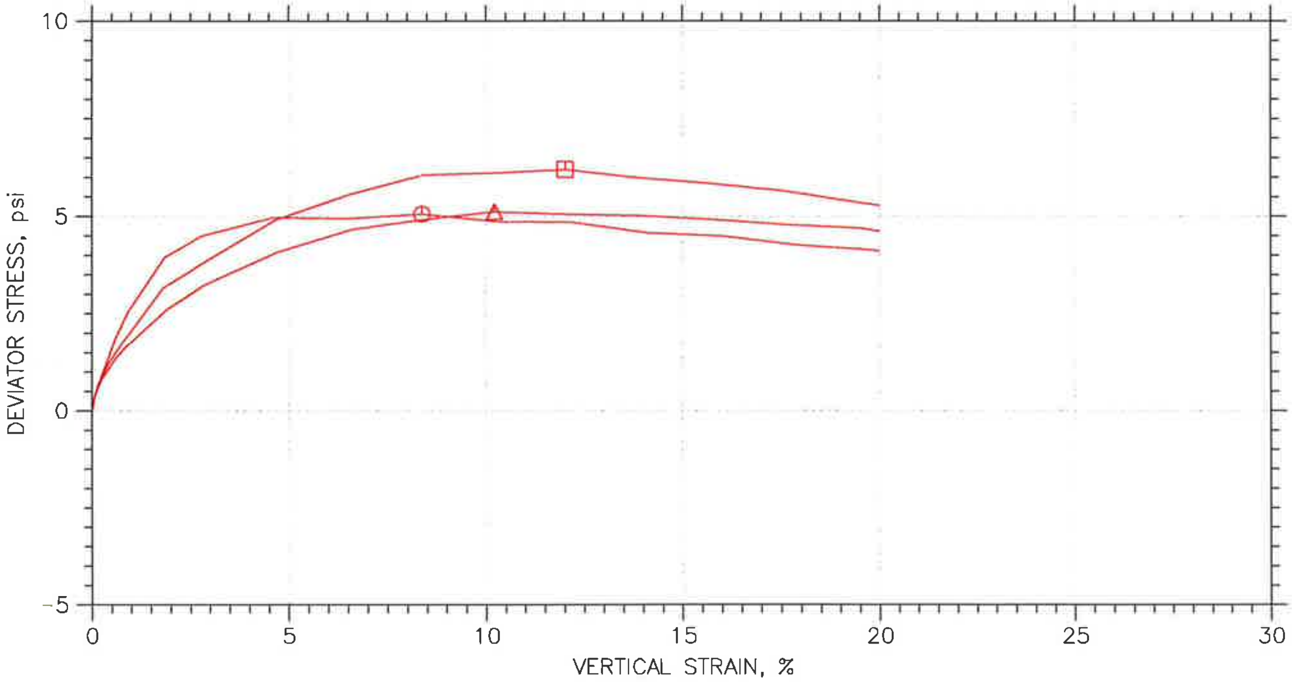


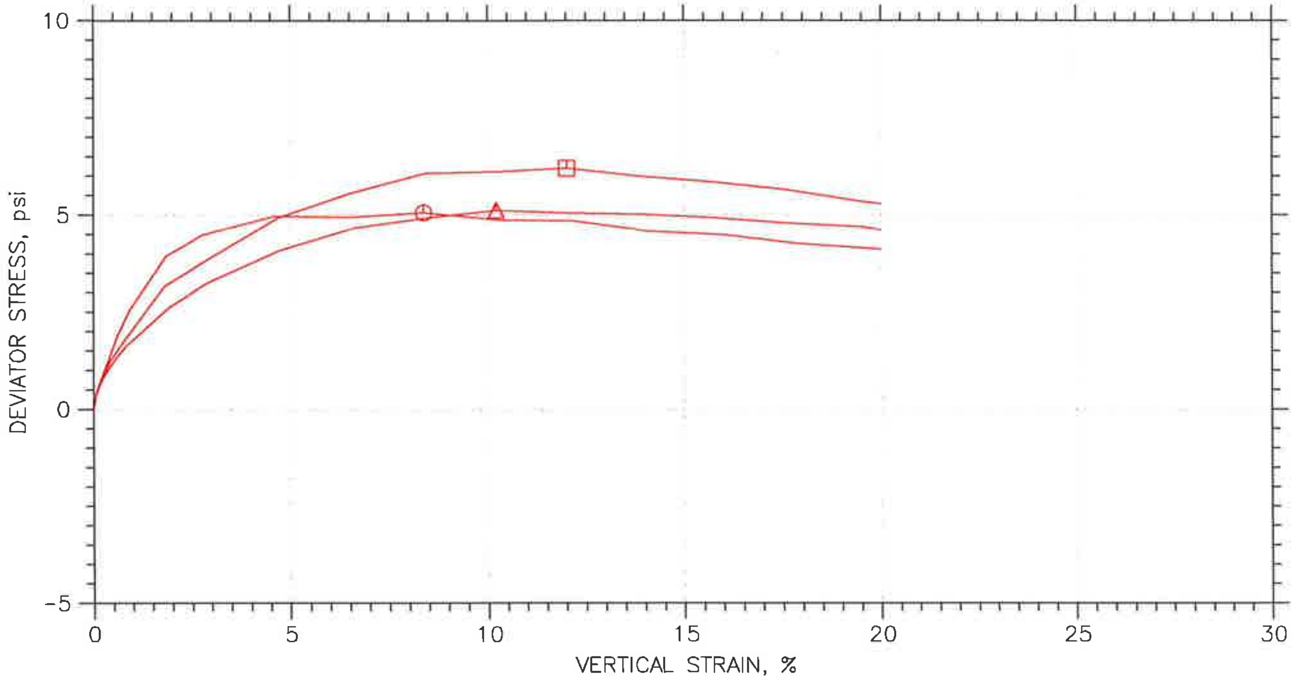
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	HB3@2.75	13-229	2.25-2.75	JMA	4/9/13			13-229 MSE.dat
△	HB3@5'	13-230	5-5.5	JMA	4/10/13			13-230 MSE.dat
□	HB4@8.6'	13-235	8.6-9.1	JMA	4/10/13			13-235 MSE.dat

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



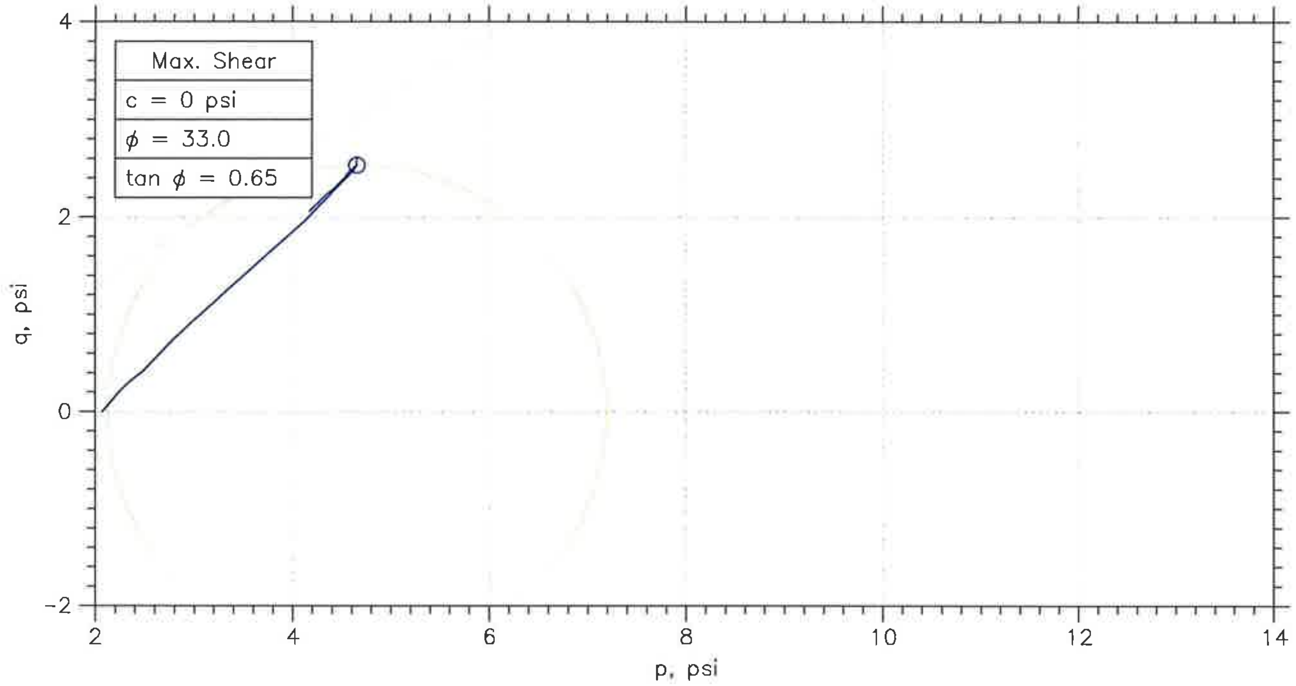


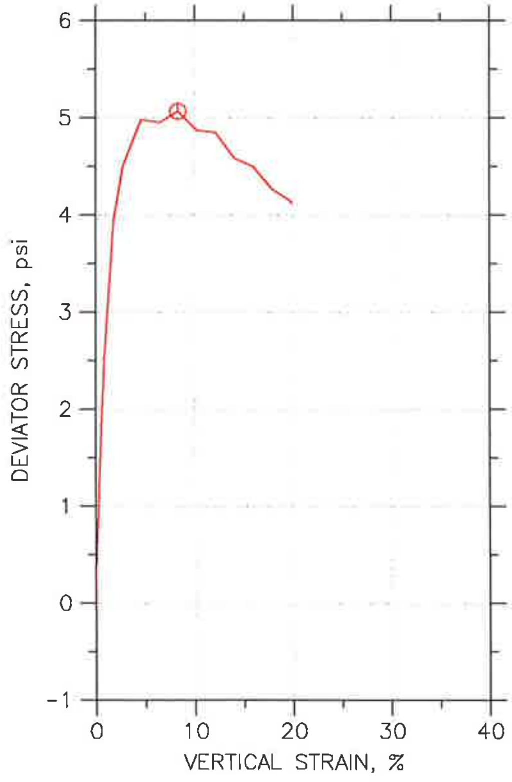
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	HB3@2.75	13-229	2.25-2.75	JMA	4/9/13			13-229 MSE.dat
△	HB3@5'	13-230	5-5.5	JMA	4/10/13			13-230 MSE.dat
□	HB4@8.6'	13-235	8.6-9.1	JMA	4/10/13			13-235 MSE.dat

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		---			

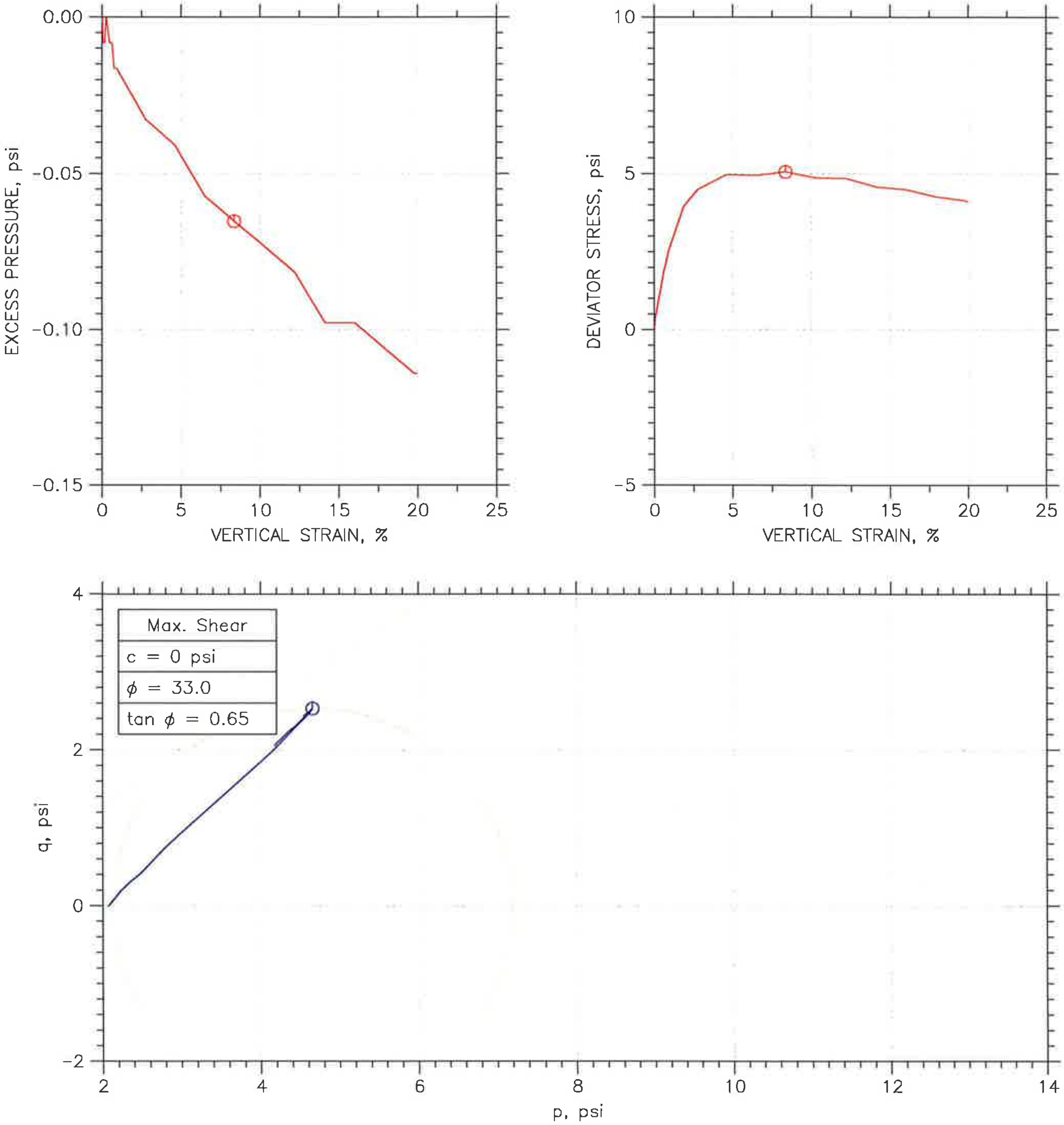
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB3@2.25
Sample Type: 2.5"calbrl
Description: SILT
Remarks: Unconsolidated Undrained

Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

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

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

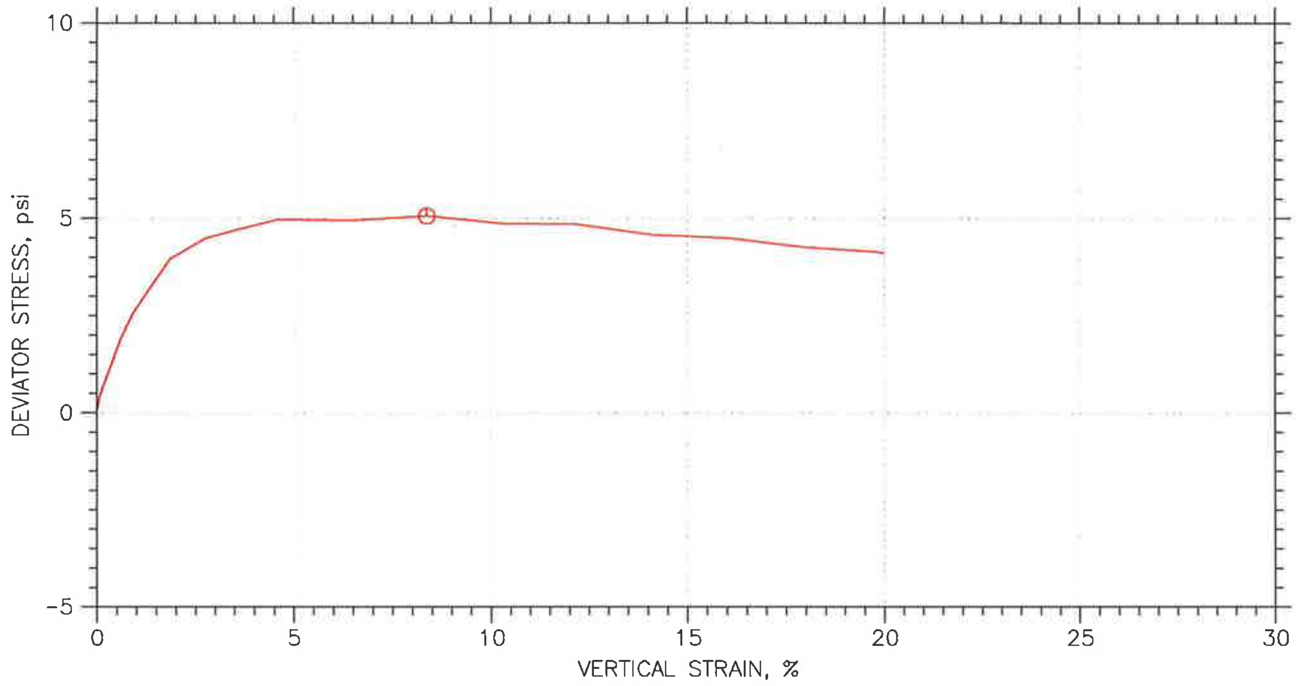


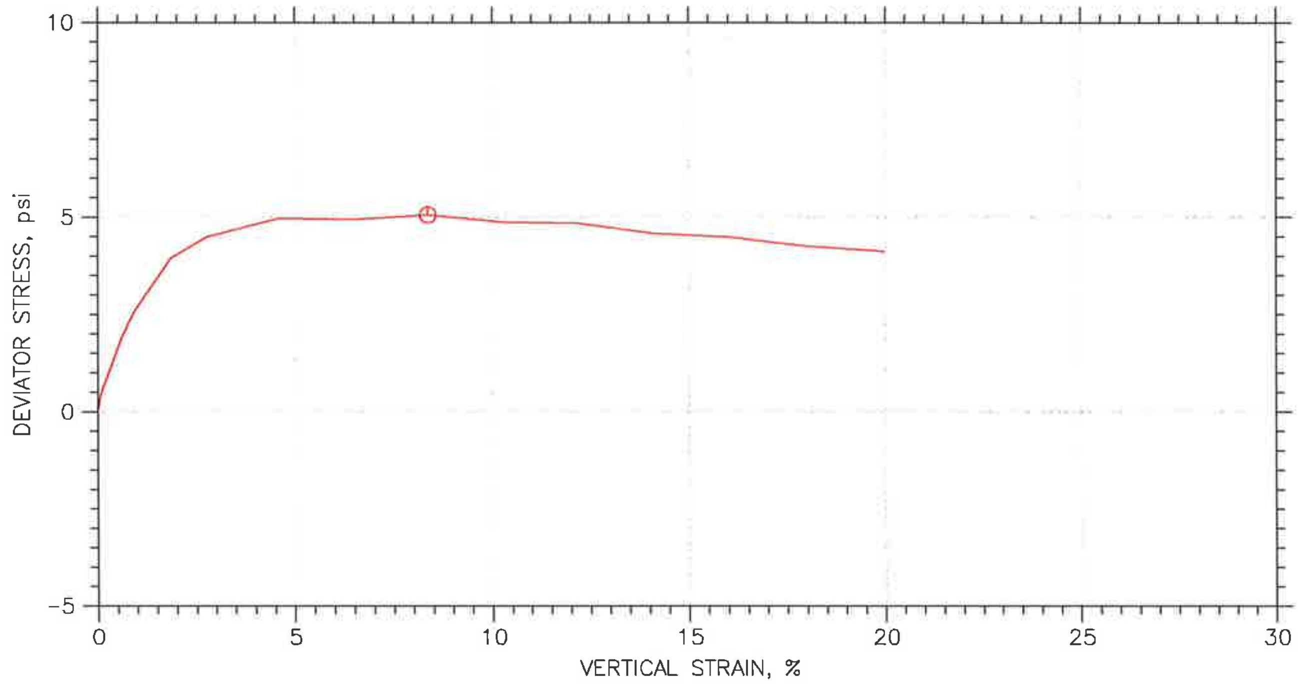
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB3@2.75	13-229	2.25-2.75	JMA	4/9/13			13-229 MSE.dat

Project: Martin Slough 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



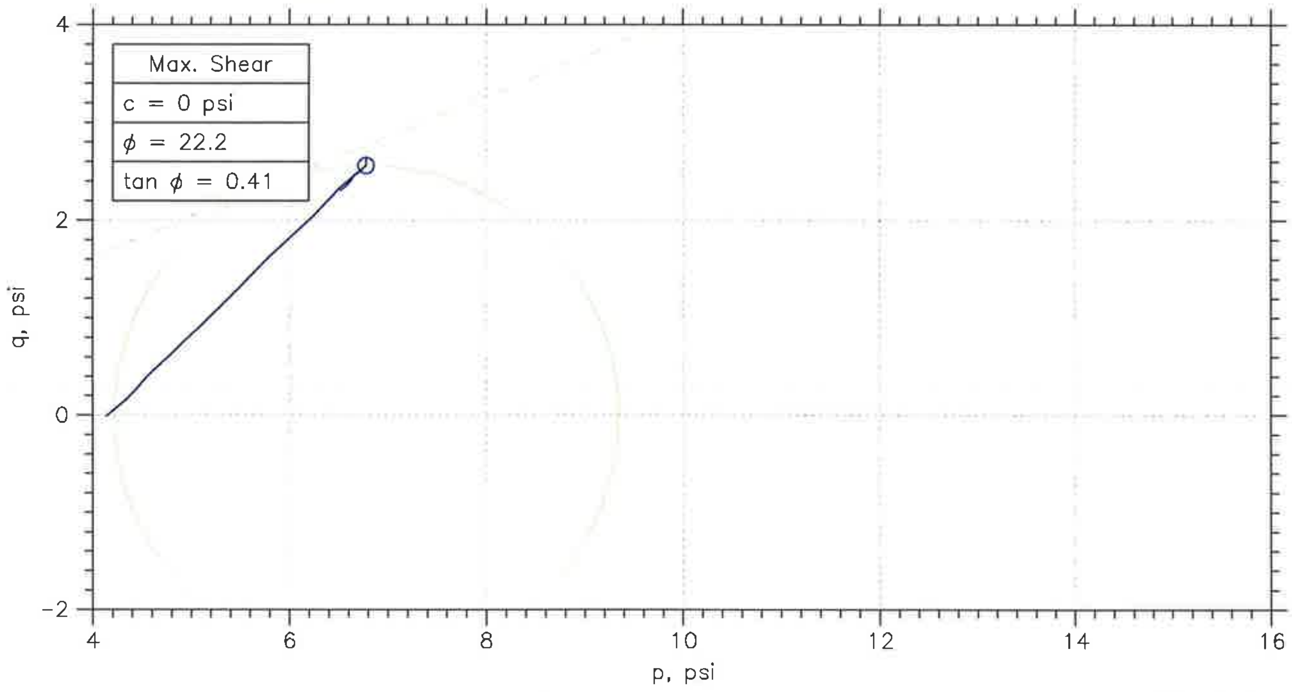


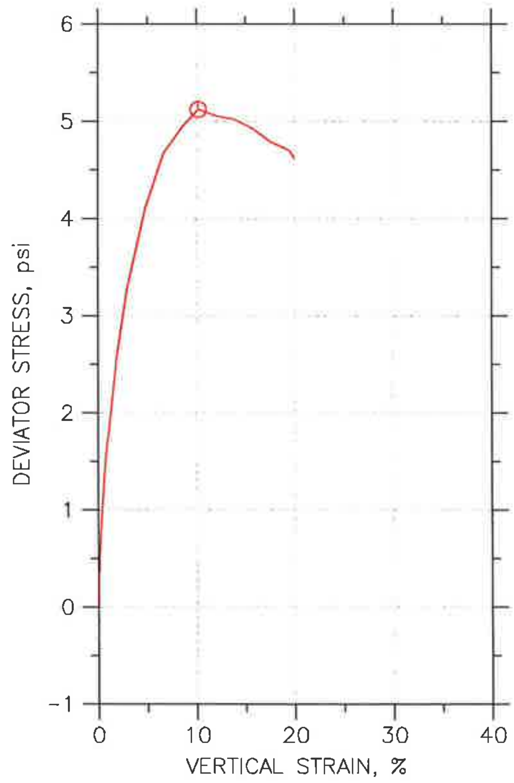
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB3@2.75	13-229	2.25-2.75	JMA	4/9/13			13-229 MSE.dat

Project: Martin Slough Enhanceme	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

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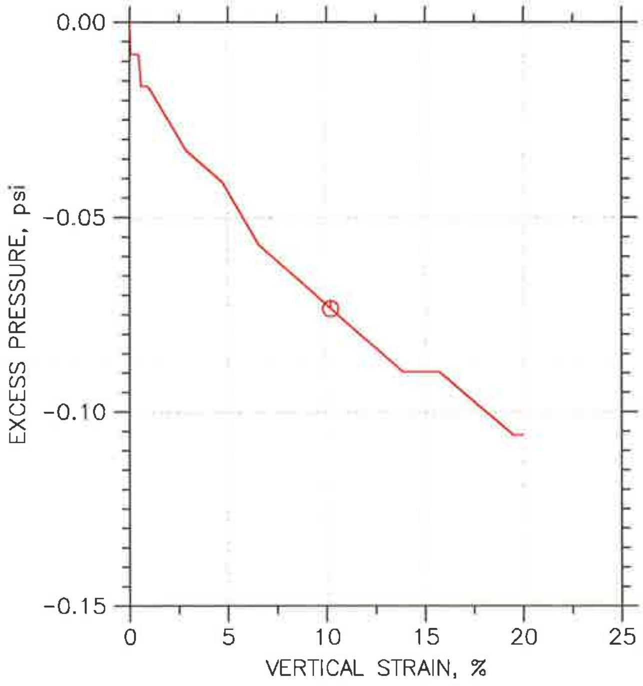
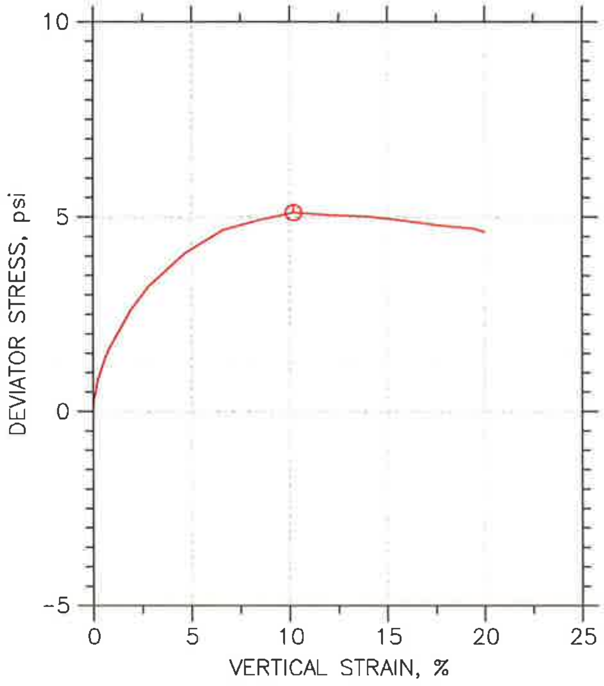
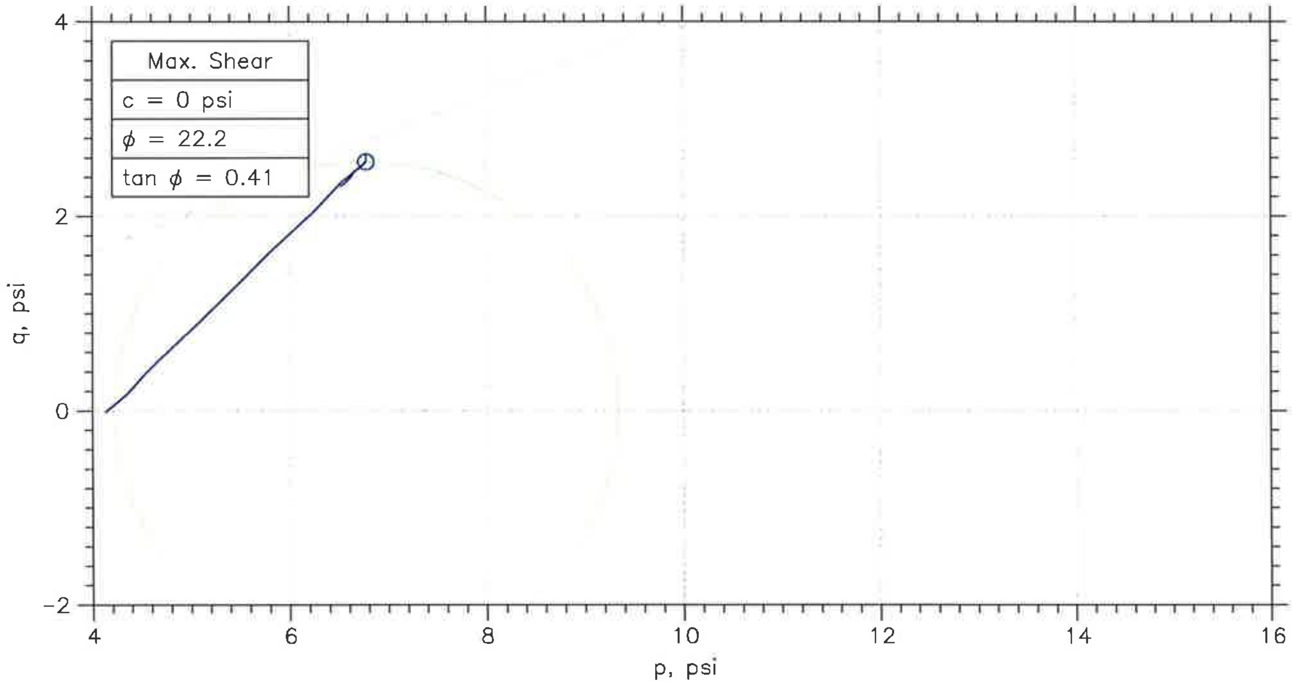
Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





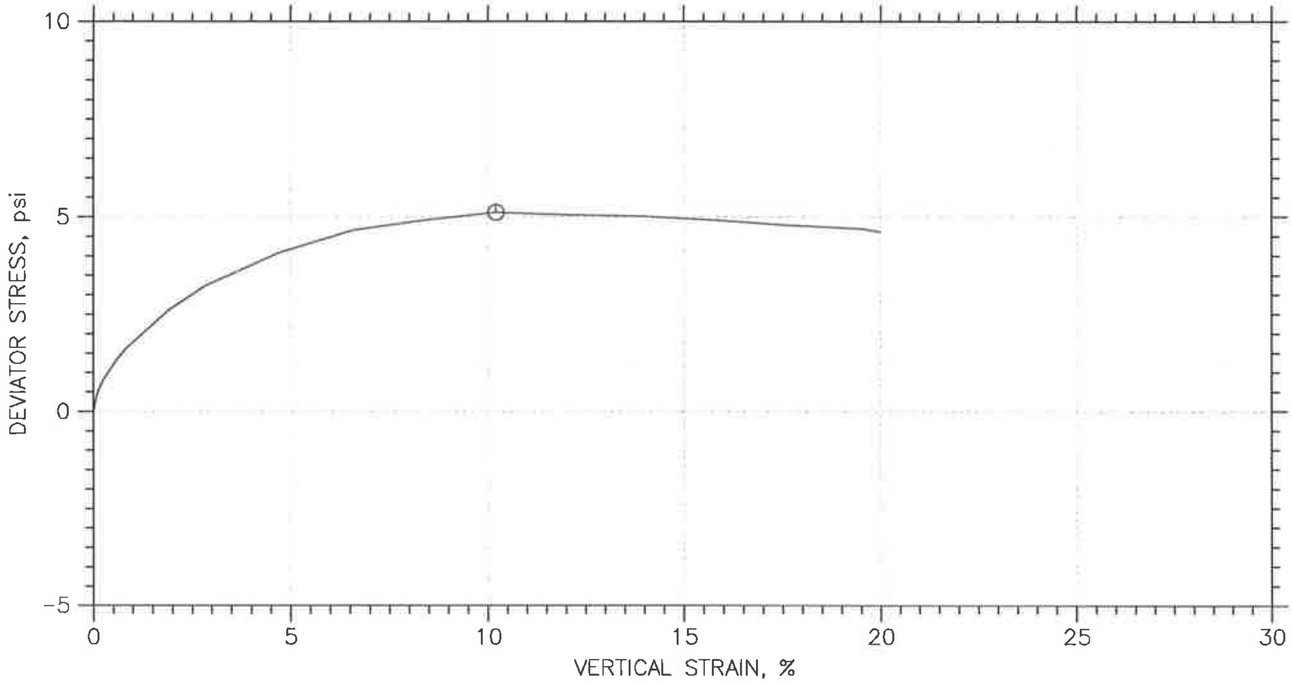


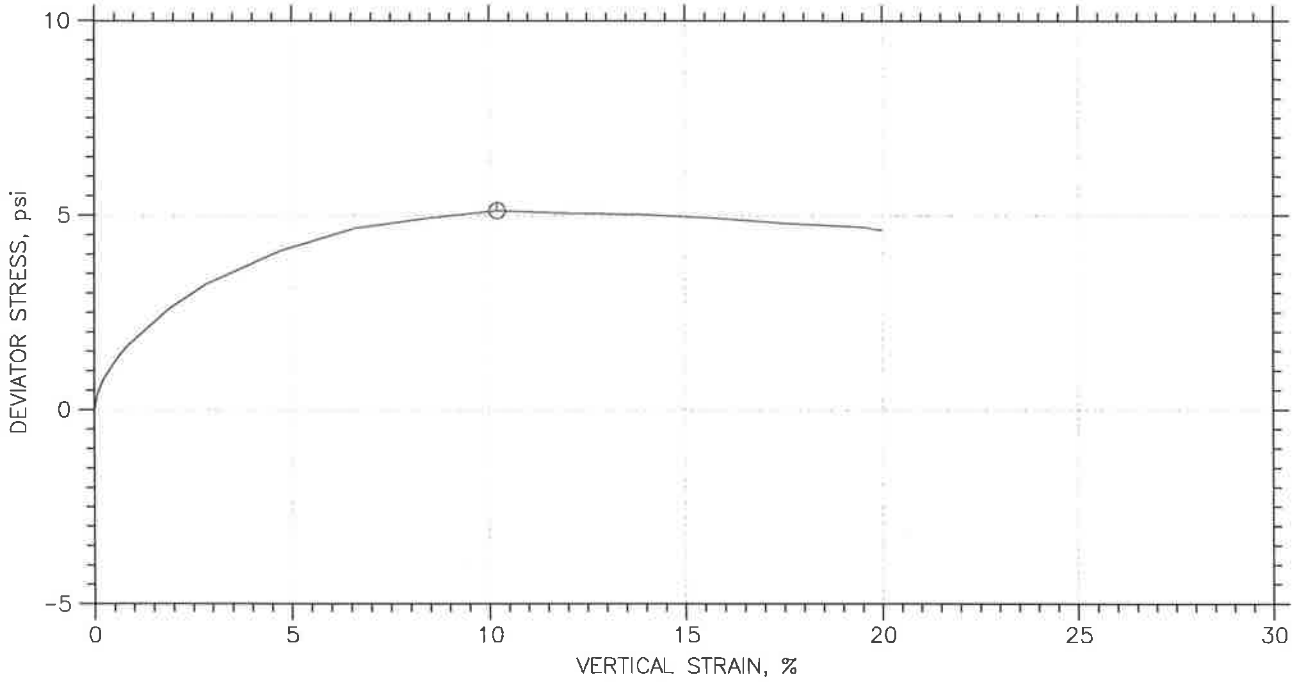
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB3@5'	13-230	5-5.5	JMA	4/10/13			13-230 MSE.dat

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
○	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		

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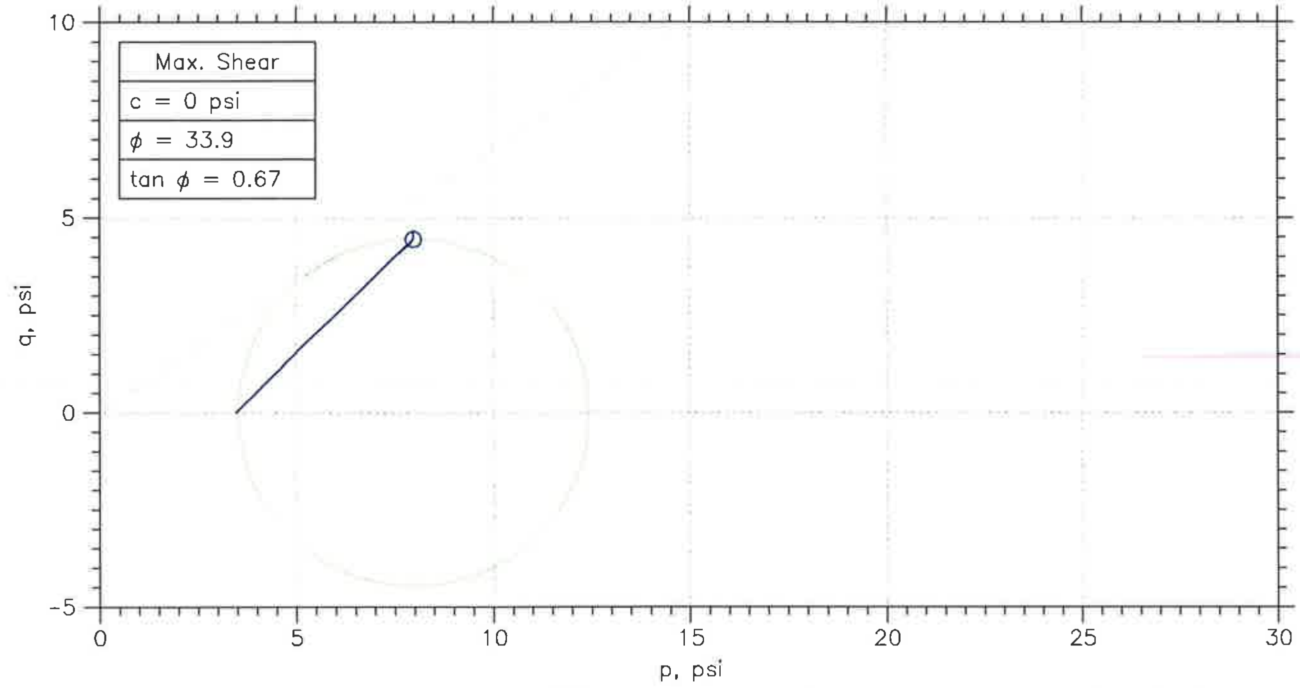
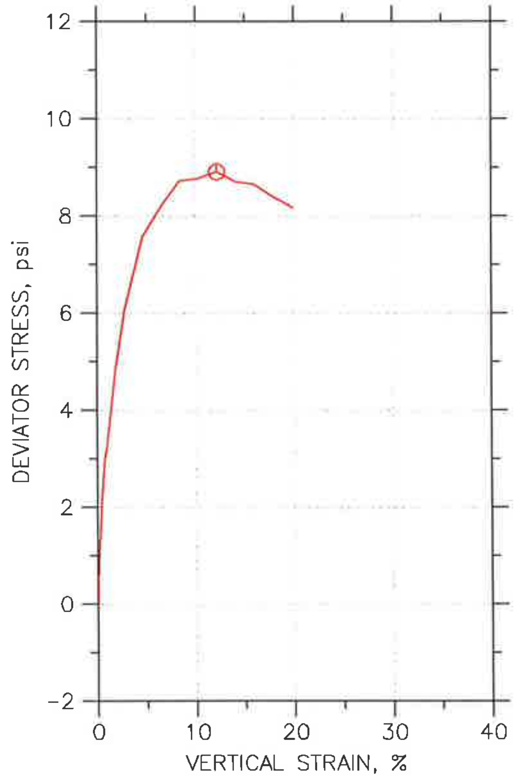
UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





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		---			

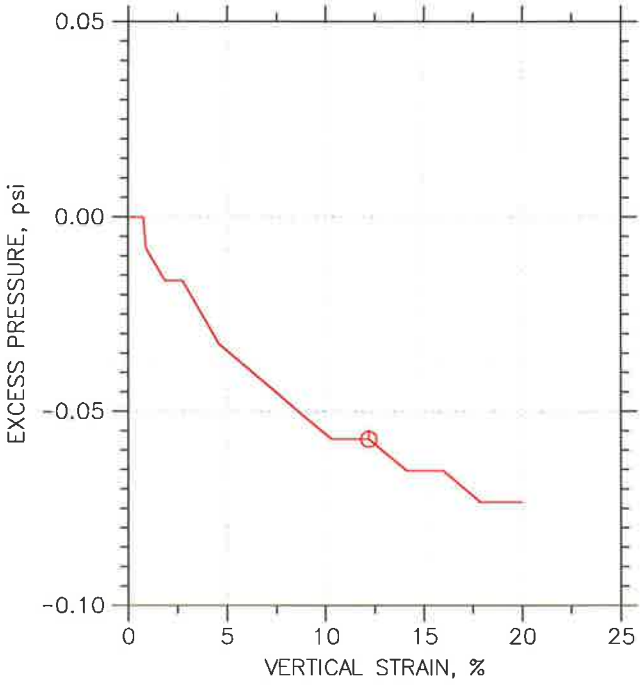
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB4@3.2-3.7
Sample Type: 2.5"calbrl
Description: SILT
Remarks: Unconsolidated Undrained

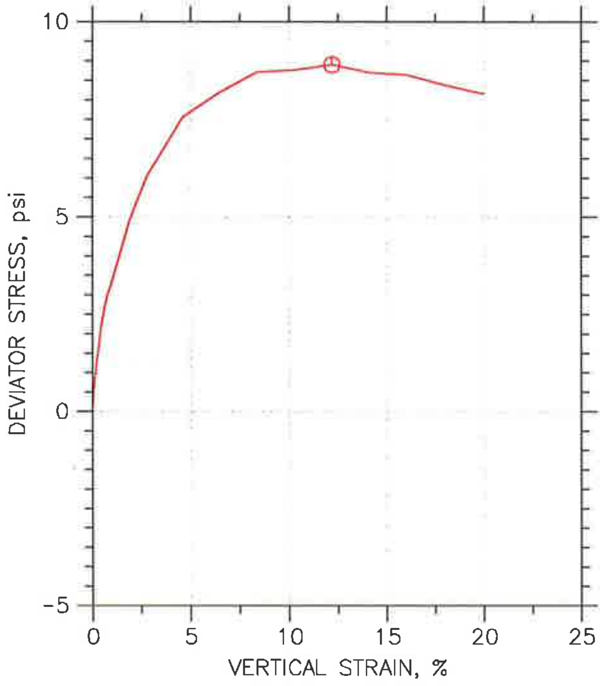
Thu, 18-APR-2013 12:24:31

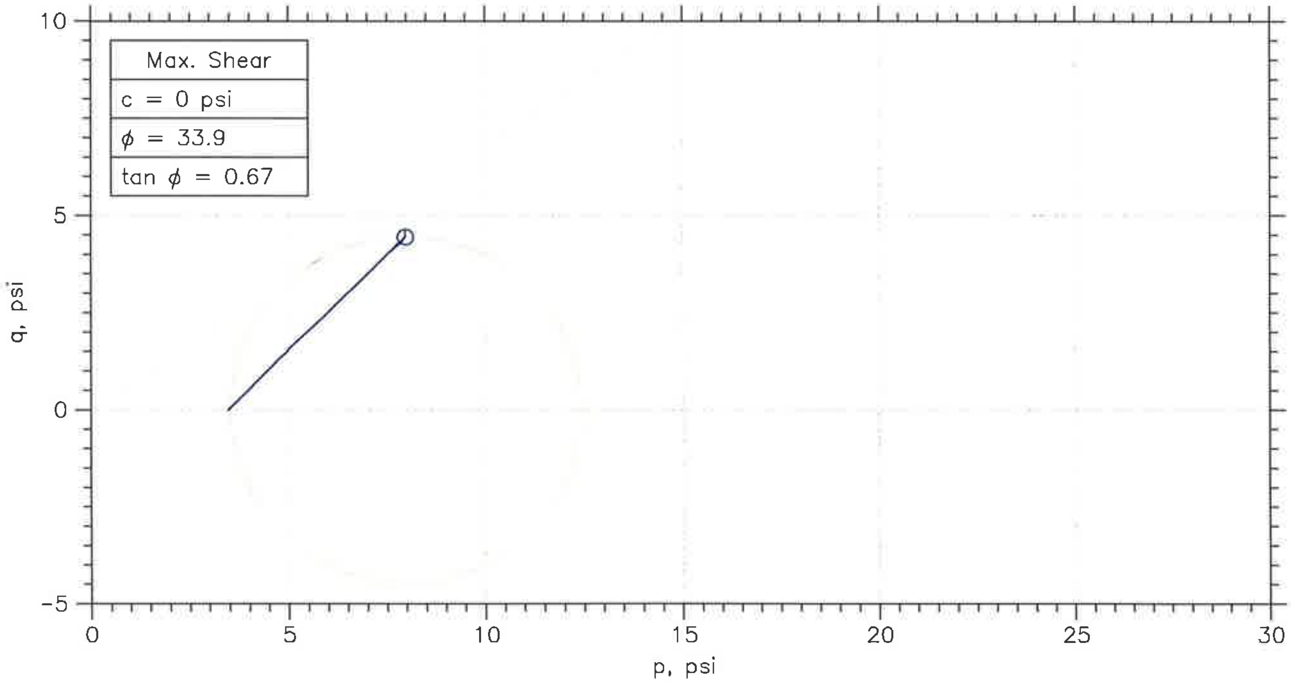
Phase calculations based on start and end of test.

* Saturation is set to 100% for phase calculations.

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





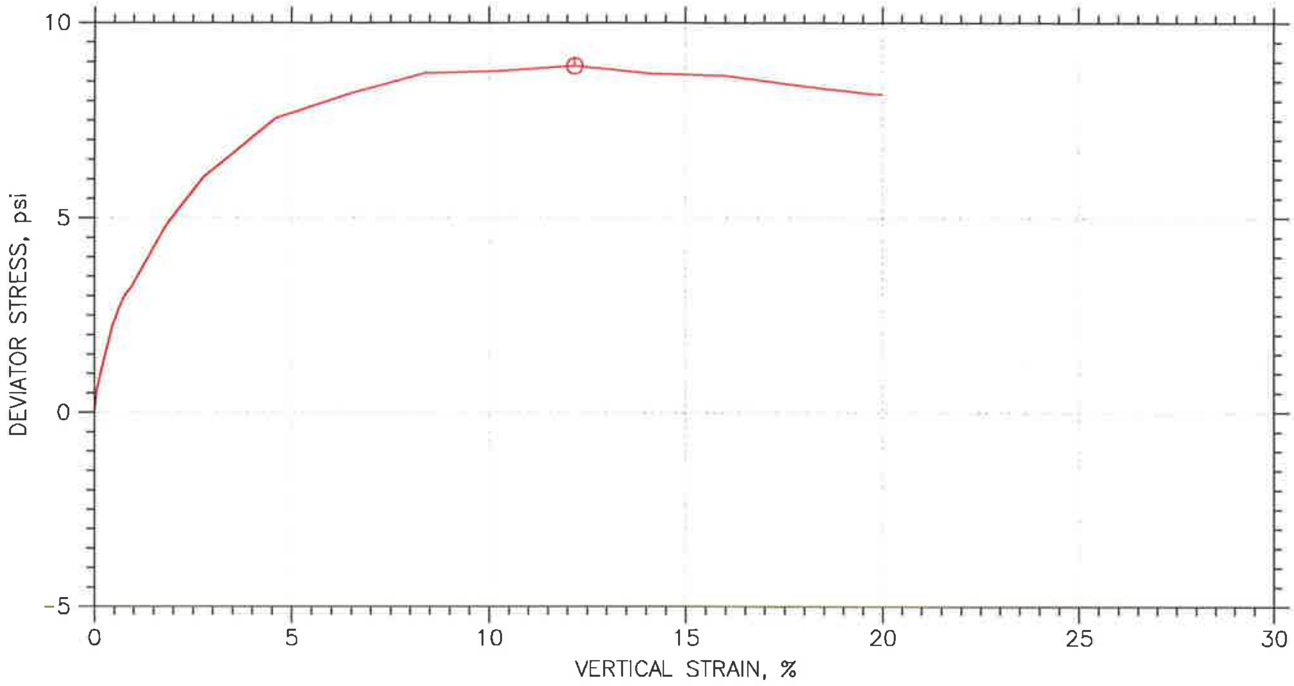


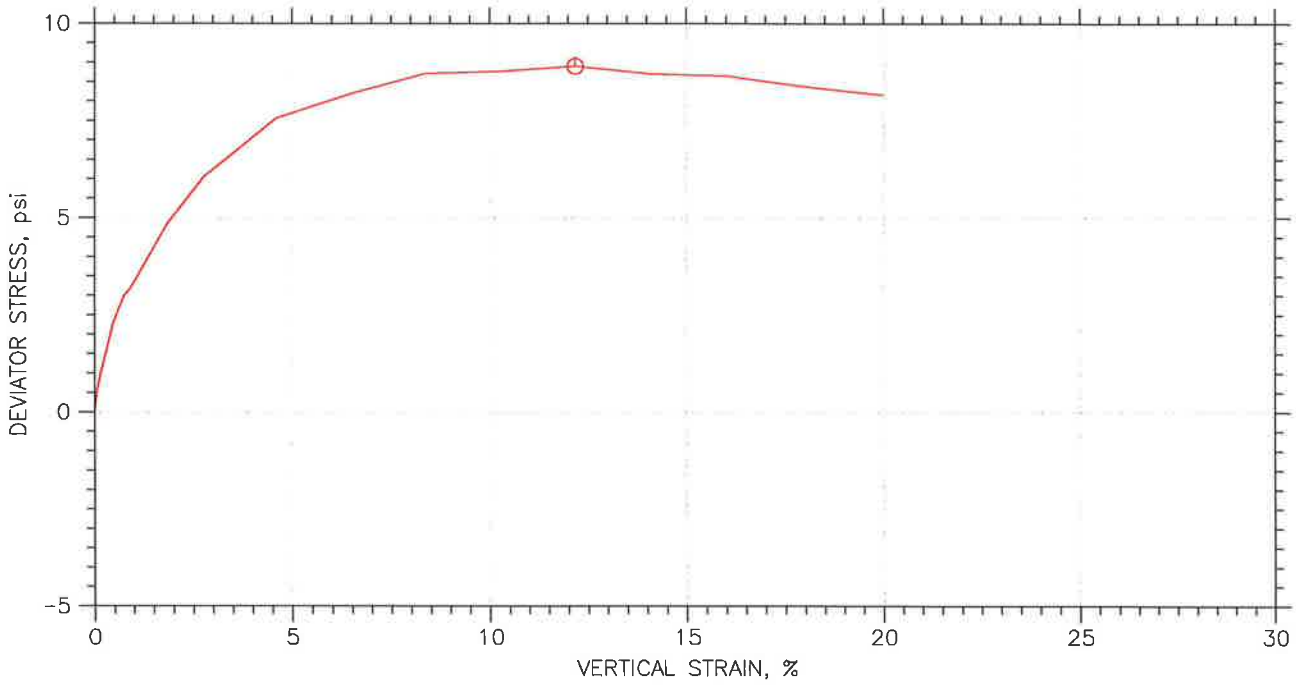
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB4@3.2	13-233	3.2-3.7	JMA	4/10/13			13-233 MSE.dat

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



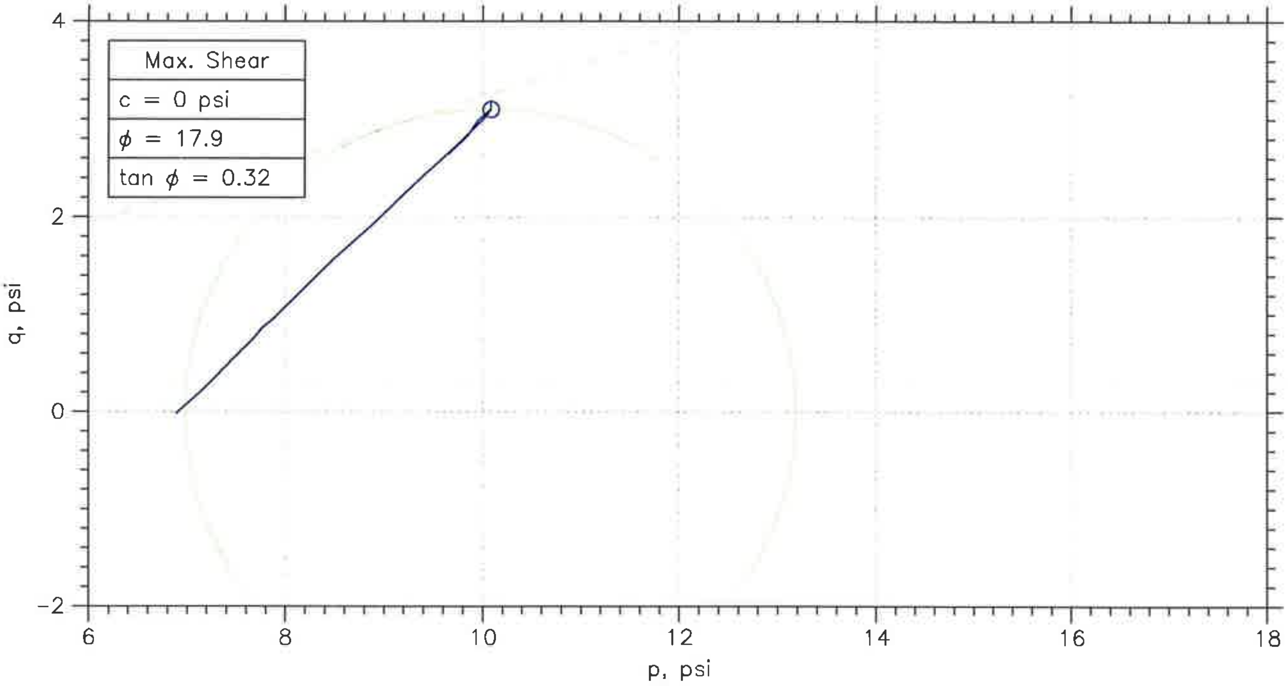


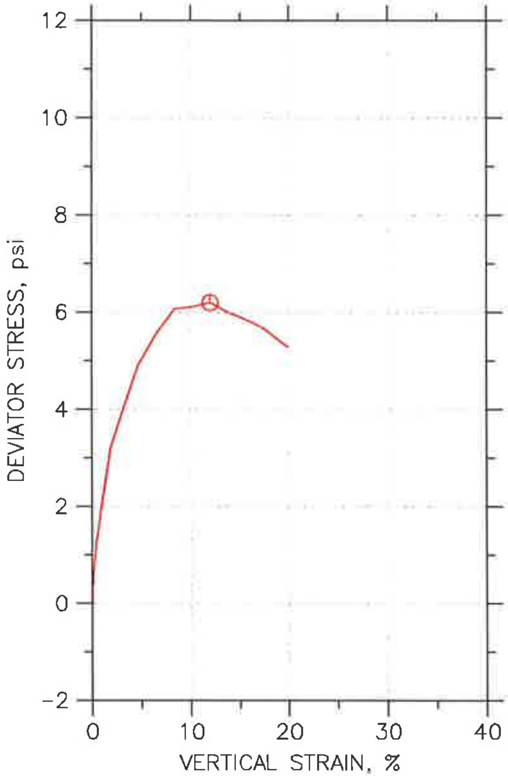
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⏀	HB4@3.2	13-233	3.2-3.7	JMA	4/10/13			13-233 MSE.dat

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		---			

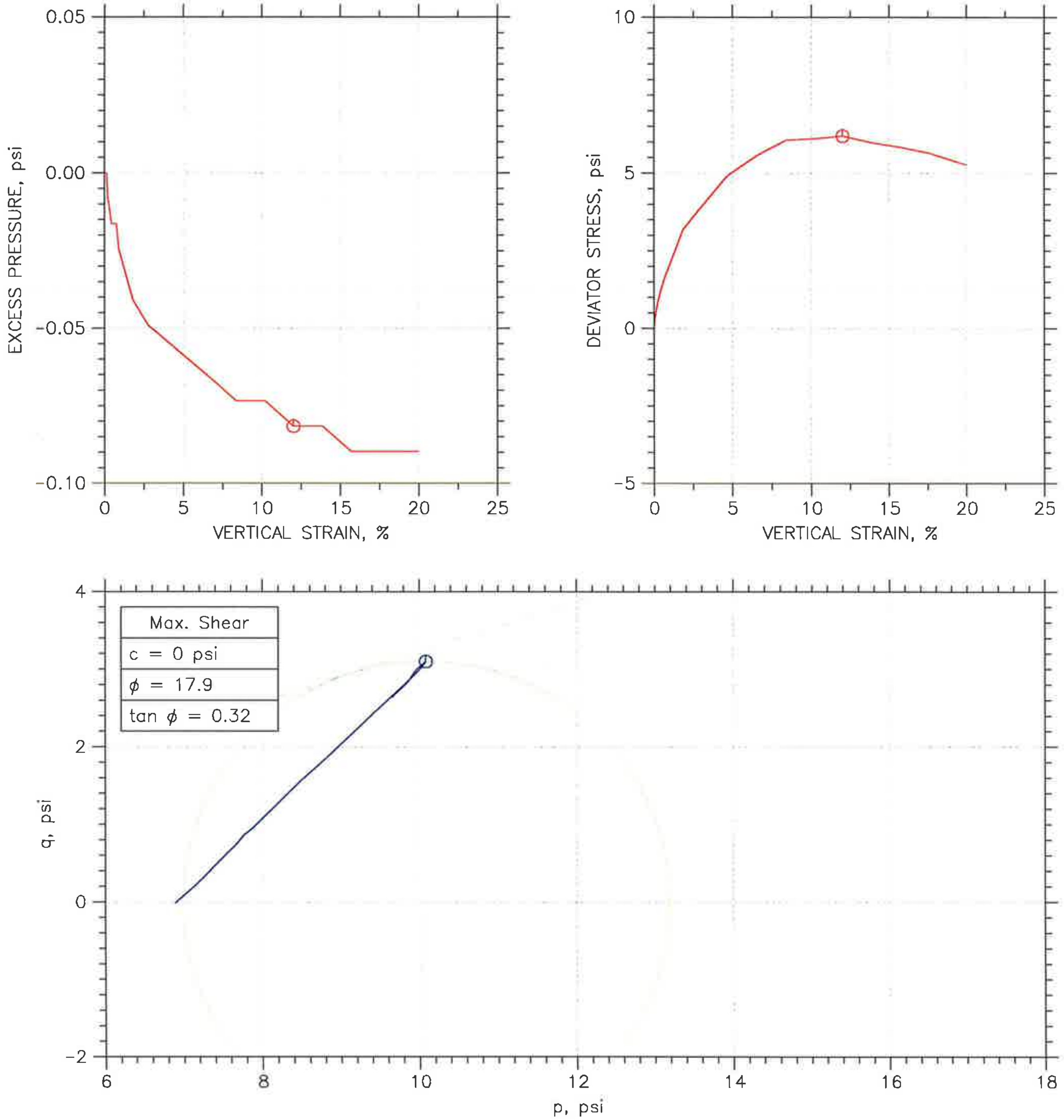
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB3
Sample Type: 2.5"calbrl
Description: Blue Gray SILT
Remarks: Unconsolidated Undrained

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

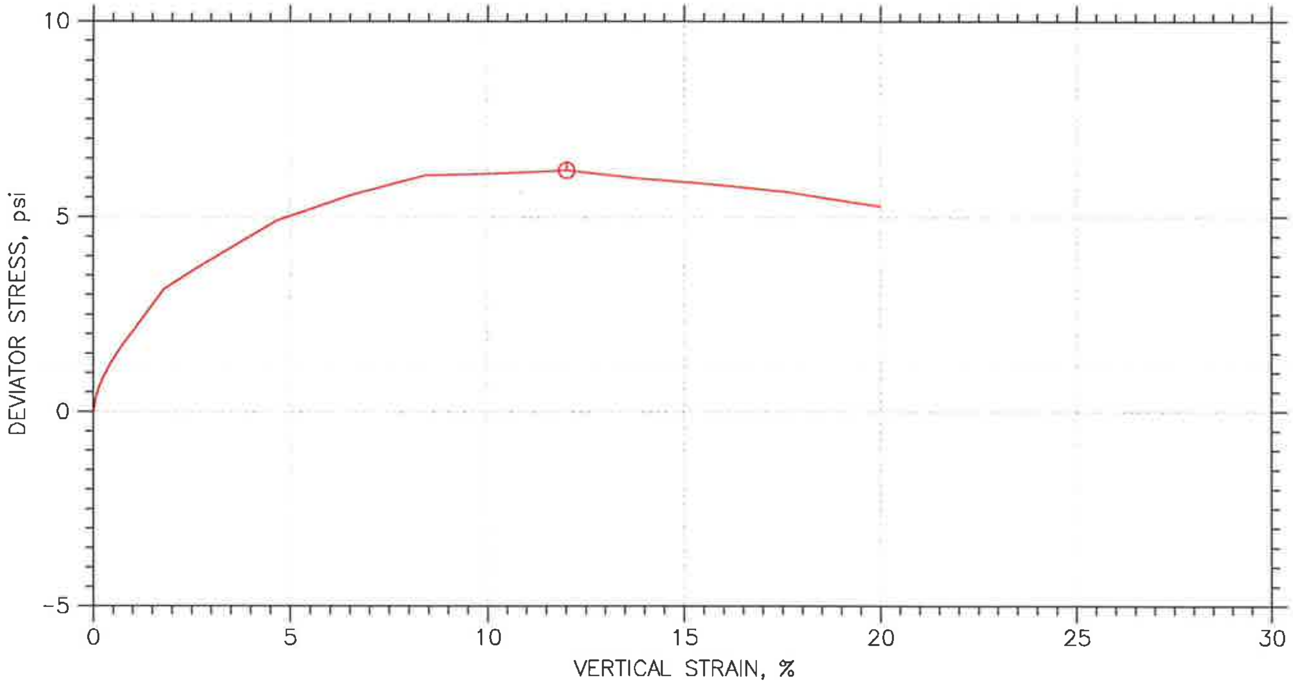


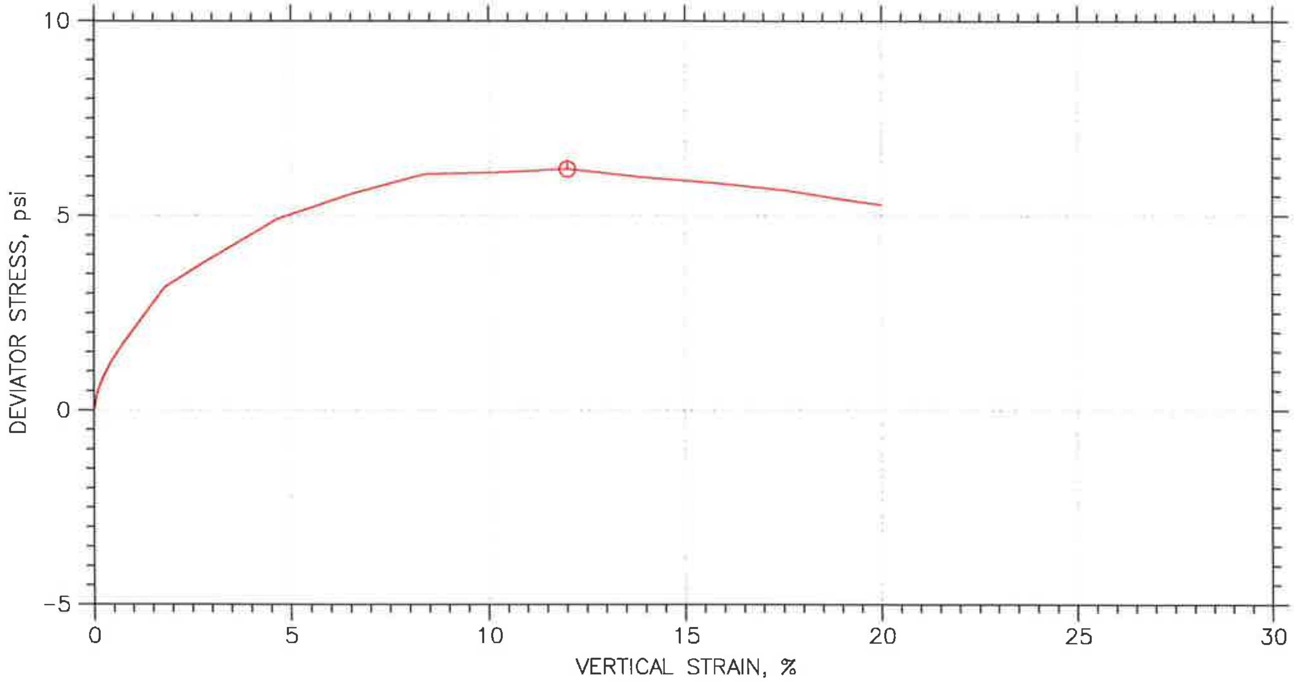
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⏀	HB4@8.6'	13-235	8.6-9.1	JMA	4/10/13			13-235 MSE.dat

Project: Martin Slough Enhanceme	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



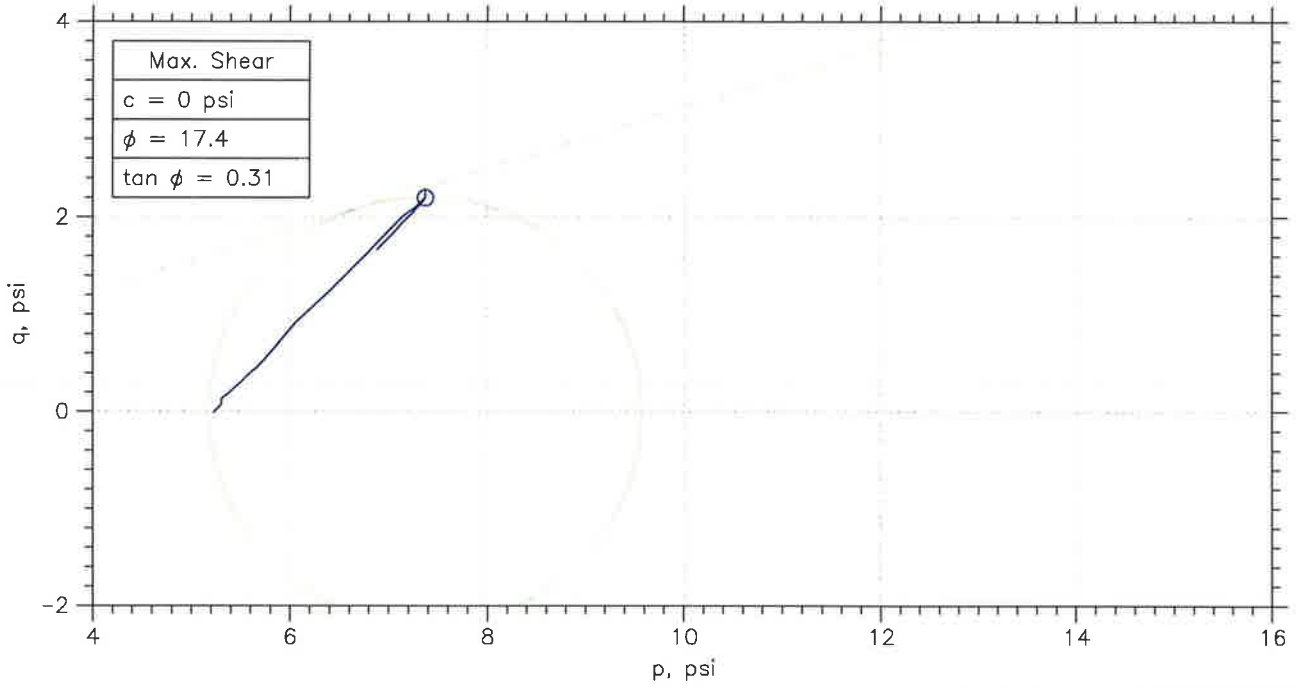


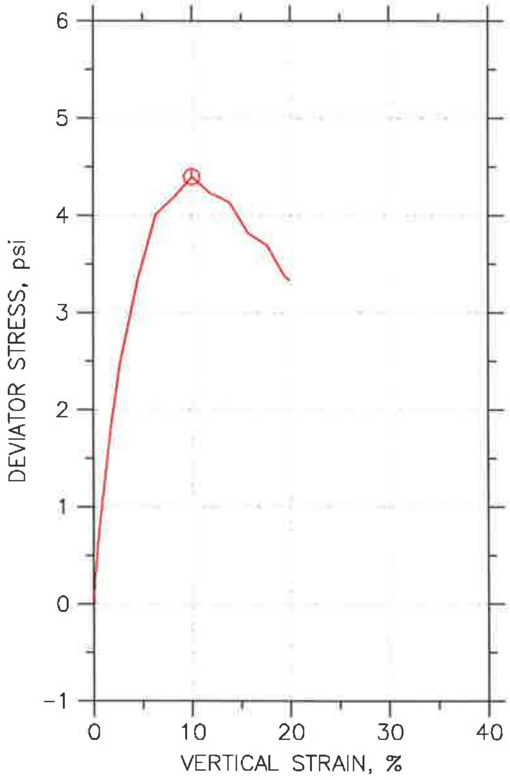
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
⦶	HB4@8.6'	13-235	8.6-9.1	JMA	4/10/13			13-235 MSE.dat

Project: Martin Slough Enhancem	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





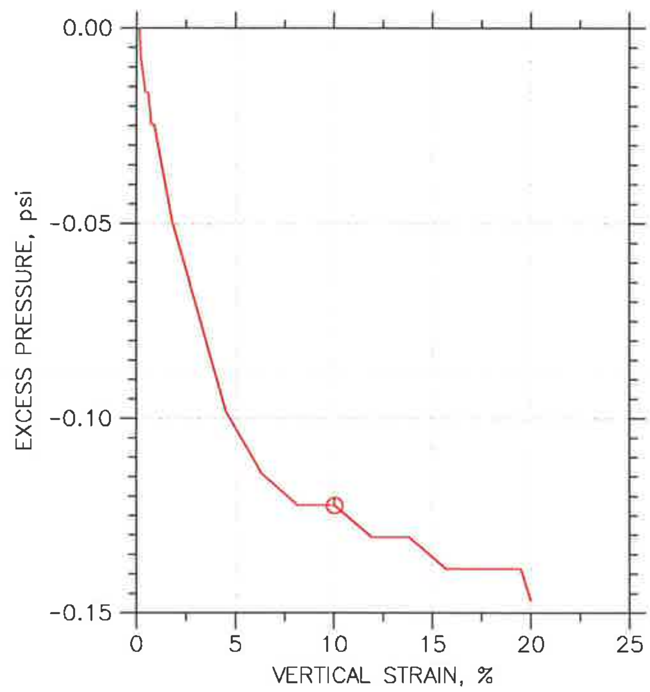
Symbol		⦶			
Sample No.		HB12@5'			
Test No.		13-243			
Depth		5-5.5			
Initial	Diameter, in	2.38			
	Height, in	5.07			
	Water Content, %	45.3			
	Dry Density, pcf	73.01			
	Saturation, %	95.6			
	Void Ratio	1.24			
Before Shear	Water Content, %	45.3			
	Dry Density, pcf	73.17			
	Saturation*, %	96.1			
	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		---			

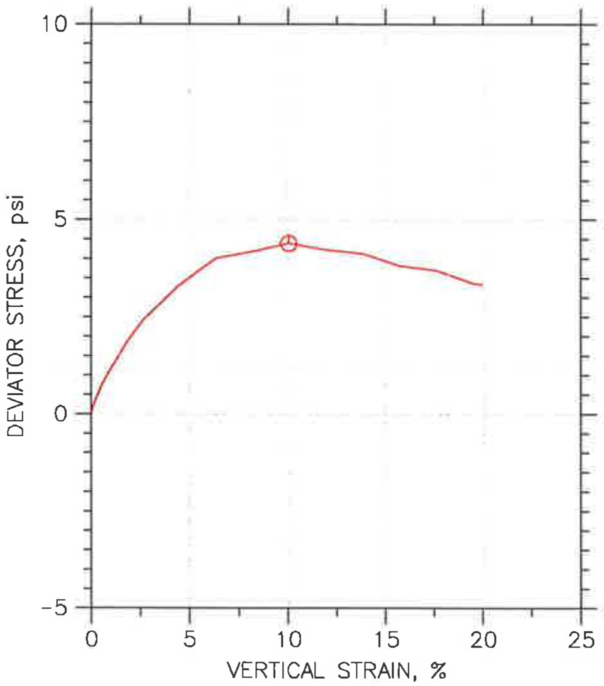
Project: Martin Slough Enhancement
Location: Eureka
Project No.: 013035
Boring No.: HB12@5-5.5
Sample Type: 2.5"calbrl
Description: Brown SILT
Remarks: Unconsolidated undrained

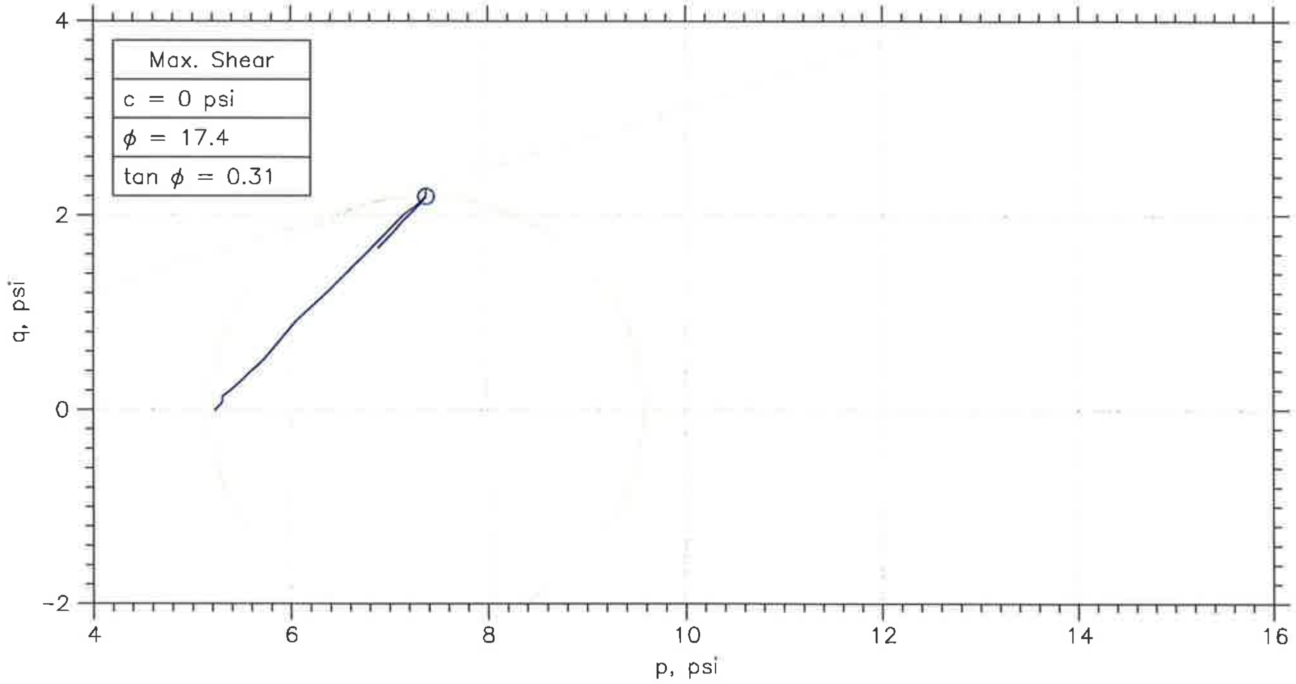
Phase calculations based on start and end of test.
* Saturation is set to 100% for phase calculations.

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

UNCONSOLIDATED UNDRAINED TRIAXIAL TEST





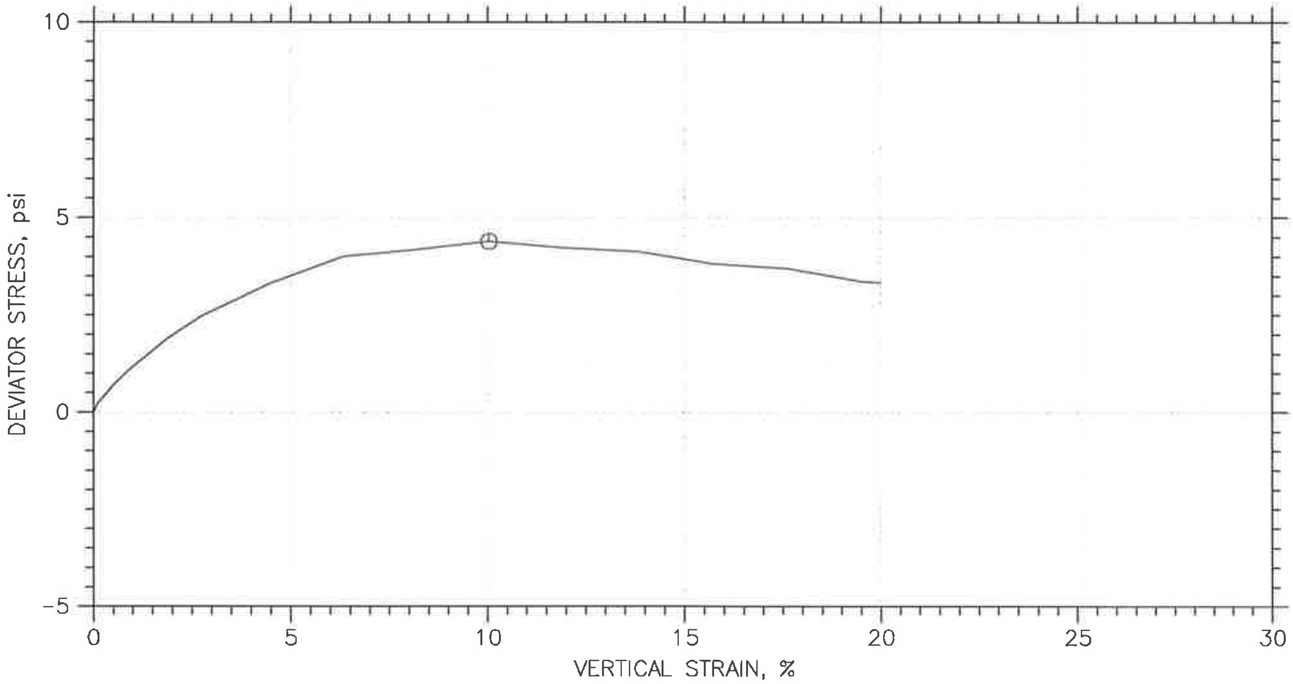


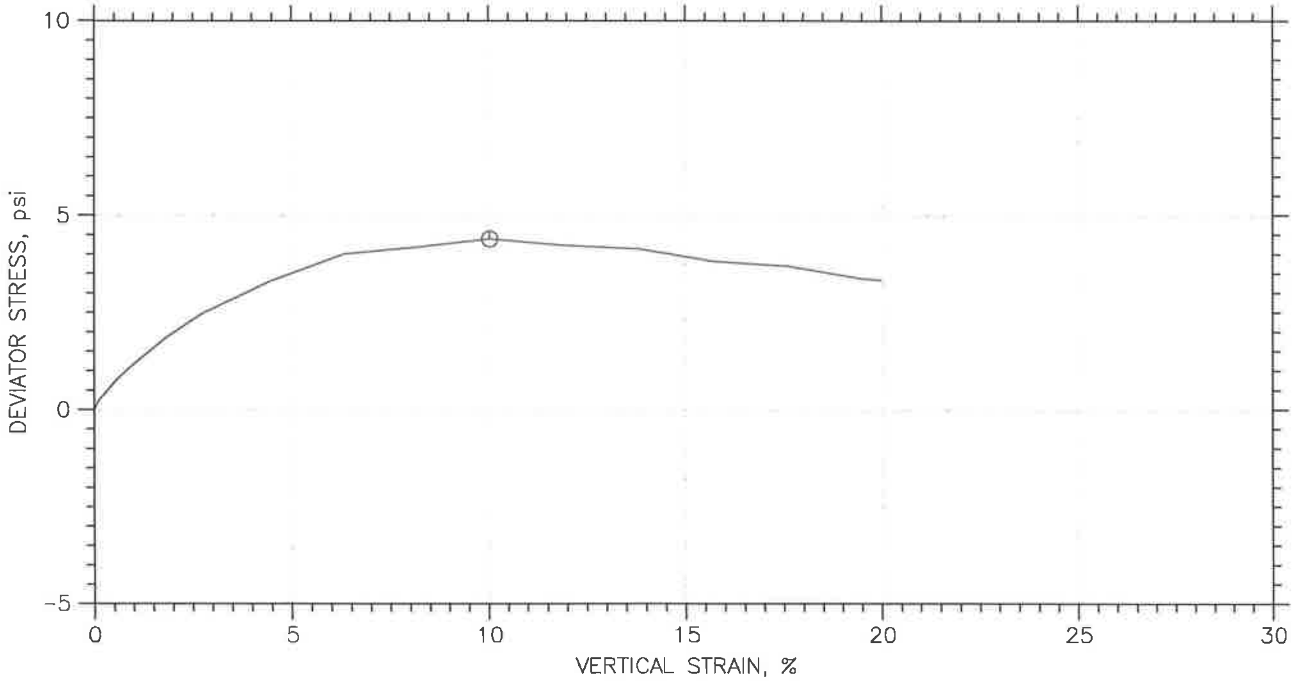
	Sample No.	Test No.	Depth	Tested By	Test Date	Checked By	Check Date	Test File
○	HB12@5'	13-243	5-5.5	JMA	4/11/13			13-243 MSE.dat

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

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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
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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:

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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 C

HEC-RAS Modeling Results 100 Year

HEC-RAS Plan: 100 year   Profile: 13FEB2003 1210

River	Reach	River Sta	Profile	Q Total	Min Ch El	W.S. Elev	Crit W.S.	E.G. Elev	E.G. Slope	Vel Chnl	Vel Left	Vel Right	Flow Area	Top Width	Froude # Chl
				(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/ft)	(ft/s)	(ft/s)	(ft/s)	(sq ft)	(ft)	
North Trib.	North Trib.	1700	13FEB2003 1210	0.10	3.39	8.81		8.81	0.000000	0.00	0.00	0.00	117.84	99.69	0.00
North Trib.	North Trib.	1600	13FEB2003 1210	0.48	2.57	8.81		8.81	0.000000	0.00	0.00	0.00	221.61	215.13	0.00
North Trib.	North Trib.	1500	13FEB2003 1210	1.17	1.76	8.81		8.81	0.000000	0.01	0.00	0.00	393.16	305.17	0.00
North Trib.	North Trib.	1400	13FEB2003 1210	2.09	1.23	8.81		8.81	0.000000	0.01	0.00	0.00	473.36	336.79	0.00
North Trib.	North Trib.	1300	13FEB2003 1210	3.10	1.12	8.81		8.81	0.000000	0.01	0.00	0.00	589.31	360.63	0.00
North Trib.	North Trib.	1200	13FEB2003 1210	4.11	1.00	8.81		8.81	0.000000	0.01	0.00	0.00	641.80	355.22	0.00
North Trib.	North Trib.	1100	13FEB2003 1210	5.17	0.89	8.81		8.81	0.000000	0.02	0.01	0.01	704.66	400.95	0.00
North Trib.	North Trib.	1000	13FEB2003 1210	6.28	0.79	8.81		8.81	0.000000	0.02	0.01	0.01	653.69	382.04	0.00
North Trib.	North Trib.	900	13FEB2003 1210	7.38	0.67	8.81		8.81	0.000000	0.02	0.00	0.01	681.77	372.45	0.00
North Trib.	North Trib.	800	13FEB2003 1210	8.50	0.52	8.81		8.81	0.000000	0.02	0.01	0.00	733.83	378.93	0.00
North Trib.	North Trib.	700	13FEB2003 1210	9.56	0.37	8.81		8.81	0.000000	0.02	0.01	0.00	678.22	327.18	0.00
North Trib.	North Trib.	600	13FEB2003 1210	10.50	0.22	8.81	0.58	8.81	0.000000	0.03	0.01	0.01	488.47	307.62	0.00
Martin	Mainstem	7500	13FEB2003 1210	376.01	4.36	9.43		9.80	0.005028	5.00		0.79	91.48	89.80	0.53
Martin	Mainstem	7400	13FEB2003 1210	376.30	2.63	9.09		9.36	0.004197	4.52	0.42	0.97	140.76	201.39	0.46
Martin	Mainstem	7250		Lat Struct											
Martin	Mainstem	7100	13FEB2003 1210	390.14	2.24	8.52		8.57	0.000890	2.38	0.23	0.57	415.59	511.99	0.22
Martin	Mainstem	7000	13FEB2003 1210	391.46	1.96	8.37		8.51	0.001689	3.46	0.30	0.80	212.71	213.88	0.31
Martin	Mainstem	6900	13FEB2003 1210	392.79	1.68	8.29		8.35	0.000830	2.59	0.48	0.61	366.62	380.46	0.22
Martin	Mainstem	6800	13FEB2003 1210	394.73	1.40	8.24		8.28	0.000530	2.24	0.56	0.54	453.99	413.28	0.18
Martin	Mainstem	6700	13FEB2003 1210	396.78	1.20	8.20		8.23	0.000383	1.94	0.37	0.53	506.34	380.36	0.15
Martin	Mainstem	6600	13FEB2003 1210	398.84	1.01	8.18		8.19	0.000238	1.54	0.33	0.48	615.73	388.33	0.12
Martin	Mainstem	6500	13FEB2003 1210	401.05	0.81	8.15		8.17	0.000337	1.85	0.37	0.50	543.84	407.91	0.14
Martin	Mainstem	6400	13FEB2003 1210	403.47	0.62	8.10		8.15	0.000524	2.15	0.55	0.50	445.14	425.65	0.17
Martin	Mainstem	6300	13FEB2003 1210	406.12	0.42	8.06		8.11	0.000478	2.14	0.08	0.44	408.38	455.54	0.17
Martin	Mainstem	6250		Lat Struct											
Martin	Mainstem	6200	13FEB2003 1210	408.91	0.36	8.02		8.07	0.000374	1.91	0.04	0.39	436.76	444.70	0.15
Martin	Mainstem	6100	13FEB2003 1210	411.61	0.31	8.00		8.03	0.000308	1.82		0.41	472.58	378.61	0.14
Martin	Mainstem	6000	13FEB2003 1210	414.09	0.25	7.95		8.00	0.000482	2.07		0.42	364.25	341.81	0.17
Martin	Mainstem	5900	13FEB2003 1210	415.76	0.20	7.83	3.52	7.94	0.000742	2.66			156.06	31.90	0.21
Martin	Mainstem	5840		Bridge											
Martin	Mainstem	5800	13FEB2003 1210	415.76	0.14	7.80		7.88	0.000548	2.40		0.36	275.01	270.89	0.18
Martin	Mainstem	5700	13FEB2003 1210	417.77	0.08	7.75		7.83	0.000545	2.40		0.37	278.39	271.42	0.18
Martin	Mainstem	5600	13FEB2003 1210	420.88	0.03	7.71		7.77	0.000526	2.23		0.40	329.24	318.81	0.18
Martin	Mainstem	5550		Lat Struct											
Martin	Mainstem	5500	13FEB2003 1210	424.64	-0.03	7.67		7.73	0.000465	2.18	0.15	0.38	381.06	411.61	0.17
Martin	Mainstem	5400	13FEB2003 1210	428.41	-0.08	7.61		7.68	0.000524	2.35		0.39	314.00	309.96	0.18
Martin	Mainstem	5300	13FEB2003 1210	431.64	-0.14	7.57		7.64	0.000376	2.15	0.18	0.33	311.87	261.10	0.15
Martin	Mainstem	5200	13FEB2003 1210	435.02	-0.17	7.55		7.59	0.000238	1.77	0.29	0.32	409.31	299.27	0.13
Martin	Mainstem	5100	13FEB2003 1210	438.49	-0.21	7.52		7.57	0.000269	1.87	0.24	0.28	378.22	349.16	0.13
Martin	Mainstem	5000.*	13FEB2003 1210	442.69	-0.25	7.49		7.54	0.000286	1.90	0.23	0.27	348.90	343.75	0.14
Martin	Mainstem	4900	13FEB2003 1210	448.26	-0.28	7.47		7.51	0.000281	1.85	0.23	0.26	390.92	434.99	0.14
Martin	Mainstem	4850		Lat Struct											
Martin	Mainstem	4800	13FEB2003 1210	462.02	-0.30	7.43		7.48	0.000278	1.90	0.25	0.27	373.40	354.02	0.14
Martin	Mainstem	4700	13FEB2003 1210	466.35	-0.32	7.39		7.46	0.000338	2.05	0.22	0.20	258.51	169.23	0.15
Martin	Mainstem	4600	13FEB2003 1210	469.24	-0.34	7.35		7.42	0.000357	2.07	0.27		248.21	98.12	0.15
Martin	Mainstem	4500	13FEB2003 1210	470.54	-0.36	7.31		7.38	0.000448	2.10			223.66	46.63	0.17
Martin	Mainstem	4400	13FEB2003 1210	471.20	-0.38	7.27		7.34	0.000392	2.13			221.57	40.45	0.16
Martin	Mainstem	4300	13FEB2003 1210	473.37	-0.40	7.23		7.30	0.000376	2.13	0.28		236.04	73.86	0.16
Martin	Mainstem	4200	13FEB2003 1210	476.39	-0.42	7.20		7.26	0.000292	1.91	0.28		332.95	226.44	0.14
Martin	Mainstem	4100	13FEB2003 1210	480.14	-0.44	7.16		7.22	0.000334	2.03	0.14		269.40	245.91	0.15
Martin	Mainstem	4000	13FEB2003 1210	484.88	-0.46	7.12		7.19	0.000344	2.05	0.16		264.83	234.36	0.15
Martin	Mainstem	3900	13FEB2003 1210	488.90	-0.47	7.09		7.15	0.000353	2.08	0.23		257.12	113.09	0.15
Martin	Mainstem	3800	13FEB2003 1210	491.81	-0.49	7.05		7.12	0.000332	2.05	0.27	0.28	297.65	186.25	0.15
Martin	Mainstem	3700	13FEB2003 1210	493.50	-0.51	7.01		7.08	0.000348	2.10	0.18	0.27	273.83	196.27	0.15
Martin	Mainstem	3600	13FEB2003 1210	494.52	-0.52	6.97		7.04	0.000368	2.14	0.18	0.26	243.91	82.96	0.16
Martin	Mainstem	3250		Lat Struct											
Martin	Mainstem	3200	13FEB2003 1210	498.73	-0.59	6.89		6.93	0.000195	1.71	0.31	0.44	350.38	97.95	0.12
Martin	Mainstem	3100	13FEB2003 1210	503.65	-0.60	6.86		6.91	0.000201	1.73	0.32	0.44	348.89	97.90	0.12
Martin	Mainstem	3050		Lat Struct											
Martin	Mainstem	3000	13FEB2003 1210	596.96	-0.62	6.81		6.87	0.000290	2.07	0.37	0.52	344.24	97.70	0.14
Martin	Mainstem	2900	13FEB2003 1210	598.07	-0.63	6.78		6.84	0.000295	2.08	0.37	0.51	341.83	97.60	0.15
Martin	Mainstem	2800	13FEB2003 1210	599.18	-0.65	6.75		6.81	0.000300	2.10	0.37	0.51	339.79	97.51	0.15
Martin	Mainstem	2700	13FEB2003 1210	600.29	-0.67	6.72		6.78	0.000304	2.11	0.37	0.51	337.72	97.43	0.15
Martin	Mainstem	2600	13FEB2003 1210	601.37	-0.68	6.69		6.75	0.000310	2.12	0.37	0.50	335.19	97.32	0.15
Martin	Mainstem	2500	13FEB2003 1210	602.40	-0.70	6.65		6.72	0.000314	2.13	0.37	0.50	333.03	97.23	0.15
Martin	Mainstem	2400	13FEB2003 1210	603.38	-0.71	6.62		6.69	0.000320	2.15	0.37	0.50	330.48	97.14	0.15
Martin	Mainstem	2300	13FEB2003 1210	604.27	-0.73	6.59		6.66	0.000325	2.16	0.37	0.49	328.17	97.03	0.15
Martin	Mainstem	2200	13FEB2003 1210	605.12	-0.74	6.56		6.63	0.000331	2.17	0.37	0.49	325.49	96.93	0.15
Martin	Mainstem	2100	13FEB2003 1210	605.93	-0.76	6.52		6.59	0.000337	2.19	0.36	0.48	323.12	96.82	0.15
Martin	Mainstem	2000	13FEB2003 1210	606.73	-0.77	6.49		6.56	0.000343	2.20	0.36	0.48	320.28	96.71	0.16
Martin	Mainstem	1900	13FEB2003 1210	607.58	-0.79	6.45		6.52	0.000349	2.21	0.36	0.47	317.78	96.60	0.16
Martin	Mainstem	1800	13FEB2003 1210	608.51	-0.81	6.42		6.49	0.000355	2.23	0.36	0.47	315.20	96.48	0.16
Martin	Mainstem	1700	13FEB2003 1210	609.58	-0.82	6.38		6.45	0.000363	2.25	0.36	0.46	312.14	96.36	0.16
Martin	Mainstem	1600	13FEB2003 1210	610.82	-0.84	6.34		6.42	0.000371	2.26	0.35	0.45	309.39	96.24	0.16
Martin	Mainstem	1500	13FEB2003 1210	612.23	-0.85	6.30		6.38	0.000380	2.28	0.35	0.45	306.16	96.11	0.16
Martin	Mainstem	1400	13FEB2003 1210	613.43	-0.87	6.26		6.34	0.000357	2.25	0.47	0.47	280.24	50.86	0.16
Martin	Mainstem	1300	13FEB2003 1210	614.79	-0.88	6.24		6.30	0.000304	2.06	0.38	0.31	333.28	100.41	0.15
Martin	Mainstem	1200	13FEB2003 1210	616.65	-0.90	6.20		6.27	0.000310	2.08	0.37	0.30	331.05	100.31	0.15
Martin	Mainstem	1100	13FEB2003 1210	618.44	-0.91	6.17		6.24	0.000316	2.09	0.37	0.30	328.30	100.20	0.15
Martin	Mainstem	1000	13FEB2003 1210	620.09	-0.92	6.14		6.20	0.000323	2.11	0.36	0.30	325.44	100.07	0.15
Martin	Mainstem	900	13FEB2003 1210	621.52	-0.93	6.10		6.17	0.000330	2.13	0.35	0.30	322.52	99.94	0.15
Martin	Mainstem	800	13FEB2003 1210	622.67	-0.94	6.07		6.14	0.000337	2.14	0.35	0.30	319.62	99.82	0.15
Martin	Mainstem	700	13FEB2003 1210	623.61	-0.94	6.03		6.10	0.000346	2.16	0.34	0.30	316.17	99.68	0.16
Martin	Mainstem	600	13FEB2003 1210	624.54	-0.95	6.00		6.07	0.000353	2.18	0.33	0.29	313.16	99.46	0.16
Martin	Mainstem	500	13FEB2003 1210	625.74	-0.96	5.96		6.03	0.000361	2.19	0.33	0.29	309.97	97.88	0.16
Martin	Mainstem	400	13FEB2003 1210	627.36	-0.97	5.92		6.00	0.000370	2.21	0.32	0.29	306.70	96.23	0.16
Martin	Mainstem	300	13FEB2003 1210	629.30	-0.98	5.88		5.96	0.000380	2.23	0.32	0.28	303.36	94.52	0.16
Martin	Mainstem	200	13FEB2003 1210	631.12	-0.98	5.84		5.92	0.000392	2.26	0.32	0.28	299.57	92.76	0.17
Martin	Mainstem	100	13FEB2003 1210	634.09	-0.99	5.80		5.88	0.000405	2.28	0.31	0.28	296.11	90.92	0.17
Martin	Mainstem	50		Lat Struct											
Martin	Mainstem	25		Lat Struct											
Martin	Mainstem	10	13FEB2003 1210	-0.62	-0.99	5.90		5.90	0.000000	0.00	0.00	0.00	306.01	95.50	0.00
Martin	Mainstem	0	13FEB2003 1210	0.10	-0.99	5.90	-0.97	5.90	0.000000	0.00	0.00	0.00	306.01	95.51	0.00

Appendix D

Fish Passage Analysis

Summary of Fish Passage: Adult High Passage Flow						
	Outflowing		Inflowing			
Condition	**2 6 ft x 6 ft Gates**	**Single 6 ft x 6 ft MTR Gate**	**Single 6 ft x 6 ft MTR Gate**	**Single 1.5 foot High x 2 ft Wide Aux Door**	**Total Upstream Passage [1]**	**Total Downstream Passage [2]**
Percent of time gates are open (A)	83.2%	83.2%	1.1%	8.5%		
Impassible Condition: Percent of time flow velocity > 6 fps when gates open AND depth >1 foot	0.0%	0.0%	0.0%	0.0%		
Impassible Condition: Percent of time flow <1 foot over Aux Door invert (B)	NA	NA	NA	0.0%		
Total Percent time passable to adults (Gates Open-Impassible Conditions) (C)	83.2%	83.2%	1.1%	8.5%	91.7%	91.7%

[1] Total Upstream Passage: Inflowing A - Inflowing B + Outflowing C

[2] Total Downstream Passage: Outflowing A + Inflowing C

Summary of Fish Passage: Adult Low Passage Flow

Condition	Outflowing		Inflowing		Total Upstream Passage [1]	Total Downstream Passage [2]
	2 6 ft x 6 ft Gates	Single 6 ft x 6 ft MTR Gate	Single 6 ft x 6 ft MTR Gate	Single 1.5 foot High x 2 ft Wide Aux. Gate		
Percent of time gates are open (A)	42.0%	42.0%	21.8%	53.5%		
Impassible Condition: Percent of time flow velocity > 6 fps when gates open AND depth >1 foot	0.0%	0.0%	0.0%	16.7%		
Impassible Condition: Percent of time flow <1 foot over Aux Door invert (B)	NA	NA	NA	**2.8%**		
Total Percent time passable to adults (Gates Open-Impassible Conditions) (C)	42.0%	42.0%	21.8%	36.8%	92.8%	78.9%

[1] Total Upstream Passage: Inflowing A - Inflowing B + Outflowing C

[2] Total Downstream Passage: Outflowing A + Inflowing C

Summary of Fish Passage: Juvenile Salmonid and Steelhead High Passage Flow

Condition	Outflowing		Inflowing		Total Upstream Passage [1]	Total Downstream Passage [2]
	2 6 ft x 6 ft Gates	Single 6 ft x 6 ft MTR Gate	Single 6 ft x 6 ft MTR Gate	Single 1.5 foot High x 2 ft Wide Aux. Gate		
Percent of Time gates are open (A)	54.2%	54.2%	14.1%	41.2%		
Impassible Condition: Percent of time flow velocity > 1.5 fps AND Gates open	4.8%	1.2%	3.5%	33.8%		
Total Percent Time Passable to Juveniles (Gates Open - Impassible Conditions) (B)	49.3%	53.0%	10.5%	7.5%	94.3%	64.7%

[1] Total Upstream Passage: Inflowing A + Outflowing B

[2] Total Downstream Passage: Outflowing A + Inflowing B

Summary of Fish Passage: Juvenile Salmonid and Steelhead Low Passage Flow

Condition	Outflowing		Inflowing		Total Upstream Passage [1]	Total Downstream Passage [2]
	2 6 ft x 6 ft Gates	Single 6 ft x 6 ft MTR Gate	Single 6 ft x 6 ft MTR Gate	Single 1.5 foot High x 2 ft Wide Aux. Gate		
Percent of Time gates are open (A)	39.7%	39.7%	24.2%	58.6%		
Impassible Condition: Percent of time flow velocity > 1.5 fps AND Gates open	2.1%	0.2%	9.3%	47.6%		
Total Percent Time Passable to Juveniles (Gates Open - Impassible Conditions) (B)	37.7%	39.5%	15.0%	11.0%	98.1%	54.7%

[1] Total Upstream Passage: Inflowing A + Outflowing B

[2] Total Downstream Passage: Outflowing A + Inflowing B

Appendix E

Geologic Setting for CEQA

Geologic Setting

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

Geologic Setting

Martin Slough Enhancement Project

Prepared for:

Redwood Community Action Agency

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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.