Hydrology, Flood Management and Infrastructure

Impact Analysis Support

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Hydrology, Flood Management and Infrastructure

Impact Analysis Support

G-1. Tidal Channel Hydraulic Geometry Analyses

MEMORANDUM

TO: Members of the South Bay Salt Pond Restoration Project Management Team

FROM: Philip Williams and Associates, Ltd. (PWA)

DATE: October 26, 2006 – *Revised January 2007*

RE: **Tidal Channel Hydraulic Geometry Analysis**

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1. INTRODUCTION

This section of Appendix G contains supporting materials for hydraulic geometry calculations made to predict both (1) slough channel geometry for the SBSP long term alternatives, and (2) potential changes in slough channel top width associated with the increase in tidal prism in Alviso and Guadalupe Sloughs resulting from Phase 1 actions at Pond A6.

2. SOUTH BAY SALT POND LONG TERM ALTERNATIVES

The long-term slough channel cross-sectional areas for Alternatives A, B and C were estimated using the hydraulic geometry relationships below, which were developed by Williams and others (2002) for San Francisco Bay. These empirical relationships are based on data from historic and existing mature San Francisco Bay salt marshes ranging in size from 2 to 5,700 ha (Williams and others 2002). The relationships relate channel depth, width and cross-sectional area to both marsh drainage area and tidal prism, and they can be used to predict potential long-term slough dimensions in response to changes in marsh drainage area and tidal prism. In the relationships below, the depth, width and cross-sectional area are represented by the dependent variable (y) and the tidal prism and marsh area are represented by the independent variable (x).

	Tidal Prism	**Marsh Area**
Depth	$y = 0.388x^{0.176}$	$y = 1.31x^{0.202}$
Width	$y = 0.147x^{0.461}$	$y = 3.44x^{0.552}$
Cross-Section Area	$y = 0.0284x^{0.649}$	$y = 2.40x^{0.772}$

A detailed discussion of the method employed in calculating the long-term channel cross-sectional areas for Alternatives A, B and C may be found in Section 3.3.2 of the Hydrodynamic Modeling Report: Alternatives Analysis (PWA 2006) (Appendix J).

Tables 1 through 5 in this appendix summarize the hydraulic geometry calculation results. Figures 1 through 5 show the locations in the sloughs for which the calculations were made. Please note that the Cross-Section ID's in Tables 3 through 5 were assigned by numbering the cross-sections from upstream to downstream. Therefore, the furthest upstream cross-section would be assigned the number one and the furthest downstream cross-section would be assigned the highest number in the slough sequence.

3. PHASE 1 ACTION AT POND A6

Phase 1 slough channel top-widths in Alviso and Guadalupe Sloughs downstream of proposed Pond A6 breach locations were predicted and compared to the minimum distances between slough levees to assess the potential for Phase 1 actions to cause levee erosion. Phase 1 action for Pond A6 would breach to both sloughs near their mouths. Phase 1 action for Pond A8 would introduce muted tidal action in Pond A8 (upstream of Pond A6) via Alviso Slough. Therefore, Alviso Slough would be affected by restoration at both Ponds A6 and A8. Estimates of slough channel geometries at the mouth of Alviso Slough capture the cumulative impacts of the Phase 1 actions on channel geometry.

Predictions of slough channel top-width are based on the empirical relationship developed by Williams and others (2002) for San Francisco Bay which relates channel top-width to both marsh drainage area and tidal prism. Because there is scatter in the empirical data set used to develop the hydraulic geometry relationships, the dimensions of measured channel cross-sections are not expected to exactly match those predicted using the equations. This is the case for Alviso and Guadalupe Sloughs, where the dimensions predicted by the hydraulic geometry based on the existing tidal prism predict deeper, narrower channel near the Pond A6 breaches. Because there may be a physical basis for the deviation from predicted dimensions, for example the effects of fluvial flows from the Guadalupe River or slough mouth dynamics, the differences between predicted and measured dimensions for existing conditions were preserved in the long-term channel dimension estimates. Long-term channel dimensions were calculated as the sum of the existing measured dimensions and the predicted incremental long-term change. Increases in each channel dimension were calculated using the derivatives. These increases in channel size were then added to the measured existing dimensions to estimate new equilibrium geometries.

Table 6 summarizes the existing and predicted short-and long-term channel top-widths for Alviso and Guadalupe Sloughs. Short-term predictions are based on the estimated Phase 1 tidal prism at the slough mouths. Tidal prisms modeled in the Far South Bay Model (Appendix G. Alviso Pond A8 Hydrodynamic Modeling and Geomorphic Analyses) were calculated for existing conditions and Pond A8 Phase 1 conditions. The modeled tidal prism under existing conditions was used in this analysis as well. To incorporate the impacts of Phase 1 actions at Pond A6 and A8, the increase in existing tidal prism under modeled Phase 1 Pond A8 conditions was added to estimates of Pond A6 tidal prism. For Guadalupe

Slough, the Pond A8 contribution to Phase 1 tidal prism is zero. Pond A6 Phase 1 tidal prism was estimated by calculating the potential storage volume of the breached drainage areas below MHHW.

Long-term slough channel top-width predictions are based on contributing marsh area in order to reflect the loss in potential tidal prism as the pond bed fills with sediment over time. Existing upstream tidal prisms modeled in the Far South Bay Model (Appendix G. Alviso Pond A8 Hydrodynamic Modeling and Geomorphic Analyses) were converted to equivalent upstream marsh areas using an empirical relationship between tidal prism and marsh area (Williams and others 2002). Phase 1 Pond A8 tidal prism was converted to upstream marsh area using the same method and was added to measured Phase 1 Pond A6 drainage areas to estimate the additional marsh area under Phase 1 conditions.

To assess the potential for Phase 1 actions to cause the erosion of adjacent levees, the minimum distances between the levees were measured downstream of the proposed Pond A6 Phase 1 breaches. These are compared to predicted channel top-widths in Table 6.

4. REFERENCES

PWA. 2006. Hydrodynamic Modeling Report: Alternatives Analysis. San Francisco, CA.: Prepared for: California State Coastal Conservancy, U.S. Fish and Wildlife Service, California Department of Fish and Game.

Williams PB, Orr MK, Garrity NJ. 2002. Hydraulic geometry: A geomorphic design tool for tidal marsh channel evolution in wetland restoration projects. Restoration Ecology 10(3):577-590.

Table 1. Predicted Existing and Long-Term Tidal Prism at the Slough Mouths

Slough	Existing Tidal Prism (ac-ft)[1]	Estimated Long-term Tidal Prism At Slough Mouth					
		Alternative A		Alternative B		Alternative C	
		Tidal Prism (ac-ft)	% Increase	Tidal Prism (ac-ft)	% Increase	Tidal Prism (ac-ft)	% Increase
Mt. Eden Creek[2]	170	960	460%	460	170%	460	170%
North Creek[2]	140	180	30%	400	190%	650	380%
Old Alameda Creek[2]	260	1,700	560%	1,990	680%	2,890	1030%
Alameda County Flood Control Channel	60	660	1010%	1,020	1630%	1,020	1630%
Ravenswood Slough	20	--	--	770	4810%	1,040	6530%
Mud Slough	500	--	--	--	--	1,210	140%
Artesian Slough	1,140	--	--	--	--	1,270	10%
Alviso Slough	670	1,390	110%	1,830	170%	2,660	300%
Guadalupe Slough[3]	11,660	12,450	10%	12,830	10%	12,830	10%
Stevens Creek	0	--	--	280	6050%	280	6050%
Mountain View Slough/Permanente Creek	20	--	--	510	2960%	510	2960%
Charleston Slough	20	--	--	90	270%	90	270%
Coyote Creek, d/s of Alviso Slough	5,430	6,350	20%	6,920	30%	10,320	90%

Notes

1- Existing tidal prism for Alviso Slough is from Shaaf & Wheeler (2000); existing tidal prism for Coyote Creek is based on Alternative A Year 0 model results; all other existing tidal prisms are estimated from hydraulic geometry relationships (Williams and others 2000) using existing channel depths and assuming sloughs are in equilibrium.

2- Existing tidal prims for Mt. Eden Creek, North Creek, and Old Alameda Creek include tidal prism from the Eden Landing Ecological Reserve tidal restoration

3- Existing tidal prism for Guadalupe Slough is estimated from the deepest point in the slough, which is just upstream of the slough mouth.

Table 2. Predicted Long-term Increases in Slough Channel Depth

Slough	Location in Slough[3]	Long-term Increase in Depth (meters)		
		Alternative A	Alternative B	Alternative C
Mt. Eden Creek	Up-slough	2.8	1.9	1.9
	Mouth	2.9	2.4	2.4
North Creek	Up-slough	--	1.7	2.2
	Mouth	1.5	2.0	2.3
Old Alameda Creek[1]	Up-slough	0.2	0.2	0.4
	Mid-slough	1.1	1.2	1.4
	Mouth	1.1	1.2	1.4
Alameda County Flood Control Channel	Up-slough	--	0.2	0.2
	Mid-slough	1.2	1.6	1.6
	Mouth	1.4	1.8	1.8
Ravenswood Slough	Up-slough	--	0.6	0.6
	Mid-slough	--	1.8	2.0
	Mouth	--	2.1	2.3
Mud Slough	Mid-slough	--	--	0.7
Artesian Slough	Mouth	--	--	0.1
Alviso Slough	Up-slough	--	0.3	0.3
	Mid-slough	0.6	0.7	0.9
	Mouth	0.7	0.8	1.1
Guadalupe Slough[2]	Up-slough	0.2	1.1	1.1
	Mid-slough	0.4	0.2	0.2
	Mouth	0.1	0.1	0.1
Stevens Creek	Up-slough	--	1.8	1.8
Mountain View Slough/Permanente Creek	Mid-slough	--	1.6	1.6
	Mouth	--	1.8	1.8
Charleston Slough	Up-slough	--	0.3	0.3
	Mouth	--	0.6	0.6
Coyote Creek	Up-slough	--	--	0.3
	Mid-slough	--	--	0.9
	Mouth	--	--	0.4

Notes

1- Scour in Old Alameda Creek assumes one channel.

2- Increase in depth for the mouth of Guadalupe Slough is the increase below the deepest cross-section along the channel, which is just upstream of the mouth.

3- These refer to general locations within the sloughs. They correspond with cross-sections but the cross-section numbers are different for each slough and for each alternative.

Table 3. Predicted Alternative A, Year 50 Slough Channel Depths

Slough	Cross Section ID	Existing Upstream Marsh Area (hectares)	Existing Channel Depth, Measured (m below MHHW)	Restored Upstream Marsh Area (hectares)	Restored Channel Depth, Predicted (m below MHHW)	Predicted Increase in Channel Depth (m)
Mt. Eden Creek[1]	1	103	1.4	312	4.2	2.8
	2	103	1.6	448	4.5	2.9
North Creek[1]	1	84	1.9	107	3.4	1.5
Old Alameda Creek[1,2]	1	76	2.8	147	3.6	0.8
	2	76	2.8	399	4.4	1.6
	3	160	2.9	731	5.0	2.0
	4	160	3.1	731	5.0	1.9
Alameda County Flood Control Channel	1	41	2.8	254	4.0	1.2
	2	41	2.8	324	4.2	1.4
Alviso Slough[3]	1	241	3.8	404	4.4	0.6
	2	325	4.5	502	4.6	0.1
	3	325	4.1	614	4.8	0.7
Guadalupe Slough[4]	1	121	3.5	149	3.6	0.2
	2	558	4.7	656	4.9	0.2
	3	558	4.6	733	5.0	0.4
	4	3788	6.9	3962	7.0	0.1
	5	3788	5.4	4008	7.0	1.6
Coyote Creek[3]	1	1973	6.4	2257	6.2	--

Notes:

1-Existing upstream marsh area for Mt. Eden Creek, North Creek, and Old Alameda Creek include marsh area from the Eden Landing Ecological Reserve tidal restoration

2-Existing depths in Old Alameda Creek are extracted from Delft model grid and therefore assume Old Alameda Creek is one channel. These were not used to calculate the existing upstream marsh area. Upstream marsh area for both channels of Old Alameda Creek were calculated and summed and the associated equilibrium channel depth was used in the hydraulic geometry calculations

3- Existing marsh area for Alviso Slough is based on Shaaf & Wheeler (2000); existing marsh area for Coyote Creek is based on Alternative A Year 0 model results; existing marsh area for Old Alameda Creek is explained in Note 2, all other existing marsh areas are estimated from hydraulic geometry relationships (Williams and others 2000) using existing channel depths and assuming sloughs are in equilibrium.

4- Existing tidal prism for Guadalupe Slough is estimated from the deepest point in the slough, which is just upstream of the slough mouth.

Table 4. Predicted Alternative B, Year 50 Slough Channel Depths

Slough	Cross Section ID	Existing Upstream Marsh Area (hectares)	Existing Channel Depth, Measured (m below MHHW)	Restored Upstream Marsh Area (hectares)	Restored Channel Depth, Predicted (m below MHHW)	Predicted Increase in Channel Depth (m)
Mt. Eden Creek[1]	1	103	1.4	103	3.3	1.9
	3	103	1.6	239	4.0	2.4
North Creek[1]	1	84	1.5	84	3.2	1.7
	2	84	1.9	212	3.9	2.0
Old Alameda Creek[1,2]	1	76	2.6	76	3.1	0.5
	2	76	2.8	147	3.6	0.8
	3	76	2.8	399	4.4	1.6
	4	76	2.9	835	5.1	2.2
	5	76	3.1	835	5.1	2.0
Alameda County Flood Control Channel	1	41	2.8	56	2.9	0.2
	2	41	2.8	404	4.4	1.6
	3	41	2.8	474	4.5	1.8
Ravenswood Slough	1	12	2.2	40	2.8	0.6
	2	12	2.2	252	4.0	1.8
	3	12	2.2	252	4.0	1.8
	4	13	2.2	371	4.3	2.1
Alviso Slough[3]	1	114	4.4	181	3.7	--
	2	128	3.8	249	4.0	0.2
	3	167	3.8	376	4.3	0.6
	4	241	3.8	574	4.7	1.0
	5	325	4.5	672	4.9	0.4
	6	325	4.1	777	5.0	0.9
Guadalupe Slough[4]	1	5	1.8	54	2.9	1.1
	2	121	3.5	208	3.9	0.4
	3	558	4.7	715	4.9	0.2
	4	480	4.6	792	5.0	0.5
	5	3788	6.9	4022	7.0	0.1
	6	3788	5.4	4112	7.0	1.7
Stevens Creek	1	5	1.8	154	3.6	1.8
Mountain View Slough/Permanente Creek	1	12	2.2	178	3.7	1.6
	2	14	2.2	260	4.0	1.8
Charleston Slough	1	12	2.2	22	2.4	0.3
	2	19	2.4	59	3.0	0.6
Coyote Creek[3]	4	762	6.38	2425	6.3	--

Notes:

1-Existing upstream marsh area for Mt. Eden Creek, North Creek, and Old Alameda Creek include marsh area from the Eden Landing Ecological Reserve tidal restoration

2-Existing depths in Old Alameda Creek are extracted from Delft model grid and therefore assume Old Alameda Creek is one channel. These were not used to calculate the existing upstream marsh area. Upstream marsh area for both channels of Old Alameda Creek were calculated and summed and the associated equilibrium channel depth was used in the hydraulic geometry calculations

3- Existing marsh area for Alviso Slough is based on Shaaf & Wheeler (2000); existing marsh area for Coyote Creek is based on Alternative A Year 0 model results; existing marsh area for Old Alameda Creek is explained in Note 2, all other existing marsh areas are estimated from hydraulic geometry relationships (Williams and others 2000) using existing channel depths and assuming sloughs are in equilibrium.

4- Existing tidal prism for Guadalupe Slough is estimated from the deepest point in the slough, which is just upstream of the slough mouth.

Table 5. Predicted Alternative C, Year 50 Slough Channel Depths

Slough	Cross Section ID	Existing Upstream Marsh Area (hectares)	Existing Channel Depth, Measured (m below MHHW)	Restored Upstream Marsh Area (hectares)	Restored Channel Depth, Predicted (m below MHHW)	Predicted Increase in Channel Depth (m)
Mt. Eden Creek[1]	1	103	1.4	103	3.3	1.9
	3	103	1.6	239	4.0	2.4
North Creek[1]	1	84	1.5	182	3.7	2.2
	2	84	1.9	323	4.2	2.3
Old Alameda Creek[1,2]	1	76	2.6	238	4.0	1.4
	2	76	2.8	309	4.2	1.4
	3	76	2.8	602	4.8	2.0
	4	76	2.9	1149	5.4	2.5
	5	76	3.1	1149	5.4	2.4
Alameda County Flood Control Channel	1	41	2.8	56	2.9	0.2
	2	41	2.8	404	4.4	1.6
	3	41	2.8	474	4.5	1.8
Ravenswood Slough	1	12	2.2	40	2.8	0.6
	2	12	2.2	313	4.2	2.0
	3	12	2.2	360	4.3	2.1
	4	13	2.2	480	4.6	2.3
Mud Slough	1	257	4.0	546	4.7	0.7
Artesian Slough	1	518	4.6	571	4.7	0.1
Alviso Slough[3]	1	114	4.4	181	3.7	--
	2	128	3.8	322	4.2	0.4
	3	167	3.8	488	4.6	0.8
	4	241	3.8	715	4.9	1.2
	5	325	4.5	881	5.2	0.7
	6	325	4.1	1071	5.4	1.3
Guadalupe Slough[4]	1	5	1.8	54	2.9	1.1
	2	121	3.5	208	3.9	0.4
	3	558	4.7	715	4.9	0.2
	4	480	4.6	792	5.0	0.5
	5	3788	6.9	4022	7.0	0.1
	6	3788	5.4	4112	7.0	1.7
Stevens Creek	1	5	1.8	154	3.6	1.8
Mountain View Slough/Permanente Creek	1	12	2.2	178	3.7	1.6
	2	14	2.2	260	4.0	1.8
Charleston Slough	1	12	2.2	22	2.4	0.3
	2	19	2.4	59	3.0	0.6
Coyote Creek[3]	1	762	4.8	815	5.1	0.3
	2	762	4.8	1456	5.7	0.9
	3	1280	5.2	1974	6.1	0.9
	4	1973	6.4	3413	6.8	0.4

Notes:

1-Existing upstream marsh area for Mt. Eden Creek, North Creek, and Old Alameda Creek include marsh area from the Eden Landing Ecological Reserve tidal restoration

2-Existing depths in Old Alameda Creek are extracted from Delft model grid and therefore assume Old Alameda Creek is one channel. These were not used to calculate the existing upstream marsh area. Upstream marsh area for both channels of Old Alameda Creek were calculated and summed and the associated equilibrium channel depth was used in the hydraulic geometry calculations

3- Existing marsh area for Alviso Slough is based on Schaaf & Wheeler (2000); existing marsh area for Coyote Creek is based on Alternative A Year 0 model results; existing marsh area for Old Alameda Creek is explained in Note 2, all other existing marsh areas are estimated from hydraulic geometry relationships (Williams and others 2000) using existing channel depths and assuming sloughs are in equilibrium.

4- Existing tidal prism for Guadalupe Slough is estimated from the deepest point in the slough, which is just upstream of the slough mouth.

Table 6. Predicted Channel Width Calculations for Phase 1 Actions at Pond A6

Short Term Channel Geometry Predictions							
Cross Section Location	Existing Upstream Tidal Prism, Modeled[1] (m^3)	Additional Contributing Tidal Prism of Phase 1, Modeled and Calculated [2,3,4] (m^3)	Cumulative Tidal Prism (m^3)	Existing Channel Top Width, Measured (m)	Predicted Increase in Top Width (m)	Predicted Channel Width (m)	Existing Width Between Levees, Measured (m^2)
Mouth of Guadalupe Slough	1.1E+06	6.2E+05	1.7E+06	57	20	77	168
Mouth of Alviso Slough	1.1E+06	1.6E+06	2.8E+06	40	45	85	213
Long Term Channel Geometry Predictions							
Cross Section Location	Existing Upstream Marsh Area[5] (ha)	Additional Contributing Marsh Area of Phase 1[2,6,7] (ha)	Cumulative Marsh Area (ha)	Existing Channel Top Width, Measured (m)	Predicted Increase in Top Width (m)	Predicted Channel Width (m)	Existing Width Between Levees, Measured (m^2)
Mouth of Guadalupe Slough	430	37	467	57	0.5	57	168
Mouth of Alviso Slough	430	107	537	40	1.3	41	213

Notes:

1 - Modeled by the Far South Bay Model (Alviso Pond A8 Hydrodynamic Modeling and Geomorphic Analysis, Appendix G)

2 - Additional contributions from Phase 1 actions at Pond A8 and A6

3 - Phase 1 Pond A8 additional tidal prism modeled by the Far South Bay Model

4 - Phase 1 Pond A6 additional tidal prism calculated from the estimated drainage area and elevation relative to MHHW

5 - Equivalent marsh area of modeled tidal prism by the Far South Bay Model

6 - Equivalent marsh area of Phase 1 Pond A8 additional tidal prism modeled by the Far South Bay Model

7 - Equivalent marsh area of Phase 1 Pond A6 additional marsh area calculated from the estimated drainage area and elevation relative to MHHW

cross-section location

breach location

figure 1

South Bay Salt Ponds Restoration Project

Hydraulic Geometry Cross-Section and Breach Locations: Eden Landing Pond Complex, Alternative A

PWA

cross-section location

breach location

figure 2
South Bay Salt Ponds Restoration Project

Hydraulic Geometry Cross Section and Breach Locations:
Alviso Pond Complex, Alternative A

PWA

CrossSectionLocations.xls / Alviso_A

cross-section location

breach location

figure 3

South Bay Salt Ponds Restoration Project

Hydraulic Geometry Cross-Section and Breach Locations:
Eden Landing Pond Complex, Alternative C

PWA

cross-section location

breach location

figure 4
South Bay Salt Ponds Restoration Project

Hydraulic Geometry Cross Section and Breach Locations:
Alviso Pond Complex, Alternative C

PWA

CrossSectionLocations.xls / Alviso_C

cross-section location

breach location

figure 5
South Bay Salt Ponds Restoration Project

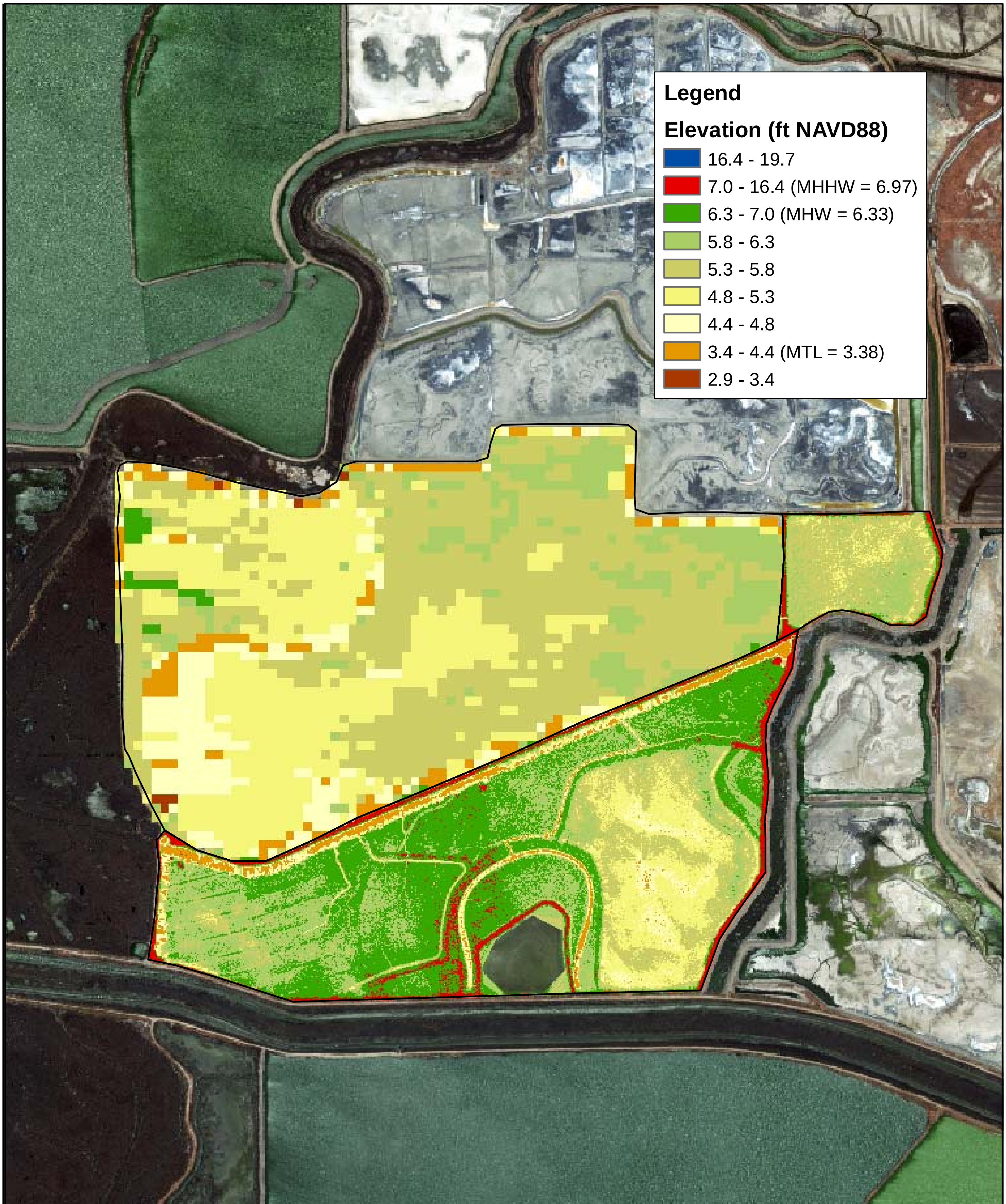
Hydraulic Geometry Cross Section and Breach Locations:
Ravenswood Pond Complex, Alternative C

PWA

Hydrology, Flood Management and Infrastructure

Impact Analysis Support

G-2. Topography of Phase 1 Action Restoration Sites



Source: USGS Sonar (Ponds E9); LIDAR, 2004 (Pond E8, E8X)

N

0 100 500 1,000 1,500 2,000 2,500 Feet

10 acres

figure **1**

South Bay Salt Ponds Restoration

Topography of Ponds E8/E8X/E9 (Tidal Ponds)

G:\1720SBSP\Phase1Actions\Projects\E8_E9Topo.mxd

Proj. # 1751.01

PWA

Source: LIDAR, 2004

10 ACRES

0 250 500 1,000 1,500 2,000 Feet

figure **2**

South Bay Salt Ponds Phase 1 Actions

Topography of Pond E12/E13 (Managed Pond)

Proj. # 1751.01

PWA

G:\1720SBSP\Phase1Actions\Projects\E12_E13Topo.mxd

Source: LIDAR, 2004

0 990 1,980 2,970 Feet

10 acres

figure 3

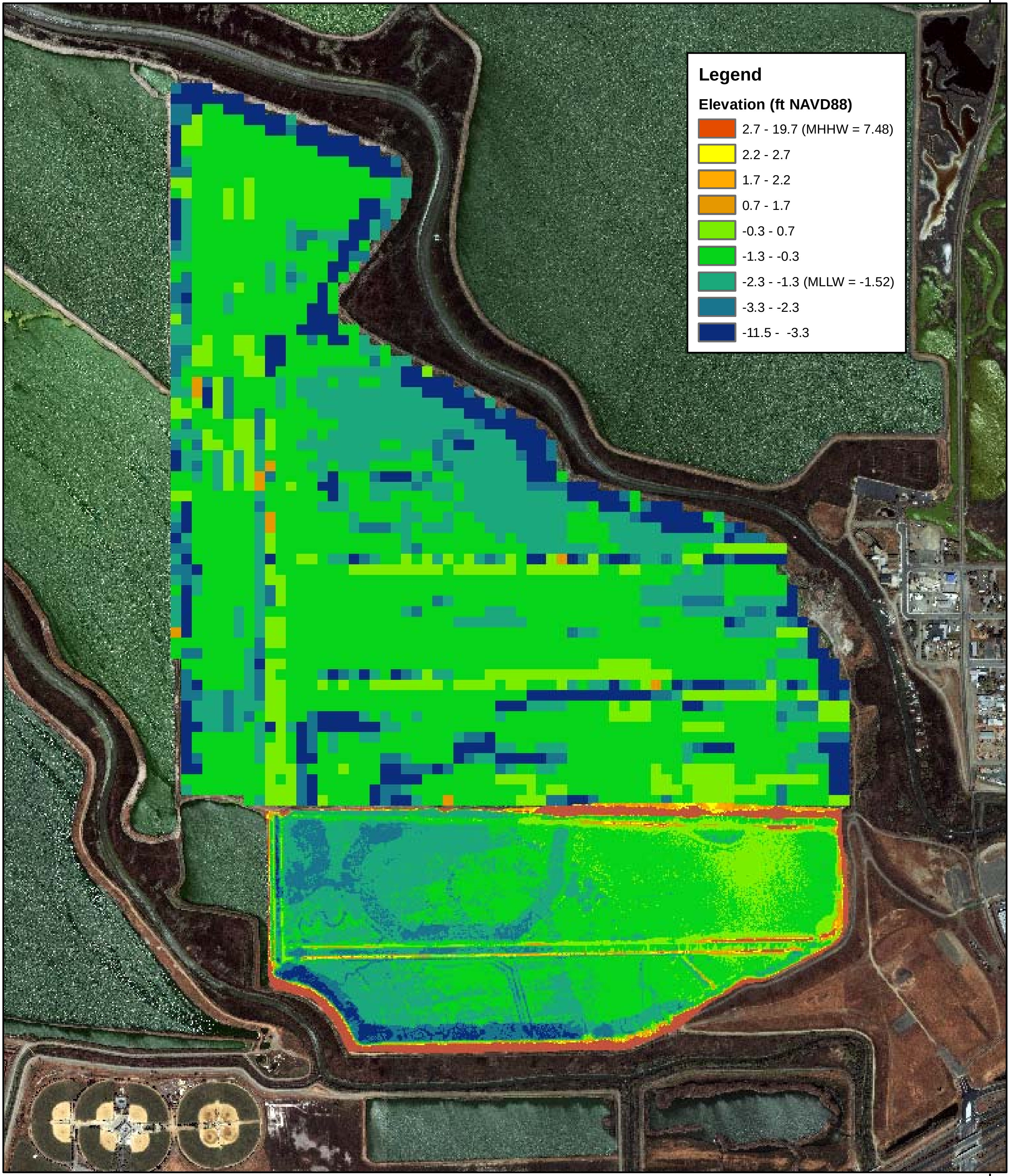
South Bay Salt Ponds Restoration Project

Topography of PondA6 (Tidal Pond)

Proj. # 1751.01

PWA

G:\1720SBSP\Phase1Actions\Projects\Pond A6v3.mxd



Source: A8 - USGS Sonar 2004
A8S - LIDAR, 2004

N

0 1,000 2,000 3,000 Feet

10 acres

figure **4**

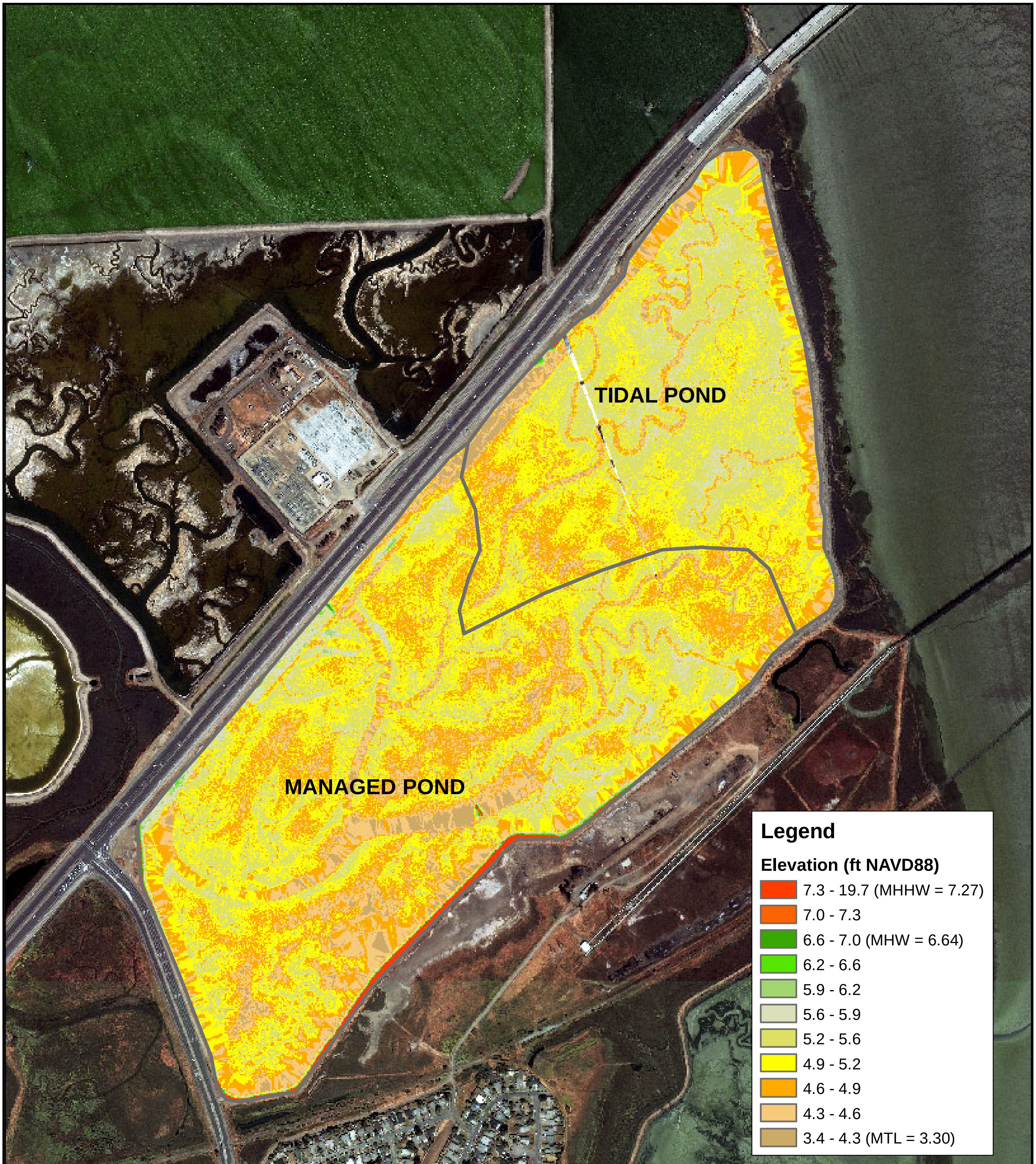
South Bay Salt Ponds Restoration Project

Topography of Pond A8 (Reversibly Tidal Pond)

Proj #1751.01

PWA

G:\1720SBSP\Phase1Actions\Projects\Pond A8v2.mxd



Source: LIDAR, 2004

10 acres

0 250 500 1,000 1,500 2,000 Feet

figure **5**

South Bay Salt Ponds Restoration

Topography of Pond SF2 (Tidal and Managed Portions)

Proj. # 1751.01

PWA

G:\1720SBSP\Phase1Actions\Projects\SF2v2.mxd

Hydrology, Flood Management and Infrastructure

Impact Analysis Support

G-3. Eden Landing Ponds E8A, E9 and E8X Hydrodynamic Modeling and Geomorphic Analysis

MEMORANDUM

TO: Members of the South Bay Salt Pond Restoration Project Management Team

FROM: Philip Williams and Associates, Ltd. (PWA)

DATE: October 22, 2006

RE: **Eden Landing Ponds E8A, E9 and E8X Hydrodynamic Modeling and Geomorphic Analysis**

Note: The hydrodynamic modeling and geomorphic analysis documented in this memorandum were performed for a previous version of the Phase 1 action in which Ponds E8A, E9, and E8X were breached to Old Alameda Creek (OAC), North Creek, and the small historic Mt. Eden Creek channel mouth to the south of Mt. Eden Creek. In this previous version of the Phase 1 action, Pond E9 was not breached to the current (larger) Mt. Eden Creek channel that was constructed as part of the Eden Landing Ecological Reserve restoration project. In the current version of the Phase 1 action, Pond E9 would be breached to the current Mt. Eden Creek channel. Breaching Pond E9 to the current Mt. Eden Creek is expected to improve tidal drainage and provide additional fluvial discharge capacity compared to the previous version of the Phase 1 action analyzed in this memorandum. Additional modeling will be performed during the design phase of the restoration to confirm the evaluation of potential impacts on OAC and Mt. Eden Creek water levels for the current version of the Phase 1 action. (October 3, 2007)

1. INTRODUCTION

Tidal wetland restoration at Ponds E8A, E8X, and E9 has been identified as an early restoration action of the South Bay Salt Pond Restoration (SBSP) Project. Figure 2-8 of the EIS/R shows the proposed restoration plan at these ponds. This memorandum describes the hydrodynamic modeling and hydraulic geometry analyses PWA performed in support of the restoration planning and environmental review (i.e., NEPA/CEQA documentation) for this 'Phase 1' action.

Computer-based simulation of tidal hydrodynamics and flood water levels were performed to assess whether the proposed restoration actions at Ponds E8A, E8X, and E9 would:

- *Increase peak water levels during a combined coastal and fluvial flood event*. The proposed restoration actions must not increase flood hazards over either the short- or long-term. The potential impact of the proposed restoration actions on flood hazards is established by comparing baseline and post-project water levels, especially adjacent to developed areas along Old Alameda Creek (OAC) upstream of the 20-gate structure.
- *Provide adequate tidal circulation within the ponds, adjacent sloughs, and the Eden Landing Ecological Reserve (ELER)*. Prolonged tidal damping could delay habitat development in the

restored ponds if elevation-inundation characteristics are substantially different from the natural tidal regime.   Evaluation of the tidal characteristics resulting from the proposed restoration actions will allow managers to evaluate trade-offs between various elements of the restoration plan.

PWA applied hydraulic geometry relationships to assess possible changes to the tidal sloughs and pond channels as the restored site evolves. Results from this analysis provided input for breach sizing and slough bathymetry in the long-term hydrodynamic model runs.

2. HYDRAULIC GEOMETRY ANALYSES

Historically, Old Mt. Eden Creek and Old Alameda Creek drained expansive areas of tidal marsh and received freshwater discharges from the watershed. Following the construction of levees to create the salt ponds, the tidal sloughs were isolated from the contributing tidal marsh area, reducing the tidal prism exchanged through the sloughs. Mt. Eden Creek and North Creek were recently excavated as part of the ELER Project. These channels were excavated to drain portions of the ELER restoration project site, which is currently planned for tidal restoration in 2007. Restoring Ponds E8A, E9 and E8X to tidal inundation by breaching the levees will increase the drainage area and tidal prism in Mt. Eden Creek, North Creek and Old Alameda Creek, inducing tidal scour as the channels adjust towards a new equilibrium in balance with the restored tidal prism.

2.1 Methods

Hydraulic geometry relationships developed by Williams and others (2002) for San Francisco Bay express channel width, depth and cross-sectional area as a function of tidal prism or marsh area. These relationships were used to predict long-term equilibrium slough dimensions and establish preliminary estimates of breach sizes. In addition to describing future restored conditions, this information provided input to the hydrodynamic modeling of the Year 50 scenarios.

Calculation of long-term equilibrium slough dimensions assumes that mature marsh develops within the ponds over the 50-year planning horizon (i.e., marsh plain elevations would be close to MHHW and tidal channel networks would be fully developed). This assumption is consistent with analysis presented in the South Bay Geomorphic Assessment (PWA in progress, Appendix F). Long-term channel depth and cross-sectional area were calculated as a function of contributing marsh area, including the ELER and SBSP Restoration Project restored marsh areas. Channel depth and cross-sectional area were assumed to be the most important dimensions affecting channel conveyance. Channel width was calculated by assuming the slough cross-section was parabolic and specifying its depth and area. This approach was repeated at multiple locations along Mt Eden Creek, North Creek, and OAC to estimate channel dimensions for the Year 50 model scenario.

Breach locations were selected such that restored tidal flows would focus on the relict channel network inside Ponds E8A, E9 and E8X and gradually re-occupy the historic drainage network. For the purposes of providing initial estimates of breach geometry for modeling purposes, the five perimeter breaches were ‘oversized’ relative to the total marsh area of the three ponds. Specifically, breaches sizes were computed from hydraulic geometry relationships assuming 100% of the restored pond area drained to Mt Eden Creek (from Pond E9), 100% to OAC (50% through each breach from Pond E8A), and 60% to North Creek (30% each from Pond E8X). This approach resulted in oversized breaches (i.e., collectively, the five perimeter breaches have the capacity to drain 260% of the long-term marsh area) but was useful for modeling purposes; it prevents the perimeter breaches from restricting tidal flows and allows the hydrodynamic model to simulate how the adjacent sloughs will convey the restored tidal prism.

2.2 Results

Existing and Year 50 predicted cross sections for two locations – (1) OAC downstream of North Creek and the Pond E8A breaches and (2) Mt. Eden Creek downstream of the Pond E9 breach – are shown in Figures

Figure **1**. These results assume that all of the restored tidal prism will scour the northern channel of the two-channel OAC due to its direct connection with the Pond E8 and North Creek. The geometry used for the Year 0 oversized breaches and the Year 50 scoured slough are presented in Table 1.

Table 1. Estimated Dimensions of Perimeter Breaches and Long-Term Slough Geometry

Location	**Thalweg Depth Below MHHW (ft)**	**Top Width (ft)**	**Cross-sectional area Below MHHW (ft^2)**
Pond E9/Mt. Eden Creek Breach	13.2	240	1862
Pond E8A/OAC Breaches	11.4	154	1091
Pond E8X/North Creek Breaches	10.3	124	224
Mt. Eden Creek	11.0	127	927
N. Creek (upstream of breaches)	10.5	113	791
N. Creek (downstream of breaches)	11.9	158	1245
OAC (upstream of breaches)	12.9	199	1705
OAC (downstream of breaches)	14.0	250	2327

3. HYDRODYNAMIC MODELING

3.1 Methods

Model simulations were performed to address the key questions listed above and included flood and dry-weather simulations for Baseline, Project Year 0, and Project Year 50 scenarios. The flood runs covered seven days, with the peak flood conditions timed to occur in the middle of the simulation. Dry-weather simulations spanned seventeen days to capture spring/neap variability in the tides. Our modeling approach builds upon previous efforts (URS, 2002a, 2002b, 2004; Kamman Hydrology and Engineering, 2001) and information from Alameda County Flood Control District staff.

<u>*Model Extent and Schematization.*</u> The one-dimensional MIKE11 representation of OAC extends from I-880 to the Bay, with former salt ponds north of the creek channel schematized as a parallel floodplain. Approximately 3.4 miles upstream of the Bay, the OAC schematization includes a 20-tide gate structure

that spans the slough (Figure 2). MIKE11 branches of Mt Eden Creek and North Creek extend, respectively, from ELER to the bay and OAC. As in previous efforts, the ELER is schematized as prismatic channels with 'additional storage' nodes (Kamman Hydrology and Engineering, 2001). Ponds E8A, E9, and E8X are schematized as a two-dimensional MIKE21 element and dynamically coupled to MIKE11 branches of OAC, Mt Eden Creek and North Creek using the MIKEFLOOD interface at the five perimeter breaches. Interior breaches between the ponds are resolved in the 5-meter MIKE21 grid directly. These breaches were sized according to long-term hydraulic geometry dimensions.

Bathymetry Data. MIKE11 components of the model was schematized with the proposed Mt. Eden Creek and N. Creek alignments, dimensions, stage-storage characteristics of the ELER marsh, and bed elevations presented in the East Bay Regional Park District's (EBRPD) Site Plan for 2000 (Kamman 2001). Existing slough geometry of OAC was developed from the cross-sectional surveys provided by EBRPD and dated August 3, 2000.

The MIKE21 component of the model (Ponds E8A, E8X, and E9) was developed using 25-meter USGS Sonar data of Pond E9 and 1-meter LIDAR data of Ponds E8A and E8X collected in 2004 as part of the SBSP Project (Foxgrover, 2005; Terrapoint, 2005). PWA resolved these data in a 5-meter rectangular grid, which balances simulation runtimes with the ability to resolve internal flow features of interest.

Boundary Conditions. Both flood and dry-weather simulations were completed for all Baseline and Project scenarios. PWA specified different time-variable water levels and hydrographs at the downstream and upstream boundaries, respectively, for each type of simulation (flood- or dry-weather).

- Flood Scenario. Coincident extreme conditions were applied at the upstream and downstream boundaries to account for combined fluvial and coastal effects. In particular, the flood simulations incorporated the 24-hour, 15-year design flood hydrograph for OAC at the upstream boundary and a 10-year tide signal at the downstream boundaries at the mouths of OAC and Mt Eden Creek. The downstream boundary was increased by 0.5 ft in all Year 50 simulations to account for future sea level rise.

  Timing of the downstream and upstream boundary conditions was selected to produce the highest peak water levels in the developed areas upstream of the 20-gate structure. PWA performed a series of model runs that determined this 'worst case' scenario resulted when the hydrograph and high tide arrived at the 20-gate structure at the same time. Since the travel time between both the fluvial and tidal boundaries to the 20-gate structure is approximately 90 minutes, the fluvial hydrograph and peak high tide were released at their respective boundaries at the same time. Figure 3 and Figure 4 show the difference in water levels that result when high tide leads or lags the fluvial hydrograph by ±3 hours. In addition to these boundary conditions, a constant discharge of 10,000 gallons per minute (gpm) was applied along Mt Eden Creek to account for the lift station that drains the industrial area to the north.

- Dry-Weather Scenario. A time series of water levels simulated by the DELFT3D model for SBSP Alternative A (Year 0) for summer 2001 conditions was applied at the mouths of OAC and

Mt. Eden Creek for the dry-weather scenario. The 17-day simulation included both spring and neap tides. A constant discharge of 15 cubic feet per second (cfs) was applied at the upstream boundary of OAC for this dry-weather scenario.

Table 2 summarizes the tidal and fluvial boundary conditions applied for both the flood and dry-weather scenarios. A list of the key simulations reported in this memorandum is provided in Table 3.

Table 2. Summary of Boundary Conditions for Hydrodynamic Modeling

	Upstream Boundary (OAC at I-880)	**Downstream Boundaries (OAC, Mt Eden Ck, and historical Mt. Eden Ck at Bay)**
Flood Scenario	15-Year flood hydrograph for a 24-hr storm. Peak discharge is 3,800 cfs (design discharge) and base flow is 156 cfs.	Measured tides from San Mateo Bridge for Jan 2001 adjusted such that peak tide matches 10-Year water level (9.5 ft NAVD88 for short-term and 10.0 ft NAVD88 for long-term).
Dry-Weather	Constant 15 cfs baseflow.	Summer 2001 tides simulated by SBSP DELFT3D model.

Notes: Measured San Mateo Bridge tides from NOAA Station # 9414458. NAVD88 conversion calculated using unpublished NOAA conversions from MLLW.

Table 3. Run Catalog for Key Simulations

Run ID	Name	Slough Morphology	Pond / Marsh Morphology	Boundary Conditions	MFLOOD links	Comments
1-D	Baseline Conditions Dry-weather Year 0	Existing	ELER marshes schematized as prismatic channels + storage nodes.	Dry-weather BCs	None	Assumes ELER Project does not scour sloughs.
1-F	Baseline Conditions Flood Scenario Year 0	Existing	Same as above	Flood BCs	None	Same as above
2-D	Project Conditions Dry-weather Year 0	Existing	No pond sedimentation; Shallow relict channels.	Dry-weather BCs	Five at perimeter breaches	Representative of short-term conditions, before sloughs scour and efficient drainage network is established.
2-F	Project Conditions Flood Scenario Year 0	Existing	Same as above	Flood BCs	Same as above	Same as above
3-D	Project Conditions Dry-weather Year 50	Sloughs scoured to long-term equilibrium	Pond sedimentation to future MHHW; Scoured pond channels.	Dry-weather with tides adjusted by 0.5-ft sea level rise adjustment	Same as above	Representative of long-term conditions; estuarine sedimentation builds marsh and tidal scour enlarges sloughs + pond channels.
3-F	Project Conditions Flood Scenario Year 50	Sloughs scoured to long-term equilibrium	Same as above	Flood BCs with tides adjusted by 0.5-ft sea level rise adjustment	Same as above	Same as above

3.2 Results

Peak water levels along Old Alameda Creek. The Phase 1 action at Ponds E8A, E9, and E8X would modify existing surface drainage patterns in the adjacent sloughs and has the potential to affect water levels during flood events. PWA evaluated the potential changes to flood water levels along OAC, Mt. Eden Creek, and North Creek over the short- and long-term by comparing model results from Baseline and Project conditions.

As shown in Figure 5, implementation of the proposed Phase 1 action results in negligible changes to peak water surface elevations along OAC under Year 0 conditions. Changes in peak water levels at Year 50 result from the assumed sea level rise and diminish with distance from the Bay. The simulated water levels immediately upstream of the 20-gate structure during the flood scenario are plotted in Figure 6 for Baseline and Project conditions. These results indicate damping of the low tide (i.e., water levels during low tide are elevated) over the short-term, presumably in response to the limitations on conveyance caused by the undersized downstream channel (see hydraulic geometry discussion above). In the long-term, channel scour improves conveyance, and low-tide water levels before and after the peak event drop below Baseline conditions.

Tidal damping. Model results suggest that both high and low tides are damped along OAC under Baseline conditions, presumably due to tidal restoration associated with the ELER Project (Figure 7). Under these conditions, high and low tide water levels during spring tides are damped up to approximately 0.75 ft and 2 ft, respectively. Damping is less substantial during neap tides. Over the short-term, implementation of Phase 1 actions would result in additional tidal damping along OAC of approximately 0.3 ft at low tide during spring tides.

Based on the Year 0 simulated water levels presented in Figure 8 and Figure 9, substantial tidal damping in the ponds is expected immediately after breaching. Poor low-tide drainage is particularly pronounced in Pond E9 due to the undersized channel at the historic mouth of Mt Eden Creek and lower pond bed elevations, and may result in extensive shallow-water ponding. Although some flow will pass to OAC via Pond E8A, poor drainage in Pond E9 is expected to persist until the channel along the historic mouth of Mt Eden Creek enlarges sufficiently.

SBSP Sonar and Lidar data suggest that the relict channel network inside the pond are relatively shallow compared to natural marsh channels. This was confirmed by elevation transects collected by PWA during a field reconnaissance on September 15, 2006, which also indicated a layer of less competent sediment along the bottom of relict channels (Figure 10 and Figure 11). These relict channels are expected to scour over time due to preferential erosion and result in a more natural tidal regime within the restored ponds and adjacent sloughs (Figure 12 and Figure 13). Based on observed rates of headcutting at former salt ponds restored to tidal action [Napa Salt Pond 3 (65 ft/month) and Cooley Landing (80 to 115 ft/month)], we expect the tidal range in Ponds E8A, E8X and E9 to gradually increase over 5 to 10 years as a more hydraulically efficient drainage network forms inside the restored ponds. Based on observed rates of headcutting at Sonoma Baylands pilot channel, the historical Mt. Eden Creek mouth is expected to scour over 10 years or more. Addition reconnaissance of the historic mouth of Mt Eden Creek and comparison with appropriate reference sites (e.g., Sonoma Baylands) will help improve this estimate during subsequent design phases.

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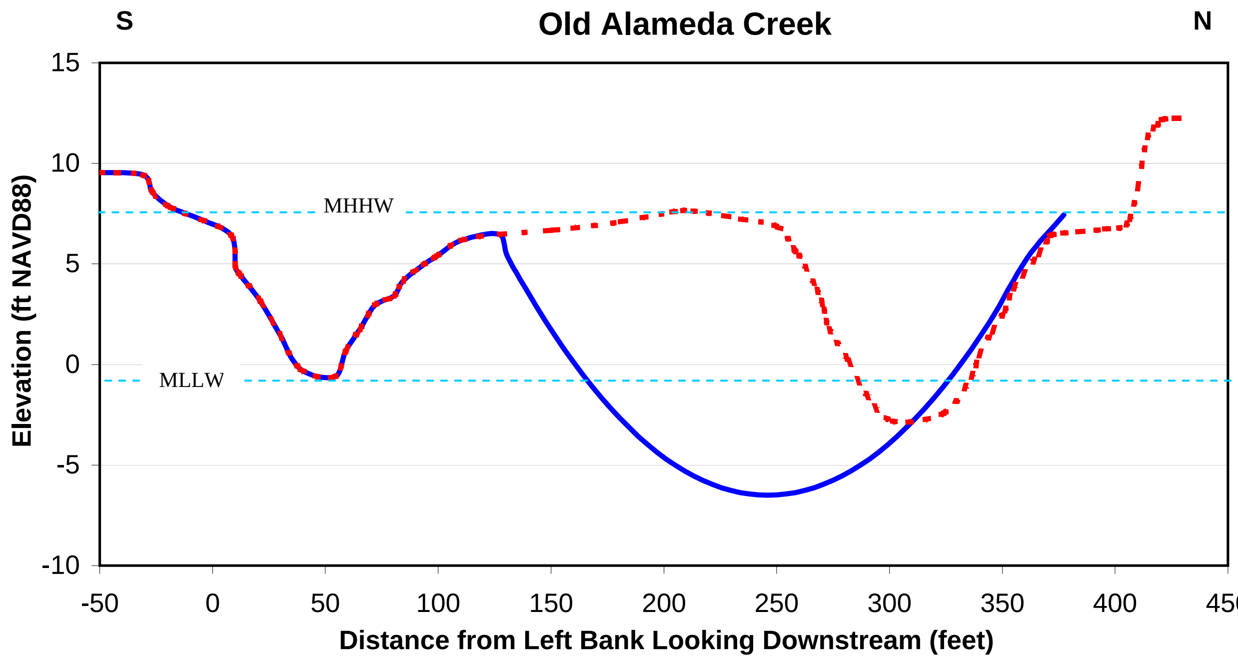
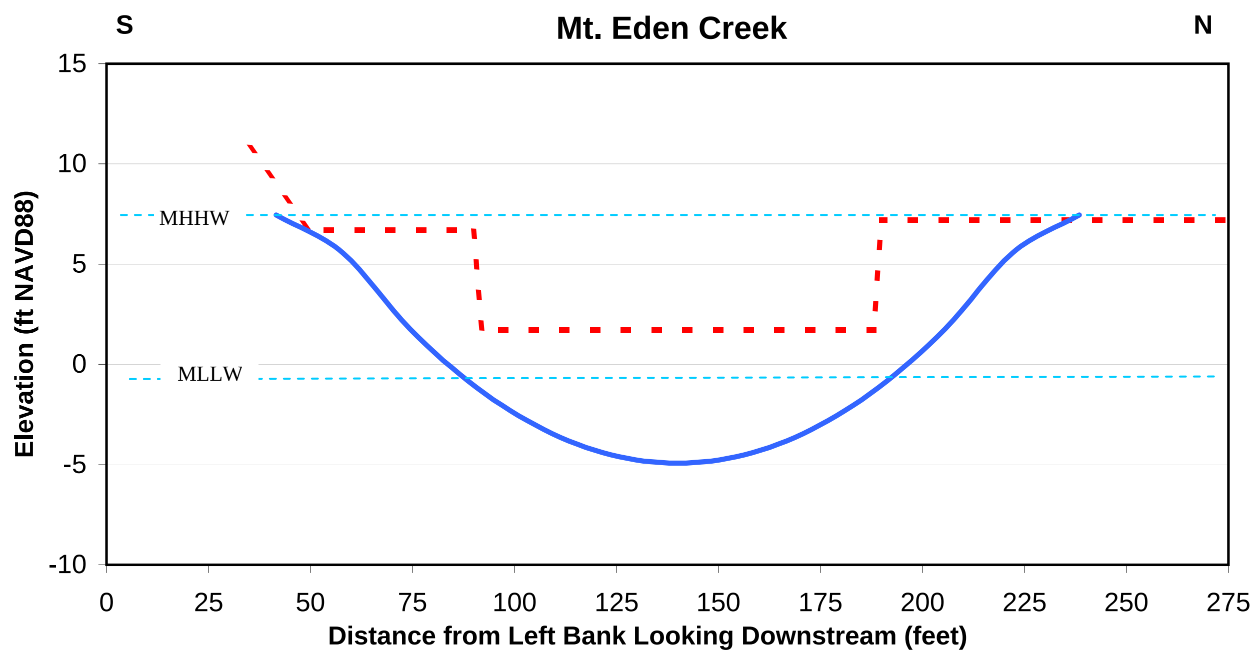
Source files for this report are located at PWA:
\\Orca\pwa\Projects\1825 Alameda County Salt Pond Integration\1825.03_OAC_phaseI\NEPA.CEQA\ModelingAppendix\

\\Orca\pwa\Projects\1825 Alameda County Salt Pond Integration\1825.03_OAC_phaseI\NEPA.CEQA\ModelingAppendix\AppendixG-3_PondsE8E9E8X.doc

FIGURES





Existing Conditions

Year 50 after Phase 1 and ELER restoration

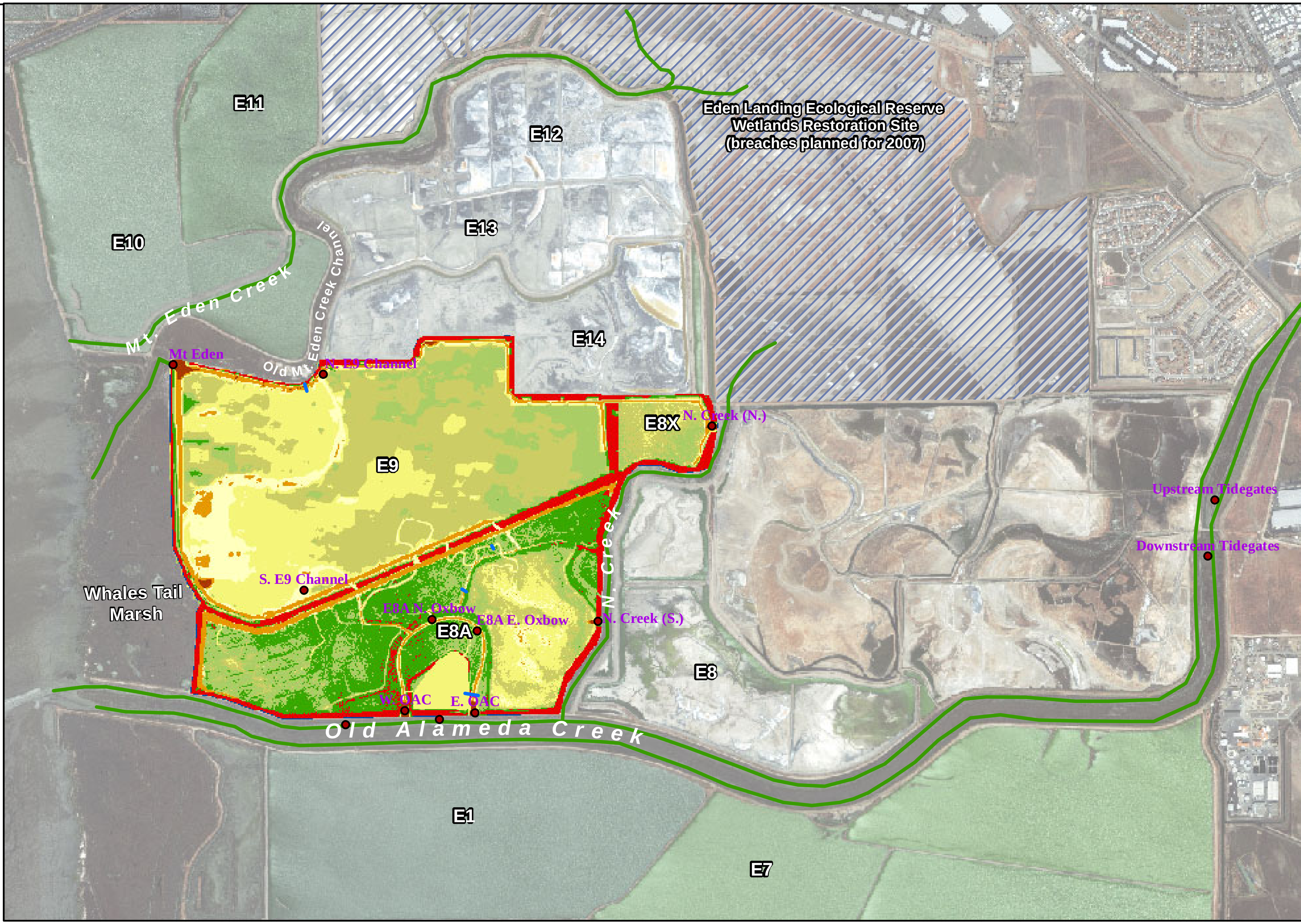
Notes: Cross-sections are downstream of all planned levee breaches. Predicted equilibrium channel geometries are for long-term conditions and assume restored marsh areas accrete to the elevation of mature marshes (MHHW). Predictions are based on hydraulic geometry relationships between mature marsh area and channel depth and area (Williams and others, 2002).

figure 1

South Bay Salt Ponds Restoration Project

Short- and Long-term Slough Geometry

PWA



Model Bathymetry Source: USGS Sonar (Ponds E9); LIDAR, 2004 (Pond E8, E8X)

0 500 1,000 1,500 2,000 2,500 Feet

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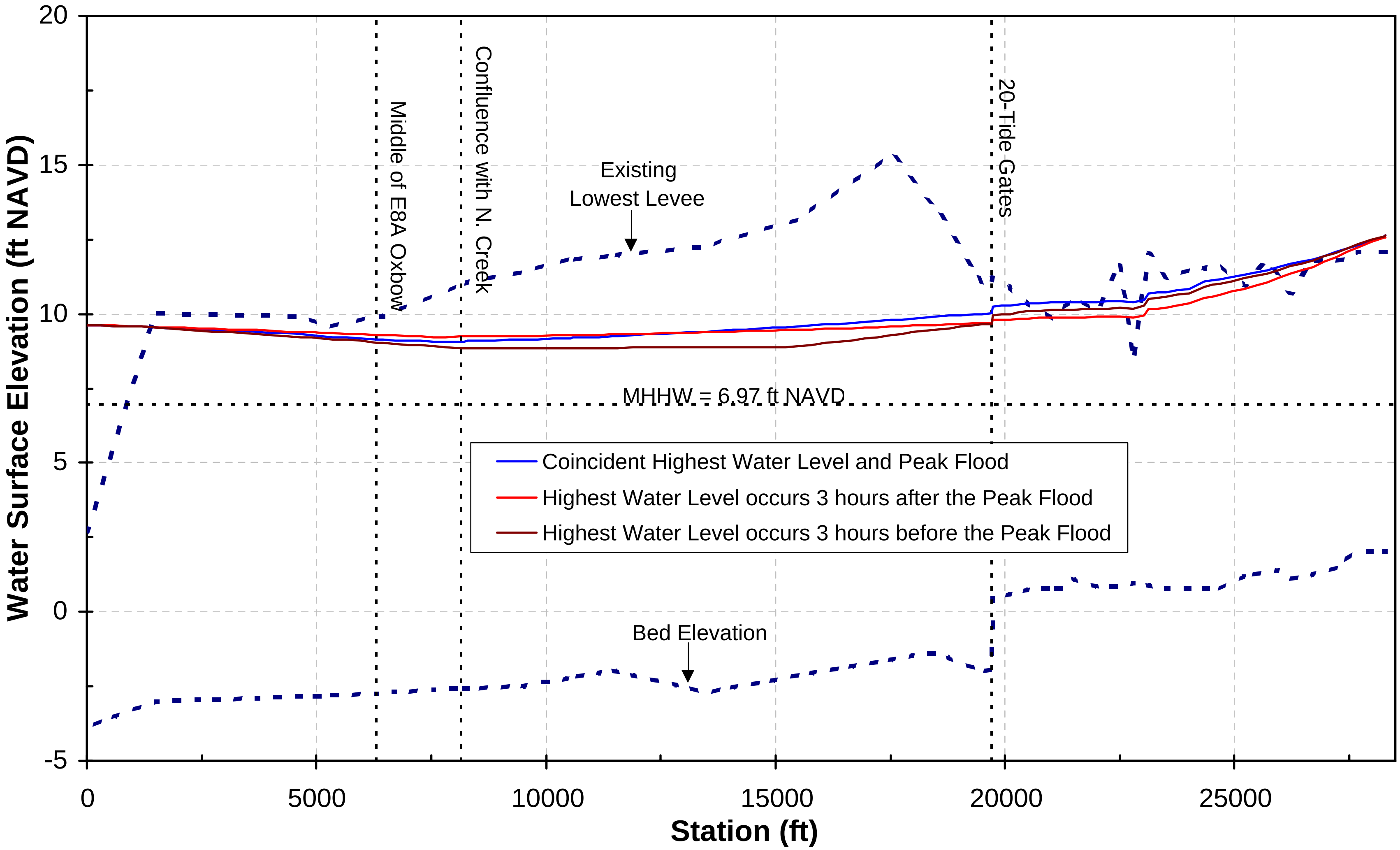
Figure 2

South Bay Salt Ponds Restoration Project

Model Extraction Points and Pond Topography

PWA Ref 1825.03

PWA



Notes:
Results simulated with MIKEFLOOD
Initial Water Surface Elevation = 6.6 ft NAVD
All simulations include existing sloughs

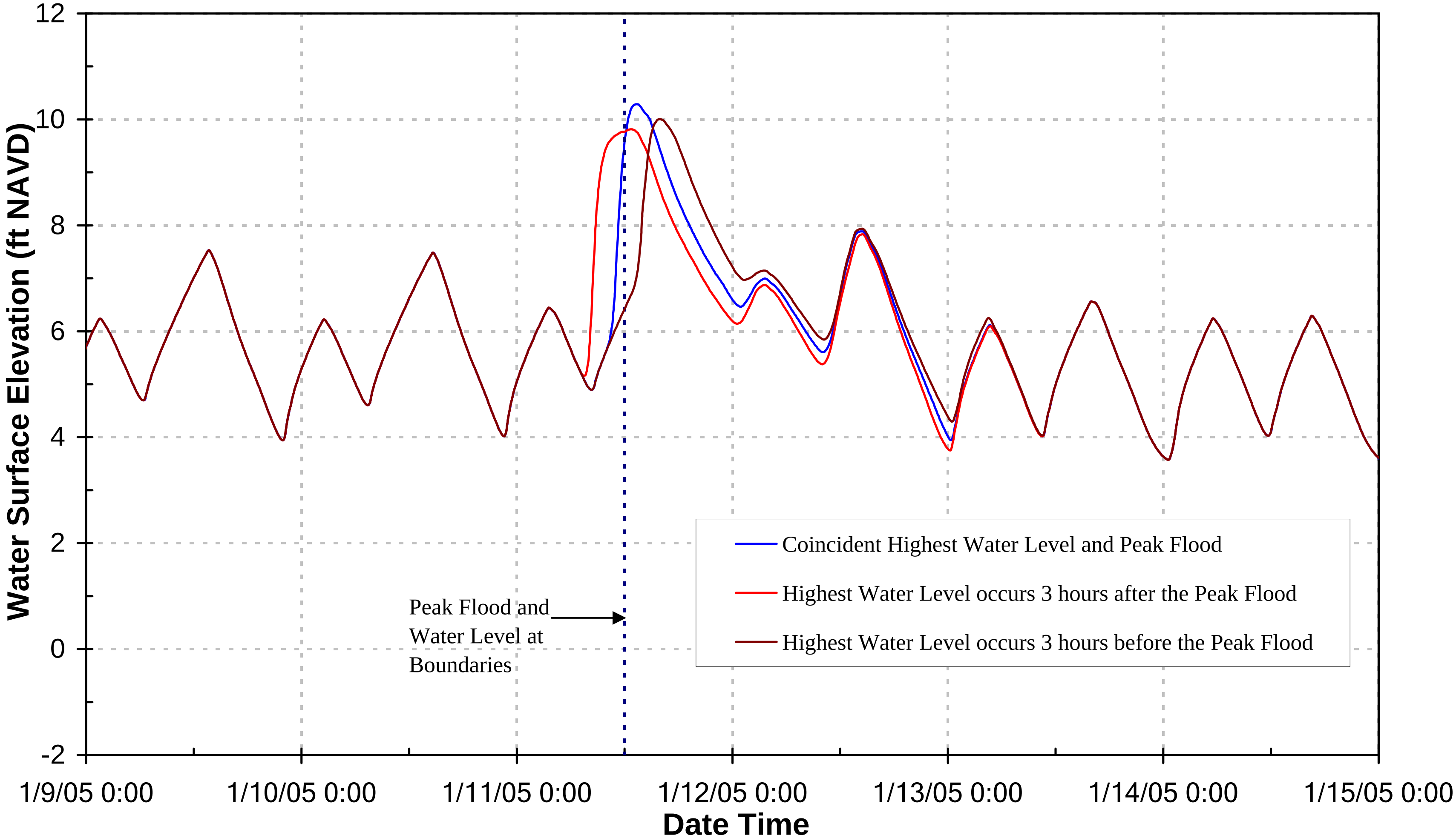
Figure 3

South Bay Salt Ponds Restoration Project

Maximum WSE along OAC for Different Flood Timing

PWA Ref 1825.03

PWA



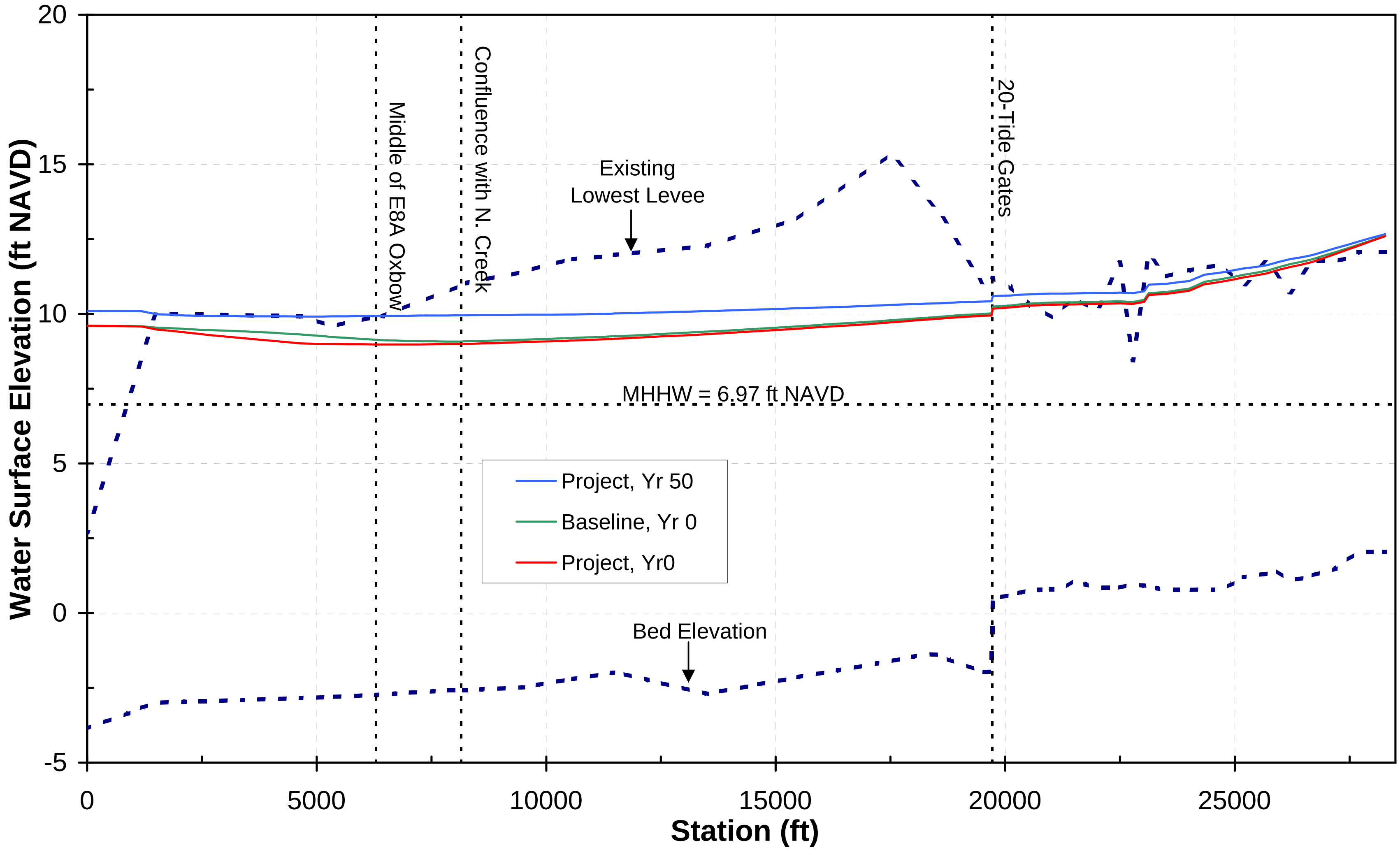
Notes:
Results simulated with MIKEFLOOD
Initial Water Surface Elevation = 6.6 ft NAVD
Peak Discharge and Tide at the Boundaries (1/11/05 12:00)

Figure 4

South Bay Salt Ponds Restoration Project

Simulated WSE Upstream of 20-Tide Gates

PWA Ref 1825.03



Notes:
Results simulated with MIKEFLOOD
Initial Water Surface Elevation = 6.6 ft NAVD
Project includes 5 perimeter breaches
Yr 0 scenarios include existing sloughs
Project, Yr 50, includes scoured sloughs, deepened channels and accreted ponds

Figure 5

South Bay Salt Ponds Restoration Project

Simulated Maximum WSE along OAC

PWA Ref 1825.03

PWA

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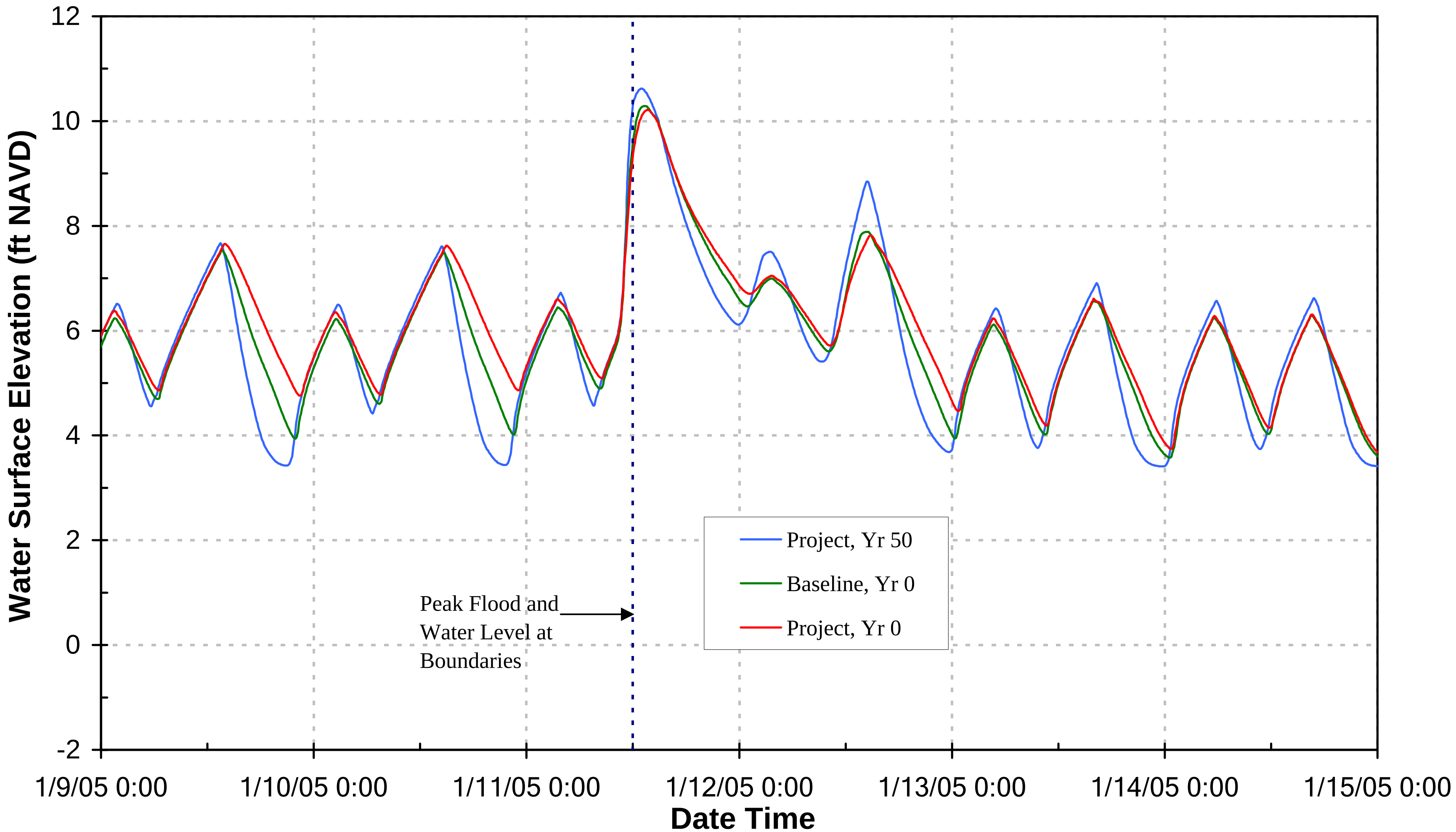
10/22/2006



Notes:
Results simulated with MIKEFLOOD
Initial Water Surface Elevation = 6.6 ft NAVD
Peak Discharge and Tide at the Boundaries (1/11/05 12:00)

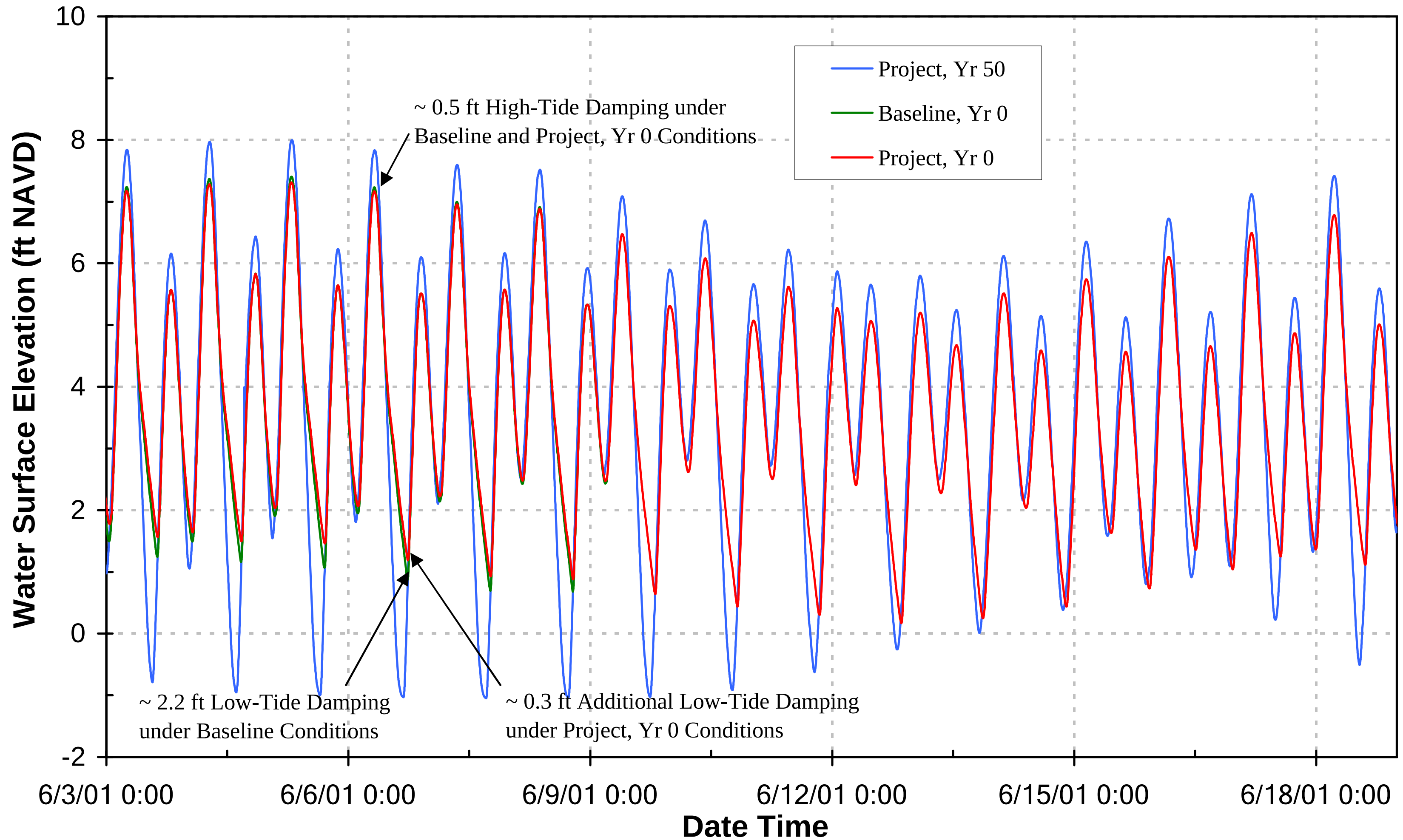
Figure 6

South Bay Salt Ponds Restoration Project

Simulated WSE Upstream of 20-Tide Gates

PWA Ref 1825.03

PWA



Notes:
Results simulated with MIKEFLOOD
Initial Water Surface Elevation = 6.6 ft NAVD

Figure 7

South Bay Salt Ponds Restoration Project

Simulated WSE Downstream of OAC Breaches

PWA Ref 1825.03

PWA

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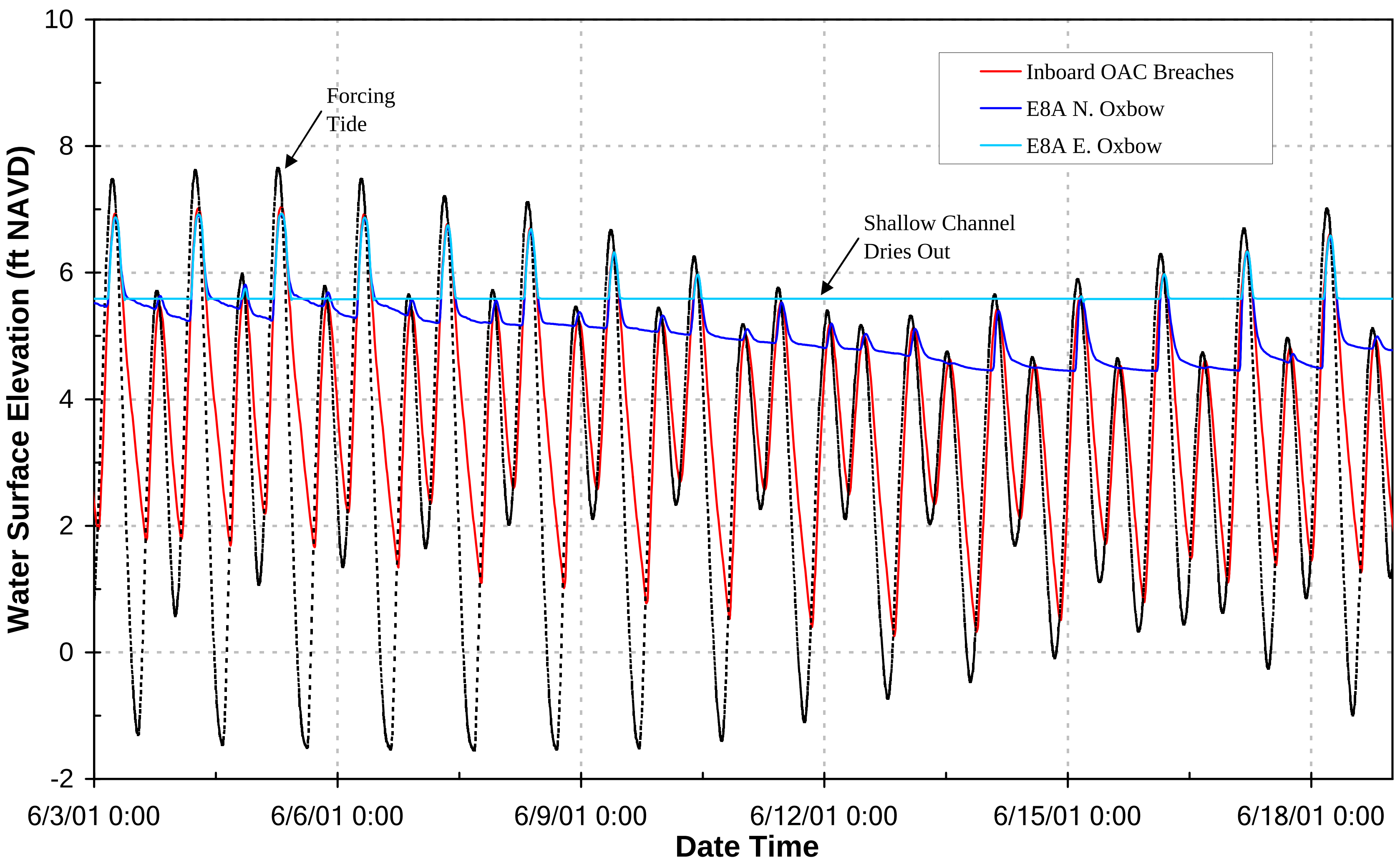


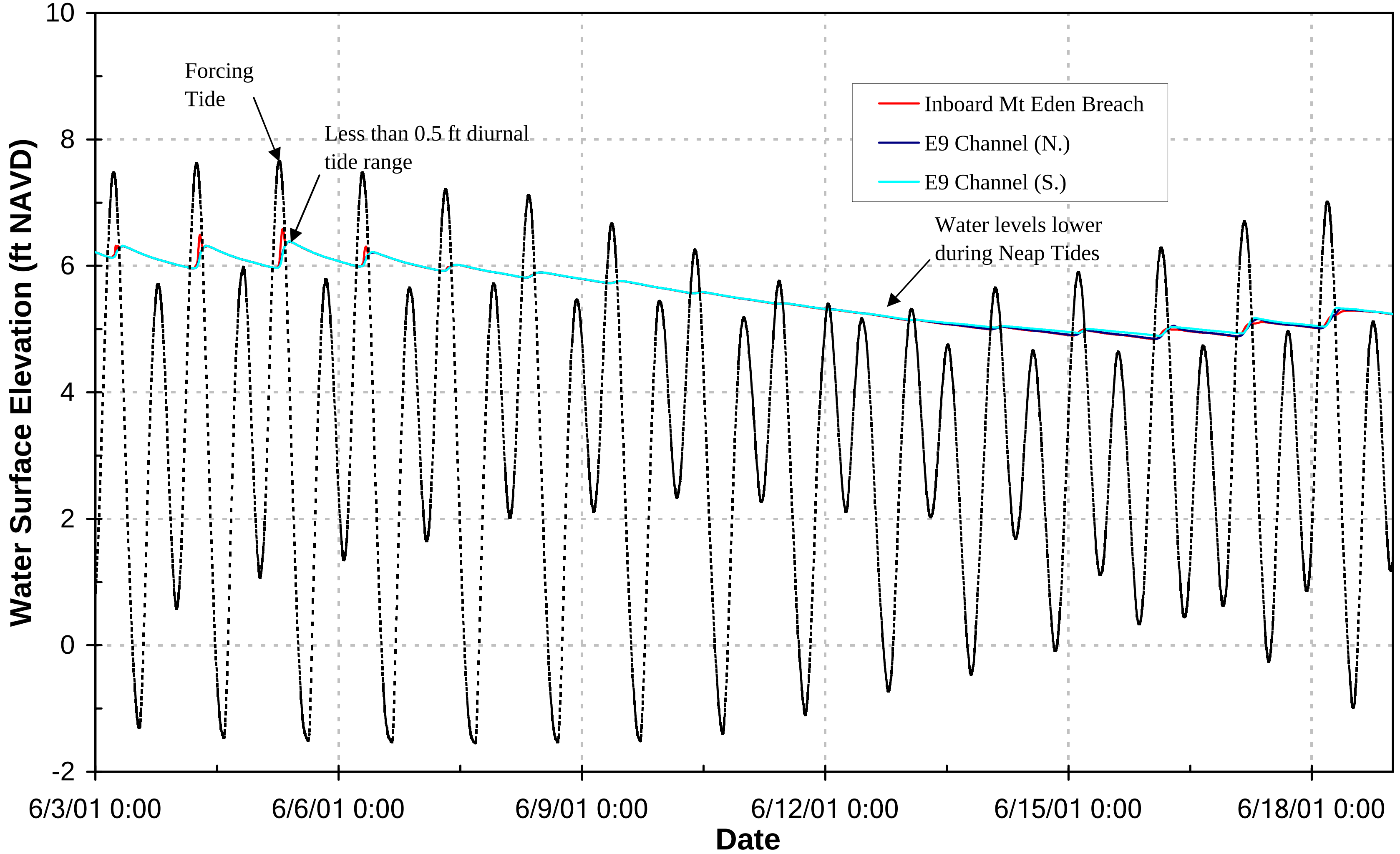
Notes:
Initial Water Surface Elevation = 6. 6 ft NAVD

Figure 8

South Bay Salt Ponds Restoration Project

Simulated WSE in E8A, Project, Yr 0

PWA Ref 1825.03



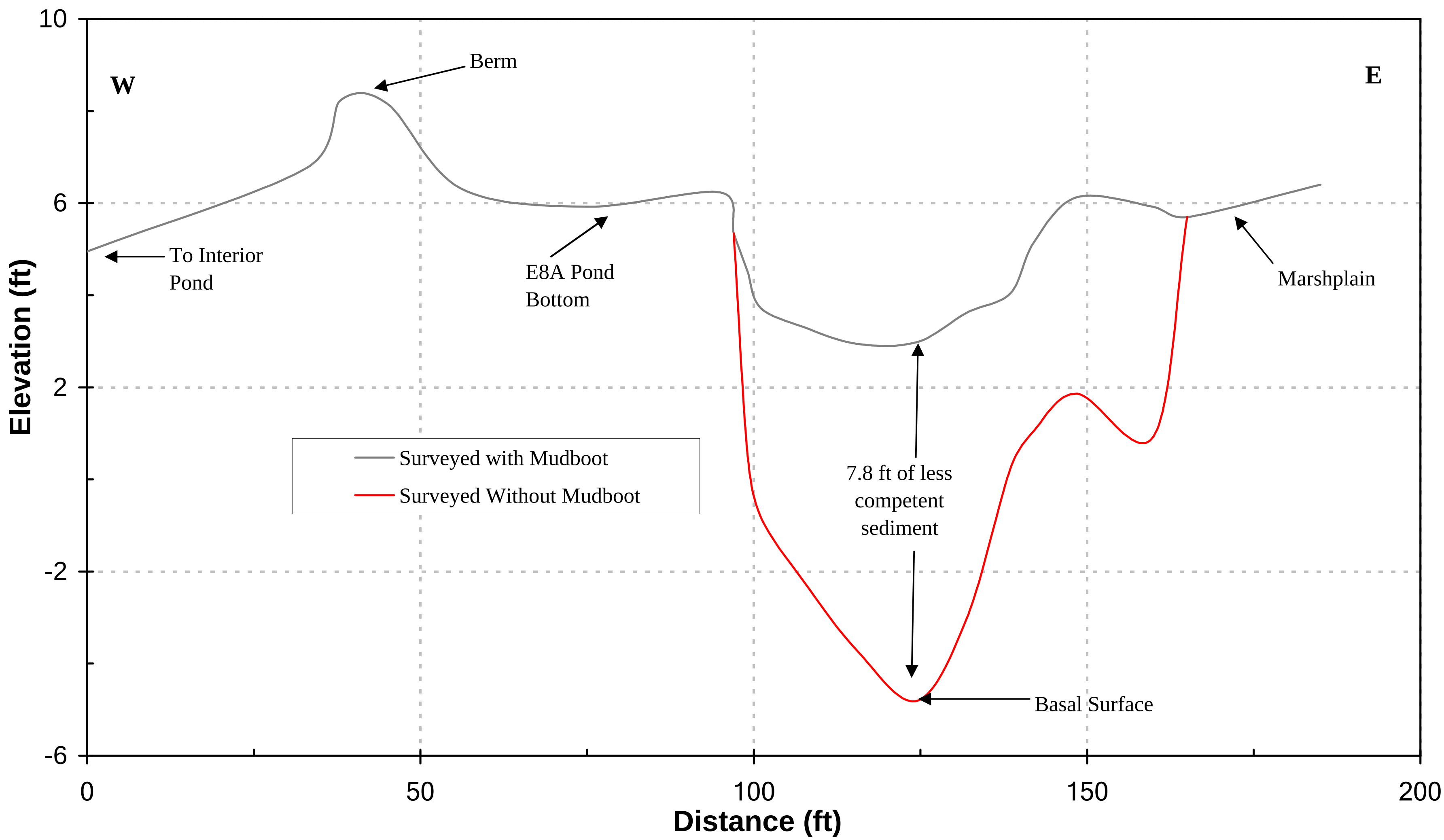
Notes:
Initial Water Surface Elevation = 6. 6 ft NAVD

Figure 9

South Bay Salt Ponds Restoration Project

Simulated WSE in E9, Yr 0, Existing Channels

PWA Ref 1825.03



Notes:
PWA performed a Reconnaissance Survey on September 15, 2006
The survey was not tied into a vertical control
The mudboot off profile in the channel may vary as much as 1.5 ft due to the hardness of the basal surface

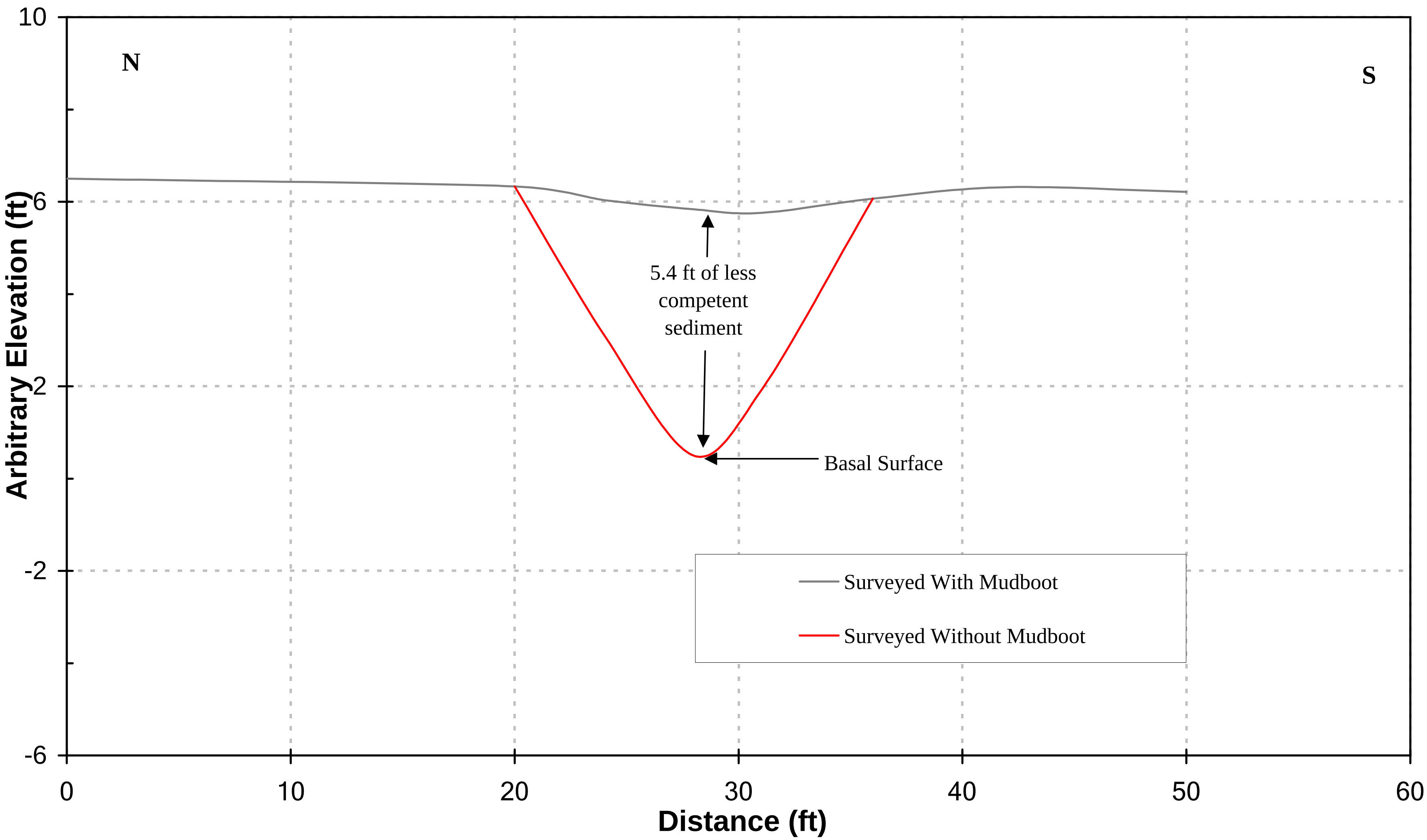
Figure 10

South Bay Salt Ponds Restoration Project

Surveyed Transect of E8A E. Oxbow

PWA Ref 1825.03

PWA



Notes:
PWA performed a Reconnaissance Survey on September 15, 2006
The survey was not tied into a vertical control
The mudboot off profile in the channel may vary as much as 1.5 ft due to the hardness of the basal surface

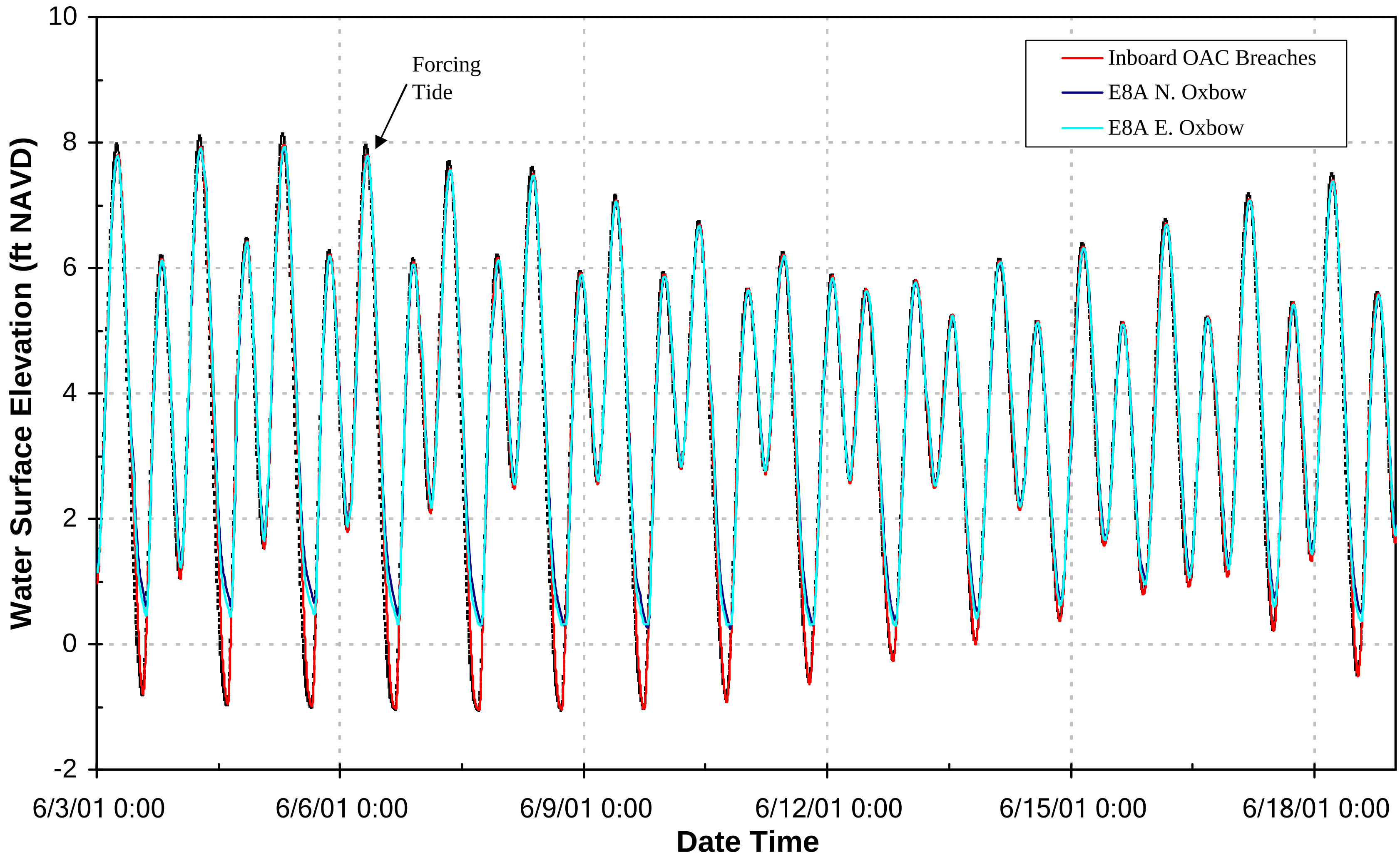
Figure 11

South Bay Salt Ponds Restoration Project

Surveyed Transect of E8A N. Historical Channel

PWA Ref 1825.03

PWA



Notes:
Initial Water Surface Elevation = 6. 6 ft NAVD

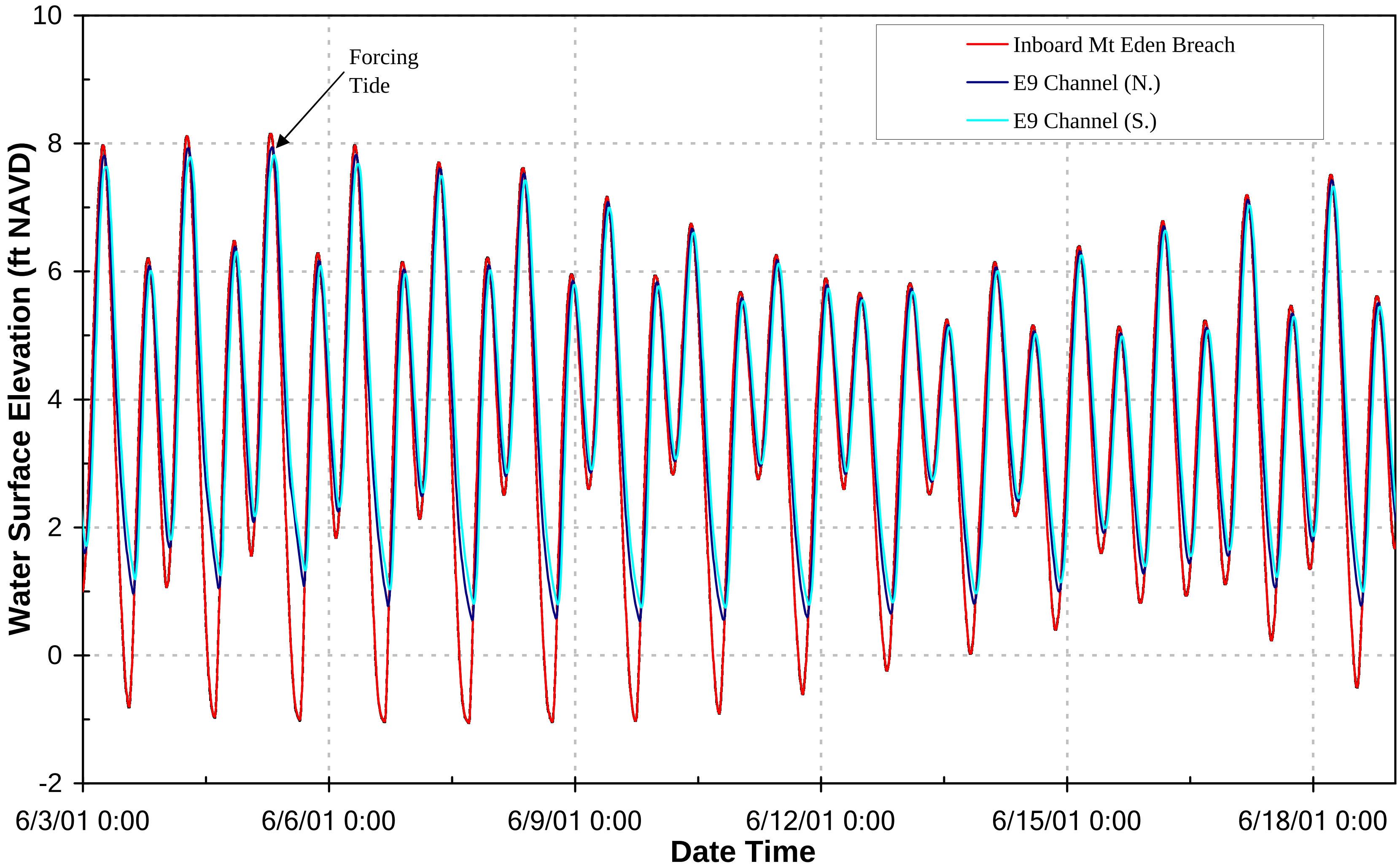
Figure 12

South Bay Salt Ponds Restoration Project

Simulated WSE in E8A, Project, Yr 50

PWA Ref 1825.03

PWA



Notes:
Initial Water Surface Elevation = 6. 6 ft NAVD

Figure 13

South Bay Salt Ponds Restoration Project

Simulated WSE in E9, Yr 50

PWA Ref 1825.03

PWA

Hydrology, Flood Management and Infrastructure

Impact Analysis Support

G-4. Eden Landing Ponds E12 and E13 Water and Salt Balance Modeling

South Bay Salt Pond Restoration Project

M E M O R A N D U M

TO: Members of the South Bay Salt Pond Restoration Project Management Team

FROM: Philip Williams & Associates, Ltd. (PWA)

DATE: October 26, 2006 – *Revised February 6, 2007*

RE: **Eden Landing Ponds E12 and E13 Water and Salt Balance Modeling**

---

1. INTRODUCTION

The South Bay Salt Pond Restoration (SBSP) Project Phase 1 restoration action at Eden Landing Ponds E12 and E13 would reconfigure these managed ponds to create shallow water habitat (approximately six inches or 15 cm deep) with a range of salinities for migratory shorebirds. Section 2.5.2 and Figure 2-10 of the SBSP EIR/S describe and show the proposed restoration plan for Ponds E12 and E13. This memorandum describes the modeling and assessments performed by PWA in support of the restoration planning and environmental review (i.e., NEPA/CEQA documentation) for this Phase 1 action.

PWA performed computer-based simulations of managed pond salinities and water depths to assess the proposed restoration action at Ponds E12 and E13. PWA also performed technical assessments of pumping intake water into Ponds E12 and E13 and the proposed high-salinity discharge mixing basin. The purposes of the simulations and assessments were to:

- *Assess whether the Ponds E12 and E13 restoration would provide target shallow water depths for migratory shorebird habitat and a range of salinities throughout the dry season.* The proposed Ponds E12 and E13 restoration action is intended to provide shallow water depths that are optimal for shorebird foraging habitat. Each pond would be divided into three cells (Figure 1) with progressively increasing salinity levels in each cell. Ponds E12 and E13 would each be managed to maintain one cell with low salinities (approximately 20 to 40 parts per thousand or ppt), one cell with moderate salinities (approximately 40 to 80 ppt), and one cell with high salinities (approximately 80 to 140 ppt) to test the importance of salinity on the density of foraging shorebirds and their prey. Target water depths in each cell range from approximately two inches (five centimeters) to one foot (0.3 meters), with an average of approximately six inches (15 cm). The average depth should vary between approximately four to eight inches (10 to 20 cm), but no more, to maintain optimal depths for foraging habitat.
- *Assess the need for active pumping of intake water into Ponds E12 and E13 to manage water depths and salinities*. Ponds E12 and E13 are high in elevation relative to the tides. The potential for gravity flows into the ponds is limited, especially during neap tides when high tides are below MHHW. The amount, cost, and schedule of pumping are important for project planning.

- *Assess the feasibility of operating the proposed discharge mixing basin to reduce high-salinity outflow from Ponds E12 and E13 to salinities below the permitted discharge salinity*. The proposed discharge mixing basin is intended to dilute and mix high-salinity outflow from Ponds E12 and E13 (approximately 80 to 140 ppt) with Bay-salinity water using gravity-flow water control structures and wind-mixing. The permitted discharge salinity in Mt. Eden Creek is 44 ppt and discharge salinities from the mixing basin should not exceed this level.

PWA used the Salt-Pond Box Model (SPOOM) developed by the U.S. Geological Survey (USGS 2004) to model the water and salt balance for the Ponds E12 and E13 reconfigured managed pond restoration (Section 2). Using the modeling results, PWA assessed pumping requirements for the Ponds E12 and E13 restoration (Section 3). PWA also performed dilution calculations for the high salinity outflow from the cells into the discharge mixing and Brown and Caldwell assessed the potential for wind-mixing in the discharge mixing basin (Section 4 and Attachment 1).

2. WATER AND SALT BALANCE MODEL

A water and salt balance model simulates the inflow, outflow, and accumulation of water and salt mass for an area over time. This type of model is not a hydraulic or hydrodynamic model and does not simulate flow rates or velocities in open-channels or water control structures using numerical methods. The water and salt balance model methods (Section 2.1) and results (Section 2.2) are described below.

2.1 Methods

2.1.1 SPOOM Model Description

The SPOOM model was developed by the U.S. Geological Survey (USGS) to simulate salinity and water volume at the Napa-Sonoma Salt Pond Complex (USGS 2004). SPOOM was written in Visual Basic and is located within an Excel file. The model represents a well-mixed box of saltwater, within a salt-pond evaporation system. The model uses the law of conservation of mass to calculate daily salinity and pond volume and includes a salt dissolution and crystallization algorithm. Model inputs include precipitation (rainfall), freshwater evaporation, infiltration, and water transfers. For the USGS simulations, infiltration is assumed to be zero and water temperature is set at 19.7 degrees Celsius. The model outputs are daily salinity, pond volume, and salt pond evaporation. The output salt pond evaporation is corrected by the temperature and salinity of the pond water using an equation from Millero and Poisson (1981). The USGS calibrated and validated the model with salinity and water-surface-elevation data from the Napa-Sonoma Salt Pond Complex. Although the model was developed for the Napa-Sonoma Salt Pond Complex analysis, SPOOM was formulated to be easily applied to other salt ponds.

PWA assessed the sensitivity of the SPOOM model to the water temperature parameter. Based on this assessment, model results are not expected to be sensitive to water temperature for the range of water temperatures expected for the Ponds E12 and E13 restoration.

2.1.2 Model Schematization

PWA linked together three separate SPOOM models to simulate three-cell scenarios for Pond E12. Each of the three cells modeled in Pond E12 were represented as ponds with flat bottoms and vertical side slopes. Acreages of the ponds and proposed cells were measured in GIS. The Pond E12 cells are 34 acres each.

PWA first manually approximated the water and salt balance by calculating the volume of inflow and outflow needed to replace water lost to evaporation (approximately 2 inches every 10 days on average) and maintain both target water depths and salinities. After entering these first approximates into the SPOOM model, PWA refined the specified inflows and outflows through successive model iterations.

PWA did not model the cells in Pond E13. The proposed Pond E13 cells are 26 acres each, or 76% of the acreage of the Pond E12 cells. The hydrology and management of Ponds E12 and E13 would be identical (e.g., cell depths, salinities, evaporation). For this reason, the volume of water transfers are expected to be directly proportional to pond areas, so that the flow volumes for Pond E13 can be estimated as 76% of the volumes modeled for Pond E12.

2.1.3 Boundary Conditions

Model input includes time series of evaporation, precipitation, and the volume and salinity of intake water to the low salinity cell, and the volume of water transfers between the cells. Consistent with the USGS simulations for the Napa-Sonoma salt ponds, PWA assumed infiltration was zero and water temperature was 19.7 degrees Celsius.

Boundary condition data for the SPOOM model are time series of surface salinity at San Mateo Bridge Station (USGS 2006) and evaporation and precipitation at Union City (Station #171) (CIMIS 2006) from 2003. PWA simulated the dry period from June through August.

2.1.4 Model Scenarios

Two potential water and salinity management scenarios were simulated:

- *Continuous Flow*. Water control structures would be set to allow flows into and between cells every day, with inflows occurring at high tides or with daily pump operation and continuous flow between the cells. For the Pond E12 and E13 model, constant flow volumes were specified every day (i.e., one day “management interval”). The initial cell salinities were 35, 55, and 100 ppt for Cell 1, 2, and 3, respectively. The initial water depth was 6 inches for all cells.
- *Periodic Flow*. Once every ten days, the water control structures would be opened to allow flows into and between each of the cells. The water control structures would be closed on the same day they were opened. The water control structures would then remain closed for the following ten days. Evaporation would decrease water depths and increase salinity levels in the cells until the structures are opened again to refill the cells and repeat the ten-day periodic flow cycle. In the Pond E12 model, constant flow volumes were specified for a single day every 10 days (i.e., ten day “management interval”). The initial cell salinities were 40, 60, and 120 ppt for Cell 1, 2, and 3, respectively. The initial water depth was 7 inches for all cells.

Both approaches assume consistent operation of water control structures through the dry season. Managing for high salinities in the rainy season is not expected to be feasible and the rainy season was not modeled for either scenario. During the rainy season, precipitation (rainfall) would likely dilute high salinity water accumulated throughout the dry season and it would not be necessary to intake water to the ponds. Water management to reach initial cell salinities was not modeled. Initial cell salinities could be achieved by filling the cells to above the target initial depths with Bay water and allowing salinity to increase as water evaporates.

2.2 Results

The results from the SPOOM model show that target salinities and depths are maintained in Pond E12 throughout the dry season with both the Constant Flow and Periodic Flow scenarios (Table 1; Figures 2 to 5.) The daily flow volumes specified for each management interval in the Pond E12 model are shown in Table 2. Flow volumes for Pond E13, estimated as 76% of the Pond E12 flow volumes, are also shown in Table 2.

Table 1 – Modeled depths and salinities in Pond E12 for two management scenarios.

Management Scenario	Figure #	Minimum water depth (inches)			Maximum water depth (inches)			Maximum salinity (ppt)		
		Cell 1	Cell 2	Cell 3	Cell 1	Cell 2	Cell 3	Cell 1	Cell 2	Cell 3
Continuous Flow (every day)	2,3	5.66	5.65	5.66	6.55	6.53	6.53	41	66	133
Periodic Flow (every 10 days)	4,5	4.76	4.77	4.92	7.32	7.28	7.20	50	75	139

Note: Cells 1, 2, 3, refer to low, moderate, and high salinity cells, respectively.

Table 2 – Daily flow volumes specified for the Pond E12 model and estimated for Pond E13 for two management scenarios.

Management Scenario	Management interval	Daily Flow Volume (acre-ft)			
		Intake into Cell 1	Cell 1 into Cell 2	Cell 2 into Cell 3	Cell 3 into Mixing Basin
Continuous Flow at E12	Once every day	1.96	1.37	0.80	0.25
Continuous Flow at E13	Once every day	1.49	1.04	0.61	0.19
Total Continuous Flow	**Once every day**	**3.45**	**2.42**	**1.41**	**0.44**
Periodic Flow at E12	One day every 10 days	22.60	16.86	11.20	5.72
Periodic Flow at E13	One day every 10 days	17.18	12.81	8.51	4.34
Total Periodic Flow	**One day every 10 days**	**39.78**	**29.67**	**19.72**	**10.06**

Notes: Daily flows occur at every management interval (i.e., every one day for Continuous Flow and every 10 days for Periodic Flow). Flow volumes for Pond E13 are estimated as 76% of the Pond E12 flow volumes.

Cell water depths for the Continuous Flow scenario varied by less than 0.9 inches and the average cell depths were 6.0 inches for the entire simulation period (Figure 2, Table 1.) The salinity varied by less than 10 ppt for Cells 1 and 2 and by less than 25 ppt for Cell 3 (Figure 3). The depths for the Periodic Flow scenario varied by less than 2.5 inches and the average depths were 6.0 inches for the entire summer period (Figure 4, Table 1). Modeled depths are within the range of target average depths to provide optimal shallow water foraging habitat for migratory birds (four to eight inches). The salinity varied by less than 25 ppt for Cells 1 and 2 and by less than 45 ppt for Cell 3 (Figure 5). Maximum salinities for both scenarios are maintained below the threshold for gypsum precipitation to occur (approximately 147 ppt). Leaving the water control structures closed so that the cells do not receive intake water for more than 10 days (i.e., management interval longer than the Periodic Flow scenario) would likely cause cell depths to fluctuate beyond the target range. The potential for salinities to reach the threshold for gypsum precipitation would also be greater.

The Continuous Flow scenario provides less variable water depths and salinities than the Periodic Flow scenario. The Continuous Flow scenario is therefore expected to provide more desirable shallow water foraging habitat for migratory shorebirds (Steve Rottenborn, H.T. Harvey & Associates, pers. comm.). Also, comparing average flow volumes over the management intervals for the two scenarios, the Continuous Flow scenario requires slightly less intake volume and produces less than half the volume of high salinity outflow. Adaptive management of the Ponds E12 and E13 restoration would test the importance of salinity on the density of foraging shorebirds and their prey and the effectiveness of different water and salinity management techniques.

Actual water and salinity management actions (e.g., opening and closing or adjusting water control structures) for the Ponds E12 and E13 could be performed with a frequency of anywhere from approximately every one to ten days (i.e., with a management interval intermediate to the Continuous Flow or Periodic Flow). If the ponds were operated less frequently than every 10 days, water level and salinity fluctuations would likely be difficult to manage and less desirable for habitat and applied studies. Opening the water control structures only every two weeks during high spring tides when the potential for gravity inflows are greatest is therefore not likely to be an effective management approach, unless adaptive management indicates that large variations in habitat conditions are acceptable and relying on spring tide inflows are most effective.

The water and salt balance model results indicate that both the Continuous Flow and Periodic Flow scenarios are feasible. As discussed in Section 4, managing discharge salinities in the proposed mixing basin is expected to be feasible for the Continuous Flow scenario, but not for the Periodic Flow scenario as formulated for this assessment. Further refinements to the Periodic Flow scenario and/or the proposed mixing basin would be required in future design phases to address discharge salinity requirements.

3. PUMPING ASSESSMENT

The average bed elevations of the Ponds E12 and E13 cells are approximately 1.3 ft below MHHW (Table 3). The potential for gravity inflows through intake water control structures may be limited during neap tides and pumping may therefore be necessary. The existing 10,000-gallon-per-minute[1] (gpm) brine pump may be used to actively pump intake water into Ponds E12 and E13. PWA assessed potential pumping costs to inform project planning and design. Pumping costs are based on a preliminary assessment of pumping costs performed by Brown and Caldwell (Attachment 2). PWA also estimated the possible pumping schedule (duration and frequency of pumping).

Table 3 – Ponds E12 and E13 elevations relative to tide levels.

Parameter	Ft NAVD
Mean Higher High Water	6.97
Mean High Water	6.33
Average bed elevations of Ponds E12 and E13 cells	5.69
Average bed elevation of the discharge mixing basin	5.31
Mean Sea Level	3.36
Mean Low Water	0.43
Mean Lower Low Water	-0.75

Sources: Tidal datums are from San Mateo Bridge, West-NOAA #9414458. Cell elevations are estimated from USGS (2004) LiDAR data.

Hydraulic modeling of the Ponds E12 and E13 restoration is recommended in future design phases to size the water control structures at the intake, between the cells, and at the outlet of the discharge mixing basin and evaluate the need for pumping. Sizing the gravity-intake water control structure would provide the basis for a refined cost estimate of installing a new intake structure, which could be compared to the cost of operating the pump. The cost-effectiveness of installing a gravity-flow water control structure versus pumping inflows could be considered as a factor in the design process. The size of the intake water control structure and the amount of pumping could be balanced to optimize project costs.

Based on the modeled flow volumes and a preliminary assessment of pumping costs performed by Brown and Caldwell (Attachment 2), pumping all of the intake water into Ponds E12 and E13 throughout the dry season (approximately April 15 to October 15) is expected to cost approximately $1,200 to $1,400 per year (2006 dollars). Pumping is therefore likely to be a cost-effective management approach. Further investigation of the reliability of the existing pump is recommended in future design phases.

If the existing pump were used to pump all of the intake water into Ponds E12 and E13, the pump would be operated for a total cumulative duration of approximately 400 hours (17 days) per year. The frequency of pumping during the dry season would depend on the water management technique (Table 4). If pumping is used only to supplement gravity flows, the duration and cost of pumping would be less (Table 4). The frequency of pumping may also be less, with pumping possibly occurring only every other week during neap tides when the potential for gravity flows is less. Pumping during the wet season would be less than in the dry season, and may occur only for a few hours once or twice per month as necessary.

[1] The flow-rate capacity of existing 10,000-gpm pump is equivalent to approximately 53 ac-ft/day or 2.2 ac-ft/hr.

Table 4 – Approximate schedule and cost of pump operation during the dry season (approximately April 15 to October 15) for two management scenarios.

Percent of intake water pumped	Volume of water pumped*	Pumping schedule		Pumping cost
		Frequent water management (Continuous Flow)	Infrequent water management (Periodic Flow)	
100%	620 to 720 ac-ft per year	2 hours every day	24 hours every 10 days	$1,200 to $1,400 per year
50%	310 to 360 ac-ft per year	2 hours every day during every other week (neap tides)	12 hours every 10 days	$600 to 700 per year

*Volume of water pumped over the dry season is estimated as the range in flow volumes modeled for the Continuous Flow and Periodic Flow scenarios (Table 3) totaled over approximately 180 days (6 months) multiplied by the percent of intake water pumped.

4. DISCHARGE MIXING BASIN ASSESSMENT

The area of the proposed discharge mixing basin is 40 acres. PWA estimated the volume of Bay-salinity water needed to reduce the salinity of high salinity outflow from Ponds E12 and E13 for both management scenarios. Brown and Caldwell also assessed the potential for high salinity dilution and wind-mixing of high-salinity and Bay-salinity water in the discharge mixing basin (Attachment 1).

PWA's estimate of the water volume needed to reduce high salinities assumed complete mixing and used the equation:

$$(V_{bay})(S_{bay}) + (V_{out})(S_{out}) = (V_{mix})(S_{mix})$$

V is the volume and S is the salinity of Bay water (bay), high salinity outflow (out), and completely mixed Bay water and high salinity water in the discharge mixing basin (mix). To obtain a conservative (high) estimate of the mixing volume, PWA assumed a Bay salinity of 30 ppt and a mixing basin salinity of 40 ppt, which is below the permitted discharge salinity (44 ppt). The depth of water in the mixing basin corresponding to the total volume of mixed water was calculated for comparison with Brown and Caldwell's estimate of the potential mixing depth (Attachment 1).

Table 5 shows estimated approximate volumes of Bay water needed to reduce the salinity of high salinity outflow from Ponds E12 and E13 to 40 ppt in the discharge mixing basin for the two management scenarios. These estimates assume complete mixing in the basin. These estimates also assume the discharge would be flushed from the mixing basin before additional high salinity outflow entered the mixing basin, and do not consider the potential for accumulating salt in the mixing basin over time. While these assumptions may not be realistic, they provide a rough approximation of the water depth in the discharge mixing basin.

Table 5 – Estimated approximate volume, depth, and water surface elevation of completely-mixed high-salinity cell outflow from Ponds E12 and E13 and Bay-salinity inflow in the discharge mixing basin.

	Salinity	Continuous Flow Scenario Volume	Periodic Flow Scenario Volume
High-salinity cell outflow into mixing basin	140 ppt	0.44 ac-ft per day	10.1 ac-ft every ten days
Bay-salinity inflow into mixing basin	30 ppt	4.4 ac-ft per day	96.3 ac-ft every ten days
Completely-mixed discharge from mixing basin	40 ppt	4.84 ac-ft per day	106.4 ac-ft every ten days
Mixing basin depth		0.13 ft (1.5 in)	2.7 ft (32 in)
Mixing basin water surface elevation		5.4 ft NAVD	8.0 ft NAVD
Mixing basin water surface elevation relative to MHHW (6.97 ft NAVD)		1.5 ft below MHHW	1.0 ft above MHHW

Brown and Caldwell's mixing basin assessment indicates that average summer winds may have the potential to mix water in the basin to a depth of approximately 1.25 ft (15 inches) (Attachment 1). Brown and Caldwell's assessment also shows that, for a range of low to high wind speeds, the mixing depth may range from 0.3 to 1.6 ft (3.5 to 19.5 inches). These estimates are also based on rough assumptions.

For the Continuous Flow scenario, these assessments indicate that the mixing basin depth could be less than the wind-mixing depth. Low to average summer wind speeds may therefore have the potential to mix high-salinity outflow from Ponds E12 and E13 with enough Bay-salinity water in the mixing basin to meet permitted discharge salinities for the Continuous Flow scenario. Based on this assessment, it may be possible to reduce the acreage of the discharge mixing basin.

The calculated mixing basin depth for the Periodic Flow scenario is deeper than the calculated wind-mixing depth, and winds may not mix water in the mixing basin. Also, the mixing basin water surface elevation for the Periodic Flow scenario is one foot above MHHW. This would not be feasible using gravity; active pumping of Bay water into the mixing basin would likely be necessary to achieve this water surface elevation. It may not be feasible to operate the discharge mixing basin for the Periodic Flow scenario to meet permitted discharge salinities, unless the periodic flows are spread over several days instead of a single day or the area of the mixing basin is increased (which is not desirable).

Further assessment, modeling, and design refinement of the discharge mixing basin is recommended in future design phases. The option of mechanically mixing water in the discharge mixing basin, possibly within a smaller footprint, may also be considered in future design phases.

5. CONCLUSIONS AND NEXT STEPS

The water and salt balance modeling and pumping and discharge mixing basin assessments for the Ponds E12 and E13 reconfigured managed pond restoration indicate:

- There are a range of management techniques, from frequent (daily) to in-frequent (approximately once every week) water management, that can provide both target shallow water depths for migratory shorebird foraging habitat and a range of salinities to test the importance of salinity on the density of foraging shorebirds and their prey.
- Pumping intake water into Ponds E12 and E13 may provide a cost-effective management technique.
- The operation of the proposed discharge mixing basin to reduce the salinities of high-salinity outflows from Ponds E12 and E13 to below permitted discharge salinities may be feasible for water management techniques that manage flows more frequently.

Hydraulic modeling and further assessment and design refinement of the discharge mixing basin are recommended in future design phases.

6. REFERENCES

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Millero, F.J., and Poisson, A. 1981. International one-atmosphere equation of state for seawater. Deep-Sea Research, v. 28, p. 625-629.

USGS. 2006. Continuous Monitoring in the San Francisco Bay and Delta. http://sfbay.wr.usgs.gov/sediment/cont_monitoring/

USGS. 2004. Salt-Pond Box Model (SPOOM) and Its Application to the Napa-Sonoma Salt Ponds, San Francisco Bay, California. Water-Resources Investigations Report 03-4199.

7. LIST OF PREPARERS

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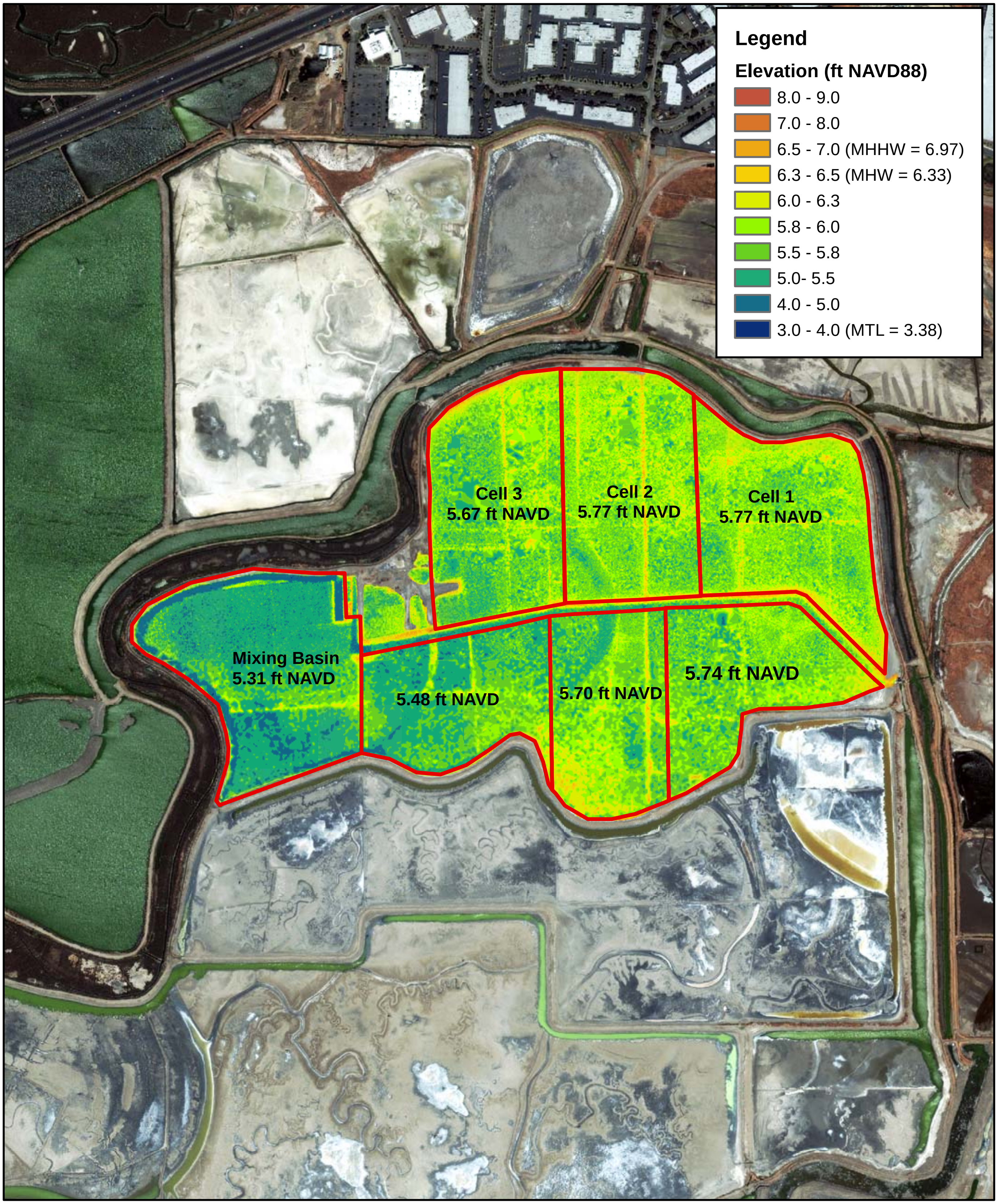
Source files for this report are located at PWA:
\\Orca\pwa\Projects\1750_South_Bay_Salt_Ponds\Phase1_Actions\Pond_E12-E13\Appendix\Final_Draft\G-4_PondsE12-13_WSmodel_appx.doc

8. LIST OF ATTACHMENTS



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N

0 500 1,000 1,500 2,000 Feet

10 ACRES

figure 1

South Bay Salt Ponds Phase 1 Actions

Cell Divisions and Elevations for Pond E12/E13

Proj. # 1751.01

PWA

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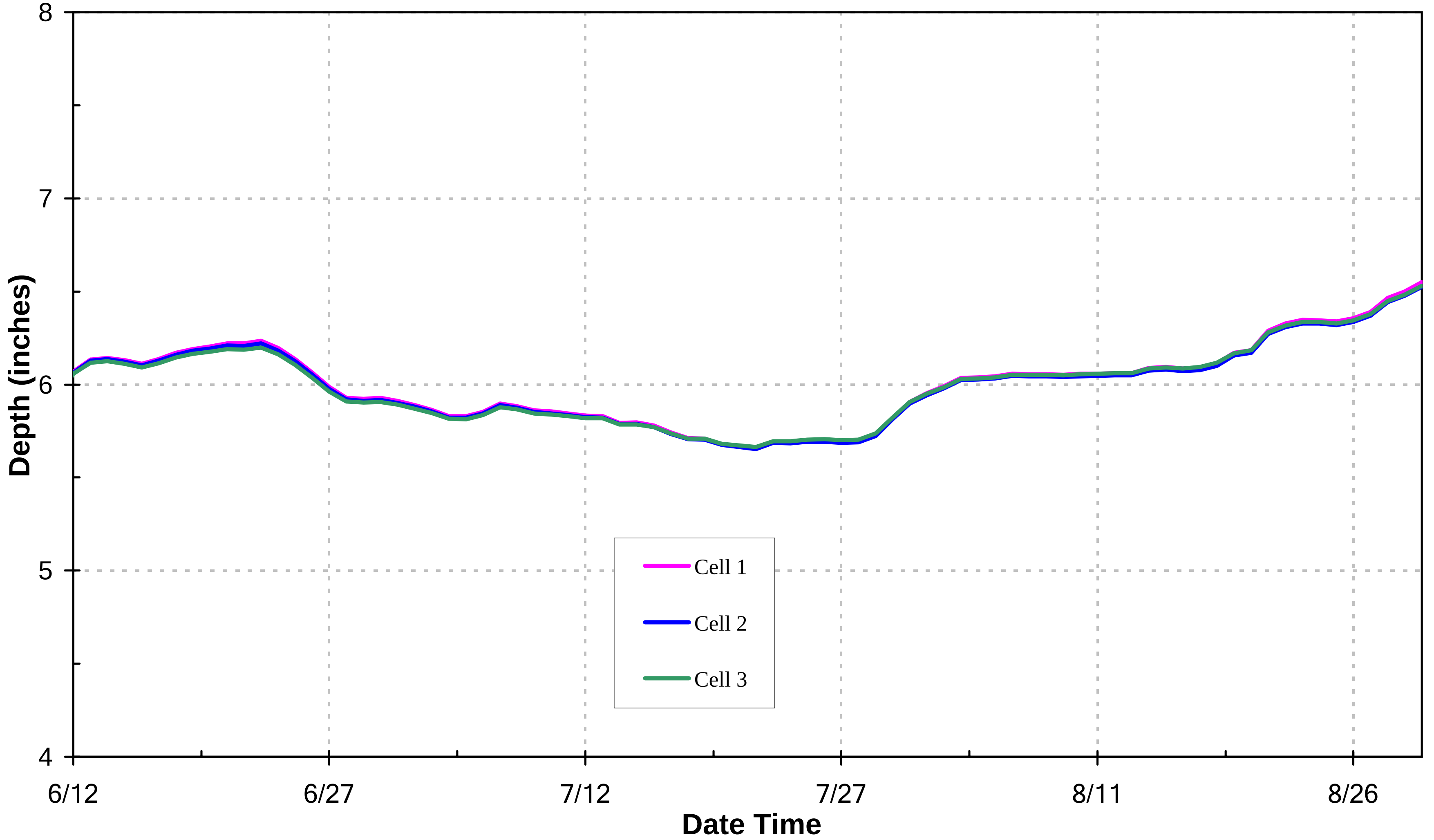
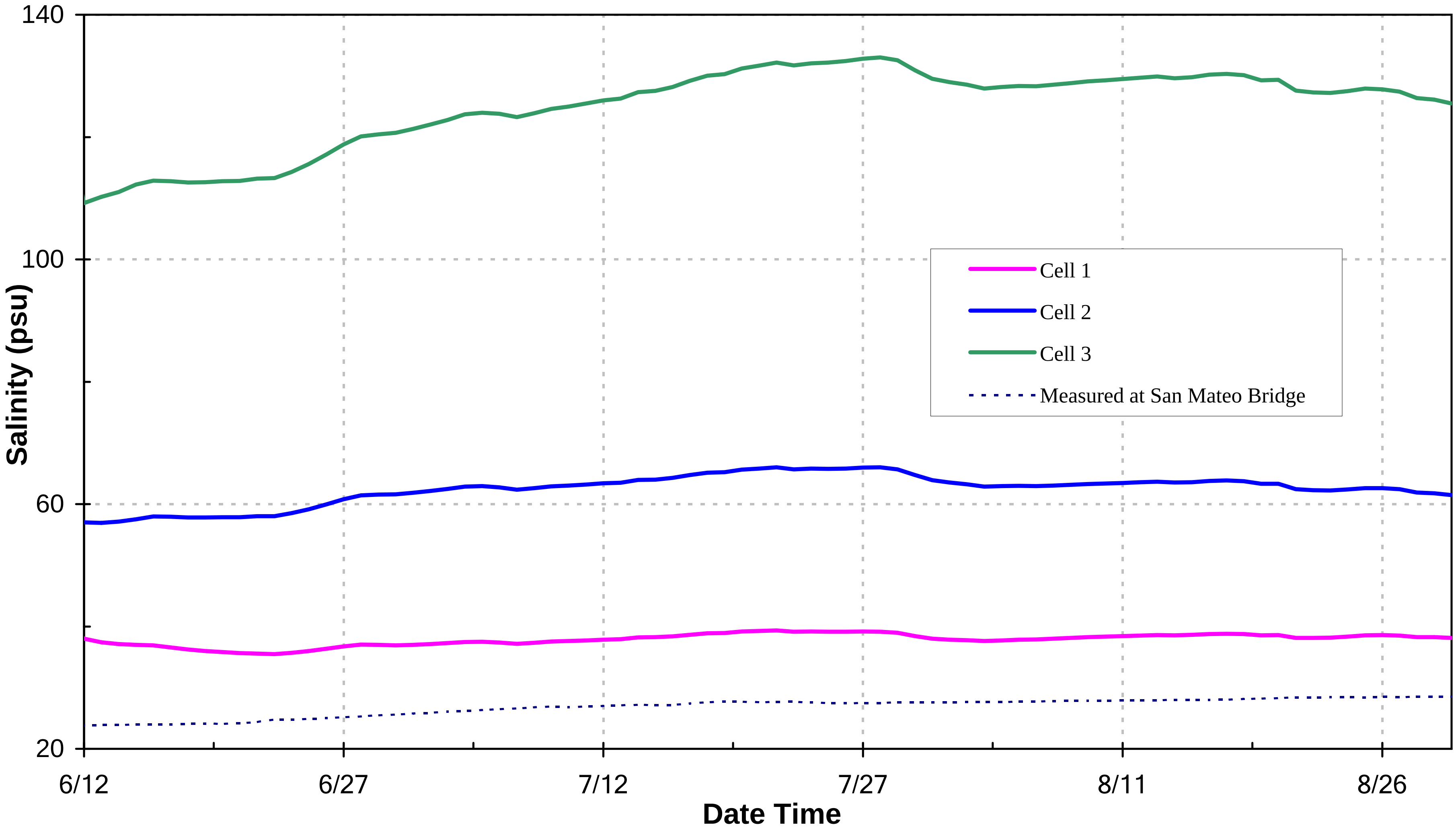


Notes: Salinity and depth were simulated with SPOOM. Each cell has an area of 34 acres and an initial depth of 6.0 inches. Bed elevations: 5.77 ft NAVD (cell 1), 5.77 ft NAVD (cell 2), 5.67 ft NAVD (cell 3). Management scheme: Every day 0.09 inch of water is released to the mixing basin and 0.69 inch of Mt. Eden Creek water is brought into the system	*Figure* 2 — *South Bay Salt Ponds Phase 1 Actions* — Depth for Continous Flow Scenaric — PWA Ref 1751.03 — PWA



Notes:
Salinity and depth were simulated with SPOOM
Each cell has an area of 34 acres and an initial depth of 6.0 inches
Bed elevations: 5.77 ft NAVD (cell 1), 5.77 ft NAVD (cell 2), 5.67 ft NAVD (cell 3)
Management scheme: Every day 0.09 inch of water is released to the mixing basin and 0.69 inch of Mt. Eden Creek water is brought into the system

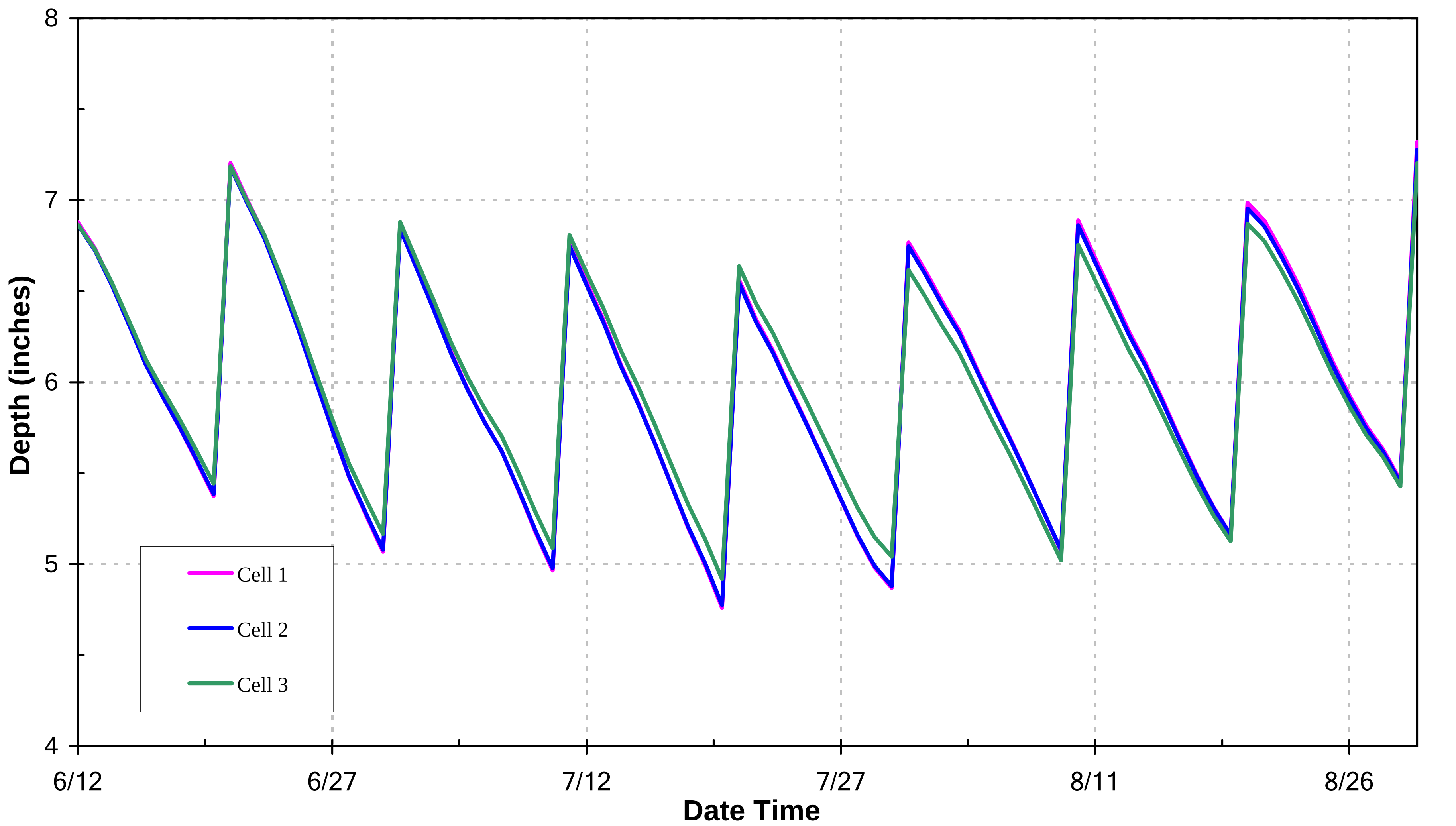
Figure 3

South Bay Salt Ponds Phase 1 Actions

Salinity for Continous Flow Scenario

PWA Ref 1751.03

PWA



Notes:
Salinity and depth were simulated with SPOOM
Each cell has an area of 34 acres and an initial depth of 7.0 inches
Bed elevations: 5.77 ft NAVD (cell 1), 5.77 ft NAVD (cell 2), 5.67 ft NAVD (cell 3)
Management scheme: Every 10 days 2.00 inches of water is released to the mixing basin and 7.91 inches of Mt. Eden Creek water is brought into the system

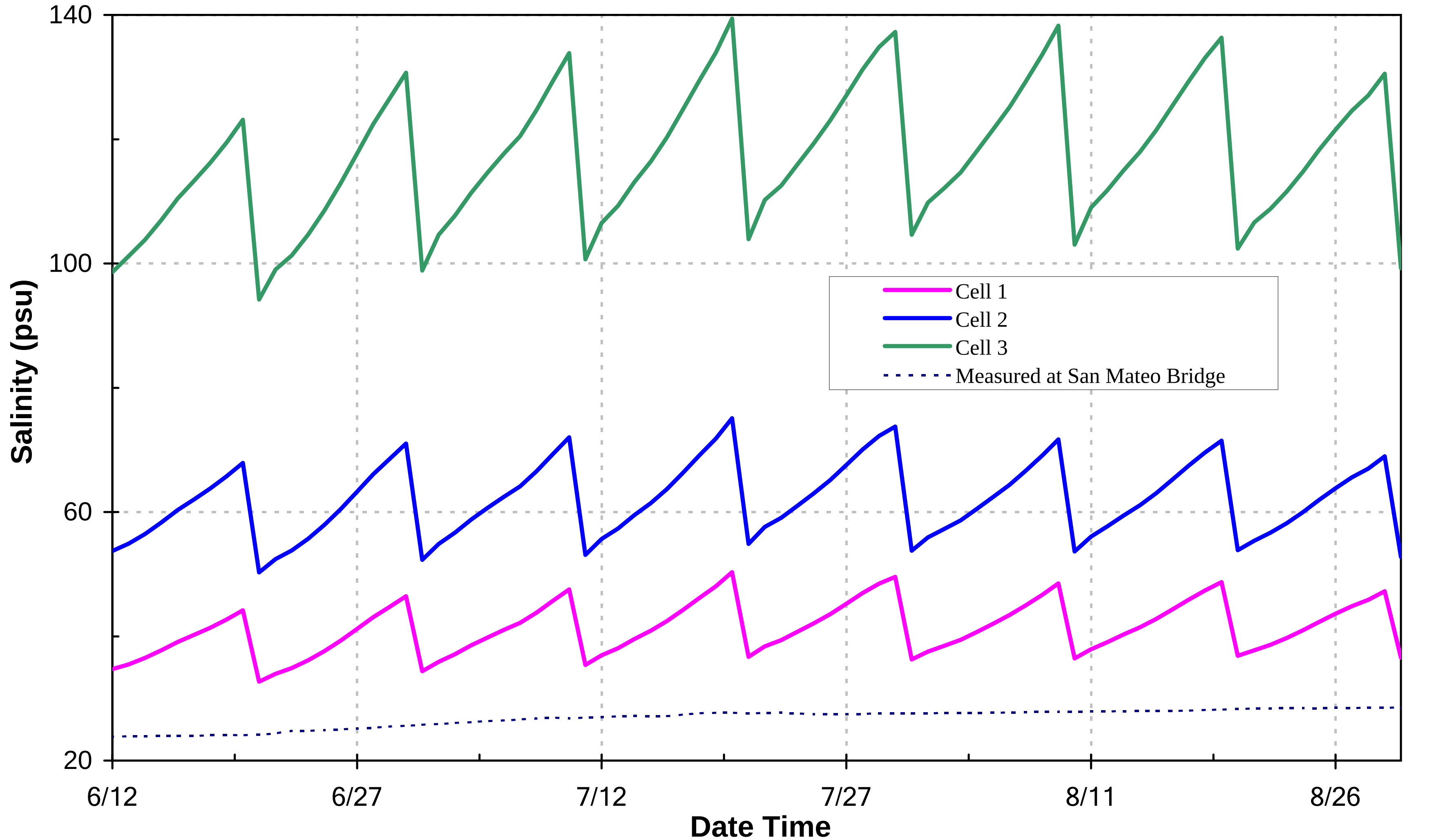
Figure 4

South Bay Salt Ponds Phase 1 Actions

Depth for Periodic Flow Scenario

PWA Ref 1751.03

PWA



Notes:
Salinity and depth were simulated with SPOOM
Each cell has an area of 34 acres and an initial depth of 7.0 inches
Bed elevations: 5.77 ft NAVD (cell 1), 5.77 ft NAVD (cell 2), 5.67 ft NAVD (cell 3)
Management scheme: Every 10 days 2.00 inches of water is released to the mixing basin and 7.91 inches of Mt. Eden Creek water is brought into the system

Figure 5

South Bay Salt Ponds Phase 1 Actions

Salinity for Periodic Flow Scenario

PWA Ref 1751.03

PWA

South Bay Salt Pond Restoration Project

M E M O R A N D U M

TO: Members of the South Bay Salt Pond Restoration Project Management Team

FROM: Brown and Caldwell

DATE: October 26, 2006

RE: **ADMINISTRATIVE DRAFT E12/13 Mixing Basin Feasibility Assessment**

Introduction

As part of the planned future management of the Eden Landing Preserve, Ponds E12 and E13 will be managed as high salinity ponds during the dry season. Salinities of 120 to 140 parts per thousand (ppt) are expected in the highest salinity cells. Conversely, maximum permitted discharge salinity to Mount Eden Creek is 44 ppt. In order to achieve this lower salinity, highly saline water will be mixed with tidal water (salinity approximately 20 to 30 ppt) in a large shallow pond of approximately 40 acres.

When highly saline water is introduced to water of lower salinity under low energy conditions, the denser highly saline water will tend to spread in a layer below the lower density, lower salinity water. Absent mixing forces, the layers will tend to remain separated. This separation of layers is known as stratification. The densities of high salinity and San Francisco Bay water are shown for comparison in Table 1.

Table 1. Salinities and Densities

	Salinity	Density at 20^{o} Celsius
San Francisco Bay	30 ppt	1020.96 kg/m^3
High Salinity Ponds discharge	120-140 ppt	1091.56-1107.94 kg/m^3

The purpose of this memorandum is to calculate if wind can induce sufficient mixing in the shallow pond (Discharge Mixing Basin) to meet discharge requirements. Preliminary calculations by PWA and Brown and Caldwell have shown that enough tidal water can be introduced to the Discharge Mixing Basin to dilute the high salinity discharge from Ponds E12 and E13, provided that thorough mixing takes place. Brown and Caldwell calculated the necessary volume and flow of 30 ppt tidal water to dilute high salinity water down to salinity discharge requirements. Based on estimated volume and flow of high salinity water into the mixing basin (provided by Philip Williams and Associates), and assuming tidal water inflows for 3 hours of high tide twice daily, it appears the necessary flows of tidal water are feasible. A more detailed assessment of the infrastructure necessary to transmit such flows will be done during the preliminary design. Refer to Figure 1 for locations of Ponds E12, E13 and the Discharge Mixing Basin.

A similar salinity mixing operation is currently operating at the Alviso pond complex south of Eden Landing. The Alviso ponds are managed by the U.S. Fish and Wildlife Service (FWS). This is the first year the FWS successfully managed for high salinity ponds, and no discharges have taken place to date. Dilution will take place by discharging the high salinity water to an adjacent pond with lower salinity water. Water will be discharged from the top of the mixing pond to the bay, which may allow for less-saline discharges if stratification occurs. To date, Fish and Wildlife has not tested whether wind induced mixing takes place in the Alviso Complex.

Theory

When wind-shear forces create waves, water particles are set into elliptical orbits. This orbital wave motion moves water particles that can create shear forces on the basin bottom great enough for resuspension of sediment. Refer to Figure 2 for the forces induced by wind that causes resuspension of sediment. It has been assumed in this memorandum that shear forces sufficient to re-suspend sediment are also great enough to induce mixing between two different salinity layers.

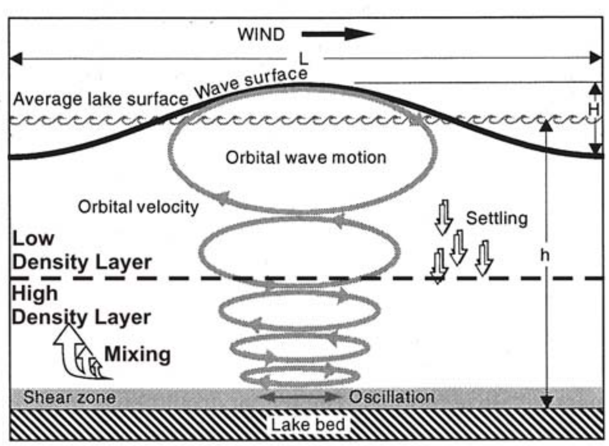


Figure 2: Forces induced by wind that cause re-suspension
(adapted from US Department of the Interior and the US Geological Survey)

According to US Department of the Interior and the US Geological Survey, (USGS – 1996) "The magnitude of the stress that causes resuspension in the shear zone along the bottom is a function of wave length and water depth, and is generally sufficient to begin resuspension when water depth is less than one-half the wave length (L), which is also the defining condition between deep-water and shallow-water waves." Wave-forecasting equations and wave theory developed in most oceanography text books are used to compute the wave height and wave length (USGS – 1996).

The following equations are used to compute the wave length.

1) $$L_o = \left(\frac{gT^2}{2\pi}\right)\tanh\left(\frac{2\pi h}{\left(\frac{gT^2}{2\pi}\right)}\right)$$

2) $$T = 7.54\left(\frac{U_A}{g}\right)\tanh\left(0.833\left(\frac{gh}{U_A^2}\right)^{0.375}\right)\tanh\left(\frac{0.0379\left(\frac{gF}{U_A^2}\right)^{0.333}}{\tanh\left(0.833\left(\frac{gh}{U_A^2}\right)^{0.375}\right)}\right)$$

where: L_o = wavelength
T = wave period
U_A = wind speed
g = gravity acceleration
F = effective fetch
h = water depth

Assumptions

Ponds E12 and E13 will only be managed for high salinities during the dry season. During the dry season, net evaporation takes place to drive up salinities to the target levels. These calculations address summer wind conditions. Wind data were obtained for the past season from a weather data recorder at the Shoreline Interpretive Center on the north side of the Eden Landing Preserve. During the summer months, on most days wind picks up during the early afternoon, and blows quite hard from the west to northwest until early to mid evening. For this reason, wind data from 2pm to 7pm were used in the calculations, as this is likely when the majority of mixing will occur. Wind speeds are presented in Table 2.

The Discharge Mixing Basin is approximately 1,400 feet (north-south) by 960 feet (east-west) and is surrounded by a levee. Refer to Figure 1 for an illustration of the Discharge Mixing Basin. Because the surrounding levee will partially block the wind, fetch is conservatively estimated at 900 feet for westerly winds. Fetch is the unobstructed distance along a water surface for wave development by wind

Using Equations 1 and 2, wave lengths were calculated under low, average and high wind conditions. It is assumed that shear forces sufficient to re-suspend sediment are also great enough to induce mixing between two different salinity layers. Knowing that re-suspension of sediment takes place when water depth is less than one-half the wave length, Table 2 summarizes the theoretical approximate wave length and mixing depth at various wind conditions:

Table 2: Wind speeds with corresponding wave lengths and mixing depths

	Wind Speed (mph)	Wave Length (inches)	Mixing Depth (inches)
Low	2	7	3.5
Average	14.8	30	15
High	24	39	19.5

Conclusion

Without mixing, introduced high salinity water will form an underlying layer beneath fresher, lower density water. With average summer wind speeds of about 15 miles per hour, wave theory predicts that mixing will take place to a depth of 15 inches. With wind speeds of 24 miles per hour mixing takes place to a depth of 19.5 inches. The conclusions in this memorandum are based on the assumptions that sufficient mixing will occur due to wind and that sediment resuspension theory applies. These assumptions and analyses should be further examined during the preliminary design. Additional calculations will also be required to determine operational parameters for the Discharge Mixing Basin and size and configuration of water control structures.

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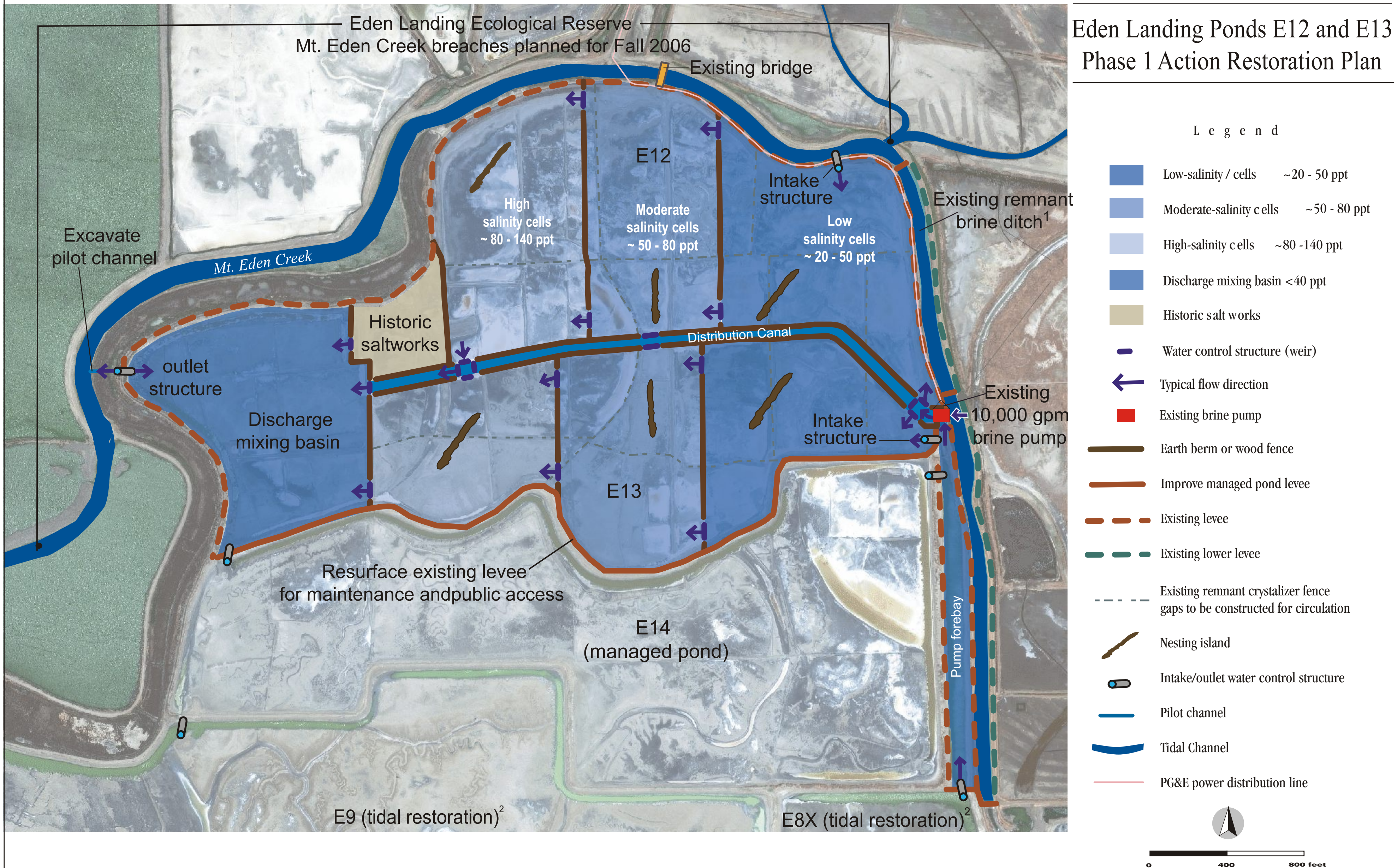
US Dept of the Interior and the US Geological Survey. US Geological Survey Open-File Report 95-414, Portland, Oregon. 1996. *Upper Klamath Basin Nutrient-Loading Study – Estimate of Wind-Induced Resuspension of Bed Sediment During Periods of Low Lake Elevation.*

US Fish and Wildlife Service and California Department of Fish and Game. *South Bay Salt Ponds Initial Stewardship Plan.* June 2003.

South Bay Salt Pond Restoration Project

Figure 1

Eden Landing Ponds E12 and E13 Phase 1 Action Restoration Plan



MEMORANDUM

TO:	Michelle Orr, Philip Williams & Associates Ltd (PWA)
	Don Danmeier, Philip Williams & Associates Ltd (PWA)
	Steve Crooks, Philip Williams & Associates Ltd (PWA)
FROM:	Michael Parenti, Brown and Caldwell (BC)
	Laura Marshall, Brown and Caldwell (BC)
SUBJECT:	Pond E12/13 Pumping – Preliminary Estimate of Probable Cost
DATE:	February 14, 2006

Process and Assumptions

Brown and Caldwell (BC) estimated the electricity costs associated with pumping for Pond E12/13 based on the preliminary estimate of pumping volume range (provided by PWA) of 160,000 to 250,000 m^3. Costs are provided as a range based on the minimum and maximum estimated volumes.

To predict electricity costs associated with pumping, BC first calculated the horsepower using the following equation:

BHP = (GPM*TDH)/(3956*Efficiency)

Where: BHP = brake horsepower
GPM = pumping rate (gallons per minute)
TDH = total dynamic head (ft)

BC then calculated a cost/hour of pump operation using the following formula:

cost/hr = (BHP)*(0.746 kW/BHP)*(cost/kWhr)

BC converted the cost/hr to cost/year based on the expected amount of pumping time each year (hrs pumping/year).

The pumping rate was estimated by two independent methods, and an associated cost was calculated for each. The first method was based on the volume range provided by PWA and an estimated amount of pumping time. The pumping time was estimated by assuming that pumping occurred during two dry months, for half of each month (below average tides), for two high tides a day, and for four hours at each high tide. This is the equivalent of 248 pumping hours each year. The second method assumed a pumping rate of 10,000 gpm which is understood to be the pumping rate of the pump located at Pond E12/13. The total pumping time for this pump was estimated by dividing the total volume to be pumped by 10,000 gpm.

The total dynamic head value was estimated as 5 feet, which includes frictional and static losses. Approximately 2.4 ft of the 5 ft TDH is for inlet, outlet, and minor losses. The change in elevation of water levels between the inlet and outlet was estimated as the difference between the water elevation in the pond and MHHW. The elevation of water in the pond was assumed to be 2.25 mNAVD, which is based on a pond elevation of 2.0 mNAVD (actual range is 1.6 to 2.0 mNAVD) and an average water depth of 3 in (0.25 ft) which was provided by PWA. MHHW was estimated by PWA to be 2.1 mNAVD in this area. The calculated elevation difference is 0.5 ft. The resulting TDH is 2.9 ft (2.4 ft + 0.5 ft), which was increased to 5 ft to account for pumping when tide levels are below MHHW, and to be conservative. Five feet is also considered a good "rule of thumb" value for this type of situation and this level of estimate.

Efficiency was assumed to be 60 percent. This value includes pump efficiency and motor efficiency. The efficiency value also takes into account the age of the equipment, which will be less efficient than new equipment.

Information on the current rate that the Department of Fish and Game (DFG) pays for electricity was not available. Therefore BC reviewed the current PG&E rates for commercial and large agricultural customers. The current cost of electricity for these users ranges from $0.10/kWh to $0.19/kWh. BC used the upper value ($0.19/kWh) for calculations. BC also estimated the electricity cost associated with pumping for the year 2011. The DFG does not have a long term contract with PG&E for rates, therefore BC used a 6% increase per year to escalate the electricity cost to $0.25/kWh for 2011. Utility companies typically report anticipated rate increases of 5% to 6% per year.

Results

The estimated electricity costs associated with pumping in Pond E12/13 range from $250/year for 160,000 m^3 to $390/year for 250,000 m^3. These costs are based on a 10,000 gpm pump, rated at 25 hp. They are slightly higher than the costs calculated using the first method of estimating gpm as described above, which resulted in a range of $210/year and $330/year. The estimated pumping cost in 2011 (based on the 10,000 gpm, 25 hp pump) ranges from $330/year for the smaller volume to $510/year for the larger volume. This level of cost estimate typically has a range of uncertainty of minus 15% to plus 30%.

BC was asked to provide information regarding the amount of pumping that could be performed for $20,000. The table below shows the number of pumping hours that can be accomplished for various increments of $5,000 up to $20,000. This table is based on 5 ft of total dynamic head, an electricity cost of $0.19/kWh, and a 10,000 gpm, 25 hp pump.

Pumping Hours	Cost (2006)
1,412	$5,000
2,825	$10,000
4,237	$15,000
5,650	$20,000

Solar Power Feasibility

Cargill has looked into solar power in the past for functions requiring significantly less horsepower than is necessary for the predicted amount of pumping. They found that a significantly large area of solar panels was necessary, and opted against it. In addition, solar panels must be kept clean to function properly. The abundance of birds in the restoration area could pose a significant maintenance challenge, as would salt residue. For these reasons, solar power does not appear to be a feasible option to decrease the electricity cost associated with pumping.

Wind Power Feasibility

BC also discussed the feasibility of wind power with Cargill Salt. There are currently no wind pumps being used by Cargill in the United States for salt production. Cargill does have a functioning Archimedes screw outside of their Newark office, but it is basically a museum piece that is for show. The wind in the Bay Area was never constant enough for Cargill to implement wind power. Cargill does use wind power in other areas of the world where the wind is more constant, such as off the coast of Venezuela.

An analysis of the feasibility of implementing wind power is often performed by looking at what wind power class an area falls into. Areas are ranked from Class 1- (least wind) to Class 7 (most wind), and wind power operations are typically feasible in areas of Class 3 or greater. A map is attached that shows wind power classes for the state of California. The South Bay is a Class 1+ and Class 2 area. Wind turbines also require a certain wind speed to begin turning, and a typical required wind speed is 15 mph. A map from the California Energy Commission is attached which shows the mean speed of wind throughout California. In most areas of the South Bay the mean wind speed is approximately 11 to 12 mph. According to the attached map there is no area in the South Bay that has a mean wind speed above 13.4 mph. For these reasons, implementing wind power as a method of reducing electricity costs associated with pumping does not appear to be feasible. In addition, wind power is known to result in bird kill due to the rotating turbines. In a restoration area known for significant bird populations, this may not be acceptable.

Conclusion

Electricity appears to be the most feasible option for a pumping power source in Pond E12/13. The present day estimated cost of electricity associated with this pumping is relatively small, and ranges from approximately $200 to $400 per year (2006 dollars). Estimated costs in 2011 range from $300 to $500 per year. At a rate of $400 a year per pond, approximately 50 ponds with identical conditions (i.e., design water depth, pond elevation, evaporation rate, acreage, residence time, and tidal conditions) could be pumped for $20,000. On an acreage basis, and given identical conditions, all areas within the SBSP Restoration Project could be pumped for $20,000.

Hydrology, Flood Management and Infrastructure

Impact Analysis Support

G-5. Alviso Pond A8 Hydrodynamic Modeling and Geomorphic Analysis

South Bay Salt Pond Restoration Project

M E M O R A N D U M

TO: Members of the South Bay Salt Pond Restoration Project Management Team

FROM: Philip Williams and Associates, Ltd. (PWA)

DATE: September 05, 2007

RE: **Alviso Pond A8 Hydrodynamic Modeling and Geomorphic Analyses**

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1. INTRODUCTION

Implementation of the proposed Phase 1 action at Pond A8 will contribute to the long-term objectives of the South Bay Salt Pond (SBSP) Restoration Project by initiating applied studies that examine how tidal restoration may affect channel scour and slough salinity. Additionally, Phase 1 action at Pond A8 will be coordinated with the South Baylands Mercury Project to assist in the evaluation of ecological effects of tidal restoration on mercury bioaccumulation in the food web. Until the ecological risks associated with mercury are better understood, tidal restoration at Pond A8 must be *reversible* to avoid potential long-term environmental impacts. This means that the Phase 1 action must be controlled so that exchange between Pond A8 and Alviso Slough can be halted if necessary.

The purpose of this memorandum is to describe the computer-based modeling and geomorphic analysis performed in support of the restoration planning and environmental impact assessment of the Phase 1 action at Pond A8. Key elements of this plan include:

- Construction of an armored ‘notch’ along the perimeter levee between Pond A8 and upper Alviso Slough to provide muted tidal action;
- Excavation of a pilot channel from the notch to the Alviso Slough channel;
- Demolition of existing electrical distribution lines, wooden piles, and pump house;
- New vehicle access along the southern boundary of Pond A8S;
- Adjustment to the existing Initial Stewardship Plan (ISP) culverts at Ponds A5 and A7 such that flow only enters the ponds on flood tides but does not discharge on ebb tides; and
- Installation of an inflatable dam or flash boards to eliminate tidal exchange across the notch during the winter and provide reversibility if necessary.

Descriptions of the technical analyses are presented in the following three sections, which describe the hydrodynamic modeling of typical tidal conditions, flood modeling and hydraulic geometry analysis. Each section contains a discussion of methods followed by a presentation of results.

2. HYDRODYNAMIC MODELING OF TYPICAL TIDAL CONDITIONS

Hydrodynamic modeling was used to inform our assessment of potential changes to the flow field, salinity transport and fate of sediment particles scoured from Alviso Slough as a result of the Phase 1 action at Pond A8. PWA used the two dimensional (depth-averaged) DELFT3D modeling platform to simulate tidal hydrodynamics of the far South Bay, Alviso Slough, and the Pond A7 System (Ponds A8, A5 and A7) at baseline and post-project conditions. To simulate project conditions immediately after implementation of Phase 1 and sometime later, simulations were carried out assuming existing and scoured conditions along Alviso Slough. Model predictions for each of these three scenarios include simulations of water level, salinity, tidal prism, velocity, bed shear stress and particle tracking.

2.1 Methods

DELFT3D was chosen as the primary hydrodynamic tool for the SBSP Restoration Project. The model developed for the programmatic modeling efforts (the South Bay Model) was therefore adapted for use in evaluating the proposed Phase 1 action at Pond A8. DELFT3D is a widely-used software package that includes both a hydrodynamic module to solve the depth-averaged equations of fluid motion and a particle tracking module which uses the flow field results to simulate the movement of individual water or sediment particles. The application of each module to the proposed Phase 1 action at Pond A8 is described below.

2.1.1 Hydrodynamic Module

The hydrodynamic module uses finite differences to simulate fluid flow on a curvilinear grid that can be fitted to the irregular shoreline of the bay and tidal sloughs. Application of a depth-averaged hydrodynamic model is suitable for the shallow and well-mixed South Bay. The construction, calibration and validation of the South Bay Model is documented in PWA (2006b). For the present analysis, the South Bay Model was cropped at the Dumbarton Bridge (Figure 1). The smaller domain of the Far South Bay Model (FSBM) reduces run time while resolving flow features within the area affected by the Phase 1 action.

One shortcoming of the South Bay Model was the energy dissipation observed in narrow sloughs, particularly those with widths represented in the model as one cell wide (PWA 2006b). The net result of the energy dissipation is an artificial attenuation of the tide range along upper (landward) reaches of narrow sloughs. Comparison with observation data at the mouth and head of Alviso Slough (Moffatt & Nichol Engineers 2005) indicate this ‘numerical damping’ is on the order of ten percent along Alviso Slough (i.e., tidal damping is exaggerated by about ten percent.). In an effort to simulate a more realistic tidal range in Alviso Slough, the width of the channel thalweg was increased to two cells. This reduced the dissipation by approximately one half (i.e., attenuation of the tide range in upper Alviso Slough is approximately five percent greater than observations) although the resolved bathymetry of the narrow channel was artificially wide.

The open boundary condition for the FSBM is derived by extracting water level output from the South Bay Model at the Dumbarton Bridge. Additional boundary condition data used in the FSBM (*e.g.*, freshwater inflow, wind, evaporation and precipitation boundary conditions) match those used in the South Bay Model (PWA 2006b).

Initial conditions for water level and salinity in the far South Bay were derived from the South Bay Model, but initial conditions in the Ponds A5, A7 and A8 were established via an iterative process that minimized the initial transients. The analysis period spans June 1 to June 15, 2001 to capture variations in the tidal and salinity regime over the spring-neap cycle. The simulations begin on May 24, 2001 to allow for one week of model spin up. An example of the convergence of salinity from the initial conditions to a tidally-averaged equilibrium is shown in Figure 2. Note the adjustment of salinity for the first five days of the spin-up period (May 24 to May 29), and the stable salinity trends during the analysis period (June 1 to June 15). The summer 2001 period is consistent with the calibration period of the South Bay Model and determined to be representative of hydrologic Baseline (fall 2006) conditions.

2.1.2 Particle Tracking Module

In addition to simulating tidal hydrodynamics, the DELFT3D model suite includes a module for tracking the path of particles immersed in the flow field. Tracking particles informs the possible fate and transport of scoured and suspended sediments and passive water quality constituents, increasing understanding of potential water-quality impacts. These issues are particularly important for the proposed Phase 1 action at Pond A8 since scour along Alviso Slough has the potential to mobilized mercury-contaminated sediment from the bottom of channel.

The particle tracking module relies upon DELFT3D hydrodynamic module output to characterize the flow field. Particles with specified characteristics are released into the flow field and tracked as the modeled tidal current transports them between the active ponds, sloughs and Bay. In addition to the mean flow field, a random displacement is applied to account for turbulent mixing not resolved by the hydrodynamic model. To replicate vertical variations in the velocity field, the depth-averaged velocity is extrapolated to a logarithmic velocity profile. Particle characteristics specified in the module include: settling velocity, critical shear stress for erosion, and critical shear stress for deposition.

Due to file size restrictions of the model output, the mean velocity field was saved from the two-week hydrodynamic simulations at 80-minute intervals and then interpolated at 20-minute intervals for the purpose of particle tracking. In some instances, the two-week flow field was repeated to simulate the movement of particles over a 28-day period. The horizontal dispersion coefficient was set to 10.8 ft^2/s (1 m^2/s) and vertical dispersion coefficient was set to 0.108 ft^2/s (0.01 m^2/s).

The following two types of particles were released at the beginning of each simulation:

- Neutrally-buoyant particles. These particles have no settling velocity and remain entrained in the water column throughout the simulation.

- Sediment particles. These particles have a prescribed settling velocity, critical erosion stress and critical deposition stress that reflect important properties of sediment dynamics. Specification of these parameters were based on data presented in McDonald and Cheng (1996) and are listed in Table 1. Sediment particles were released from either along Alviso Slough (at 100-m intervals from the Bay to the model boundary) or inside Ponds A8 and A8S (at 19 points equally spaced at 300-400 m).

Table 1 – Sediment parameters used in particle tracking module

Settling Velocity	1.64×10^{-3} ft/s (0.5×10^{-3} m/s)
Critical stress for deposition	2.09×10^{-3} lbf/ft^2 (0.1 N/m^2)
Critical stress for erosion	7.31×10^{-3} lbf/ft^2 0.35 N/m^2

Although these simulations are helpful in characterizing the possible transport of sediments scoured from the bottom of Alviso Slough, it is important to note that these are not capture all of the sediment dynamic processes. For instance, since particles were only released in Alviso Slough, neither the influx of sediment from the Bay on flood tides, nor the influx of suspended sediment with freshwater discharge from the Guadalupe River are simulated. These larger-scale processes, particularly estuarine sediment dynamics, will ultimately determine long-term patterns of net erosion and deposition. Also, the sediment particles are represented as a single class of particles with a constant set of settling and critical stress parameters. Actual cohesive estuarine sediments span a range of size classes and alter their settling velocity and critical stress characteristics in response to flocculation and consolidation. These limitations must be considered when interpreting results of the simulations.

2.1.3 Setup of Model Scenarios

Three scenarios were modeled to characterize the potential effects of the proposed Phase 1 action on the tidal and salinity regimes and the transport of conservative particles. Details of the model setup for these three scenarios are presented below.

- Baseline Conditions. This modeling exercise builds upon the programmatic modeling efforts and utilizes the calibrated and validated South Bay Model. Since managed ponds were not included in programmatic modeling, additional calibration was required to simulate baseline conditions in the Pond A7 System. This included adjusting the culvert parameters in DELFT3D associated with the culverts between Pond A5 and Guadalupe Slough and Pond A7 and Alviso Slough. The operation of these culverts under U.S. Fish & Wildlife Service (USFWS) implementation of the ISP has varied in an effort to achieve discharge requirements. As of May 2006, both culverts were set to allow two-way flow on both flood and ebb tides (Eric Mruz, USFSWS, personal communication). The culvert parameters in DELFT3D were configured to replicate a bi-directional flow and result in pond water levels that match observed values (approximately 3 ft NAVD with fluctuations of less than 0.1 ft).

- Phase 1, No Scour. This scenario used existing (measured) cross-sections for Alviso Slough and added the notch to connect Alviso Slough and Pond A8. The grid cells in the model are approximately 65 feet wide, which is larger than the proposed 40-foot width of the armored notch. Therefore, in the grid cell containing the notch, a 65-foot weir with the same crest elevation as the proposed notch was added to the model. The friction coefficient of the weir was then calibrated to obtain the expected discharge of a 40-foot notch. The HEC-RAS model used for flood modeling of this system (Section 4) provided the expected discharge for the notch. A comparison of the HEC-RAS rating curve and the calibrated DELFT3D rating curve for the notch is shown in Figure 3. A pilot channel extending from the notch to the slough was also added to the model. This channel is aligned with the grid and is one grid cell wide. The bed elevation of the pilot channel varies linearly from the notch crest elevation of 0.42 ft NAVD to slough's bed elevation.

The internal levees between Pond A8, A5 and A7 were modeled as weirs rather than bathymetric features due to the size of the model's grid cells. The elevation of the weirs was set according to the 1 m LIDAR data collected for the SBSP Restoration Project (Foxgrover and Jaffe 2005). These weirs span the full width of a grid cell, unlike the notch-approximation weir previously described; therefore the friction coefficients were set to standard values (WL | Delft Hydraulics 2003).

- Phase 1, With Scour. The modeled channel thalweg along Alviso Slough was modified to estimate the response of the increased tidal prism on channel geometry. Analyses presented in Section 3 below predict that implementation of Phase 1 action would deepen and widen the upper (landward) half of Alviso Slough. To account for these potential changes, the modeled Alviso Slough channel was deepened immediately downstream of the notch. Cross-sections were linearly interpolated in the upper half of Alviso Slough, from just downstream of the notch to the existing (measured) cross-section half-way down Alviso Slough. Existing (measured) cross-sections were used for the lower half of Alviso Slough. Section 3 provides predicted channel dimensions.

2.2 Results

Water levels, salinity, tidal prism, current speed and bed shear stress predictions from the hydrodynamic model were analyzed to determine potential impacts associated with the proposed Phase 1 action and support the restoration planning effort. Locations of the stations and transects used in these analyses are shown in Figure 4.

2.2.1 Pond water levels

Under existing conditions, water levels in Pond A5 and A7 fluctuate less than 0.1 ft about a mean water level just over 3 ft NAVD (Eric Mruz, USFWS, personal communication). Pond A8 is not hydraulically connected to the Bay and its water level is set by evaporation, precipitation and occasional pumping to/from Pond A7. Implementation of Phase 1 would raise water levels in all three ponds such that their internal levees would almost always be submerged. Water levels would be nearly uniform across Ponds A8, A5 and A7 and fluctuate ~0.5 ft about a mean elevation of approximately 4 ft NAVD (Figure 5). These higher water levels would result in typical water depths of approximately 2 feet in Ponds A5 and A7. Water depths in Pond A8 would be on the order of 5 feet due to its lower bed elevation. Pond water levels do not change measurably when channel scour in Alviso Slough is considered, indicating the hydraulic capacity of the armored notch controls the amount of exchange between Pond A8 and Alviso Slough.

2.2.2 Slough water levels

Friction losses and interactions with channel geometry result in gradual attenuation of the tidal wave as it propagates up Alviso Slough. Observed water levels collected in 2004 suggest that the tidal damping upstream in Alviso Slough under baseline conditions is on the order of ten percent (Moffatt & Nichol Engineers 2005). Results from the DELFT3D model of baseline conditions produce similar, but not identical results due to additional numerical damping, as discussed above.

Modeling results for the without-scour conditions indicate that the proposed Phase 1 action at Pond A8 would further damp the tidal range along the landward half of Alviso Slough (Figure 6). The largest changes are expected to occur during low tides, when water levels are elevated approximately 2 ft above baseline conditions. The low-tide damping is attributed to continued discharge from Pond A8 during ebb tides. Modeling results indicate that damping also occurs at high tides, although the level of damping is less than one foot. Changes to the tidal regime along the bayward half of Alviso Slough are negligible. Results from the with-scour scenario show a similar amount of tidal damping, although low-tide water levels along upper Alviso Slough are approximately 0.5 feet lower than observed under without scour conditions. This suggests that tidal damping in the slough is largely insensitive to the channel scour predicted to occur in response to the Phase 1 action.

2.2.3 Slough salinity

The longitudinal salinity profile in Alviso Slough is a result of the mixing of the more saline bay water and the freshwater inflows from the Guadalupe River into the head of Alviso Slough. Under baseline conditions, the mean (time-averaged) salinity varies by about 15 ppt over the length of Alviso Slough (Figure 7a), from 20 ppt at the mouth to less than 5 ppt upstream. As shown in Figure 7a, implementation of the Phase 1 action at Pond A8 would increase the mean salinity profile to slightly above 15 ppt over much of Alviso Slough. Changes to the longitudinal profile during lower low water (LLW) and higher high water (HHW) are plotted in Figure 7b and Figure 7c, respectively.

The increase in slough salinity is attributed to two mechanisms. First, the flood-only culverts at Ponds A5 and A7 route relatively saline bay water into the pond system where evaporation continues to elevate the salinity of pond water before it is eventually discharged into upper Alviso Slough at the Pond A8 notch. Second, the restored tidal prism increases the mixing, or tidal dispersion, between the slough and the Bay.

2.2.4 Tidal prism

Tidal prism at several locations was estimated as the total volume of water passing through a transect during each phase of the tide and is plotted in Figure 8. Results for baseline conditions are comparable to the 195 acre-ft estimate predicted in previous modeling of Alviso Slough (Schaaf & Wheeler 2004). Model results indicate that implementation of Phase 1 action could increase the mean diurnal tidal prism in Alviso Slough near the notch by a factor of three, from about 200 ac-ft to approximately 620 ac-ft. Tidal prism increases at the mid-slough transect by about 20% (from 630 ac-ft to about 760 ac-ft) but drops slightly at the mouth transect (from 1220 ac-ft to 1130 ac-ft).

The decrease in tidal prism at the mouth of the slough is counter-intuitive, but the time series of water level and tidal prism in Figure 8 help explain this drop. When compared with the water level time series, it is apparent that the decline in tidal prism at the mouth occurs on the flood tide after LLW. Under Phase 1 conditions, LLW is also a period when the pond water levels are significantly higher than slough water levels, and discharge from Pond A8 into upper (landward) Alviso Slough is greatest. This suggests that slough storage of drained pond water suppresses the subsequent flooding tide. This suppression on every other flood tide is sufficient to produce a slight decrease in the tidally averaged tidal prism at the mouth of the slough.

2.2.5 Tidal current speed and bed shear stress

Longitudinal profiles of mean (time-averaged) current speed and time series of instantaneous current speed at Station 2 are presented in Figure 9. Under baseline conditions, tidal currents generally decrease in speed with distance from the slough mouth, although variations occur due to localized changes in channel geometry (Figure 9a). The time series of current speed at Station 2 (Figure 9b) reveals an asymmetry between flood and ebb tides, with floods tending to be shorter and have larger peak currents while ebbs are longer and have lower peak currents.

The Phase 1 action would increase current velocities, most appreciably in the upper half of Alviso Slough where currents more than double. The time series indicates that the largest increase would occur on ebb tides and reduce the flood/ebb asymmetry compared to baseline conditions (Figure 9b). Once the slough has scoured in response to the Phase 1 action, current velocities would decline, but still remain above baseline conditions in the upper half of the slough. Near the mouth of the slough, the model predicts that mean current speeds will declined slightly as a result of the Phase 1 action. This decline is consistent with the slight decrease in tidal prism discussed above.

Bed shear stress, which determines the erosion and deposition of sediment, is estimated as a quadratic function of current speed. Therefore, the profiles and time series of bed shear stress (Figure 10) exhibit shapes similar to the current speed predictions, only the slopes are steeper. The change in flood/ebb bed shear stress asymmetry as a result of the Phase 1 action is particularly relevant to the likely changes to sediment dynamics along Alviso Slough. Under baseline conditions, the bed shear stress on ebb tides is always predicted to be less than the typical value for the critical deposition stress of 0.1 N/m^2 (McDonald and Cheng 1996), even during the spring tides shown in Figure 10b. This asymmetry is consistent with net sediment transport into Alviso Slough (Dyer 1986) and observed shoaling along its channel. In contrast, the predicted bed shear stress under Phase 1 conditions would exceed the critical deposition stress on both ebb and flood tides and lack the strong asymmetry of baseline conditions. Hence, the bed shear stress regime after implementation of the Phase 1 action would likely favor scour of channel sediments.

2.2.6 Particle tracking

Particle tracking can be thought of as a virtual experiment in which a number of particles are released into the flow field and their paths recorded. Particle concentration can be computed by dividing the number of particles in any grid cell by the water volume. Particles can be used to simulate conservative, neutrally-buoyant scalars or can be used as a surrogate for sediment transport to investigate the fate and transport of water quality constituents adsorbed onto sediments. Although informative, particle tracking does not give a complete picture of the long-term erosion or deposition patterns associated with sediment dynamics because only part of the sediment field is considered (e.g., no estuarine sediments enter Alviso Slough during flood tides) and important processes (e.g., consolidation and flocculation) are neglected. It should be noted that the actual sediment dynamics in the Bay are complicated by the range of particle sizes and the spatial variability in sediment properties; specification of a single settling velocity and critical stresses for deposition and erosion is a simplification.

The results presented below provide synoptic views of the entire model domain and an examination of how the total mass of particles disperses across the slough, Bay and ponds over time.

2.2.6.1 Synoptic view of particle tracking

Figure 11 depicts the predicted locations of neutrally buoyant particles that are released at the beginning of a two-week simulation and then dispersed by tidal currents. In this simulation of Phase 1 conditions, only six of the 97 particles initially released at the beginning of the run have left Alviso Slough by the fourth day (June 4). Particles continue to gradually move into the open Bay over the successive days, but even after two weeks the majority of the particles remain in Alviso Slough or within Ponds A5, A7 and A8. These results suggest a relatively small tidal excursion within the ponds and upper slough and suggest that particles only reach the Bay after mixing over multiple days. This finding helps in the interpretation of sediment particle simulations, as described below. Note that this simulation did not include a continual release of particles at the upstream boundary since only the fate and transport of existing sediments along the channel bed were of interest.

In order to characterize potential changes to sediment transport processes, month-long simulations were carried out under baseline and Phase 1 (no scour) conditions. Each of these simulations consisted of releasing 250,000 particles and then tracking their movement. Particle concentrations can be computed by summing the number of particles in each cell and dividing by the water volume. Simulations were performed using both neutrally buoyant and sediment particles. Results from the baseline simulation are shown in Figure 12, which shows sediment particle concentrations at the bed after one, two, three and four weeks after the initial release on June 1. Results for the same simulation under Phase 1 conditions are shown in Figure 13. Under Phase 1 conditions, greater dispersion within Alviso Slough and transport into the Ponds A8, A5 and A7 result in smaller concentrations of sediment particles along the slough at the of the 28-day simulation.

2.2.6.2 Mass fraction by geographic region

A quantitative approach to aggregate the results of the synoptic maps is to sum the mass of particles in regions of interest. For this case, the three regions of interest are the ponds (Ponds A5, A7 and A8), Alviso Slough, and the remainder of the Bay. The total mass of particles found in each of these regions was determined by multiplying the particle concentration in each grid cell by the water depth and then summing the mass across all cells in the region. This mass was then scaled by the entire mass in the model to determine the mass fraction.

Results of this analysis are shown in Figure 14 for particles released into Alviso Slough under baseline conditions. Since the ponds are not connected to the Bay, the entire mass is distributed between only the slough and the Bay. For the case with sediment particles, nearly 70% of the particles remain in the slough after 28 days. The case with neutrally-buoyant particles, which represents the most extreme extent of particle dispersion, only about 35% of the particles remain in slough after 28 days.

Results of the mass fraction analysis for Phase 1 conditions are presented in Figure 15. Two simulations of sediment and neutrally buoyant particles were performed to assess the sensitivity of releasing particles at slack low and high water. These four different runs illustrate the range of possible responses of particles released into the system at different times and with different particle characteristics. Figure 15a and Figure 15b depict the evolution of the mass fraction for sediment particles released at slack low and high water, respectively. Because a flood tide follows the low water release, the distribution of mass on Day 2 in Figure 15a exhibits an initial pulse of particle mass from the slough into the ponds. For the high water release shown in Figure 15b, the first ebb tide sweeps a larger fraction into the bay. This initial difference between ponds and bay of approximately 10% persists through the remainder of the sediment particle simulations. By last day of the simulations, Day 28, both scenarios predict that 35-40% of the mass would remain in the slough, 35-40% of the mass would be transported to the ponds, and 30-20% of the mass would be transported to the bay.

To provide an upper bound for the dispersion of particles out of the slough, the same slack low and high water scenarios were performed assuming neutrally buoyant particles. The mass fraction distributions for these scenarios are shown in Figure 15c and Figure 15d. Since these particles remain in the water column throughout the simulation, their fate represents the largest possible extent of sediment dispersion. As evidenced by the distributions on Day 28 of the right column panels, only 10-15% of the neutrally buoyant particles would remain in the slough. For a release at low water, 35% of the particles end up in the bay and 55% end up in the ponds. In contrast, for the high water release, these quantities are reversed. As in the case for sediment particles, the effects of the initial pulse, either into Pond A8 or into the far South Bay, persists throughout the simulation.

Particle tracking simulations were also performed assuming an initial release of neutrally buoyant particles in Pond A8. (Sediment particles were not simulated since the low tidal current velocities in the ponds would result in deposition only – critical thresholds for erosion would not be achieved.) Mass fraction analyses of releases at slack low and high water are presented in Figure 16a and Figure 16b, which show only a 5-10% sensitivity to release time. Dispersion of particles into the bay is slow, with particles just begun to arrive in the bay seven days into the simulation. Fitting the predicted mass fraction in the pond to the exponential decay equation $c / c_0 = e^{-kt}$ yields a range of estimates for the decay constant k from 0.014 to 0.27 day^{-1}. Based on this range of decay constants, the time for one half of the water to be flushed from the ponds is 25 to 50 days and the time for 99% of the water to be flushed from the ponds is 170 to 330 days.

3. HYDRAULIC GEOMETRY ANALYSIS

3.1 Methods

The hydraulic geometry analysis was used to predict long-term channel dimensions for Alviso Slough with the Pond A8 Phase 1 action implemented. The predicted channel dimensions were used in both the analysis of the Pond A8 Phase 1 action impact on the potential for levee erosion downstream of the proposed notch (Phase 1 Impact 3.3.4 – Pond A8) and in the hydrodynamic modeling of typical tidal conditions, Phase 1 With Scour model simulation (DELFT3D; Section 2). A separate analysis was conducted to examine the joint Pond A6 and A8 Phase 1 action impacts on potential for levee erosion near the mouth of Alviso Slough which is discussed in Appendix G-1. This analysis of Phase 1 action impact is for the impacts due to Phase 1 action at Pond A8 only. Additionally, predicted long-term dimensions were not used in the flood modeling (HEC-RAS; Section 4), which utilized a more conservative scenario of no channel scour.

Historically, Alviso Slough drained an expansive area of tidal marsh. However, construction of salt pond levees isolated Alviso Slough from its tributary tidal marsh and joined it to the Guadalupe River by a dredged extension at the head of the slough. This activity reduced the tidal prism within Alviso Slough and introduced freshwater flows. Over time, the slough channel adjusted to the reduced tidal prism through sediment deposition and shoaling within the channel, which lead to the development of fringe marsh along the channel. Restoring muted tides in Pond A8 will increase the tidal prism in Alviso Slough, inducing tidal scour as the channel adjusts towards a new equilibrium.

The scoured Alviso Slough channel dimensions expected to result from the Phase 1 action at Pond A8 were predicted using hydraulic-geometry relationships developed by Williams and others (2002) for San Francisco Bay. These empirical relationships are based on data from historic and existing mature San Francisco Bay salt marshes ranging in size from 5 to 14,000 acres. The relationships relate channel depth, width and cross-sectional area to tidal prism.

Long-term channel dimensions were predicted for two possible Phase 1 implementation scenarios. Under the first implementation scenario, the proposed Pond A8 notch is constructed with a width of 20 ft. Under the second scenario, the proposed Pond A8 notch is constructed with a width of 40 ft. The FSBM was run for both of these possible Phase 1 implementation scenarios. FSBM results show that approximately 54% of the tidal prism passing from Alviso Slough through a 40 ft notch into Pond A8 would pass through a 20 ft notch into Pond A8. For the purposes of predicting long-term channel dimensions, the tidal prism mobilized in Alviso Slough by a 20 ft notch into Pond A8 is considered to be 50% of that mobilized by a 40 ft notch into Pond A8.

Channel width, channel depth and cross-sectional area were estimated for five locations along Alviso Slough, the notch, downstream of the old County marina, at the Pond A8 bulge, mid-slough, and mouth transects shown in Figure 4. Tidal prisms modeled in the FSBM at three of these locations (the notch, mid-slough and the mouth), were used to inform the hydraulic geometry analysis. The tidal prism for the remaining transect locations (old County marina and the Pond A8 bulge) were linearly interpolated along the channel from those tidal prisms that were modeled. The model simulations used to predict tidal prism use existing pond bathymetry (assumes that limited exchange across the notch will result in negligible sedimentation in the ponds over the expected life of Phase 1) and existing slough bathymetry (Phase 1 no scour). The tidal prism of this DELFT simulation was estimated for all three locations by first calculating the instantaneous discharge as the product of velocity and water depth. These instantaneous discharges were then integrated for each higher high flood tide to estimate the total volume of water entering through that cross-section during each tide. The estimated tidal prism is the average of all the higher-high flood tide volumes which occurred during the analysis period. Tidal prisms calculated for the Phase 1 no scour model simulation were compared to those calculated for the Phase 1 scour model simulation and there were negligible differences. Therefore, the tidal prisms calculated for the Phase 1 no scour model simulation were used to inform the Phase 1 hydraulic geometry predictions.

Because there is scatter in the empirical data set used to develop the hydraulic geometry relationships, the dimensions of measured channel cross-sections are not expected to exactly match those predicted using the equations. This is the case for Alviso Slough where the dimensions predicted by the hydraulic geometry based on the existing tidal prism predict a shallower, wider channel at the upstream cross-section and a deeper, narrower channel at the mid-slough cross-section (Table 2). Because there may be a physical basis for the deviation from predicted dimensions, for example the effects of fluvial flows from the Guadalupe River, the differences between predicted and measured dimensions for existing conditions were preserved in the long-term channel dimension estimates. Long-term channel dimensions were calculated as the sum of the existing measured dimensions and the predicted incremental long-term change. Increases in each channel dimension were calculated using the derivatives of the hydraulic geometry equations. These increases in channel size were then added to the measured existing dimensions to estimate the new equilibrium geometry of Alviso Slough. The equation below was used to evaluate the channel dimensions resulting from Phase 1 actions:

$$D_{P1} = D_{EC_{measured}} + \left(\left. \frac{\partial D}{\partial TP} \right|_{TP_{EC} + \frac{TP_{P1}}{2}} \right) \left(TP_{P1} - TP_{EC} \right)$$

where D=channel depth, TP=tidal prism, subscript EC refers to existing conditions and subscript P1 refers to Phase 1. The units for the above equation are metric. Calculations were completed in metric units and converted to feet for the table below.

To assess the potential for the Phase 1 action at Pond A8 to cause the erosion of adjacent levees, the distances between the levees were measured at the transect locations downstream of the proposed Pond A8 Phase 1 notch. As discussed in Sections 2.2.4 and 2.2.5, modeled tidal prism increases in Alviso Slough after Phase 1 action at Pond A8 decrease from the notch to mid-slough, where increases are negligible and continue to be so to the mouth. The analysis of potential levee erosion therefore focused on this reach in order to capture the vicinity for which the most potential for erosion is predicted.

3.2 Results

The measured channel dimensions, modeled tidal prism and final channel dimensions estimated for Phase 1 actions are summarized in Table 2 assuming both 40-ft and 20-ft notch widths. The distance between the levees at each transect is noted for comparison with predicted channel widths. See Figure 4 for transect locations.

Table 2 does not summarize conditions at the mouth transect because the changes in tidal prism and channel geometry due to Phase 1 actions at Pond A8 were negligible at this location. In general, the results suggest that in the vicinity of the notch, the Alviso Slough channel may deepen by approximately 2 ft and widen by almost 100 ft. Predicted changes at the mid-slough transect are minimal (~20 ft), suggesting scour associated with Phase 1 will likely be confined to the landward half of the slough. This finding is consistent with model results that show increases in tidal current velocity (Figure 9) and bed shear stress (Figure 10) are greatest along upper Alviso Slough.

Table 2 – Summary of Existing and Predicted Slough Geometry

Scenario	**Additional Tidal Prism (ac-ft)**	**Tidal Prism in Slough (ac-ft)**	**Channel Depth (ft)**	**Channel Width (ft)**	**Channel XS Area (ft^2)**
Notch transect (328 feet between levees)					
Measured Existing Conditions	0	208	14.5	131	1210
Predicted Existing Conditions	0	208	11.3	160	1050
Predicted with Phase 1 Actions (20 ft notch)	205	413	15.7	186	1754
Predicted with Phase 1 Actions (40 ft notch)	410	618	16.6	226	2206
Downstream of old County marina (230 feet between levees)					
Measured Existing Conditions	0	276	13.9	131	1086
Predicted Existing Conditions	0	276	11.9	182	1263
Predicted with Phase 1 Actions (20 ft notch)	182	458	14.6	177	1549
Predicted with Phase 1 Actions (40 ft notch)	365	641	15.3	209	1936
A8 Bulge (308 ft between levees)					
Measured Existing Conditions	0	436	13.1	157	1366
Predicted Existing Conditions	0	436	12.9	226	1711
Predicted with Phase 1 Actions (20 ft notch)	128	564	13.7	184	1658
Predicted with Phase 1 Actions (40 ft notch)	257	693	14.2	207	1924

Mid-Slough Transect (755 feet between levees)					
Measured Existing Conditions	0	632	13.5	246	1905
Predicted Existing Conditions	0	632	13.7	269	2183
Predicted with Phase 1 Actions (20 ft notch)	63	695	13.6	257	2034
Predicted with Phase 1 Actions (40 ft notch)	126	758	13.8	268	2160

Notes: 1) Predicted existing conditions derived from modeled tidal prism at each cross section
2) Phase 1 action channel dimensions are the sum of the a) additional dimensions calculated from the derivative of hydraulic geometry relationships; and b) measured existing conditions

4. FLOOD MODELING

Hydraulic modeling was used to assess peak flood water levels in Alviso Slough, Guadalupe Slough, and Ponds A5, A6, A7, and A8 (the Pond A7 system) during a 100-year combined fluvial and tidal flood event, defined as a 100-year fluvial flood concurrent with 10-year tidal event. PWA used the one dimensional, unsteady-state HEC-RAS software developed by the U.S. Army Corps of Engineers. Simulations were run for baseline conditions and for Phase 1 conditions, which had a 40’ notch connecting Alviso Slough and Pond A8 and conservatively assumed no post-project channel scour in Alviso Slough downstream of the notch. Including predicted scour would likely lower simulated flood water levels. Additional sensitivity simulations were run with 30’ and 20’ notches to assess how smaller notch sizes would affect flood peak water levels.

4.1 Methods

PWA developed the HEC-RAS model by updating and adapting two existing Santa Clara Valley Water District (SCVWD) hydraulic models: the Guadalupe River / Alviso Slough HEC-RAS steady-state model and the unsteady-state Baylands UNET model. The Alviso Slough and Pond A7 systems portions of the model received extensive analysis and review as part of PWA’s Flood Analyses Report (PWA 2006a). Guadalupe Slough and its lateral connections to the Pond A7 system and ponds to the west were imported from UNET with minor modification to broaden the breadth of the analysis and assess the Phase 1 impacts on the adjacent slough. The Flood Analyses Report (PWA 2006a) provides additional detail on model methods for baseline conditions.

4.1.1 Baseline Conditions

The baseline conditions model was used to characterize flooding conditions in Alviso and Guadalupe Sloughs and the adjacent ponds. The main components of the hydraulic model include the model geometry, the model parameters, and the boundary conditions.

4.1.1.1 Model Geometry

The geometric extent of the baseline conditions model includes Alviso Slough from Highway 237 to Coyote Creek, Guadalupe Slough from Highway 237 to Coyote Creek, all of the Ponds in the A7 system, as well as Ponds A3N, A3W, A9, A10, A11, and A12. Channel survey data from 2004 was used to update the cross sections of Alviso Slough. This data included in-channel and levee portions of Alviso Slough for the reach downstream of the old Alviso Marina, and just the in-channel portions of the slough for the reach between the old marina and Gold Street Bridge. Cross sections were spaced in approximately 400 foot (120 m) intervals.

Cross section data from the outside of the levees (i.e. the portion of the levees adjacent to the ponds and the pond bottom elevations) were disregarded, creating cross sections that spanned only from left levee top to right levee top. Levee top profiles were added to the model as lateral weirs connecting the slough channel to the adjacent ponds, allowing the exchange of water over the levees during periods of high water levels. Alviso Slough was connected by a series of lateral weirs to Ponds A6, A7, A8, A9, A10, A11, and A12. The Pond A8 engineered weir was defined as a lateral weir connection in the model using data from post-construction survey drawings provided by SCVWD, dated March 28, 2005. Guadalupe Slough was connected to Ponds A3N, A3W, A5, and A8.

Ponds were modeled using the area time depth method (i.e. vertical sidewalls). Pond areas and bottom elevation were adopted from the UNET model. The model geometry assumes Pond A6 is intact with the outboard levees maintained at their current elevations. Ponds were connected through storage area connections using the elevation profiles from the UNET model.

By connecting Ponds A6, A7, and A8 with Pond A5, Alviso Slough and Guadalupe Slough were effectively linked in the model through lateral weir connections and storage area connections. From this, the model had the ability to quantify the extent of influence of Guadalupe River (Alviso Slough) flooding on Guadalupe Slough. Exterior and interior levee low points can be seen in Figure 17.

4.1.1.2 Model Parameters

Roughness values were established in the UNET model through calibration of the March 1995 flood event. Manning's n values typically range from 0.27 to 0.30 in the channel and from 0.11 to 0.20 in overbank areas and on the levees. In the area upstream of the Pond A8 engineered weir, overbank area roughness values were adjusted to 0.40 and 0.50 to account for vegetation removal by SCVWD. For the lateral structures connecting the channel to the ponds, the weir flow coefficients (Cd) used were 2.60 ft^2/s (1.44 m^2/s). For the engineered weir, Cd was set to 3.08 ft^2/s (1.7m^2/s) since this weir has been designed to pass water more efficiently than other levees.

4.1.1.3 Boundary Conditions

The HEC-RAS model was run using an unsteady-state flow regime, with the 100-year flow hydrograph at the upstream boundary and the 10-year tide signal at the downstream boundary. The flood hydrographs from in the UNET model were used in the HEC-RAS model. Guadalupe River flows were applied to Alviso Slough at Highway 237. Guadalupe Slough received flow from San Tomas Aquinas at Highway 237, as well as additional lateral inputs further down the channel from Calabazas Creek, the Sunnyvale East Canal, and the Sunnyvale West Canal.

A 10-year storm water surface elevation time series was provided as the downstream boundary condition from the UNET model (measured in March, 1995 and adjusted to the mouth Alviso Slough). The maximum water level of this time series, however, was less than that of the 10-year storm tide used in SCVWD's steady-state HEC-RAS model of 10.17 ft (3.1 m) NAVD88. Therefore, all water levels in the time series were adjusted by the difference between these two water levels, so the maximum water level in the time series reached a value of 10.17 ft (3.1 m) NAVD88.

The model was run at 1-minute computational timesteps beginning February 17, 2000 at 2:00 am and ending March 7, 2000 at 12:00 am. This period extends the beginning and end of the existing UNET model to assess how the system functioned during dry conditions and also to assess how fast the ponds drained after the 100-year flood event. The flow hydrographs were extended using the constant baseflow values from the beginnings and ends of the UNET flood hydrographs and the water surface elevation time series were extended using results from the baseline conditions hydrodynamic modeling.

4.1.2 Phase 1 Conditions

4.1.2.1 Model Geometry

The Phase 1 conditions model included the proposed armored notch in the levee separating Alviso Slough from Pond A8. This notch was implemented in the model as a lateral weir connection just upstream of the existing engineered weir. The notch was 40' in width with vertical sidewalls and an invert elevation of 0.42 ft (0.13 m) NAVD88. The weir coefficient for the notch was 3.0 ft^2/s (1.66 m^2/s).

In addition to the 40' notch simulation, the Phase 1 model was run with a 30' notch and a 20' notch to estimate the flood response on Alviso and Guadalupe Sloughs to smaller notches. Beginning the Phase 1 implementation with a smaller notch as part of the adaptive management process would allow managers to observe the effectiveness of the notch in mobilizing tidal prism to scour Alviso Slough.

4.1.2.2 Model Parameters and Boundary Conditions

The model parameters and boundary conditions used in the Phase 1 model were the same as those in the baseline conditions HEC-RAS model.

4.2 Results

Model water level and flow output from baseline and Phase 1 conditions were compared to evaluate the flood response of the Phase 1 action at Pond A8.

4.2.1 Existing Fluvial Flooding

Developed areas upstream of the former salt ponds rely on the engineered weir at Pond A8 and off-line flood storage in the Pond A7 system to reduce water levels in Guadalupe River during high fluvial flood events. Diversion of flood flows over the engineered weir occurs when water levels in Alviso Slough exceed 11.5 ft (3.5 m) NAVD88. The Baseline Conditions Model shows that during a 100-year flood event, water is routed from Pond A8 to Ponds A5 and A7 and eventually to Pond A6 by overtopping internal pond levees. The results of our model show water levels in the Pond A7 system reaching up to 10.9 ft (3.33 m) NAVD88. Schaaf & Wheeler (2004) estimate that the ISP culverts would require 40-60 days to drain the ponds (Ponds A6 and A8 would require pumping or evaporation to completely drain). Alviso Slough maximum water surface elevation results from this analysis show no difference to water surface profile results from previous HEC-RAS modeling of Alviso Slough and the Pond A7 system that did not include Guadalupe Slough in the geometry.

4.2.2 Phase 1 Fluvial Flooding

4.2.2.1 Alviso Slough and Pond A7 System flooding

Implementation of the proposed Phase 1 action would open Ponds A5, A7 and A8 to muted tidal flow through the armored notch and overtopping of internal levees. These changes would reduce the available off-line flood storage volume in the Pond A7 system. The HEC-RAS modeling results indicate that the Phase 1 notch improves flood routing during a 100-year flood event resulting in lower peak water levels on Alviso Slough compared to baseline conditions.

Since the invert of the proposed notch would be significantly (approximately 11 ft) lower than the existing engineered weir crest, stormwater from the Alviso Slough 100-year flood hydrograph would flow into the pond sooner than in baseline conditions. The low elevation of the notch crest allows for water to be diverted from the channel before and during the peak stage of the flood hydrograph, decreasing the peak in-channel flow rate in Alviso Slough downstream of the proposed notch. Peak flow downstream of the engineered weir was estimated at 10,135 cfs (287 cms) in the Baseline Conditions Model and 8,263 cfs (234 cms) in the Phase 1 Flood Model. The modeling results show that with the Phase 1 action, Alviso Slough water levels are lowered in the vicinity of Pond A8 (see Figure 18). Under the Phase 1 action, less water would spill over the existing engineered weir compared to baseline conditions, but a significant volume of water would flow from the channel through the armored notch resulting in a greater total diversion to Pond A8 (see Figure 19).

As stormwater from Alviso Slough flows into Ponds A5, A7, and A8, the pond water levels would increase and eventually spill into Pond A6. Under baseline conditions, the volume of 100-year stormwater entering Pond A6 from Ponds A5 and A7 is less than the capacity of Pond A6 (no spill from Pond A6 out to the Bay or adjacent sloughs). Under the Pond A8 Phase 1 action, Pond A6 would fill to the level of the lowest elevation (10.5 ft NAVD88) along the levee separating the pond from Alviso Slough and begin to spill back into Alviso Slough and out to the Bay. However, since more stormwater would be diverted upstream and conveyed through the Pond A7 system, the net result would be less volume conveyed through Alviso Slough. By more effectively transferring more water through the Pond A7 system and utilizing the available offline storage of Pond A6, the Phase 1 action would lower peak in-channel flows and water levels compared to baseline conditions (see Figure 20). Compared to baseline conditions, Phase 1 pond water levels peak at a higher elevation, but drain faster following a 100-year flood event. See Figure 21, Figure 22, Figure 23, and Figure 24 for water levels in Ponds A8, A7, A5, and A6, respectively.

4.2.2.2 Guadalupe Slough flooding

During the 100-year fluvial flood event, water is exchanged between Guadalupe Slough and Pond A5 over the slough's eastern levee. Initially, a small volume of water is passed to Pond A5 in response to the rising flood hydrograph on Guadalupe Slough. Soon after, the much higher-volume event on Alviso Slough fills Pond A5 to an elevation above the Guadalupe Slough / Pond A5 levee and forces water to spill into Guadalupe Slough. This additional flow increases peak water levels in Guadalupe Slough (see Figure 25).

4.2.2.3 Sensitivity Simulations

The Phase 1 HEC-RAS model was run with notch width of 30’ and 20’ to assess how smaller notch widths would affect flooding in Alviso Slough, Guadalupe Slough, and the Pond A7 system. Results from these simulations are shown in Figure 20 through Figure 25. By reducing the notch width, the decrease in water levels on Alviso Slough is less than with the 40’ notch. However, water level increases on Guadalupe Slough also become less with a smaller notch width. With a 20’ notch, increases in flood water levels on Guadalupe Slough are negligible, yet a flood reduction is still seen on Alviso Slough.

4.2.2.4 Pond A6 Phase 1 Actions

Implementation of Phase 1 actions at Pond A8 would most likely occur in conjunction with the implementation of Phase 1 actions at Pond A6, which calls for converting Pond A6 to a tidal system. This would result in the loss of flood storage of Pond A6 and would bring Bay tides to the existing A5/A6 and A7/A6 levees. The HEC-RAS model was modified to assess how water levels in Alviso Slough would respond as a result of converting Pond A6 to a tidal system.

The loss of Pond A6’s flood storage volume would not adversely affect flooding in Alviso Slough because water overtopping the A5/A6 and A7/A6 levees would spill directly into the Bay which has a much greater storage volume than Pond A6. Bay tides would overtop the A5/A6 and A7/A6 levees during a 10-year tidal event, but the volume of this “reverse-flow” water entering Ponds A5 and A7 would be minimal since the duration of time overtopping occurs would be less than about two hours. This slight reduction in storage volumes of Ponds A5 and A7 is not significant compared to the total storage volume of the ponds and would not affect peak water levels in Alviso Slough during a 100-year fluvial event. These results are summarized in Figure 26 and Figure 27.

4.3 Seasonal Notch Operations and Adaptive Management

As stated above, the notch would be designed to be reversible to avoid potential long-term ecological impacts associated with mercury bioaccumulation. Additionally, the notch could be operated seasonally – open during the dry season and reduced in width or closed during the rainy season – which would allow beneficial Alviso Slough scour while open and maintain existing flood management operations and avoid fish trapping while closed (partially or completely).

The results of the flood modeling indicate that keeping a 40-ft notch open during the rainy season would improve levels of flood protection on Alviso Slough, but would increase flood water levels in Guadalupe Slough. A notch with multiple bays, adds operational flexibility. HEC-RAS modeling results presented here indicate that flood protection in Guadalupe Slough could be maintained by closing the notch completely or reducing its width (e.g., 20-ft) seasonally.

In the long-term, scour on Alviso Slough may increase conveyance in Alviso Slough sufficiently that a 40-ft notch could be open year round without increasing Guadalupe Slough flood water levels. This long-term scenario has not been modeled. Hydraulic geometry calculations, which provide a rough estimate of potential long term channel dimensions, predict the slough would widen by approximately 100 ft (30 m) and deepen by approximately 2 ft (0.6 m) just downstream of the notch. The enlarged channel cross-section would improve fluvial conveyance in the channel and further reduce flood levels below the short-term estimates.

In order to avoid excessive channel widening and possible unintended effects to perimeter levees along Alviso Slough and the former salt ponds, the notch would initially be operated with only a single bay open during the dry season. Depending on the amount of the amount of observed channel widening, and the amount of fringing marsh remaining, notch width could be increased incrementally up to 40 ft. If monitoring of channel widening downstream of the notch indicates a substantial risk to the structural integrity of perimeter pond levees, additional channel scour could be halted by reducing the restored tidal prism. Closing one or more of the multiple bays provides this flexibility in the long-term operation of the Phase 1 action. Initially, the notch would be operated seasonally to avoid fish trapping during periods of migration along Alviso Slough and lower Guadalupe River. Results from the fish trapping study described in Section 2.5 of the EIR/S would be used to inform the decision of whether or not to maintain an open notch year-round.

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P:\Projects\1750_South_Bay_Salt_Ponds\Task15_PondA8\EISR\Pond_A8_TechApp_2007_09_05.doc

LIST OF ATTACHMENTS

Figure 24 Pond A6 Water Surface Elevations

Figure 25 Guadalupe Slough Maximum Water Surface Elevations

Figure 26 Pond A5 Water Levels with Pond A6 Tidal

Figure 27 Pond A7 Water Levels with Pond A6 Tidal

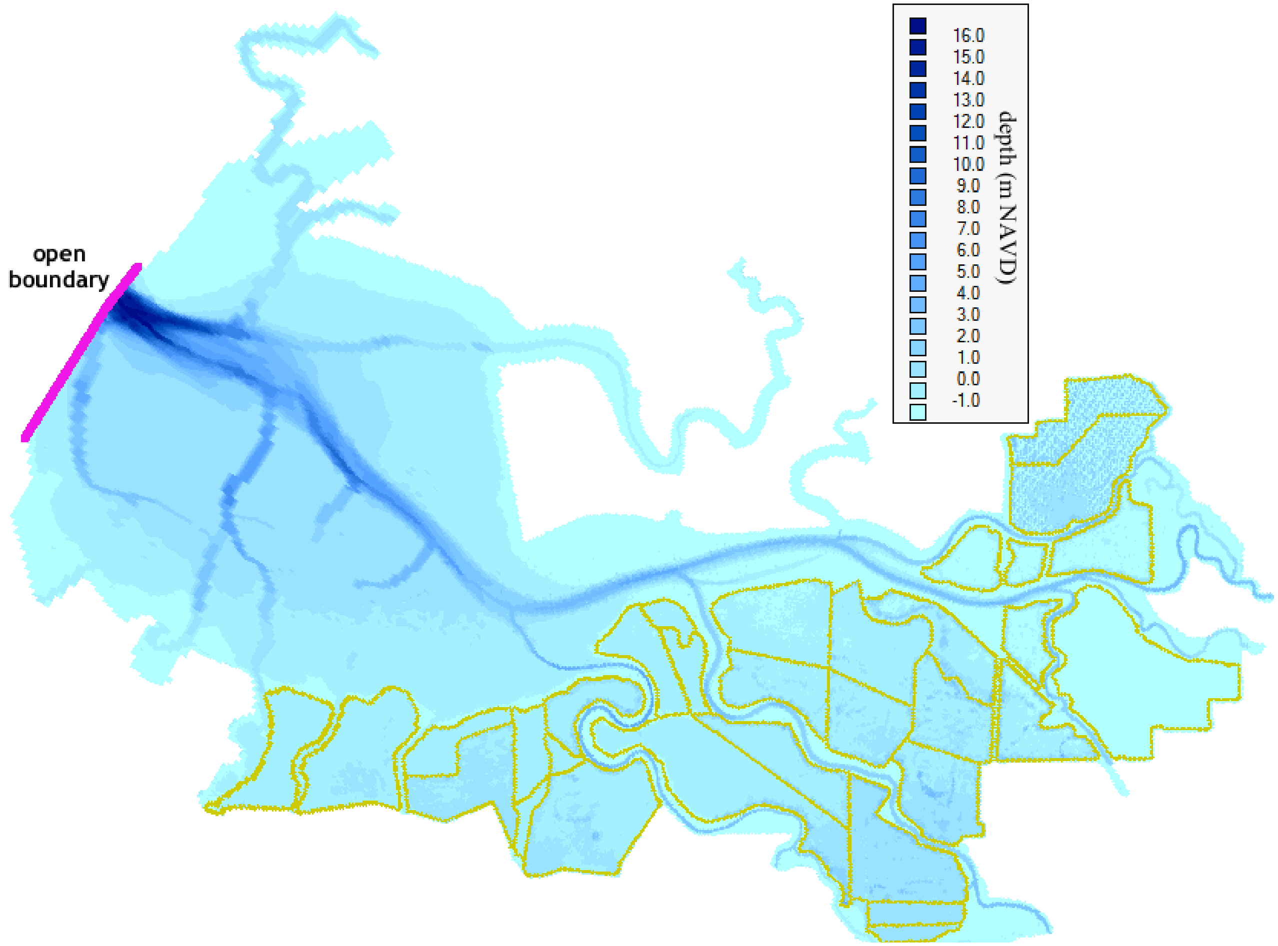
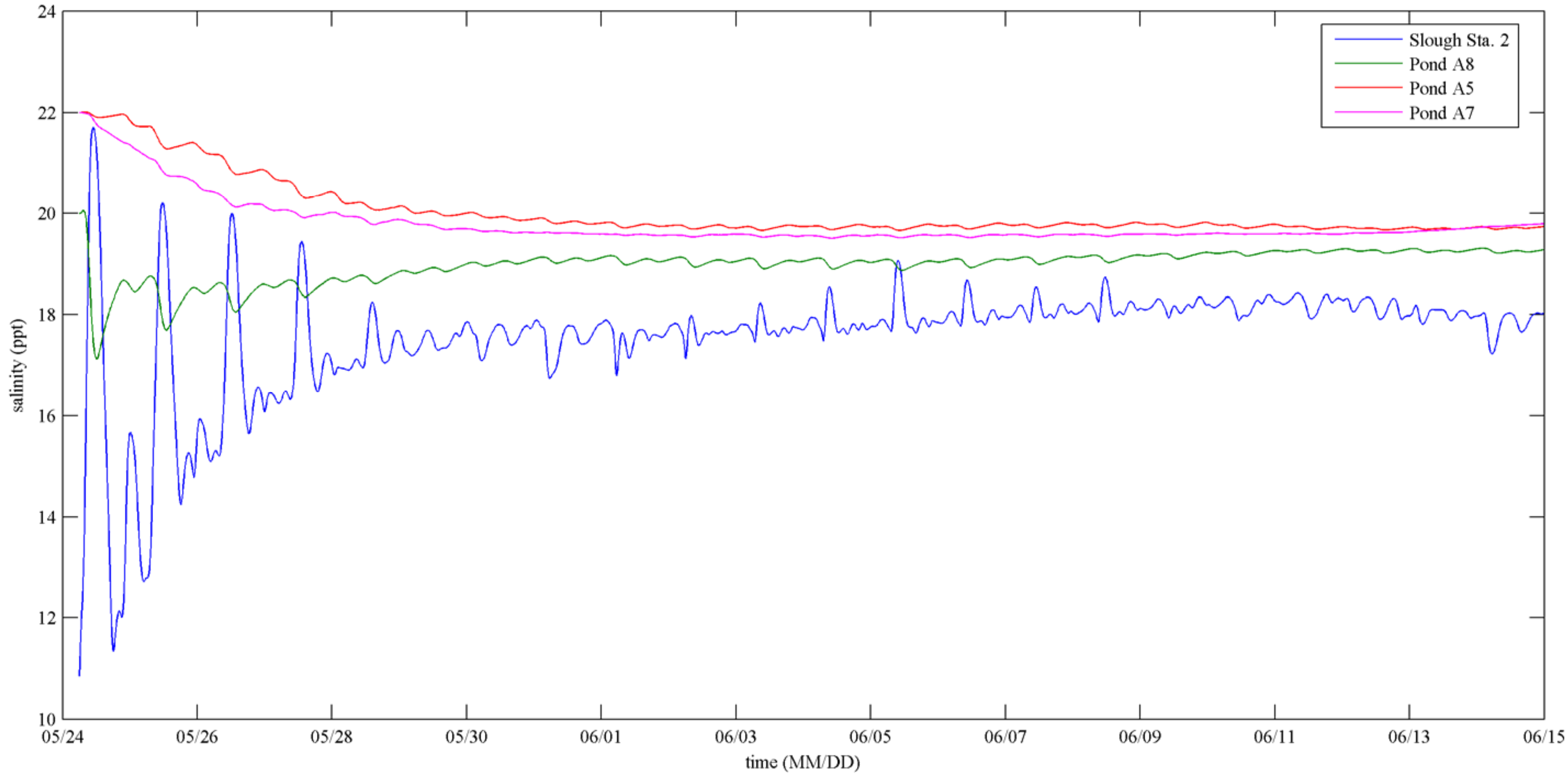


figure 1

South Bay Salt Pond Restoration Project

Far South Bay DELFT3D model bathymetry

PWA Ref# 1750.15



Source: Output from DELFT3D model of far South San Francisco Bay

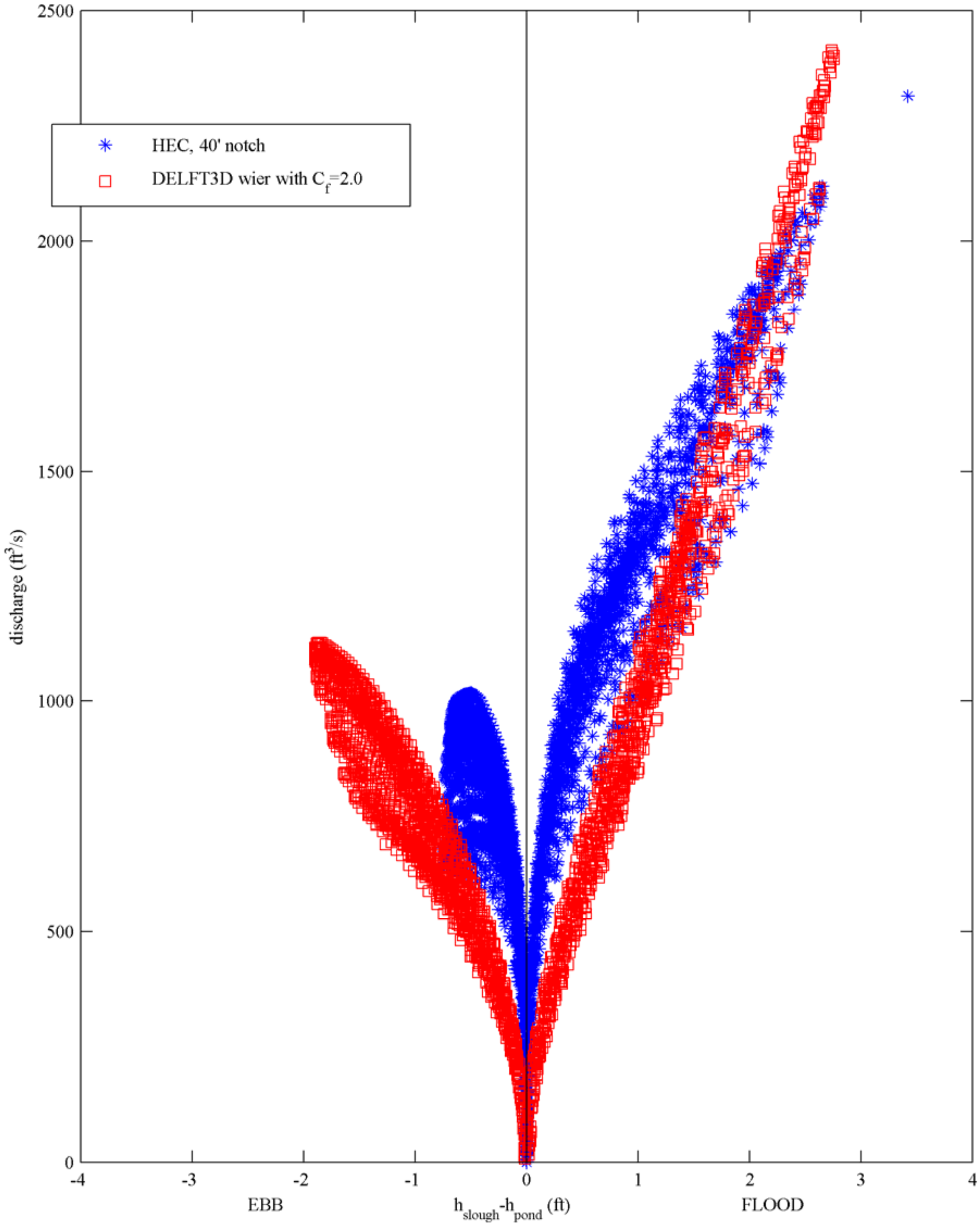
figure 2

South Bay Salt Pond Restoration Project

Spin-up of salinity initial conditions

PWA Ref# 1750.15

PWA



Source: Output from DELFT3D model of far South San Francisco Bay and HEC-RAS model for the Guadalupe River and Alviso Slough

figure 3

South Bay Salt Pond Restoration Project

HEC-RAS and DELFT3D rating curve

PWA Ref# 1750.15

PWA

figure 4

South Bay Salt Pond Restoration Project

Location of structures, stations and transects

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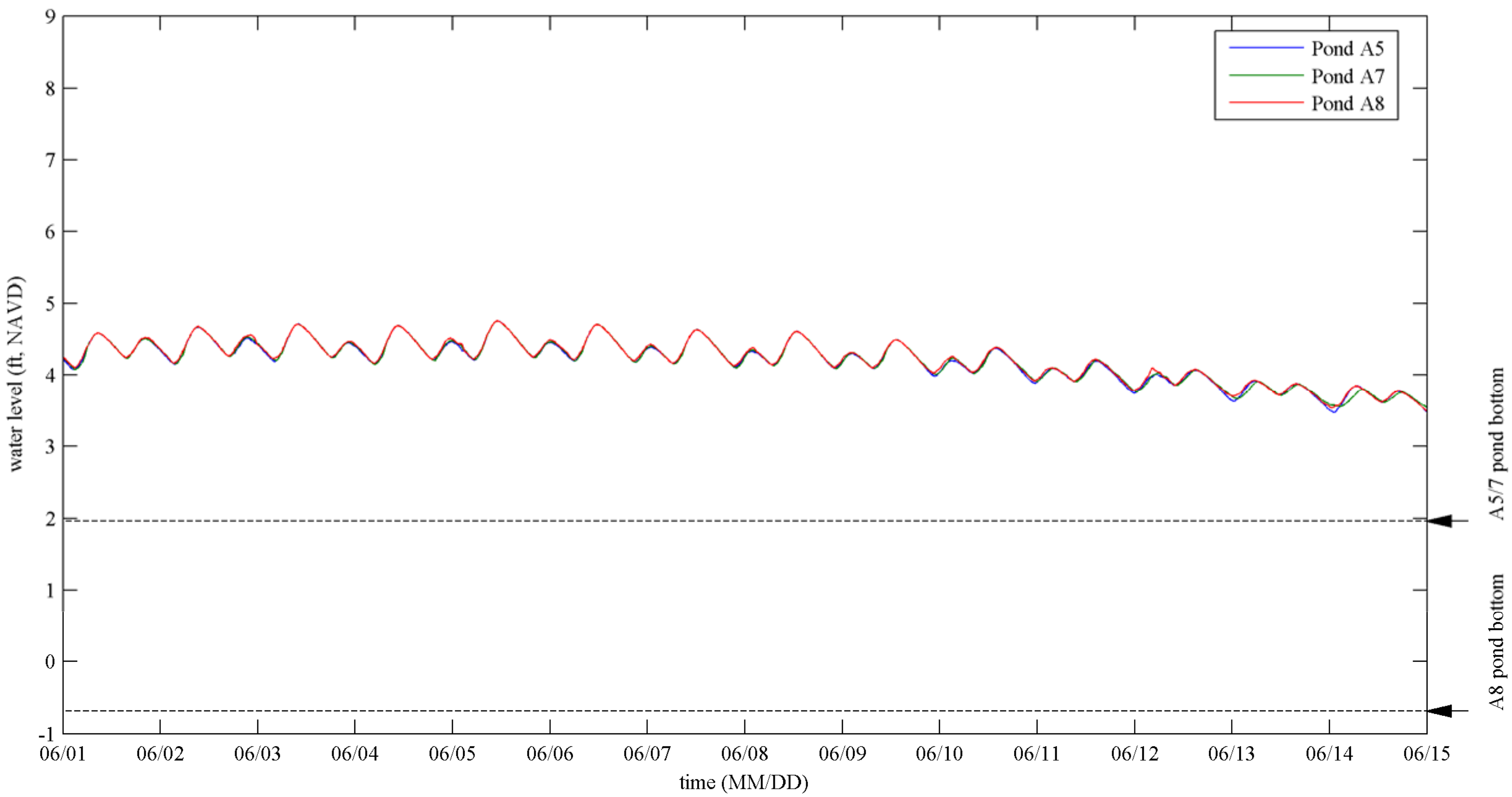


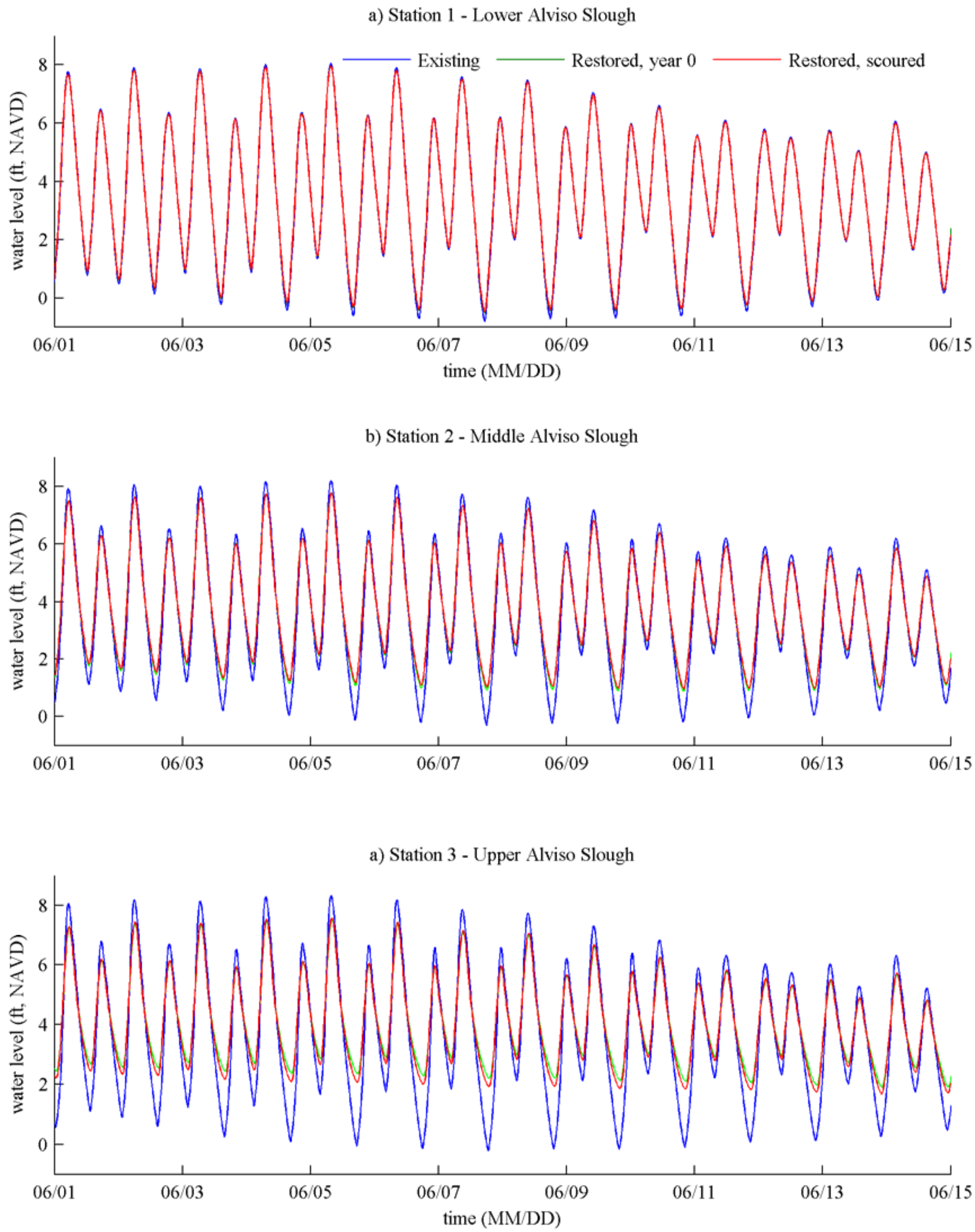
Source: Output from DELFT3D model of far South San Francisco Bay

figure 5

South Bay Salt Pond Restoration Project

Pond water levels under restoration action, year 0

PWA Ref# 1750.15



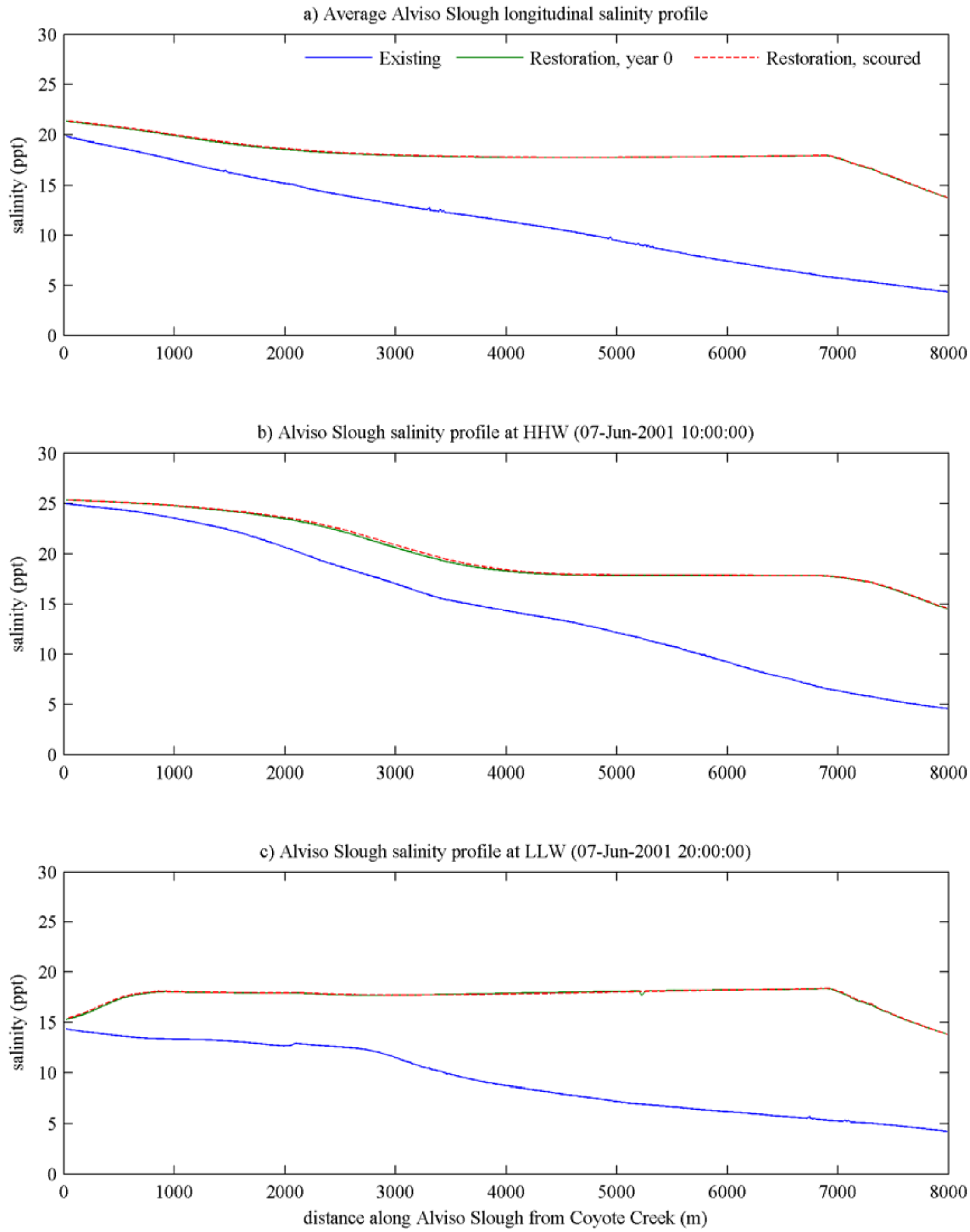
Source: Output from DELFT3D model of far South San Francisco Bay

figure 6

South Bay Salt Pond Restoration Project

Water levels in Alviso Slough

PWA Ref# 1750.15



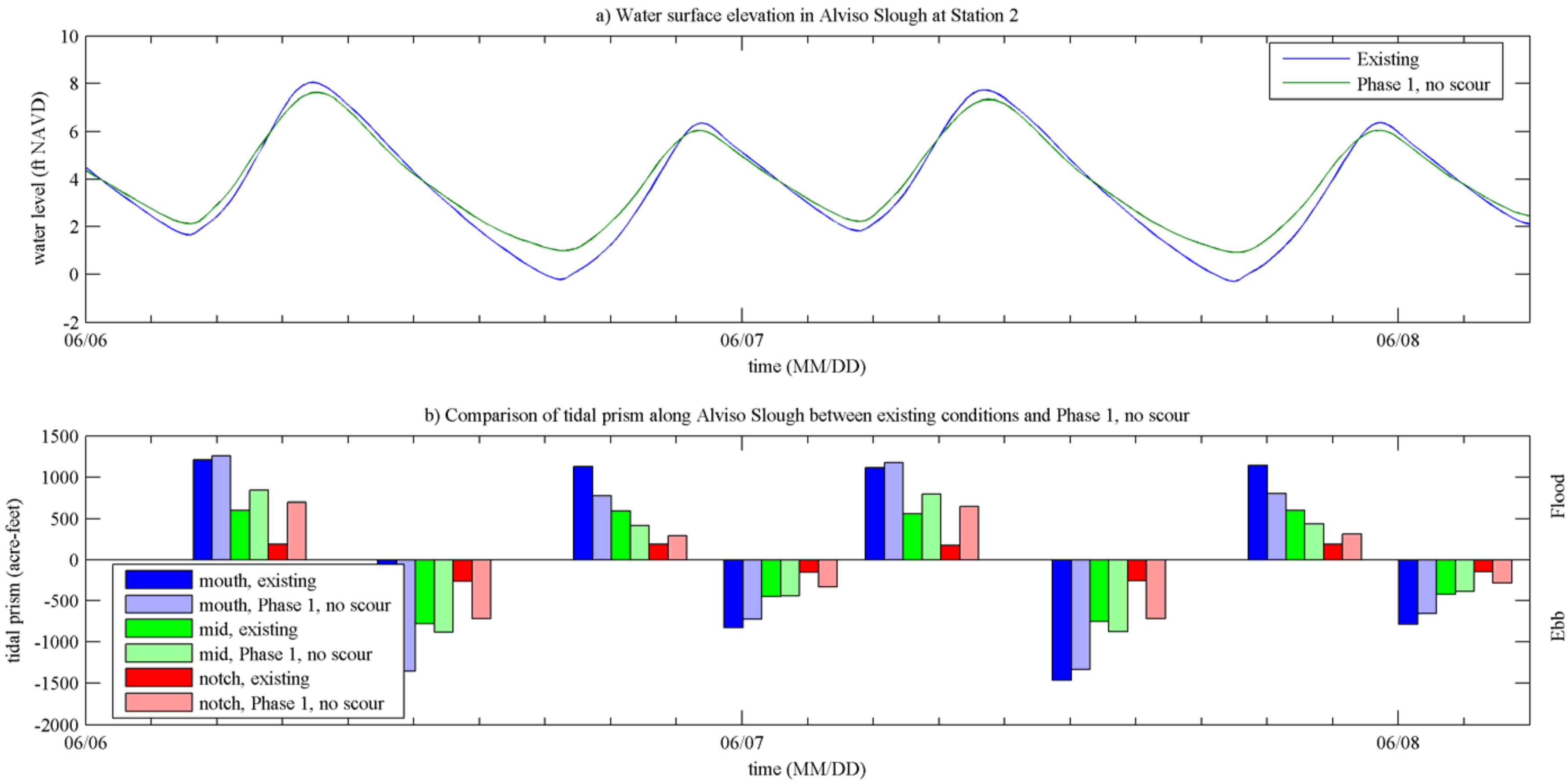
Source: Output from DELFT3D model of far South San Francisco Bay

figure 7

South Bay Salt Pond Restoration Project

Salinity profiles in Alviso Slough

PWA Ref# 1750.15



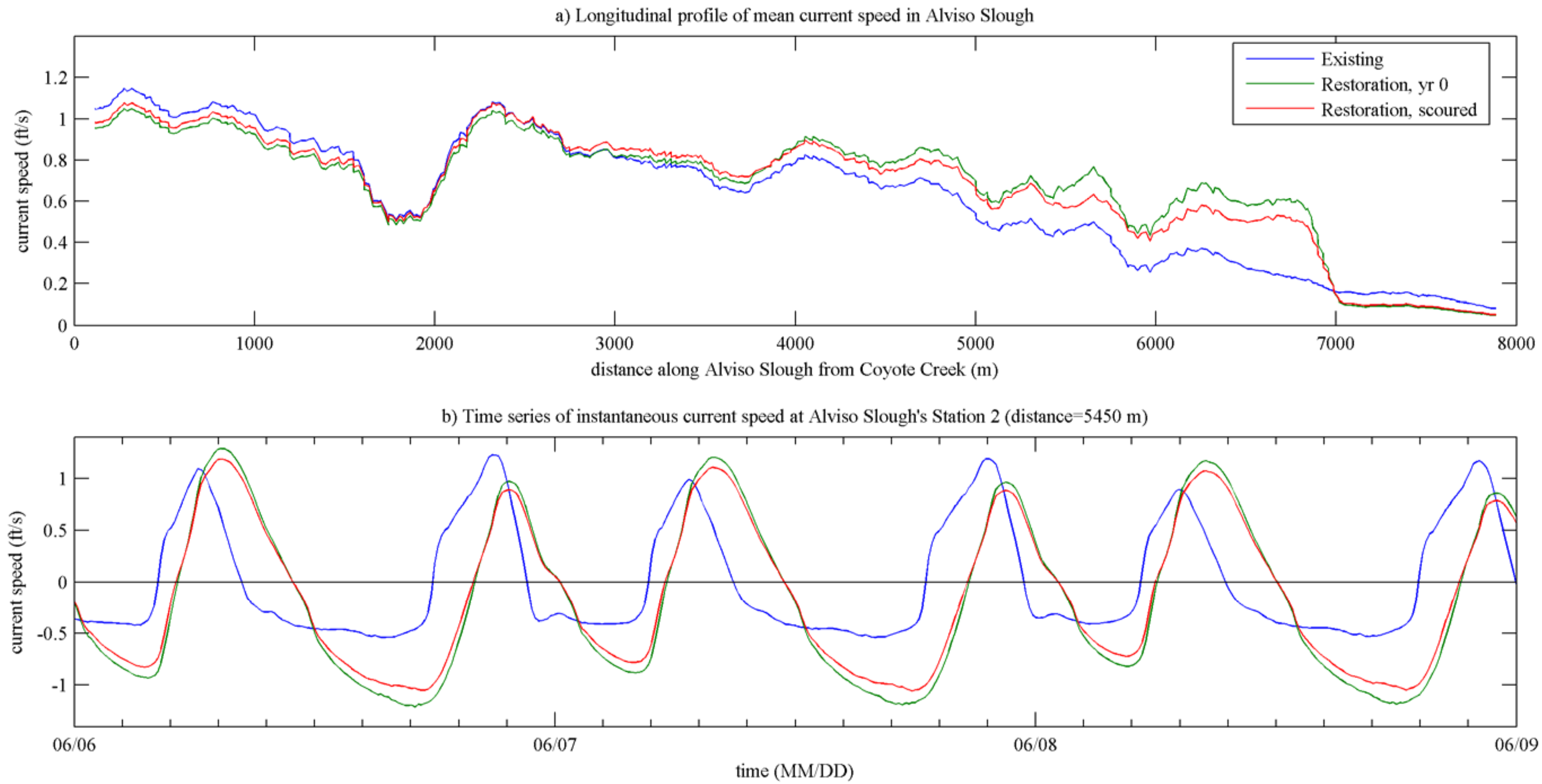
Source: Output from DELFT3D model of far South San Francisco Bay

figure 8

South Bay Salt Pond Restoration Project

Tidal prism, existing conditions vs. Phase I, no scour

PWA Ref# 1750.15



Source: Output from DELFT3D model of far South San Francisco Bay

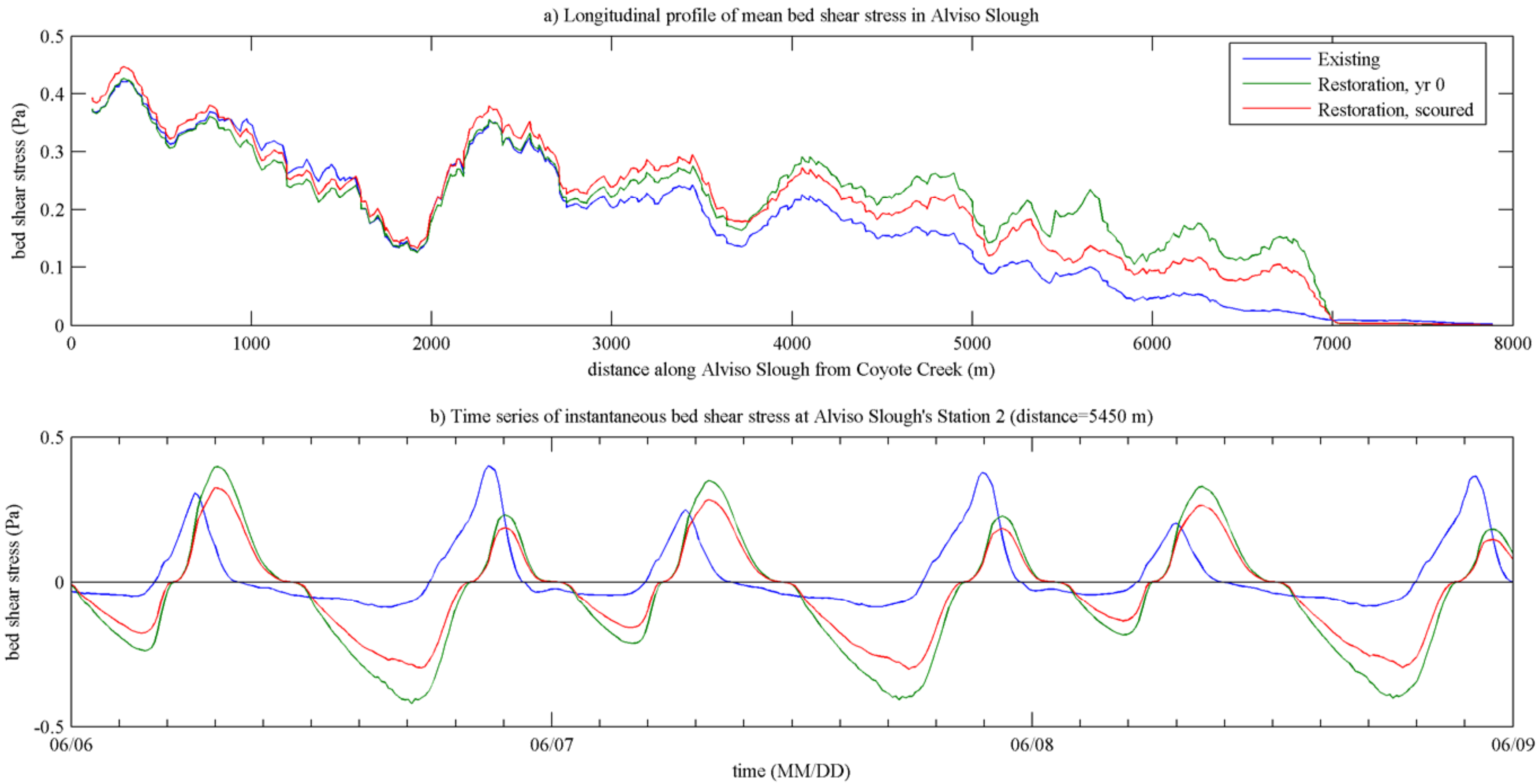
figure 9

South Bay Salt Pond Restoration Project

Comparison of modeled current speed

PWA Ref# 1750.15

PWA



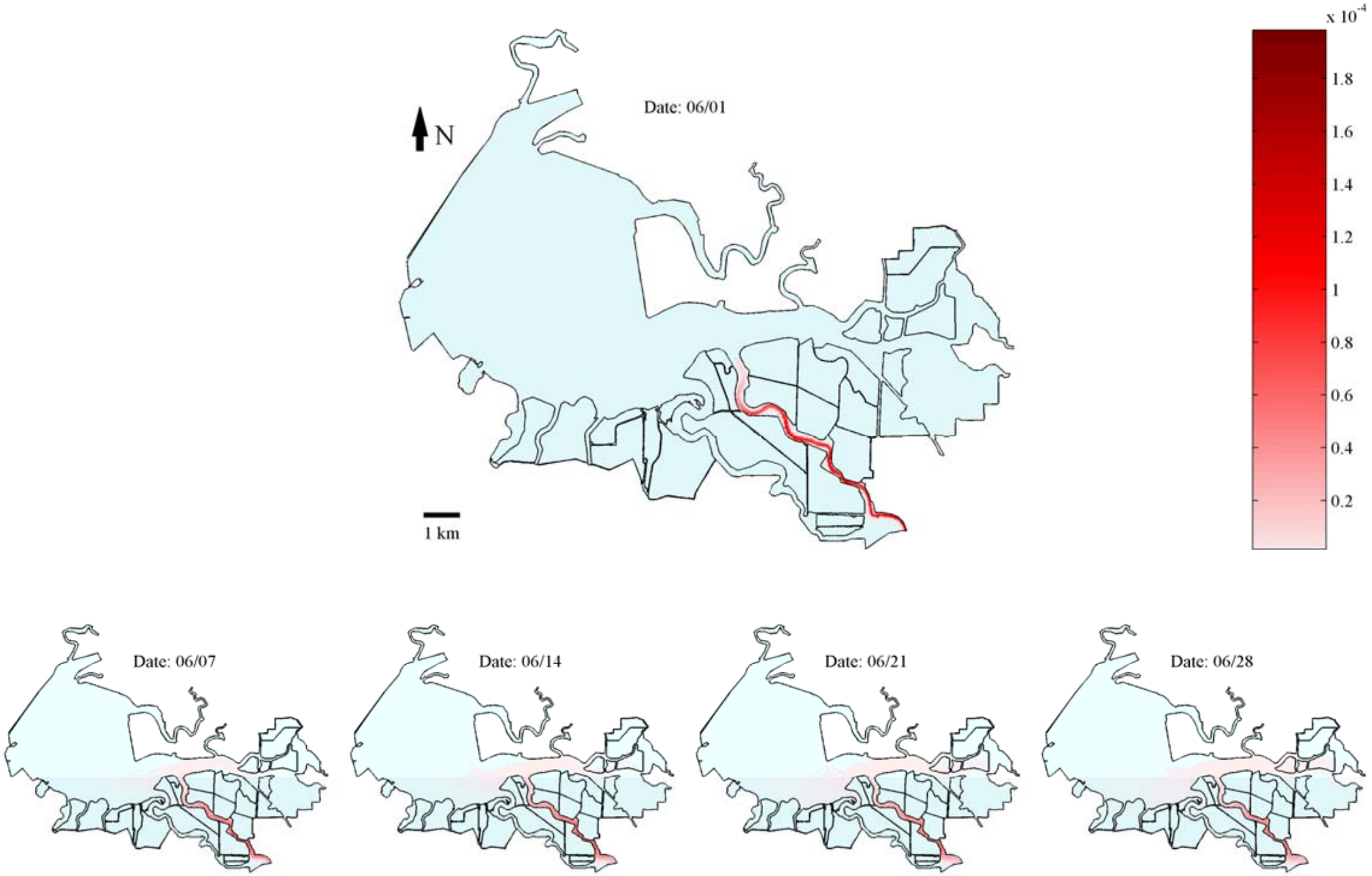
Source: Output from DELFT3D model of far South San Francisco Bay	*figure 10* *South Bay Salt Pond Restoration Project*
	Comparison of modeled bed shear stress
	PWA Ref# 1750.15

Source: Output from DELFT3D model of far South San Francisco Bay

figure *11*
South Bay Salt Pond Restoration Project

Neutrally buoyant particle locations

PWA Ref# 1750.15



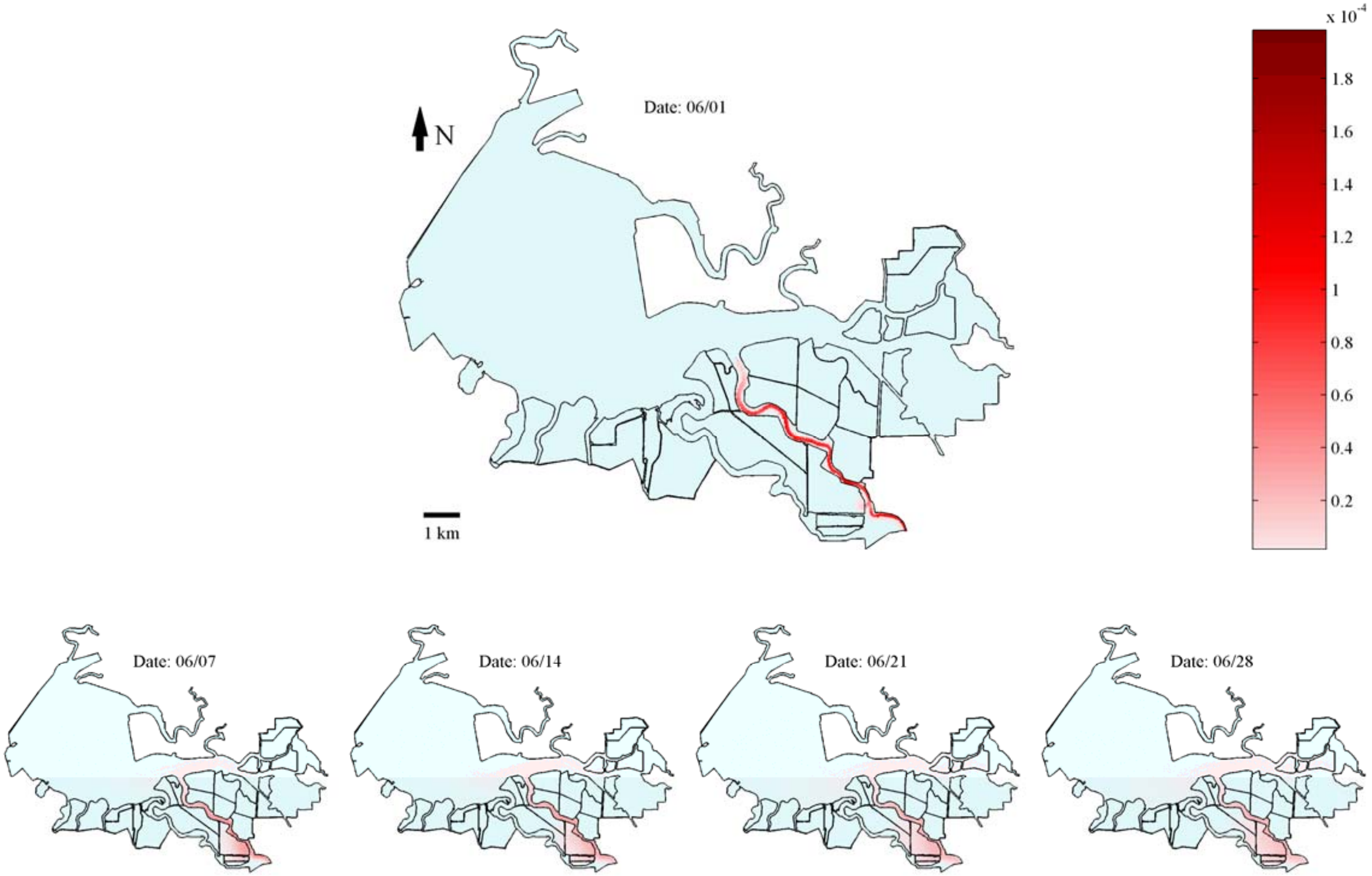
Source: Output from DELFT3D model of far South San Francisco Bay

figure 12

South Bay Salt Pond Restoration Project

Bed layer sediment concentration, exising conditions

PWA Ref# 1750.15



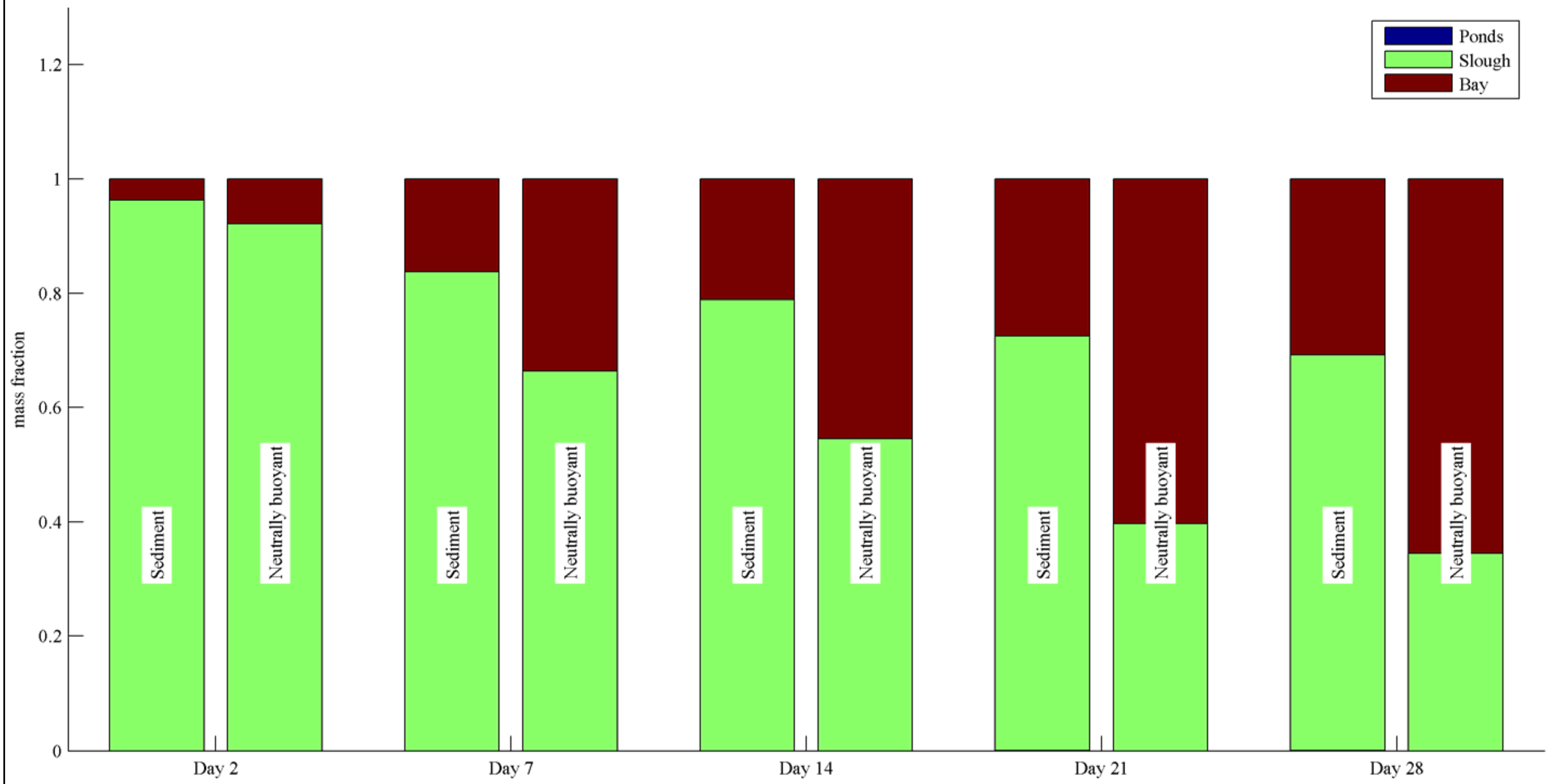
Source: Output from DELFT3D model of far South San Francisco Bay

figure 13

South Bay Salt Pond Restoration Project

Bed layer sediment concentration, Phase 1

PWA Ref# 1750.15



Source: Output from DELFT3D model of far South San Francisco Bay

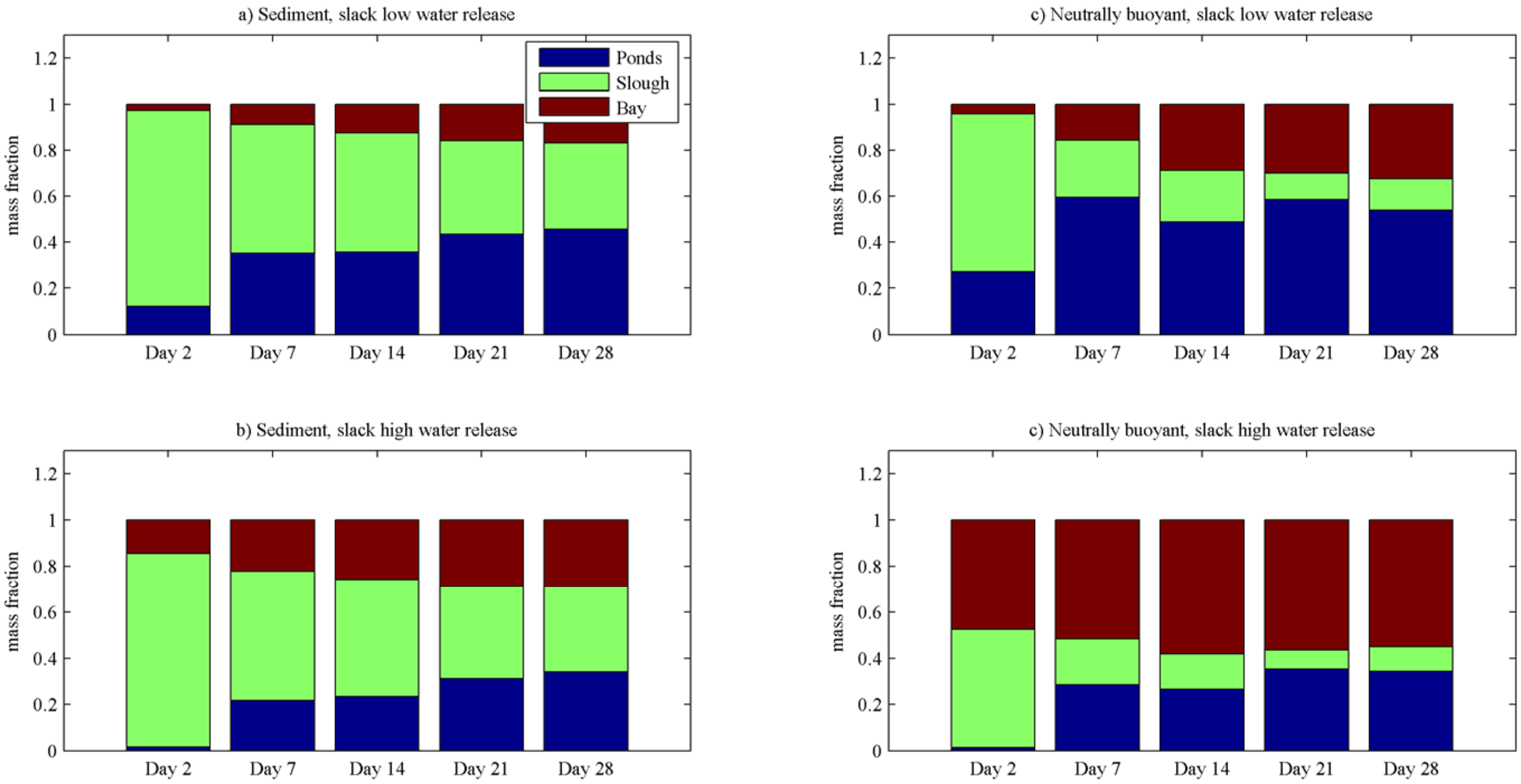
figure 14

South Bay Salt Pond Restoration Project

Distribution of Alviso Slough particle mass, existing conditions

PWA Ref# 1750.15

PWA



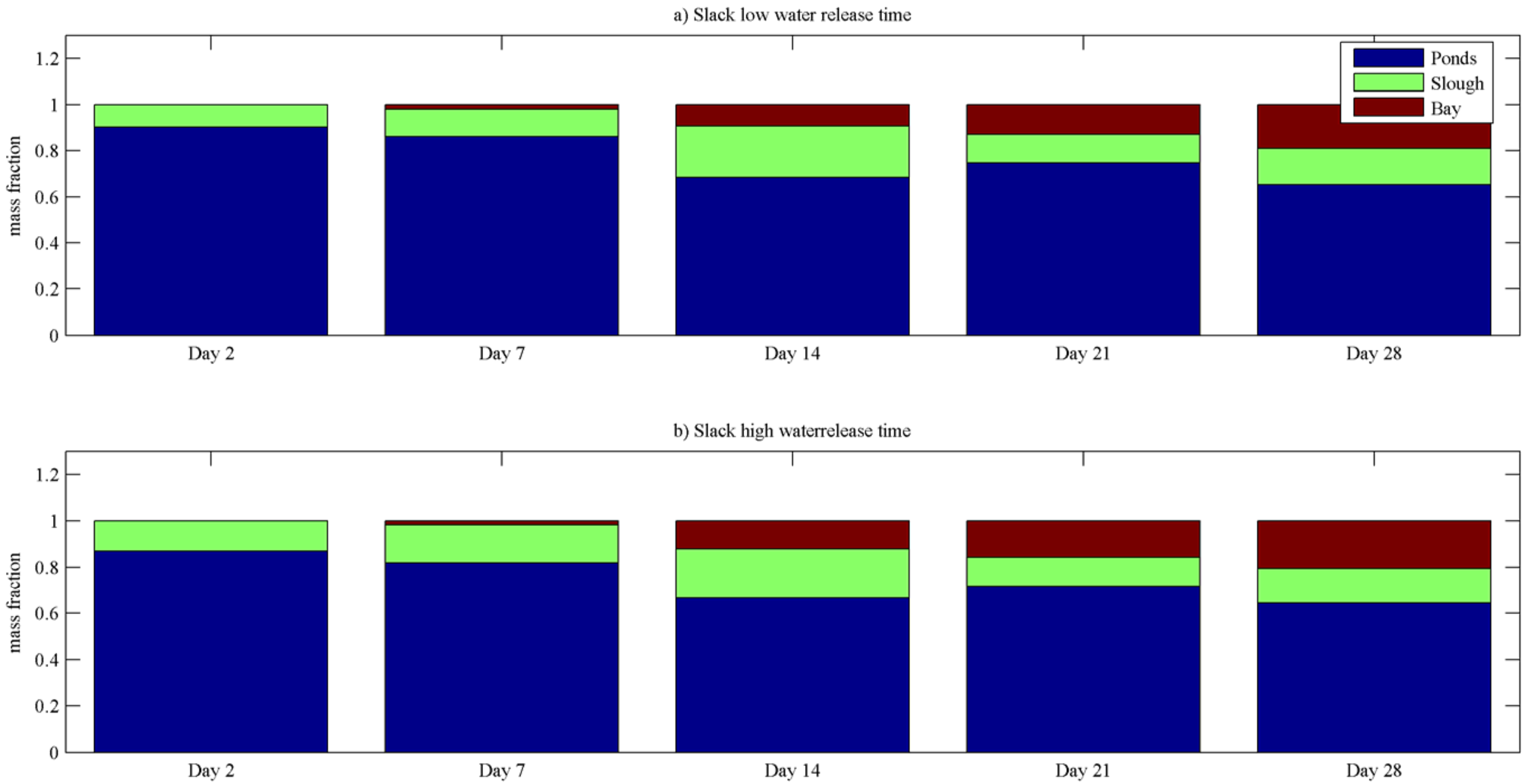
Source: Output from DELFT3D model of far South San Francisco Bay

figure 15

South Bay Salt Pond Restoration Project

Distribution of Alviso Slough particle mass, Phase 1

PWA Ref# 1750.15



Source: Output from DELFT3D model of far South San Francisco Bay

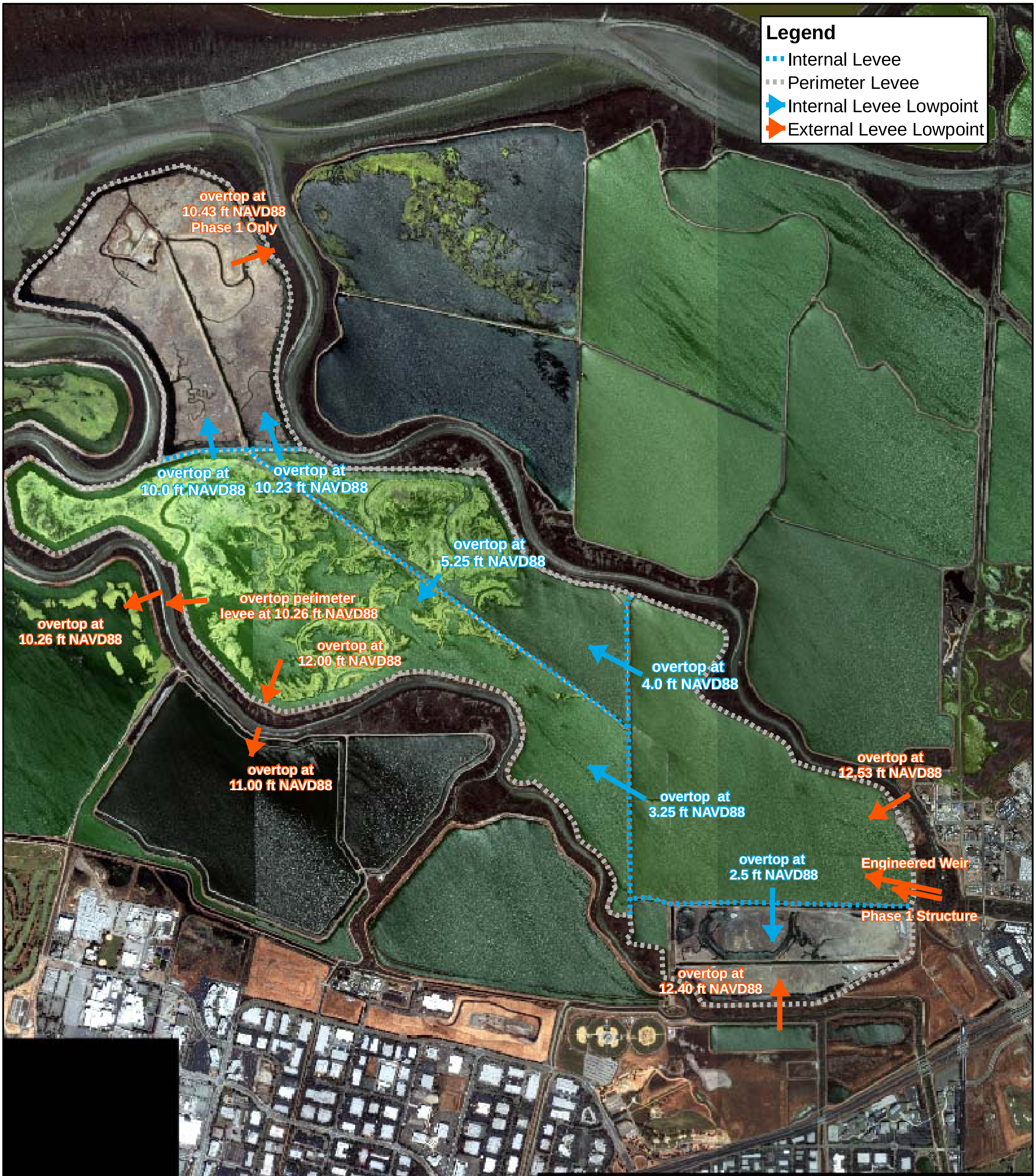
figure 16

South Bay Salt Pond Restoration Project

Distribution of Pond A8 particle mass, Phase 1

PWA Ref# 1750.15

PWA



N

0 1,000 2,000 3,000 4,000 Feet

10 acres

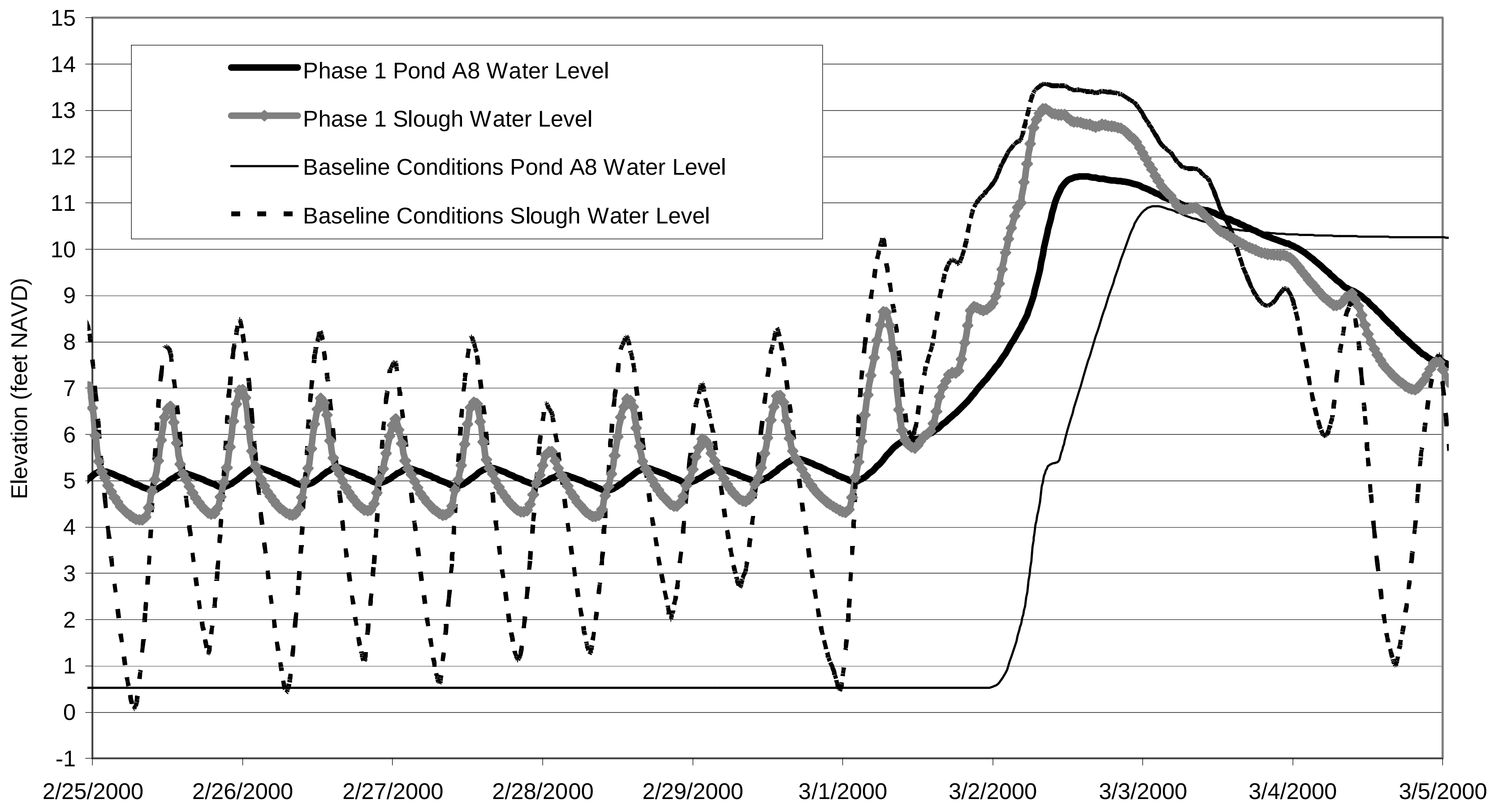
figure 17

South Bay Salt Pond Restoration Project

Levee Overtopping within Pond A7 System

Proj #1751.15

PWA



Source: Output from HEC-RAS model
Comment: Water levels in the slough at the engineered weir and Pond A8 water levels for Baseline and Phase 1 (40') conditions

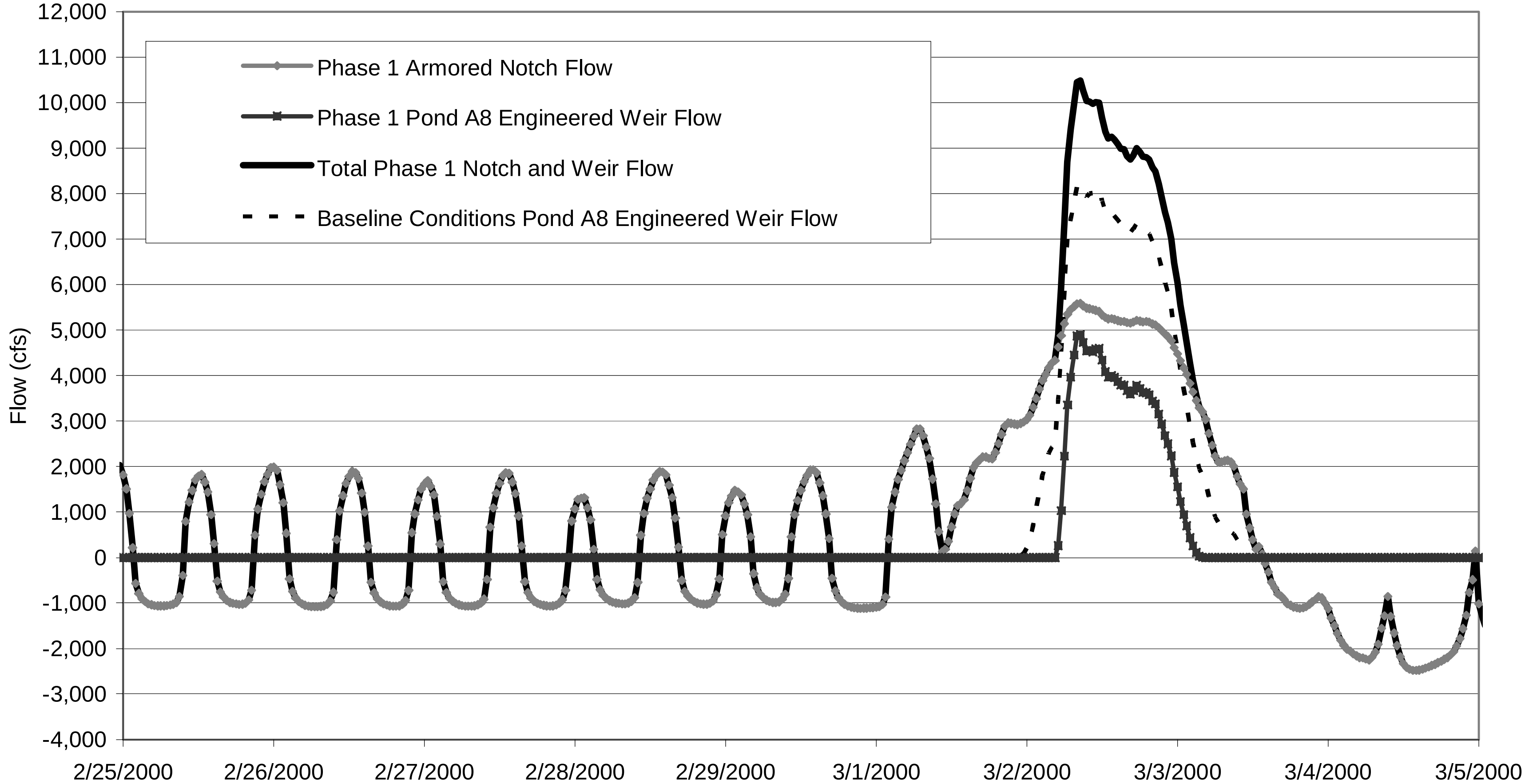
figure 18

South Bay Salt Pond Restoration Project

Pond A8 and Alviso Slough Water Levels

PWA Ref# 1750.15

PWA



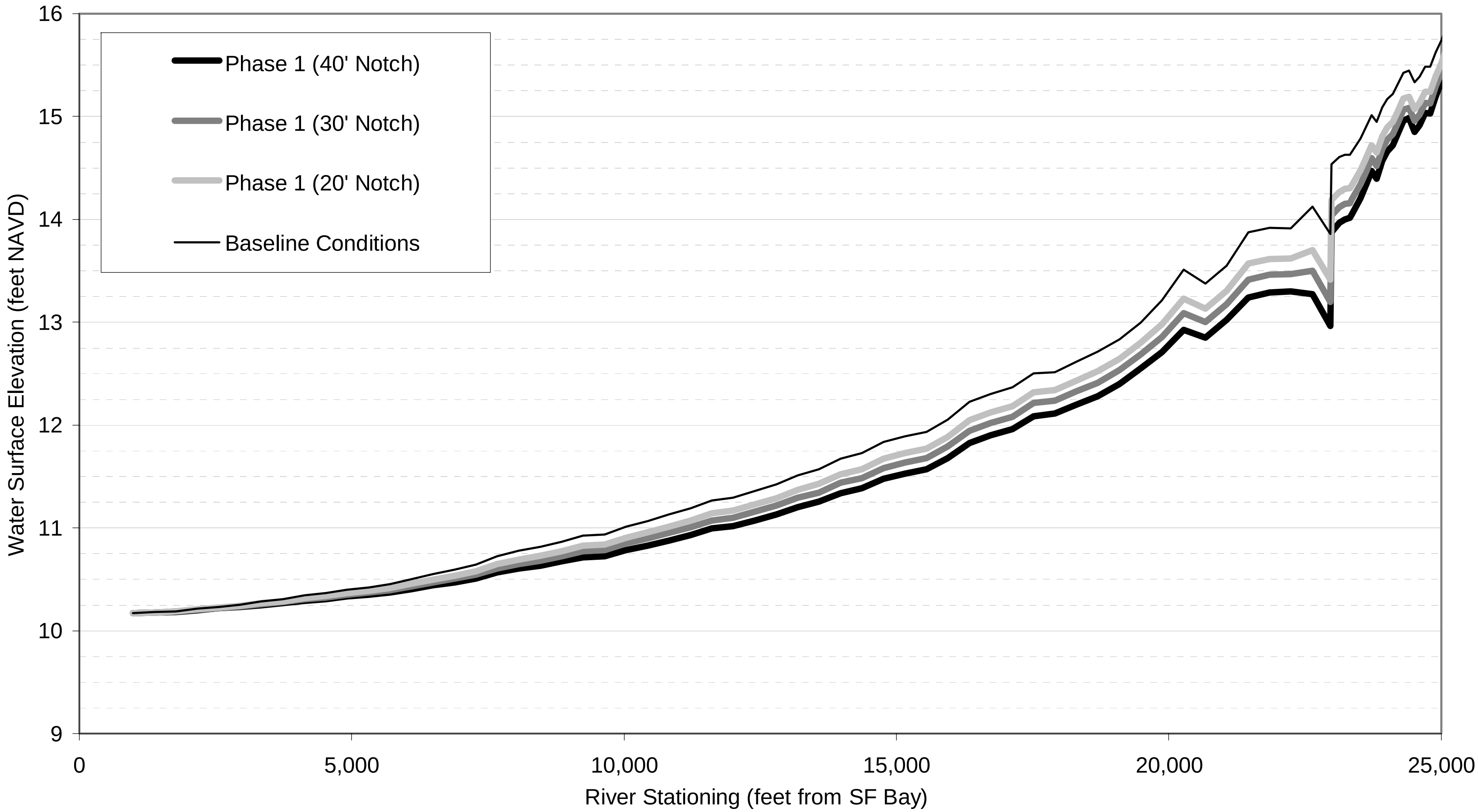
Source: Output from HEC-RAS model
Comment: Flow rates in Alviso Slough and over the engineering weir for Baseline and Phase 1 (40') conditions, and flow rates over the Phase 1 notch for Phase 1 conditions

figure 19
South Bay Salt Pond Restoration Project

Pond A8 and Alviso Slough Flow Rates

PWA Ref# 1750.15

PWA



Source: Output from HEC-RAS model

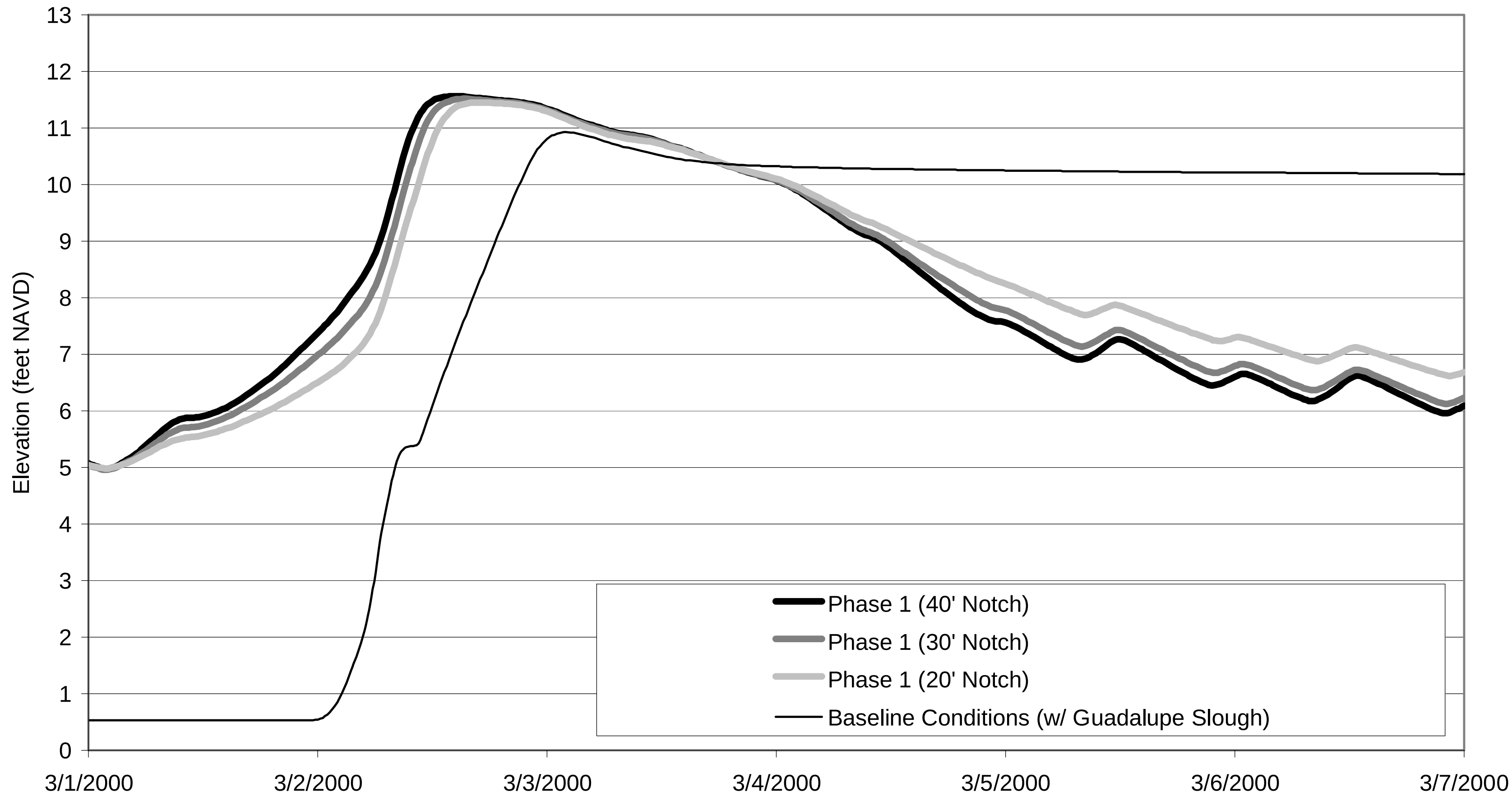
figure 20

South Bay Salt Pond Restoration Project

Alviso Slough Max Water Surface Elevations

PWA Ref# 1750.15

PWA



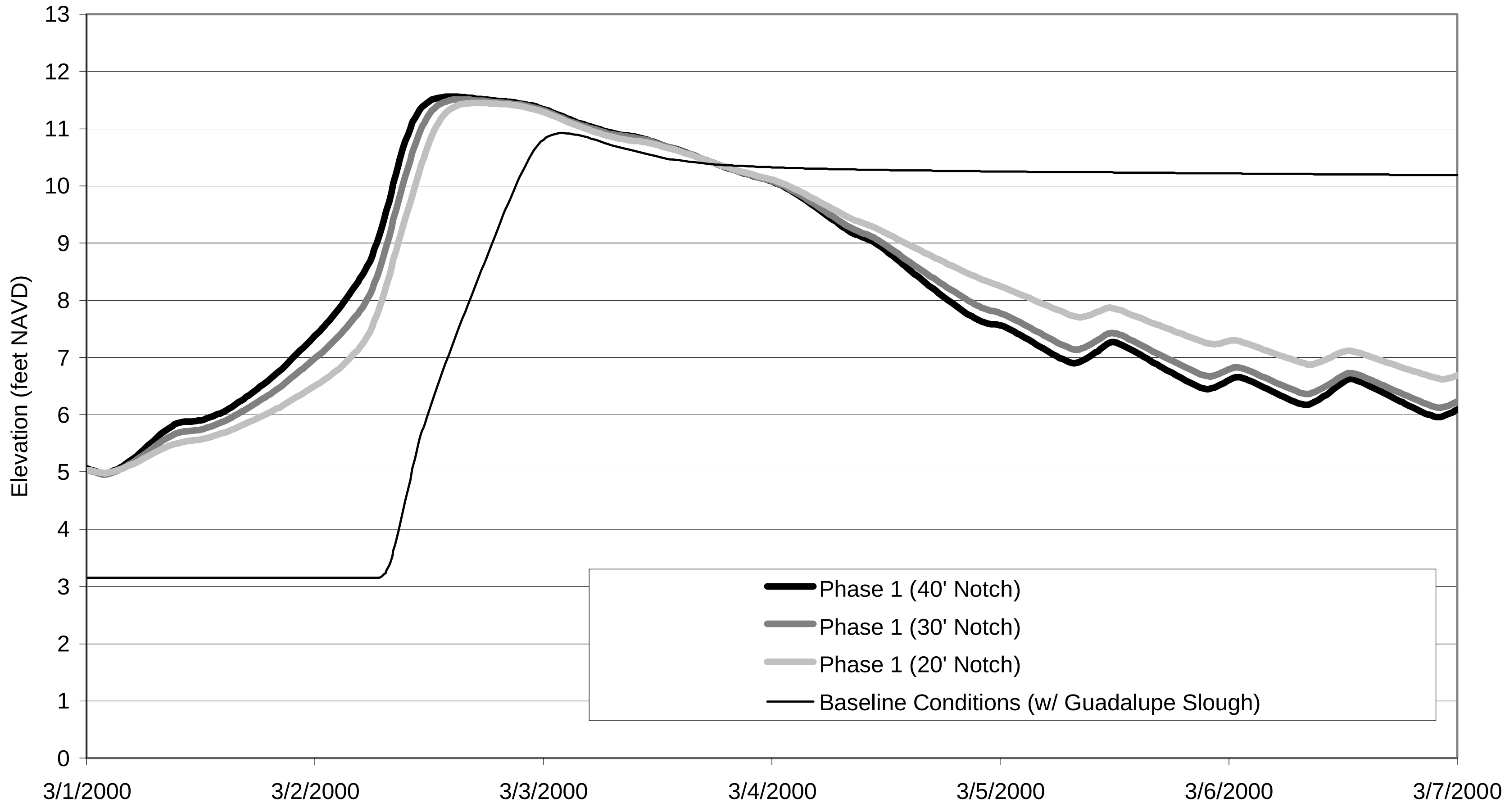
Source: Output from HEC-RAS model

figure *21*

South Bay Salt Pond Restoration Project

Pond A8 Water Surface Elevations

PWA Ref# 1750.15



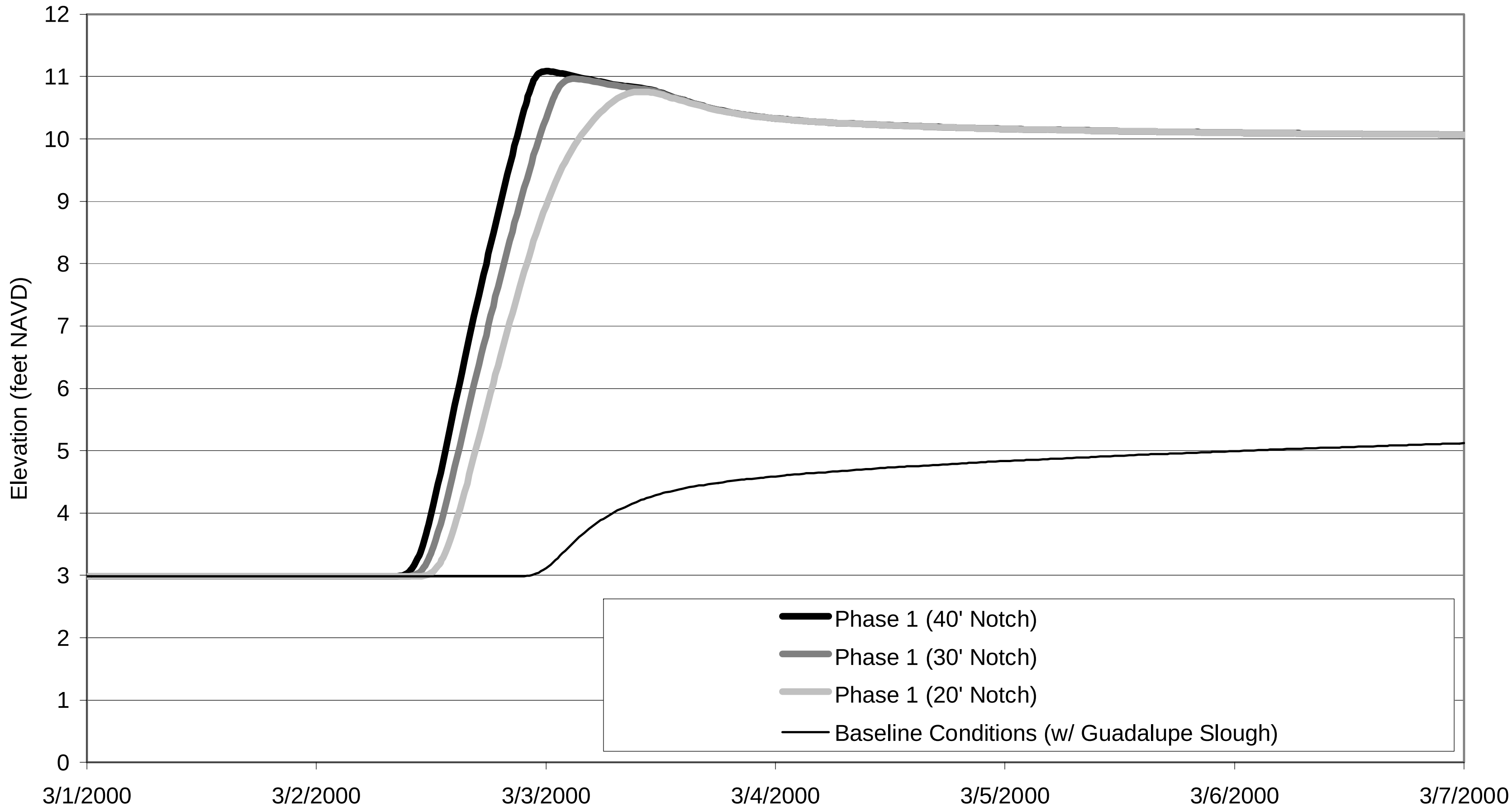
Source: Output from HEC-RAS model

figure 22

South Bay Salt Pond Restoration Project

Pond A5 Water Surface Elevations

PWA Ref# 1750.15



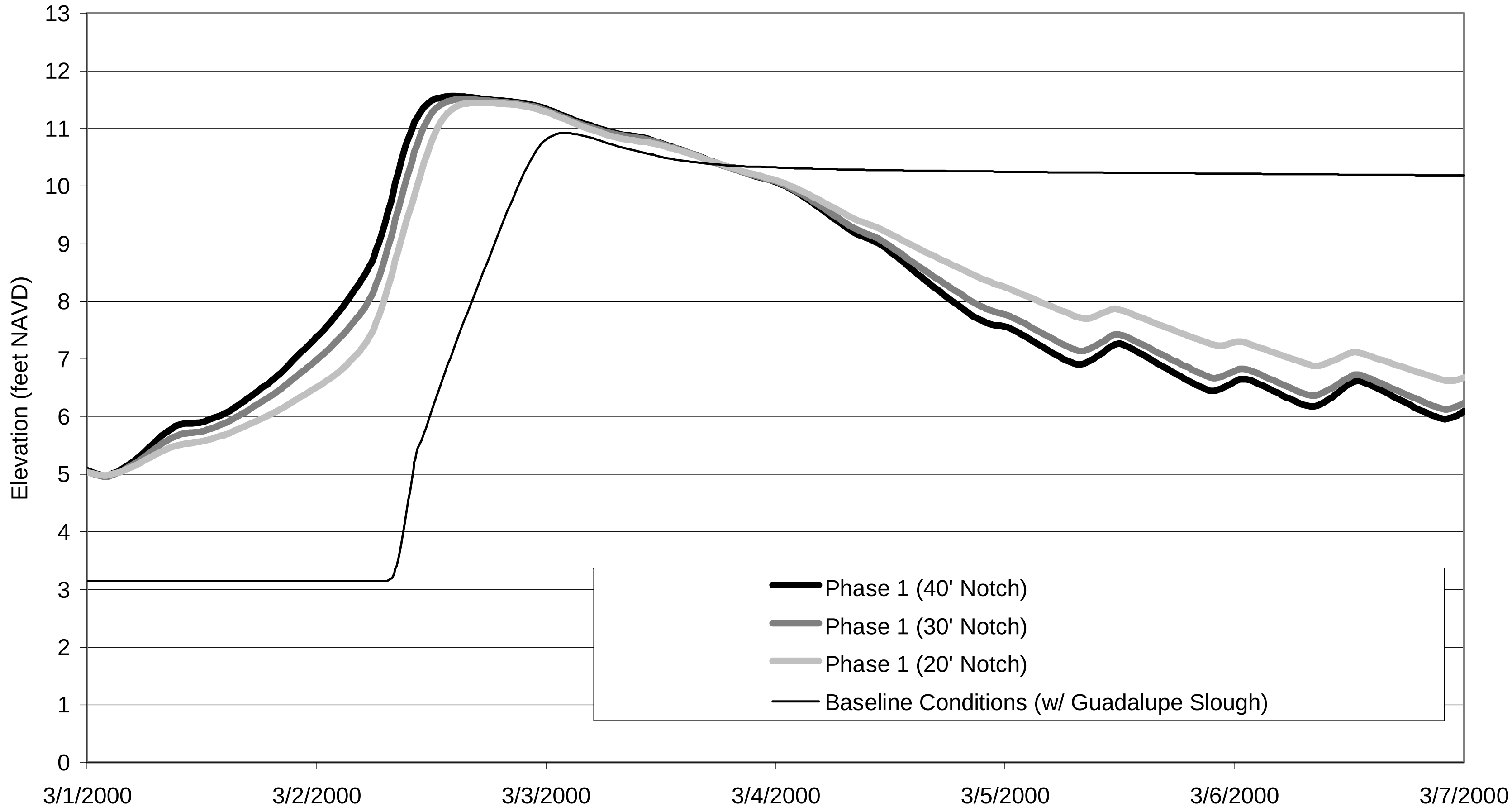
Source: Output from HEC-RAS model

figure *23*

South Bay Salt Pond Restoration Project

Pond A6 Water Surface Elevations

PWA Ref# 1750.15



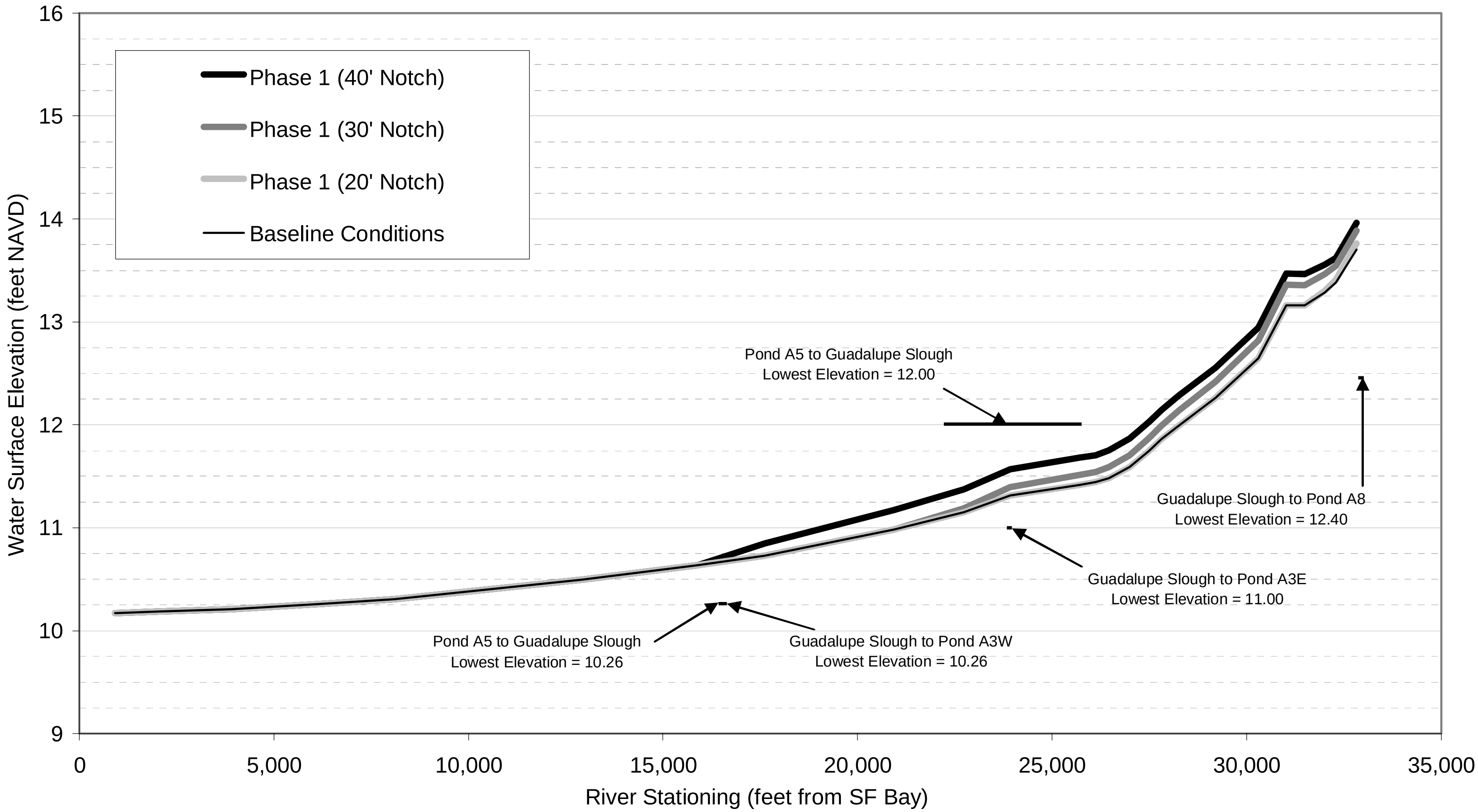
Source: Output from HEC-RAS model

figure 24

South Bay Salt Pond Restoration Project

Pond A7 Water Surface Elevations

PWA Ref# 1750.15



Source: Output from HEC-RAS model

figure 25

South Bay Salt Pond Restoration Project

Guadalupe Slough Max Water Elevations

PWA Ref# 1750.15

PWA

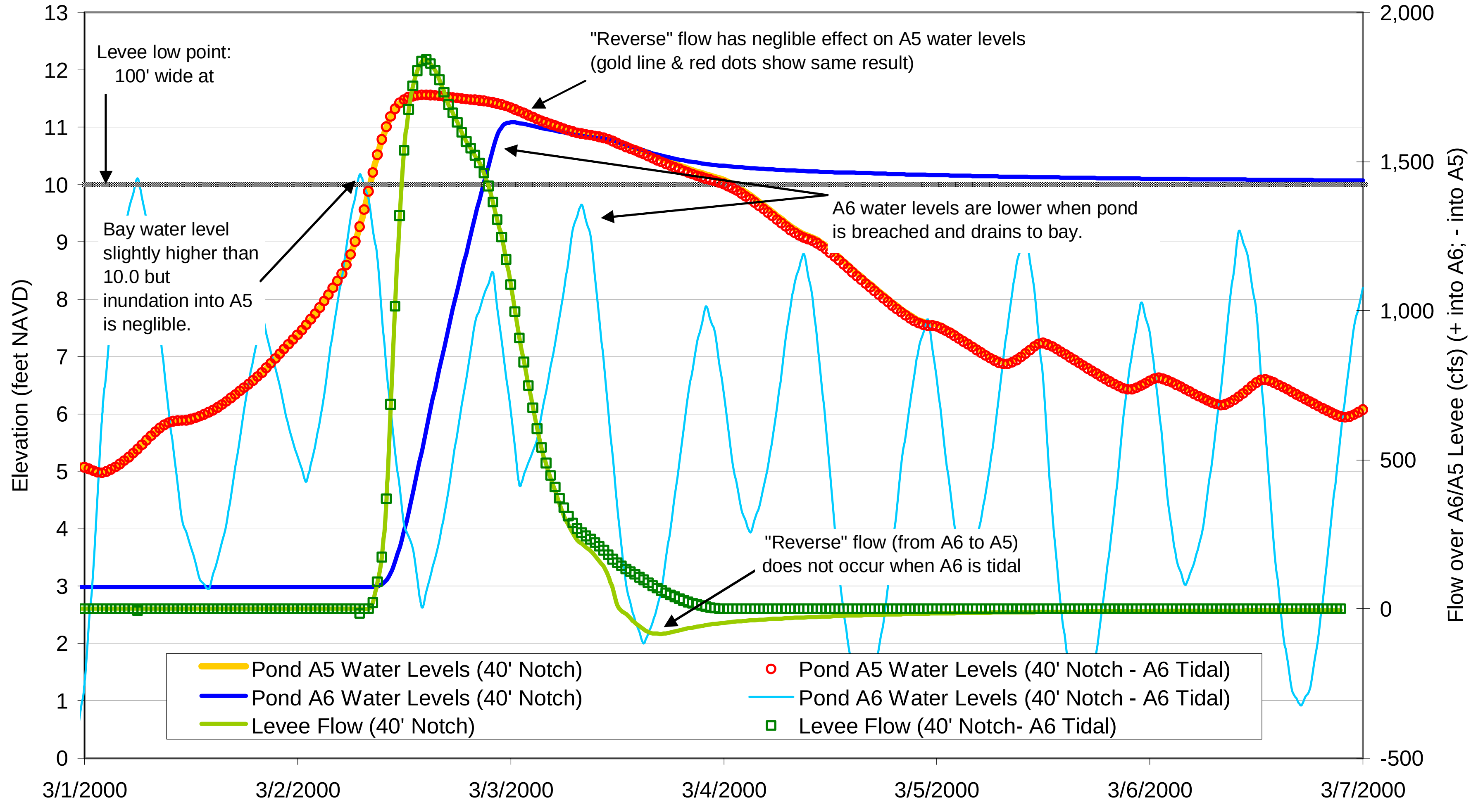


figure 26

Comments / Sources:

HEC-RAS results for 100-year fluvial and 10-year tidal simulations. Pond A5 water levels are shown for Phase 1 conditions with Pond A6 intact ("40' Notch") and with Pond A6 tidal ("40' Notch - A6 Tidal"). Pond A6 water levels are shown for both cases (for the A6 tidal case, water peaks at the 10-year elevation of 10.17 ft NAVD). Flow over the A5-A6 levee is shown for both conditions. Positive flow is from A5 to A6.

South Bay Salt Pond Restoration Project

Pond A5 Water Levels with Pond A6 Tidal

PWA

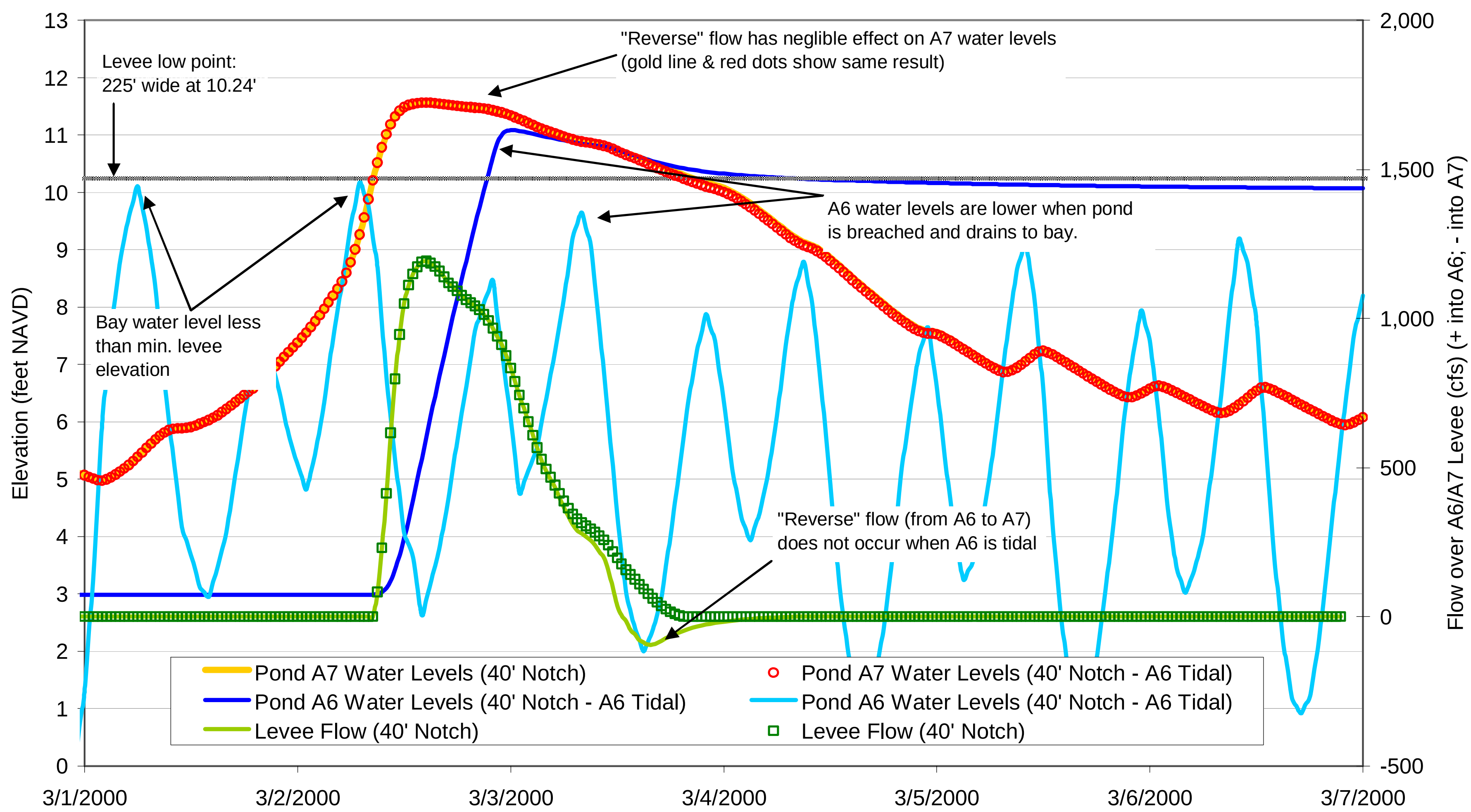


figure 27

Comments / Sources:

HEC-RAS results for 100-year fluvial and 10-year tidal simulations. Pond A7 water levels are shown for Phase 1 conditions with Pond A6 intact ("40' Notch") and with Pond A6 tidal ("40' Notch - A6 Tidal"). Pond A6 water levels are shown for both cases (for the A6 tidal case, water peaks at the 10-year elevation of 10.17 ft NAVD). Flow over the A7-A6 levee is shown for both conditions. Positive flow is from A7 to A6.

South Bay Salt Pond Restoration Project

Pond A7 Water Levels with Pond A6 Tidal

PWA

Hydrology, Flood Management and Infrastructure

Impact Analysis Support

G-6. Alviso Pond A16 Hydraulic Modeling

M E M O R A N D U M

TO: Members of the South Bay Salt Pond Restoration Project Management Team

FROM: Philip Williams & Associates, Ltd. (PWA)

DATE: May 4, 2007

RE: **Attachment 3. Alviso Pond A16 Restoration Preliminary Design: Managed Pond Hydraulic Modeling Appendix**

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1. INTRODUCTION

PWA performed one-dimensional hydraulic modeling and a residence time analysis for the Pond A16 Phase 1 reconfigured managed pond restoration for the South Bay Salt Pond Restoration (SBSP) project Environmental Impact Statement/Report (EIS/R) (PWA et al., 2007). For the preliminary design of the Pond A16 restoration, PWA refined the Pond A16 hydraulic model by adjusting the number and size of Pond A16 cells and performed additional modeling to investigate different combinations of hydraulic control structures. Modeling performed during preliminary design provides a basis for the restoration design and hydraulic criteria for water control structure design. This memorandum is an appendix to the Pond A16 Preliminary Design Memorandum and documents both the model setup performed for the EIS/R and additional modeling performed for the Pond A16 Preliminary Design.

The purposes of the computer-based model simulations of managed pond hydraulics for the Pond A16 restoration were to:

- *Assess whether the Pond A16 restoration will provide target shallow water depths for migratory shorebird habitat throughout the daily and monthly (spring-neap) tide cycles*. The Pond A16 restoration action is intended to provide shallow water depths throughout the year that are optimal for shorebird foraging habitat. Pond bottom elevations vary from approximately 1 to 5 ft NAVD (0.3 to 1.5 m NAVD). Pond A16 would be reconfigured by constructing berms to: separate higher elevation areas from lower elevation areas into three cells, allow water levels to vary between different cells, and create cells with similar shallow water depths over the sloping pond bottom. The target water depth is between approximately four to eight inches (10 to 20 cm), with an average of six inches (15 cm), to maintain optimal depths for foraging habitat. Due to variations in bed elevation there will be some areas of shallower and deeper depths within each cell.
- *Estimate hydraulic residence times within the reconfigured managed pond to inform the water quality assessment*. Managed pond water quality depends in part on the hydraulic residence time of water flowing through the managed pond. PWA used the results of the hydraulic modeling to estimate approximate residence times in Pond A16 assuming complete mixing within the cells.

- *Assess summer pond salinity for water quality discharge to Artesian Slough*. PWA estimated the increase in salinity relative to intake salinities at Coyote Creek using typical summertime evaporation rates and residence times from the hydraulic modeling.

The hydraulic modeling and residence time analyses also provide an estimate of the number, size, and design of water control structures needed to effectively manage water levels and flows for the Pond A16 restoration.

2. METHODS

PWA calculated areas and volumes for the cells and canals proposed for the Pond A16 reconfigured managed pond restoration (Section 2.1). PWA then constructed a one-dimensional hydraulic model to simulate water levels and flows in the reconfigured managed pond (Section 2.2). Using the hydraulic model results, PWA estimated approximate residence times in Pond A17 and Pond A16 for water quality assessment (Section 2.3).

2.1 Volume and Area Calculations

PWA measured approximate areas for the canals and cells proposed for the Pond A16 restoration in GIS. The open water area in each cell was calculated by subtracting the total estimated surface area of the proposed nesting islands. Typical bottom elevations (mode or most common) for each cell were estimated from sonar bathymetry data collected by the USGS under contract with the California State Coastal Conservancy. Design water levels in each cell were set at six inches (15 cm) above the typical cell bottom elevations, to meet a target ponding depth of six inches (15 cm). Note that some areas will be either deeper or shallower than the target depth due to variations in pond bed elevation. Water volumes were calculated in GIS for each cell as the difference between the existing grade of the pond bottom and the design water level. Nesting islands would be graded by borrowing material from on-site, creating deeper borrow areas within the cells. The net increase in cell water volume due to on-site borrow and island grading (cut and fill) was estimated as the volume of borrow material placed above the water level to create the islands (approximately 1,480 cubic yards (1130 m^3) per island).

2.2 Hydraulic Model Setup

PWA constructed an unsteady one-dimensional hydraulic model using the US Army Corps of Engineers HEC-RAS software (version 3.1.3) to simulate water levels and flows for the Pond A16 restoration. The unsteady simulations in HEC-RAS are performed by a modified version of the UNET (Unsteady NETwork model) program.

The Pond A16 HEC-RAS model schematic is shown in Figure 1. The model extent includes Ponds A16 and A17, Coyote Creek from the Newby Island Landfill downstream to the Union Pacific Railroad Bridge (UPRR), and Artesian Slough from the southern edge of Pond A16 downstream to the slough's junction with Coyote Creek. The Pond A16 model uses metric units. As described below, the main components of the model include the model configuration, the model parameters, and the tidal and discharge boundary conditions.

The one-dimensional HEC-RAS model of the Pond A16 restoration represents Pond A17 and the Pond A16 cells and canals as water storage basins. The model simulates water levels in the basins and flows between the basins, but does not model flow or water velocities within the basins. As a result, the model cannot represent mixing within the cells or variations in the flow between islands in a cell.

Bed Friction Sensitivity. In the Pond A16 hydraulic model, Pond A17 and the cells and canals in Pond A16 are modeled as storage volumes, which do not include the effects of bed friction. Bed friction can cause water surface gradients and reduce flows, whereas storage volumes are modeled with a flat water surface and flows are controlled only at the intake and outlet (i.e., like a bathtub). To assess the relative importance of bed friction on the shallow water flows within the Pond A16 cells, the storage elements representing Pond A17 and Cells 1-3 were converted to prismatic open channel reaches. Given the deeper water depths in the intake and outlet canals, these elements were retained as storage areas. Representative cross sections were constructed based on cell hypsometry to preserve the distribution of area at each elevation relative to existing conditions. Model runs were then executed at typical and high roughness values to assess the influence of bed friction on water levels and flow rates.

2.2.1 Model Configuration

The following discussion describes how water control structures and flow are schematized in the model, which is also shown in Figure 1.

Water from Coyote Creek flows into Pond A17 through culverts that can be adjusted for either one-way (intake-only or discharge-only) or two-way flow. Pond A17 is connected to the Pond A16 intake canal through one-way culverts. The intake canal has two weir connections each to Cells 1, 2, and 3. Cells 1, 2, and 3 each have two weir connections to the outlet canal. The outlet canal is connected to Artesian Slough by discharge only culverts near the southeastern corner of Pond A16.

PWA modeled three scenarios with multiple configurations of water control structures and hydraulic parameters. The culvert configurations and cell representations are summarized below:

- *Summer Operations*
    - Three 4-ft intake-only and one existing 4-ft two-way (muted tidal) culverts between Coyote Creek and Pond A17
    - Pond A17 storage area
    - Three 4-ft intake-only culverts between Pond A17 and A16
    - Intake and outlet canal storage areas
    - Seven 4-ft discharge-only culverts to Artesian Slough (including one existing)
- *Winter Operations*
    - One 4-ft intake-only culvert between Coyote Creek and Pond A17
    - Pond A17 storage area

  - Two 4-ft intake-only culverts between Pond A17 and A16
  - Intake and outlet canal storage areas
  - Seven 4-ft outlet culverts to Artesian Slough (including one existing)
  - Rainfall simulated based on representative CIMIS time series at San Jose, CA
  - Storm surge simulated by adding 0.5 ft to the Coyote Creek tidal boundary condition for the duration of the simulation
- *Bed Friction Sensitivity*
  - Three 4-ft intake-only culverts between Coyote Creek and Pond A17
  - Prismatic channel representing Pond A17
  - Three 4-ft intake-only culverts between Pond A17 and A16
  - Intake and outlet canal storage areas
  - Prismatic channels representing Pond A16 cells
  - Six 4-ft outlet culverts to Artesian Slough (including one existing)

Table 1 includes detailed information on the configuration of water control structures in the model scenarios. Each scenario includes the existing culverts located between Coyote Creek and Pond A17 and between Pond A16 and Artesian Slough. One-way flow limitations are applied to these culverts in the model. Invert elevations for the existing culverts were obtained from as-built drawing obtained from USFWS (2006).

The weir lengths and culvert sizes were initially set and the weir elevations and number of intake/outlet structures were then adjusted iteratively to achieve target water levels and depths (approximately 0.15 m or 6 in) in the three cells. This process resulted in the water control structure configurations for the model scenarios described above and in Table 1.

Table 1. Configuration of water control structures for recommended pond operations.

Weir or Culvert Condition	Summer Operations	Winter Operations
Intake culverts between Coyote Creek and Pond A17	Three 1.22 m (4 ft) culverts, intake-only; one 1.22 m (4 ft) two way culvert, invert elevations at 0.38 m NAVD	One 1.22 m (4 ft) culvert, intake-only; invert elevation at 0.38 m NAVD
Intake culverts between Pond A17 and Pond A16 intake canal	Three 1.22 m (4-ft) intake only culverts; invert elevations at 0.38 m NAVD	Two 1.22 (4 ft) intake only culverts; invert elevations at 0.38 m NAVD
Number of weirs between canals and Cell 1	Four 1.22 m (4-ft) rice checks	Three 1.22 m (4-ft) rice checks (intake) Two 1.22 m (4-ft) rice checks (outlet)
Elevation of Cell 1 weirs	1.22 m NAVD (intake) 1.22 m NAVD (outlet)	1.22 m NAVD (intake) 1.22 m NAVD (outlet)
Number of weirs between canals and Cells 2 & 3	Six 1.22 m (4-ft) rice checks	Four 4-ft (1.22 m) rice checks
Elevation of Cell 2 weirs	1.18 m NAVD (intake) 1.02 m NAVD (outlet)	1.18 m NAVD (intake) 1.045 m NAVD (outlet)
Elevation of Cell 3 weirs	1.16 m NAVD (intake)	1.16 m NAVD (intake)

	0.73 m NAVD (outlet)	0.755 m NAVD (outlet)
Outlet culverts between outlet canal and Artesian Slough	Seven 1.22 m (4 ft) discharge-only culverts: one existing with invert elevation at 0.38 m NAVD, six with invert elevations at -0.3 m NAVD	Seven 1.22 m (4 ft) discharge-only culverts: one existing with invert elevation at 0.38 m NAVD, six with invert elevations at -0.3 m NAVD

For the summer and winter operations, the Pond A17 and the Pond A16 canals and cells are modeled as storage basins with vertical side slopes, giving volumes equal to depth times area.

For all model scenarios, cross section information for Coyote Creek and Artesian Slough is constructed from the South Bay bathymetric survey data collected by the USGS and Sea Surveyor under contract with the California State Coastal Conservancy. Cross sections are spaced at approximately 100 meters for both channels. The bathymetry data does not include the overbank marshplain and adjacent levees. Channel bank elevations and overbank (marshplain) elevations are assumed to be at MHHW, or 2.25 m NAVD88. Left and right overbank (marshplain) widths are estimated to be 100 meters for Artesian Slough, 300 meters for Coyote Creek upstream of its junction with Artesian Slough, and 400 meters for Coyote Creek downstream of the junction with Artesian Slough. Levee elevations are set above the highest tide level in Coyote Creek so that the simulated tide levels do not overtop the levees. These assumptions are consistent with conditions in Coyote Creek and Artesian Slough and are expected to have little effect on the model results.

2.2.2 Model Parameters

Roughness (Manning's n) values for in-channel and overbank areas in Coyote Creek and Artesian Slough were set at 0.03. The hydraulic model parameters for the structures are shown in Table 2 and summarized below.

Weirs. The Pond A16 preliminary design includes rice box intake/outlet structures for the cells. Due to limitations of the HEC-RAS model, cell intake/outlet structures were modeled as broad-crested weirs, but are expected to behave more like submerged sharp-crested weirs. To compensate for the higher discharge over a sharp-crested weir, a weir coefficient (C_d) of 1.83 $m^{0.5}/s$ (3.31 $ft^{0.5}/s$) was used. The HEC-RAS model reduces flow over the weir to account for submergence. To simulate the effects of flow contraction through the weir (and provide a conservative estimate of flow through the structure), an effective width of 3.85 ft was used for each 4-ft weir specified in the design.

Culverts. All modeled culverts were 40 ft (12.2 m) in length with a roughness value of 0.013, based on an assumed construction material of high density polyethylene (HDPE). Culverts were modeled with zero slope from the upstream to downstream invert. A maximum allowable value of 1.0 was chosen for the entrance and exit loss coefficients to account for head loss through the trash racks and tide flap gates at the Coyote Creek-Pond A17 intake structure. The A16 intake and Artesian Slough outlet culverts were modeled with entrance and exit loss coefficients of 0.9 and 1.0, since the culverts will not have trash racks at the entrance. Trash racks, tide flap gates, and fish screens have the potential to reduce head and flows below modeled values. To calibrate the entrance and loss coefficients, flow through the existing Pond

A17 intake structure was modeled. The modeled flow rate averaged approximately 15 cfs (0.42 m^3/s), which agrees well with the average flow reported by USFWS (2006). Therefore, it is believed that the entrance and loss coefficients are represented correctly in the model.

Fish Screen. Flow through the fish screen at the Coyote Creek-Pond A17 intake structure was not modeled at this phase in the design.

Table 2 – Model Parameters for Water Control Structures

Culverts	
FHWA Chart #	2 – Corrugated Metal Pipe Culvert
FHWA Scale #	3 – Pipe Projecting from Fill
Length	40 ft (12.2 m)
Diameter	4 ft (1.22 m)
Slope	0.0
Entrance/Exit loss coefficients	0.9 or 1.0 / 1.0
Manning's n	0.013 (for HDPE, non-corrugated)
Weirs	
Weir type	Broad-crested weir
Weir coefficient, C_d	3.31 $ft^{0.5}/s$ (1.83 $m^{0.5}/s$)

Bed Friction Sensitivity. For the bed friction sensitivity model, two runs were executed to investigate the effects of bed roughness on flows and water surface slopes within the cells. The first run used a "typical" bed roughness value of 0.03. The second run used a "high" roughness value of 0.06.

2.2.3 Boundary Conditions

The Pond A16 HEC-RAS model has unsteady boundary conditions. The upstream boundary of Artesian Slough is a flow hydrograph using daily discharge data from the San Jose/Santa Clara Water Pollution Control Plant in San Jose. The upstream boundary of Coyote Creek is a daily flow hydrograph generated from three Santa Clara Valley Water District gauges: #1 on Penitencia Creek at Piedmont Road, #58 on Coyote Creek at Edenvale, and #75 on Thompson Creek and Quimby Road. The sum of these three gages is considered a reasonable estimation for flows in Coyote Creek (PWA 2006). The downstream boundary of the model is located on Coyote Creek at UPRR. PWA collected water level data in 2000 and 2001 in Coyote Creek at UPRR. This boundary condition data is supplied to the model in 15 minute intervals.

To simulate the effect of rainfall during the winter season, a lateral discharge hydrograph was applied to each storage area (Pond A17, intake canal, Cell 1, etc.) based on observed rainfall representative of a typical winter month in South San Francisco Bay. Rainfall from the CIMIS San Jose meteorological station for December 2001 (total monthly rainfall = 3.3") was selected for this analysis and converted to an equivalent hourly discharge to each cell based on the cell's area.

The effect of storm surge in controlling cell water levels and inflow from Coyote Creek to Pond A17 and outflow from the outlet canal to Artesian Slough was examined in an additional model run by adding 0.5 ft to the Coyote Creek tidal boundary condition. This model run included the combined effect of rainfall and storm surge.

The model's computational time period begins at 12:00 am on June 1, 2001 and ends at 12:00 am on June 30, 2001. Measured data is available for all boundary conditions during this period. The simulation period includes more than a full spring-neap tide cycle to assess how spring and neap tides affect water levels and flows in the Pond A16 restoration. The computational interval is 5 minutes and output data is recorded every 30 minutes.

2.3 Residence Time Analysis

Residence time is defined as the time for the volume of a water body to be completely replaced by new inflow. For a water body with continuously varying through-flow, the residence time can be estimated by calculating the cumulative discharge (i.e., integrating discharge) into the water body until the cumulative volume of the integrated discharge equals the volume of the pond. The duration of the integration period then approximates the residence time. This method was implemented for the ponds, canals and cells of the Pond A16 restoration using the calculated volumes and the discharges predicted by the HEC-RAS model. Changes in pond volume due to fluctuations in water level are small and are therefore neglected.

This estimate assumes that inflow replaces outflow without mixing (i.e., "plug flow"). This assumption greatly simplifies the actual conditions expected. Actual residence times may be higher or lower due to degree of mixing. Spatial variations in circulation (e.g., lower than average residence times in areas along more efficient flow paths and higher than average residence times in pockets of poor circulation) will also alter actual residence times. While the calculated residence times are not expected to be accurate, they provide a method to compare residence times for different alternatives relative to each other. This simplified method of comparison is appropriate for the level of modeling and analysis within the scope of the Pond A16 restoration preliminary design.

3. RESULTS AND DISCUSSION

3.1 Volumes and Areas

The calculated areas, depths, and volumes of Pond A17 and the Pond A16 canals and cells are shown in Table 3 (attached). These volumes are used in the residence time estimates (Section 3.3).

3.2 Hydraulic Model Results

A general discussion of the pond hydraulics is provided (3.2.1) followed by the model results for summer (3.2.2) and winter (3.2.3) operations. The results for the bed friction sensitivity analysis are presented in Section 3.2.4. Tidal slough water levels and datums are presented in Section 3.2.5.

3.2.1 Managed Pond Hydraulics

Inflow from Pond A17 to Pond A16 will be continuously driven by a relatively small (muted) hydraulic head (approximately 2 to 8 inches) compared to inflows into Pond A17 driven by high tides in Coyote Creek. Peak inflows into Pond A16 are therefore less than peak inflows into Pond A17. As a result, water level fluctuations in Pond A16 are less and target water levels within the cells are easier to achieve.

Water levels in the cells are primarily controlled by the elevation of the weirs. Cell water levels will tend to increase during spring tides when inflows are higher and exceed cell outflows. Similarly, cell water levels will tend to decrease during neap tides when inflows are lower than cell outflows. Due to variations in bed elevation, different portions of each cell will be within the target depth range over the course of the spring-neap cycle. To maintain relatively constant water levels, inflow to the cells is approximately equal to outflow from the cells. The rate of flow through the cells is primarily controlled by the water level in the intake canal and the elevation of the intake weirs. Maintaining higher water levels in the intake canal provides the hydraulic head to continuously drive flow through the cells and minimize residence time. Outflow from the outlet canal to Artesian Slough must be great enough to maintain water levels in the outlet canal that are below the cell water levels, so that water can continuously flow out of the cells. If the outlet canal water level exceeds the cell water levels, water may “back up” in the cells, potentially causing cell depths to exceed target levels. This is most likely to be observed during spring tides when the combined flow through the cells approaches the discharge capacity of the outlet structure at Artesian Slough.

Water levels and inflow for the summer operations are shown in Figure 2. Water levels and inflow for winter operations, winter operations with rain, and winter operations with rain and storm surge are shown in Figures 3-5, respectively. Weir inflow results for the modeled scenarios are shown in Figure 6.

3.2.2 Summer Operations

Inflow to each cell is approximately proportional to the cell volume to yield similar residence times in each cell. Inflow is proportioned by adjusting the number and elevation of the weirs between the canals and the cells. Cell depths for the summer scenario are shown in Figure 2c and range from 3.0 to 8.5 inches (7.5 to 21.5 cm), with an average of six inches (15 cm). Cell inflows during summer operations are shown in Figure 6a. Cell inflows and water levels fluctuate both over the daily and neap-spring (two week) tide cycles; however, cell depths are maintained near the target range (four to eight inches or 10 to 20 cm). The mean flow rates through Cells 1, 2, and 3 are 7.1 ft^3/s (0.2 m^3/s), 23.3 ft^3/s (0.66 m^3/s), and 28.3 ft^3/s (0.8 m^3/s).

3.2.3 Winter Operations

Cell inflows during winter operations are shown in Figure 6b. For the winter scenario, inflows are lower and fluctuate less over the spring-neap cycle than inflows for the summer scenario. The mean flow rates through Cells 1, 2, and 3 are 2.2 ft^3/s (0.06 m^3/s), 9.9 ft^3/s (0.28 m^3/s), and 13.4 ft^3/s (0.38 m^3/s). Cell depths for the winter scenario range from 3.5 to 8.0 inches (9 to 20 cm). Cell water depths are shown in Figure 3c and average six inches (15 cm) in Cells 2 and 3 and five inches (12 cm) in Cell 1. Cell inflows for the winter scenario are approximately three to four times less than for the summer scenario, as the capacity of the Pond A17 and Pond A16 intake culverts is reduced. In general, cell water level fluctuations are greatly damped relative to the summer operations, except in the case of an intense rainfall event. Figure 4 shows the water level and inflow for the winter operations with rainfall and Figure 5 shows the water level and inflow for the winter operations with rainfall and storm surge. In general, rainfall tends to elevate cell water levels for approximately 1-2 days following a typical event. For the storm surge simulation, cell water levels were elevated by approximately one inch for a storm surge of 0.5 ft.

3.2.4 Bed Friction Sensitivity

In general, inclusion of bed friction exerted three primary influences on the modeled water levels relative to the storage element model:

- Water surface slope from upstream to downstream within each cell: The maximum water surface slope of 0.07 in (1.8 mm) occurred in Cell 2 (the longest reach), equivalent to roughly one percent of the target water depth.
- Generally elevated water levels within each cell: Cell water levels were elevated by a maximum of 0.08 in (2 mm) relative to the storage element model.
- Delayed pond drainage on falling tides: Frictional effects accounted for a small phase lag in cell drainage on the order of 10-30 minutes.

While these effects could be significant over longer reaches, the generally short reach lengths (<3600 ft or 1100 m) and cell velocities (0.4-1.6 in/s or 1-4 cm/s) at this site render them negligible. These results indicate that the influence of bed friction on water levels and flows is not significant. It is therefore reasonable to model the restoration site as a system of storage areas.

3.2.5 Slough Water Levels

Table 4 shows a comparison of measured and modeled tidal datums in Coyote Creek and Artesian Sloughs. In general, the modeled tides in Artesian Slough are lower than the observed tides from the 2007 dataset. The MHHW, MHW, and MSL datums for the simulation period are lower than the PWA datums from observations during 2007. The MLW and MLLW datums are slightly higher than observations. Therefore, the simulated diurnal range (7.51 ft) is approximately 0.3 ft less than observed. This is most likely due to the tidal boundary condition at Coyote Creek, which is derived from PWA observed water levels in June 2001. The diurnal range for the boundary condition is approximately 0.4 ft less than the published NOAA tidal datums. This suggests that actual water levels in the sloughs may be greater than predicted by the model. Higher water levels in Coyote Creek will increase flow into Pond A17, while higher water levels in Artesian Slough will limit drainage from the Outlet canal culverts.

Table 4. Comparison of NOAA tidal datums, PWA measured tide data, hydraulic model boundary condition, and simulated water levels in Artesian Slough and Coyote Creek.

	Artesian Slough			Coyote Creek		
Tide Level	PWA Measured[1]	HEC-RAS Simulated[2]	Difference	NOAA Tidal Datums[3]	PWA Measured[4]	Difference
	ft, NAVD	ft, NAVD	ft	ft, NAVD	ft, NAVD	ft
MHHW	7.27	7.1	-0.17	7.48	7.05	-0.43
MHW	6.69	6.37	-0.32	6.9	6.33	-0.57
MSL	3.40	3.04	-0.36	3.4	2.72	-0.68
MLW	0.05	0.15	0.01	-0.28	-0.72	-0.44
MLLW	-0.54	-0.41	0.13	-1.52	-1.64	-0.12

[1]Source: PWA measured tides, 30 January 2007 – 30 March 2007.
[2]Source: HEC-RAS modeling results for Pond A16 complex.
[3]Source: NOAA tidal datums (1983-2001 Tidal Epoch), NOAA unpublished MLLW-NAVD conversion.
[4]Source: PWA measured tides, 1-30 June 2001. HEC-RAS Pond A16 Coyote Creek tidal boundary condition.

3.3 Residence Time Results

Figures 7 and 8 show time series of the estimated residence times for the summer and winter operations, respectively. For each date and time point on the horizontal axis, the residence time was calculated based on discharges centered around this point. Residence times were not calculated at the start and end of the model simulation period because the integration interval extends beyond the simulation period for these times. Estimated residence times are approximate and assume complete mixing within Pond A17, the canals, and each cell. The calculated residence times do not account for areas that may be isolated from the main flow paths in the cells and canals, such as deep areas and sheltered areas between the nesting islands. As discussed below, these factors have the potential to increase residence times in any potentially isolated areas. Similarly, some areas may experience lower than average residence times due to enhanced circulation. While the calculated residence times are not expected to represent the actual physical processes, they provide a method to compare residence times for different alternatives and inform the restoration design and water control structure configuration.

Table 5 summarizes the estimated residence times in each pond/cell and the total residence time for possible flow paths from Coyote Creek to Artesian Slough. In each cell, the longest residence time occurs during or following the neap tides (June 11 to June 19) and the shortest residence time occurs during the spring tides (June 3 to June 8 and June 21 to 25). In Pond A17, the residence time increases by slightly more than a factor of two from neap tide to spring tide for both summer and winter operations. Total residence times are estimated as the sum of residence times calculated for Pond A17, Pond A16 intake canal, a single cell, and the outlet canal. The total residence times through each cell path are shown for summer and winter operations in Figures 7 and 8, respectively.

For summer operations, the shortest residence times are predicted for Pond A17, intake canal, Cell 1, and the outlet canal. Cells 2 and 3 had somewhat longer residence times due to larger cell volumes, although intake weir elevations were adjusted to equalize residence times in the cells. For winter operations, the shortest residence times are predicted for the intake and outlet canals. Cell 1 displays the longest residence time. Neap tide residence times for the winter operations are approximately twice the residence times for the summer operations because the flows for the winter are lower. Spring tide residence times in the cells are approximately three times greater than residence times during summer spring tides. It should be noted that rainfall during the winter months will act to increase flushing of the cells and decrease the overall residence time of the pond system.

These residence time estimates assume that water is well-mixed throughout Pond A17, the canals, and each cell. If this is not the case, residence times may be longer for regions within a cell that are not well-mixed. A simple estimate for the increased residence time can be made by defining a “well-mixed region” and an isolated “backwater region”. The residence time in the backwater region would increase by a factor equal to the ratio of its percentage of the total cell volume to its percentage of the total cell discharge. For example, suppose a backwater region consisting of 90% of the pond volume only receives 10% of the discharge. Then the residence time in that backwater would be nine times larger than the residence time of the entire cell. Similarly, the residence time of the well-mixed region will decrease as a result of more flow through a smaller volume. The residence time of the entire cell, or the outflow water, would not change because water from the well-mixed and backwater regions would be combined, giving a weighted-average residence time equal to the residence time of the entire cell.

Table 5. Summary of Estimated (approximate) Residence Times in Days for the Pond A16 Reconfigured Managed Pond Restoration.

	Residence Times (Days)	
	Summer Operations **12-ft weirs** **3 intake-only A17 intake** **1 two-way A17 intake** **3 intake-only A16 intake** **7* outlet culverts**	**Winter Operations** **4-ft and 8-ft weirs** **1 intake-only A17 intake** **2 intake-only A16 intake** **7* outlet culverts**
Spring tide (shortest RT during the month) *(June 3-8 & 21-25)*		
Pond A17	1	4
Intake Canal	1	3
Cell 1	1	3
Cell 2	2	4
Cell 3	3	5
Outlet Canal	1	3
Total**	**4-6**	**13-15**
Neap tide (longest RT during the month) *(June 11 – 19)*		
Pond A17	3	8
Intake Canal	3	6
Cell 1	4	9
Cell 2	4	8
Cell 3	5	8
Outlet Canal	3	5
Total**	**13-14**	**27-28**

*Includes one existing intake/outlet culvert

**Total residence times are estimated as the sum of residence times calculated for: A) Pond A17, intake canal, Cell 1, outlet canal; and B) Pond A17, intake canal, Cells 3, outlet canal. The range in total residence times (i.e., A to B) is due to, and equal to, the difference in residence times for Cell 1 and Cell 3.

3.4 Salinity Assessment

Methods. During the summer months, evaporation will act to decrease water depths within the cells and increase the salinity of water discharged to Artesian Slough. To assess the increase in salinity in each cell, simplified calculations were used. In each cell, a constant rate of evaporation was assumed over a duration equal to the cell's longest summer residence time (typically 3-6 days). This loss of water was translated to an equivalent increase in salinity based on conservation of mass. The outflow salinity for the cell was set as the inflow salinity for the following cell so that the increase in salinity was additive.

Results. Based on salinity modeling for the Eden Landing Ponds E12 and E13 a typical evaporation rate of 2 inches per 10 days (0.017 ft/day) was assumed. For example, this translated to a 0.05 ft decrease in water level in Pond A17 over three days. This represented a loss of volume of 7 acre-ft and a salinity increase of 1.1 ppt relative to the initial Coyote Creek salinity of 25 ppt. The calculations are shown in Appendix A. For the path from Coyote Creek to Pond A17 to intake canal to Cell 3 to outlet canal to Artesian Slough, the salinity increased from 25 to 32 ppt, which is below the permit discharge threshold of 44 ppt.

4. CONCLUSIONS

The hydraulic modeling and residence time analysis for the Pond A16 reconfigured managed pond restoration indicate:

- Water management using gravity-flow water control structures (culverts and weirs) can provide target shallow water depths for migratory shorebird foraging habitat throughout the daily and monthly (spring-neap) tide cycles for summer and winter conditions, including typical rainfall events and periods prolonged storm surge.
- Hydraulic residence times in the Pond A16 restoration depends on the capacity of the water control structures. Modeling of a summer scenario—including four 4-ft culverts between Coyote Creek and Pond A17 (three intake, one two-way), three 4-ft culverts between Ponds A17 and A16, 8-ft and 12-ft weirs between the Pond A16 canals and cells, and seven 4-ft culverts from the outlet canal to Artesian Slough (including the existing outlet culvert)—gives approximate estimated residence times of 4-14 days. Modeling of a winter scenario—including one 4-ft culvert between Coyote Creek and Pond A17, two 4-ft culverts between Ponds A17 and A16, 4-ft and 8-ft weirs between the Pond A16 canals and cells, and seven 4-ft culverts from the outlet canal to Artesian Slough (including the existing outlet culvert)—gives approximate estimated residence times of 13-28 days. The residence time of water in isolated portions of the cells and canals may be lower or higher than these estimates.

5. REFERENCES

PWA. 2006. Hydrodynamic Modeling Report: Alternatives Analysis. San Francisco, CA. Prepared for: California State Coastal Conservancy, U.S. Fish and Wildlife Service, California Department of Fish and Game.

USFWS (U.S. Fish and Wildlife Service). 2006. E-mail from Eric Mruz to Nick Garrity on June 22.

USFWS (U.S. Fish and Wildlife Service). 2006. Pond A16 Operations Plan. RWQCB Final Order No. R2-2004-0018 Self-Monitoring Program. January 2006.

6. LIST OF PREPARERS

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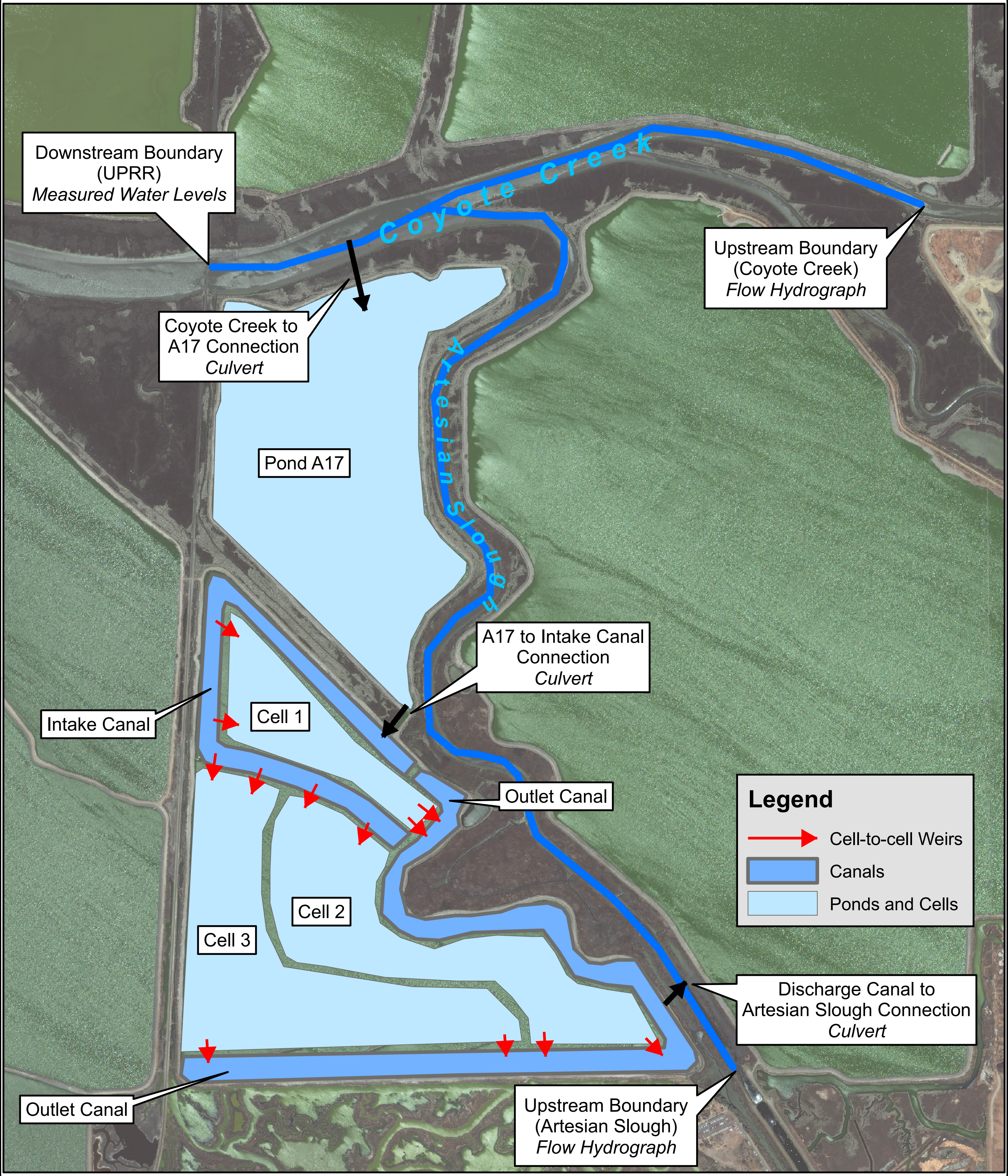
Source files for this report are located at PWA:
P:\Projects\1750_South_Bay_Salt_Ponds\Task05_Design\Stage_4\Task5c3_Pond_A16\2-Modeling\Modeling Report\Pond_A16_Modeling_Appendix_v4.doc

7. LIST OF ATTACHMENTS



Note: Pond outlines, cell outlines, weir widths, and weir locations not to scale

figure 1

South Bay Salt Ponds Restoration Project

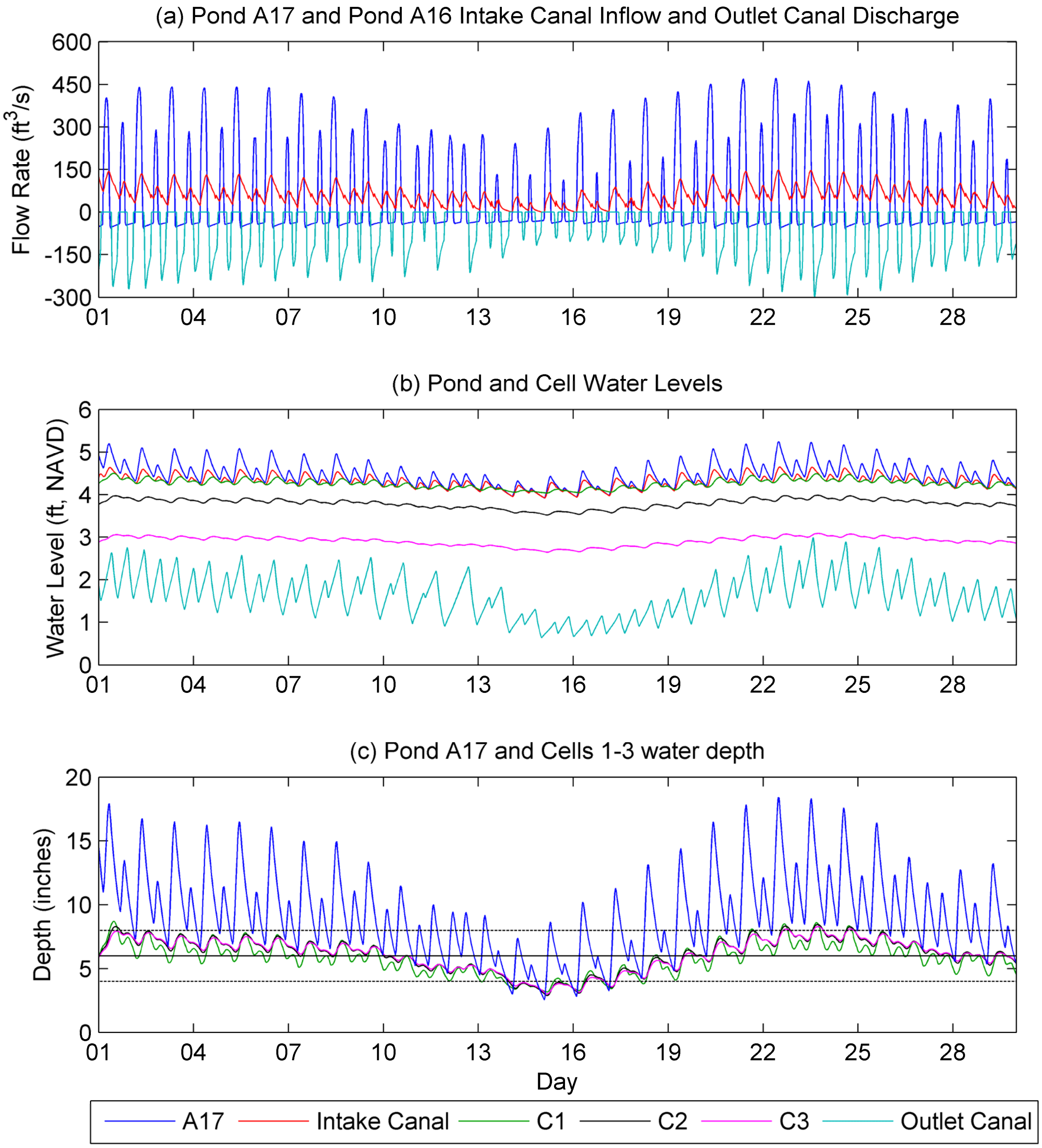
Pond A16 HEC-RAS Model Schematic

0 480 960 1,440 1,920 Feet

N

10 acres

PWA



Source: HEC-RAS modeling results for the configuration of water control structures during the summer season (Three flood-only and one two-way intake culverts at Pond A17. Three flood-only culverts at Pond A16).

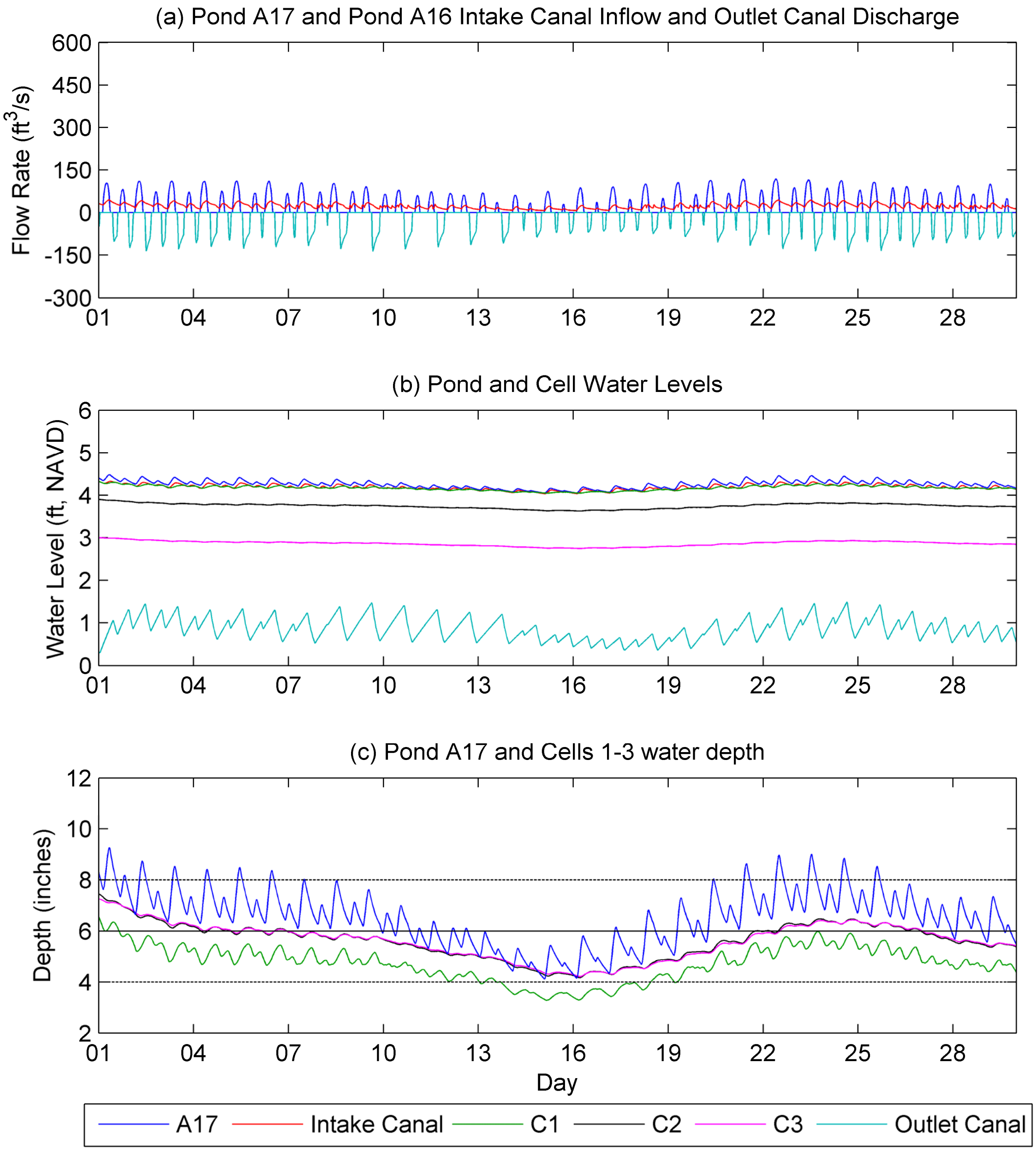
figure 2

SBSP Pond A16 Restoration

Modeled Flows and Water Levels – Summer Operations

PWA Ref# 1752-05

PWA



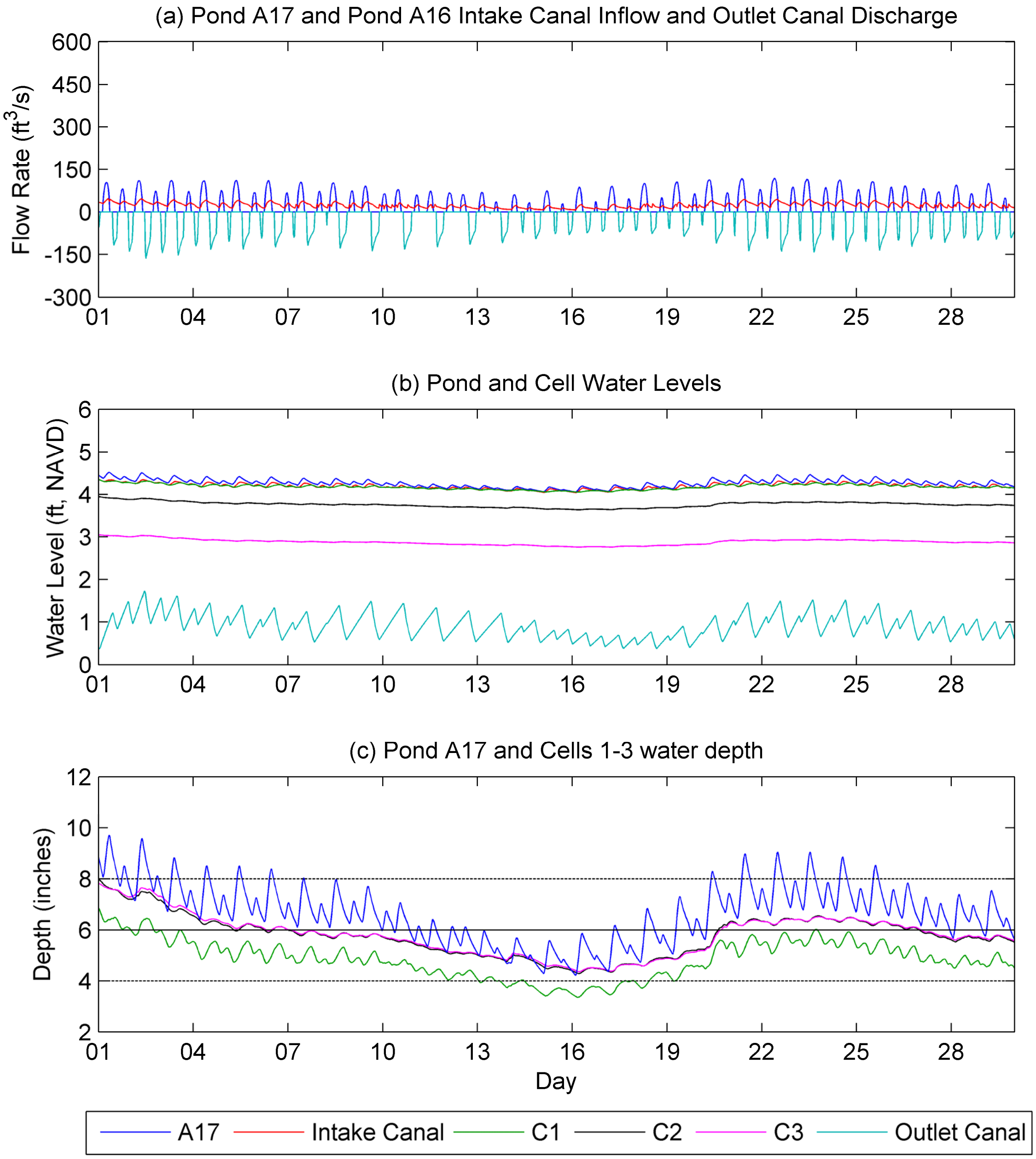
Source: HEC-RAS modeling results for the configuration of water control structures during the winter season (One flood only culvert at Pond A17. Two flood only culverts at Pond A16).

figure 3
SBSP Pond A16 Restoration

Modeled Flows and Water Levels – Winter Operations

PWA Ref# 1752-05

PWA



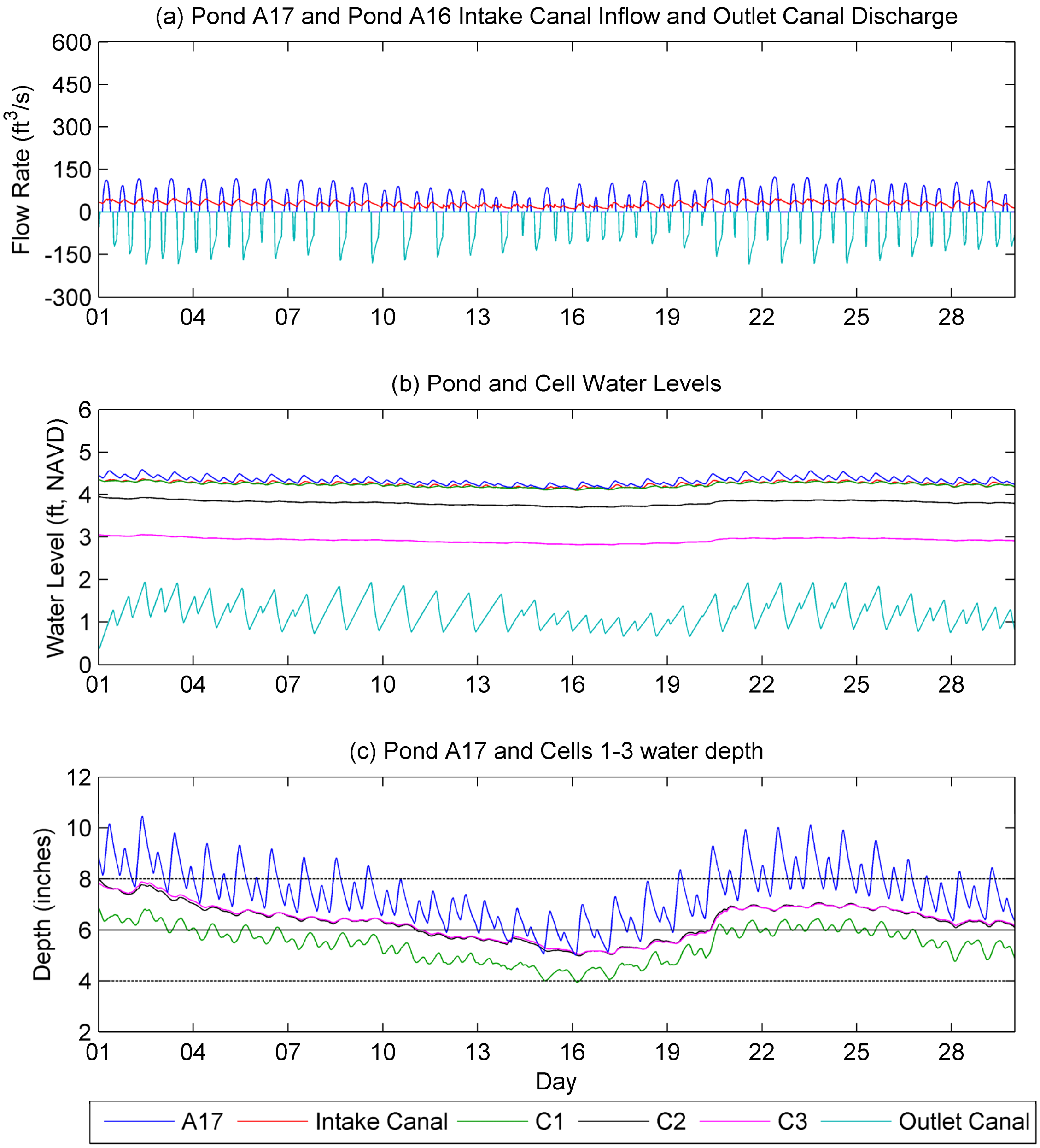
Source: HEC-RAS modeling results for the configuration of water control structures during the winter season with typical rain (One flood only culvert at Pond A17. Two flood only culverts at Pond A16).

figure 4
SBSP Pond A16 Restoration

Modeled Flows and Water Levels
Winter Operations with rain

PWA Ref# 1752-05

PWA



Source: HEC-RAS modeling results for the configuration of water control structures during the winter season with typical rain and 0.5 ft storm surge (One flood only culvert at Pond A17. Two flood only culverts at Pond A16).

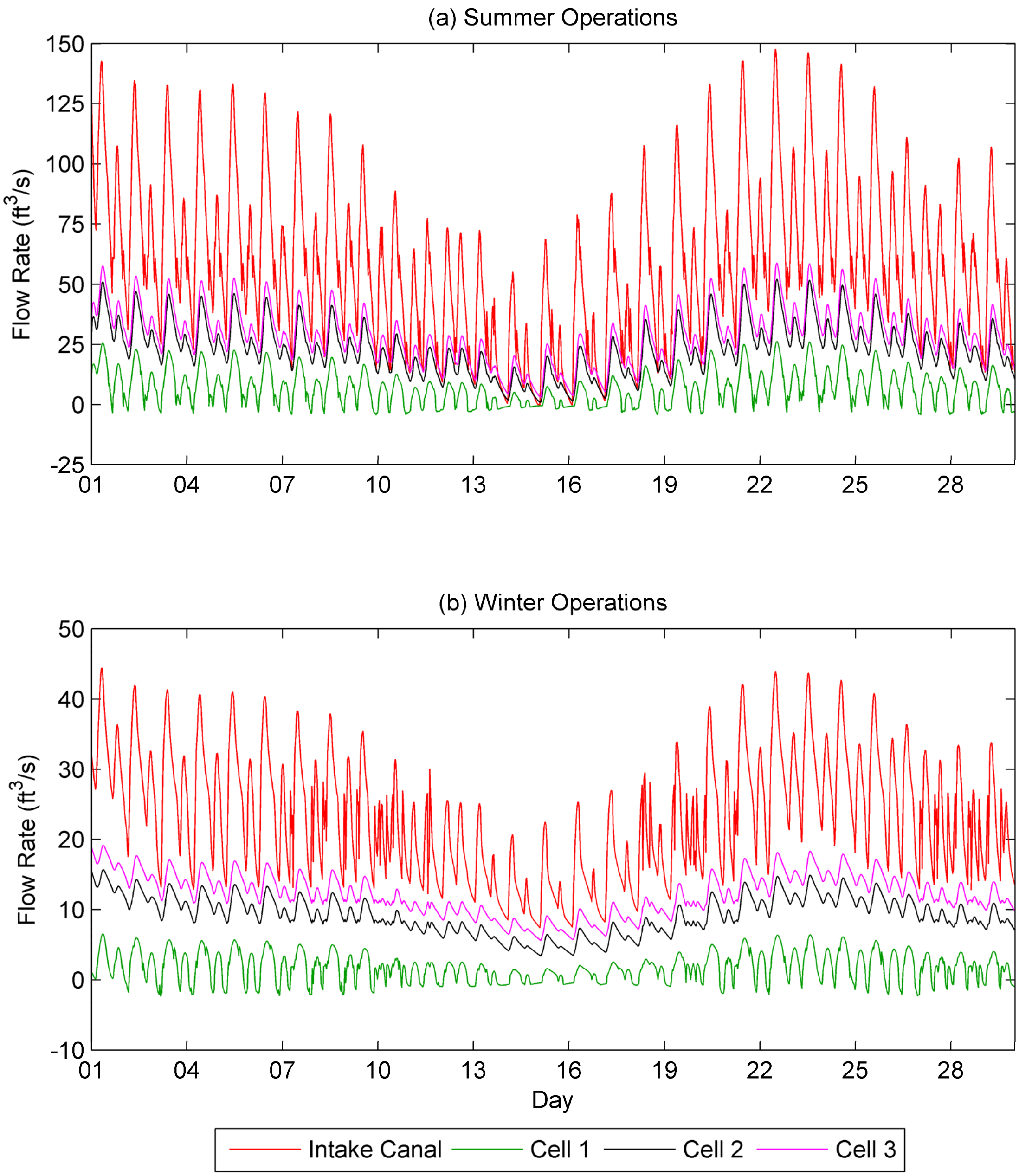
figure 5

SBSP Pond A16 Restoration

Modeled Flows and Water Levels
Winter Operations with rain and storm surge

PWA Ref# 1752-05

PWA

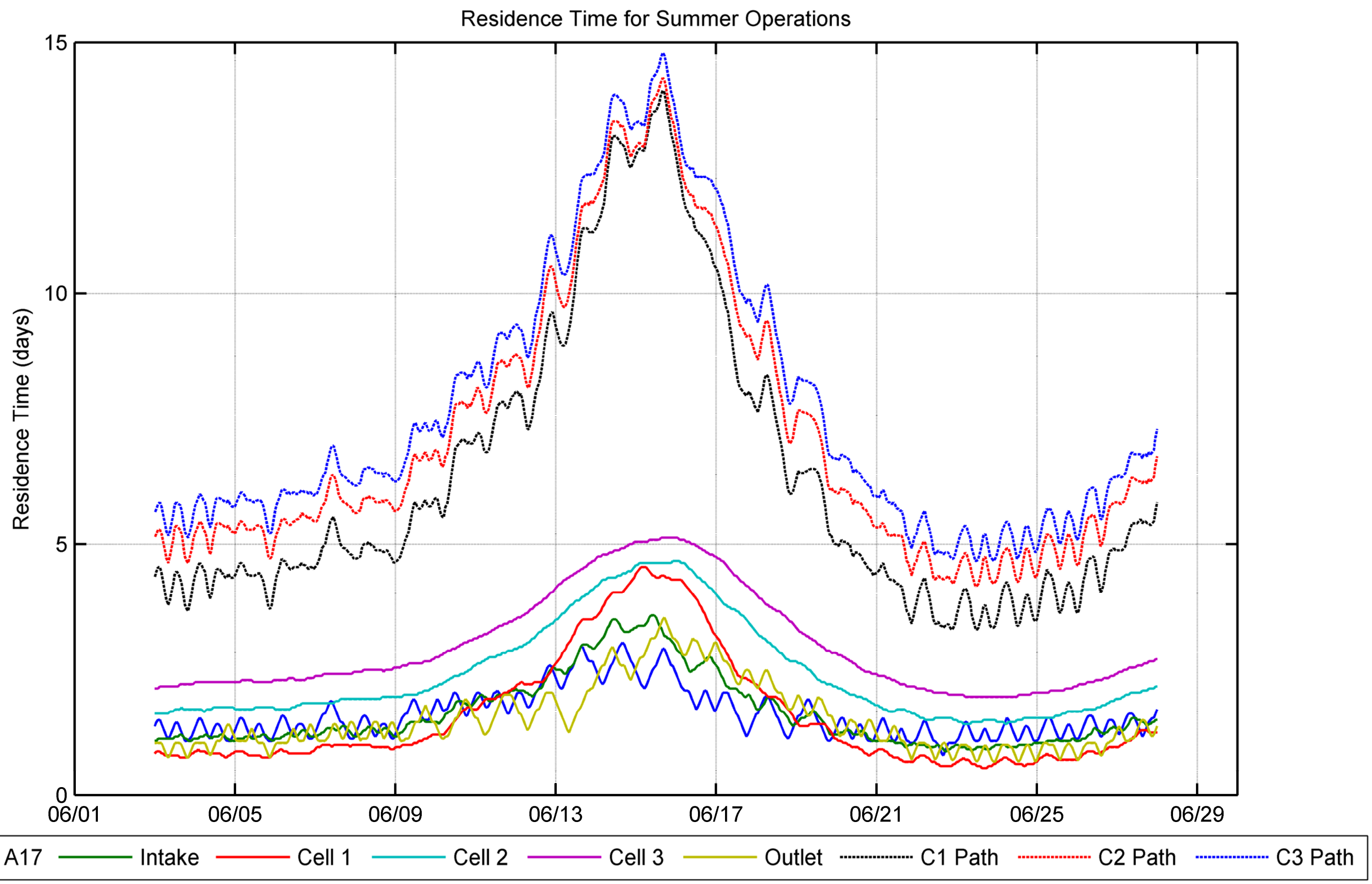


Source: HEC-RAS modeling results for the configuration of water control structures during the (a) summer and (b) winter seasons.

figure 6
SBSP Pond A16 Restoration

Modeled Weir Inflow – Summer and Winter Operations

PWA Ref# 1752-05



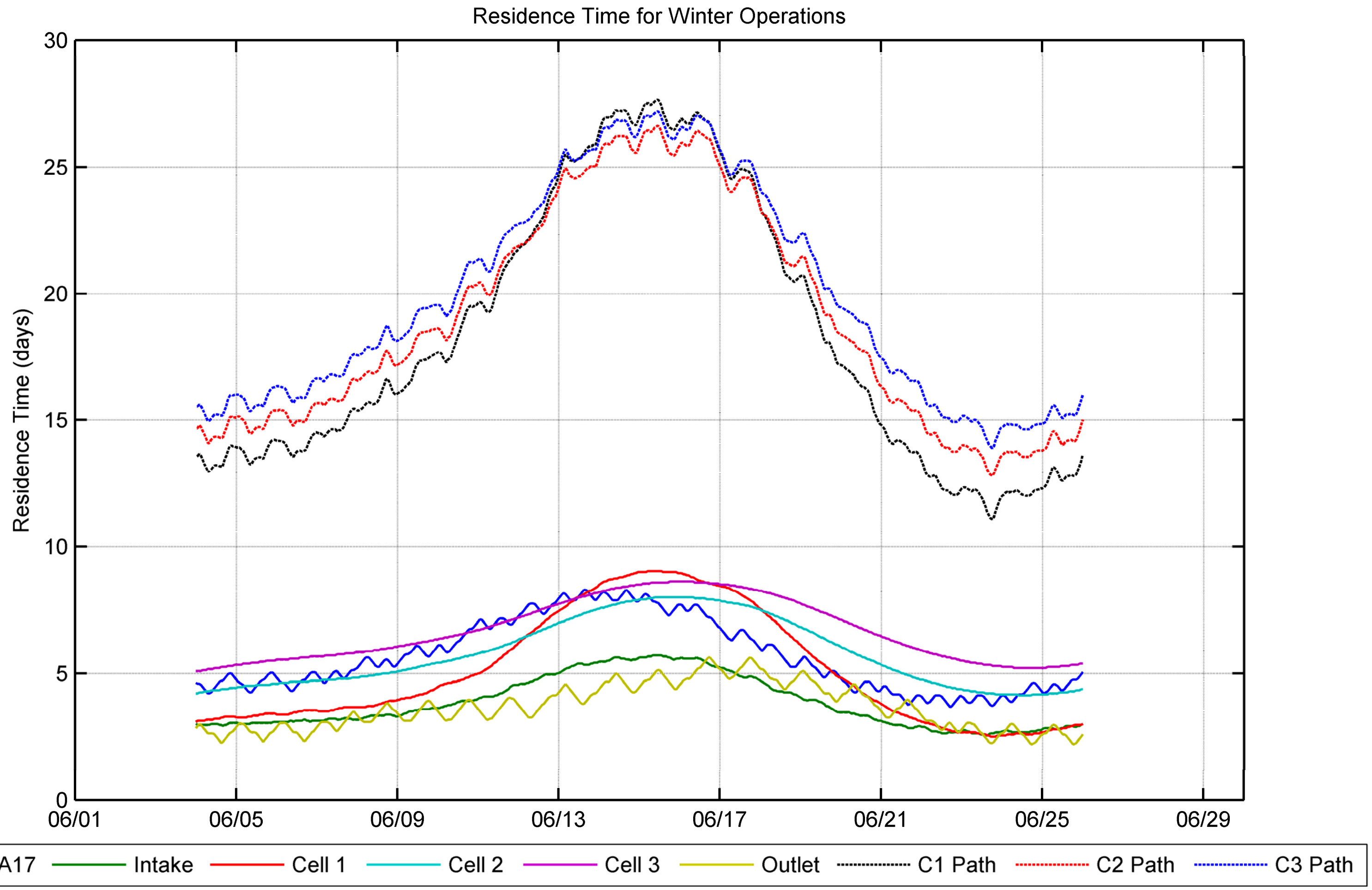
Source: HEC-RAS modeling results for the configuration of water control structures during the winter season (Three flood-only and one two-way intake culverts at Pond A17. Three flood-only culverts at Pond A16).

figure 7
SBSP Pond A16 Restoration

Pond and Cell Residence Times, Summer Operations

PWA Ref# 1752-05

PWA



Source: HEC-RAS modeling results for the configuration of water control structures during the winter season (One flood-only culvert at Pond A17. Two flood-only culverts at Pond A16).

figure 8

SBSP Pond A16 Restoration

Pond and Cell Residence Times, Winter Operations

PWA Ref# 1752-05

PWA

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Table 3. Estimated areas and volumes for the Alviso Pond A16 Phase 1 action reconfigured managed pond restoration

a) Metric units

	Pond/canal/ cell area	No. of nesting islands	Individual Island surface area	Total Island Surface Area	Open water area[1]	Typical elevation	Design water level[2]	Typical depth[3]	Max depth	Water volume between existing grade and design water level[4]	Net change in water volume due to island grading[5]	Design volume
	m^2	#	m^2	m^2	m^2	m NAVD	m NAVD	m	m	m^3	m^3	m^3
A17	530,000	0	0	0	530,000	1.13	1.36	0.23	5.0	221,400	-	221,400
Intake canal	104,700	0	0	0	104,700	-0.52	1.31	1.83	4.0	192,100	-	192,100
Cell 1	126,400	12	1,390	16,680	109,700	1.15	1.30	0.15	2.7	22,300	13,600	35,900
Cell 2	257,700	24	1,390	33,360	224,300	1.00	1.15	0.15	4.0	92,300	27,200	119,500
Cell 3	357,000	24	1,390	33,360	323,600	0.73	0.88	0.15	4.9	159,800	27,200	187,000
Outlet canal	167,800	0	0	0	167,800	-0.52	0.50	1.02	5.6	172,000	-	172,000
TOTAL	**1,543,600**	**60**		**83,400**	**1,460,100**					**859,900**	**68,000**	**927,900**

b) English units

	Pond/canal/ cell area	No. of nesting islands	Individual Island surface area	Total Island Surface Area	Open water area[1]	Typical elevation	Design water level[2]	Typical depth[3]	Max depth	Water volume between existing grade and design water level[4]	Net change in water volume due to island grading[5]	Design volume
	ft^2	#	ft^2	ft^2	ft^2	ft NAVD	ft NAVD	ft	ft	ft^3	ft^3	ft^3
A17	5,703,100	0	0	0	5,703,100	3.7	4.5	0.8	16	8,155,000	-	8,155,000
Intake canal	1,126,700	0	0	0	1,126,700	-1.7	4.3	6.0	13	6,778,100	-	6,778,100
Cell 1	1,359,900	12	15,000	180,000	1,179,900	3.8	4.3	0.5	9	788,300	480,000	1,268,300
Cell 2	2,772,800	24	15,000	360,000	2,412,800	3.3	3.8	0.5	13	3,257,800	960,000	4,217,800
Cell 3	3,841,500	24	15,000	360,000	3,481,500	2.4	2.9	0.5	16	5,640,300	960,000	6,600,300
Outlet canal	1,805,300	0	0	0	1,805,300	-1.7	1.6	3.3	18	5,944,700	-	5,944,700
TOTAL	**16,609,300**	**60**		**900,000**	**15,709,300**					**30,564,200**	**2,400,000**	**32,964,200**

[1] The cell bottom surface area is assumed to be the same as open water surface area, (i.e. steep berm side slope would not significantly change the areas)

[2] Water levels in Pond A17, intake canal, and outlet canal would vary due to muted tidal action; approximate average water levels are tabulated. See preliminary model results for water level data.

[3] Target shallow water habitat depths would range from 2 - 12 inches and average approximately 6 inches; the existing remnant historic tidal channels and borrow ditches would remain as deeper areas and would not provide shallow water habitat; the restoration design would maximize the shallow water area with a typical depth of 6 inches, however the average depth in the cells would be greater due to the deeper areas.

[4] Measured in GIS from DEM

[5] Islands will be graded by borrowing material from on-site. The "cut" and "fill" grading volumes will balance. The net change in water volume in a cell due to grading will be an increase due to the placement of borrow matieral above the water level to create the islands. This net increase is estimated from the construction/grading volumes calculated by PWA.

Appendix A Pond A17-A16 Salinity Assessment

Input Parameters			
S_0	25	ppt	Coyote Creek Initial Salinity
dh/dt	0.01667	ft/day	(Based on 2" evaporation per 10 days)

Pond/Cell	**Depth (m)**	**Depth (ft)**	**Res. Time (days)**	**dh (ft)**	**d_2 (ft)**	**S_{in}**	**S_{out}**
A17	0.37	1.21	3	-0.050	1.164	**25.0**	26.1
Intake	1.86	6.10	3	-0.050	6.052	26.1	26.3
C1	0.15	0.49	4	-0.067	0.425	26.3	30.4
C2	0.15	0.49	4	-0.067	0.425	26.3	30.4
C3	0.15	0.49	5	-0.083	0.409	26.3	31.7
Outlet	1.09	3.58	3	-0.050	3.526	31.7	**32.1**

Note: Outlet Canal inflow salinity set to C3 outflow salinity for conservative assumption of inflow salinity.

Conservation of mass: $S_{out} = d_1 * S_{in} / d_2$