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**NAPA-SONOMA MARSH RESTORATION
FEASIBILITY STUDY
NAPA AND SOLANO COUNTIES, CALIFORNIA**

**HYDRODYNAMIC MODELING ANALYSES OF
EXISTING CONDITIONS – PHASE 1
Contract No: DACW07-97-D-0002**

Prepared for

U.S. Army Corps of Engineers
San Francisco District Office

Prepared by

Philip Williams & Associates, Ltd.

with

DHI Water and Environment

March 13, 2002

PWA Ref. # 1174 – 8



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March 13, 2002

Craig Conner
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RE: **Napa-Sonoma Marsh Restoration Feasibility Study – Hydrodynamic Modeling Analyses of Existing Conditions – Phase I Report**
PWA Ref. # 1174-8

Dear Craig:

Enclosed is a CD which has PWA's final version of the Phase 1 report in Microsoft Word format and accompanying figures in pdf format. Also enclosed here are three bound color copies and one unbound color copy of the report.

Please contact me if you have further questions.

Sincerely,
PHILIP WILLIAMS & ASSOCIATES, LTD.

Chris Bowles, Ph.D.
Senior Associate

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1. INTRODUCTION

1.1 CONTEXT AND RATIONALE FOR THE STUDY

Cessation of salt production at the Napa Salt Ponds has resulted in both challenges and opportunities. The present condition is unsustainable due to salt accumulation within the ponds and eroding levees. However, restoration of natural habitats could alleviate these problems owing to inherent sustainability. A Feasibility Study for restoration is being undertaken through the US Army Corps of Engineers, San Francisco District (USACE) in conjunction with a partnership of State agencies: the California State Coastal Conservancy (CSCC) and the California Department of Fish and Game (CDFG) (who own the site). The site consisting of 6,370 acres of inactive salt ponds and nearly 3,500 acres of associated remnant sloughs and wetlands were purchased by the State of California from the former salt pond operators, Cargill Inc., in 1994. This report documents the initial phase of the larger Feasibility Study.

The ultimate objectives of the larger Feasibility Study are:

1. Evaluate a range of salinity reduction measures in the former salt ponds, so that the site can be restored to tidal action with minimal impacts on the surrounding aquatic environment.
2. Accomplish restoration of the former salt ponds to a “mosaic” of habitats present in the area historically.

A preferred management alternative will be identified for both objectives, and the second objective should be naturally self-sustaining to the extent feasible.

The existing physical conditions characterized in this Phase 1 of the larger study include parameters such as water surface elevation, and salinity transport and sediment transport, using a combination of one- and two- dimensional computational modeling. One-dimensional (1-D) computational modeling is used to describe the predominantly 1-D flow through the network of slough channels and rivers (the Napa River and Sonoma Creek), and two-dimensional (2-D) computational modeling is used to describe the predominately 2-D mixing processes in the former salt ponds. This study is closely connected to other recent projects upon which this study draws information:

- The investigation of the US Geological Survey (USGS) and University of California, Davis (UCD) to collect velocity, salinity, depth, suspended sediment concentration and temperature data at a series of seventeen monitoring locations in the slough channels and rivers across the site between September 1997 and March 1998 (Warner, 2000). These data were used to provide boundary data for the existing conditions model and for calibration and validation purposes of the 1-D computational model of the slough channels and rivers that extend across the site of the former salt ponds.
- The contract undertaken by Towill, Inc. of San Francisco to produce a Digital Terrain Model (DTM) of the slough channels, rivers, salt ponds and marsh plains of the site, using a combination of topographical and bathymetrical surveying, aerial photography and photogrammetry (Towill,

Inc., 2001). The DTM was used by PWA to construct the geometrical information required for the hydrodynamic model.

- The contract undertaken by Philip Williams & Associates (PWA) to physically measure the stage-discharge relationships for a selection of siphon water conveyance structures connecting the former salt ponds (PWA, 2001b). This information was used to characterize and identify the flow through the siphons to supplement theoretical discharge relationships that have been developed as part of the 2-D modeling of the former salt ponds.
- Napa-Sonoma Marsh Restoration Feasibility Study, Intermediate Scoping (Phase 1B) for Phase 2 (PWA, 2001a). This scoping report identified issues associated with the influence of the proposed restoration of Skaggs Island and Cullinan Ranch, technical issues identified by the Modeling Technical Advisory Group (MTAG) and the process of Restoration Alternative Selection.

1.2 LIMITATIONS OF THE STUDY

The purpose of this study (Phase 1 of the Feasibility Study) is to characterize the baseline or existing hydrodynamic conditions and construct a hydrodynamic model to simulate these conditions. In addition, geomorphic interpretation of the response of slough channels to the tidal restoration of the marsh system is investigated. No development of alternatives for restoration or modeling of these alternatives is undertaken in this phase. Succeeding phases of the Feasibility Study will investigate salinity reduction and habitat restoration alternatives. Similarly, no new field measurements were undertaken, although some key parameters have been identified and may be the subject of additional field studies in future phases.

1.3 ACKNOWLEDGEMENTS

The Project Study Team comprising the USACE, CCC and CDFG should be acknowledged as providing significant assistance throughout this study. Specifically, from the USACE, Mr. S. Nicholson, Mr. C Conner, Mr. K. Knutti, Ms. B. Dyer, Mr. B. DeJaeger, Ms. S. Von Rosenberg (GAIA Consulting, Inc.), from the CSCC, Ms. A Hutzler and Ms. N. Hitchcock, and from CDFG, Mr. L. Wyckoff, Mr. T. Huffman, Mr. J. Swanson and Mr. M. Rugg. In addition, the major role played by the MTAG should also be acknowledged, specifically, Dr. D. Schoelhammer, Professor. G. Schladow, Professor R. Sobey, Professor P. Goodwin and Mr. D. Trivedi. Finally, Dr. J. Warner and Mr. M. Barad should be acknowledged as providing assistance for this study.

2. CONCLUSIONS AND RECOMMENDATIONS

2.1 DATA COLLECTED TO CHARACTERIZE EXISTING CONDITIONS

- Limited field data and observations by agency scientists indicate that vertical salinity stratification is not important in the ponds except for short periods of time in the vicinity of the zone of initial mixing of inflow of lower salinity waters into the ponds. This implies that vertical mixing (due to the strong winds in the area) quickly break down any stratification. The 2-D modeling of the ponds is therefore adequate to answer most questions on the circulation patterns in the ponds.
- Wind and short period wave action drive vertical mixing, but the 2-D modeling appears to show less importance of the wind in driving horizontal mixing and determining horizontal salinity gradients. This could be due to the strong diurnal nature of the wind patterns and the fact that the duration of the wind is insufficient to allow large-scale circulation patterns to develop. Field measurements would be necessary to track the freshwater plume under a range of discharge and meteorological conditions to verify this finding.
- The hydrographic data, namely the water surface elevation time series, include errors that limit the accuracy of the calibration and validation exercise. The errors appear to be systematic vertical elevation offsets on the order of one foot for some locations.

Additional fieldwork is recommended to:

1. Investigate the spatial extent of stratification beyond release points in the ponds.
2. Investigate the role of winds and waves in breaking down this stratification.
3. Confirm circulation patterns in the ponds – particularly due the influence of winds.
4. Improve the hydrographic data sets used for calibration and validation by collecting additional field data and removing systematic elevation errors for the water surface time series. Also, the revised hydrographic data would then be used to refine the model calibration and validation.

Tidal boundary conditions have been shown to be very important, so the influence of boundary conditions and their location is the subject of the Napa-Sonoma Marsh Restoration Feasibility Study Phase 2 Stage 1 (PWA, in progress).

2.2 ADEQUACY OF SIMULATION MODEL

- The integration of the 1-D and 2-D models used in the analysis is shown to be a viable approach to simulating the complex flow field in large open ponds (2-D), small slough channels (1-D), and rivers (1-D and 2-D).
- The existing conditions model is adequate to serve as a baseline to describe the “no project” alternatives and compare to “with project” alternatives in subsequent phases of the study. Model

parameters derived from the existing conditions model are generally suitable for use in modeling of alternative future salinity reduction and tidal wetland restoration.

- The importance of integrating the computer models and detailed field data collection program has been demonstrated. The field data are essential to understand what processes need to be included in the computer models as well as providing calibration and validation data for the model. In turn, the model can be used to explain the field data, identify difficulties such as equipment malfunction, settling of equipment or benchmarks following installation, and simple survey errors. It should be stressed that the project area is an immensely challenging area for field data collection due to the harsh environment, poor ground conditions and large spatial scale. The model can be used to plan future field data collection programs and check that field observations are reasonable. Problems can be identified and checked in the field whilst the equipment is still deployed.
- The calibration and validation of the model also highlighted the difficulties of comparing measured velocities, salinity and suspended sediment concentration data at a point to simulated averaged data over a cross-section produced by the 1-D analyses. High salinities, fluctuating environmental conditions and wind could all contribute to inconsistencies in the field observations that limit the accuracy of the calibration.
- Single layer cohesive sediment transport (CST) modeling techniques have inherent inaccuracies. Recent studies on other restoration systems in the San Francisco Bay Region have identified that multi-layer CST is able to simulate these processes with far greater accuracy. To this end, only qualitative CST calibration, validation and sensitivity studies were undertaken for this study.
- The sensitivity analyses show that flow characteristics are very sensitive to the tidal elevations in San Pablo Bay and Carquinez Straits at low Napa River and Sonoma Creek flows. The circulation characteristics are less sensitive to inaccuracies in the discharges in Sonoma Creek and Napa River.

Further model development is recommended to:

1. Transfer momentum across the boundaries between the 1-D and 2-D models.
2. Extend the model boundaries into the San Pablo Bay to include the likely extents of the restored tidal prisms.
3. Improve modeling of density effects of elevated salinity.
4. Improve the sediment transport modeling, perhaps using a multilayer, cohesive sediment transport model.
5. Develop the model for 2-D or 3-D modeling of the Lower Napa River and Mare Island Strait – if the management strategies adopted warrant additional information of the mixing in this region.
6. Verify the horizontal salinity gradients in the ponds are as predicted in the model (for example, rapid vertical mixing, and the small effect of winds in the evolution of the plume).
7. Add water quality components to the model to allow the analysis of potential impacts of pond releases into receiving waters with reference to temperature, dissolved oxygen, turbidity and heavy metals.

8. Modify the existing conditions model further to provide a tool for adaptive management techniques that may be employed during the implementation of the future project.

These recommendations are described in part in an accompanying report, Napa-Sonoma Marsh Restoration Feasibility Study: Intermediate Scoping (Phase 1B) for Phase 2 (PWA, 2001a).

2.3 MODELING OF “WITHOUT PROJECT” CONDITIONS

- Without project conditions may result in a high probability of a levee failure releasing large volumes of saline water into the Lower Napa River and San Pablo Bay.
- If the existing pumps that move water from San Pablo Bay and the Napa River into the ponds became in-operational for mechanical or fiscal reasons, the pond elevation would drop and salinities would increase dramatically over current levels. Some ponds would dry out leaving salt panne exposed for some periods of the year. The difference between this artificial salt panne and naturally occurring salt panne, is that the concentrations could be far higher in the ponds.
- The benefits of pumping are minimal in the summer months due to lack of precipitation, in which evaporation greatly exceeds the ability to pump water into the system. Pumping also adds salt water to a continuing salt sink.
- Similarly, failure of the siphon structures connecting the ponds could eliminate exchange of flows between ponds and resulting in drying of some ponds. This is particularly prevalent after June when the benefit of pumping can be severely restricted due to the inability to force flows through the siphon structures against a large salinity gradient.
- Continuing the existing water management regime will result in gradually deteriorating conditions in terms of salinity concentration and levee integrity.

2.4 GEOMORPHIC ANALYSIS

- Hydraulic geometry relationships have been developed and updated for the northern San Francisco Bay area and were used to predict potential changes in the Napa slough channels in response to the tidal restoration of the marsh system.
- The existing slough channels are undersized compared to potential tidal flows for restored conditions.
- The magnitude of the increase in the potential restored tidal prism and the predicted response in channel geometry suggests that the evolving morphology of the interior slough channel network will be highly dynamic, especially within the first decade following restoration.
- The potential exists for a combination of significant slough channel erosion, possible levee erosion, and tidal damping within the marsh system.

Future investigation and geomorphic analyses are recommended to:

1. Define restoration alternatives for analyses.
2. Define tidal wetland components of the restoration design.

3. Reduce uncertainties and develop further methodologies to estimate habitat evolution of restoration alternatives.
4. Evaluate potential impacts and mitigation approaches for the restoration alternatives.

3. SETTING

3.1 HISTORICAL PERSPECTIVE

The Napa-Sonoma Marsh historically encompassed nearly 40,000 acres (Goals Project, 1999) in the Napa and Sonoma watersheds. The original marshes in the study area were first diked for hay production and cattle grazing in the 1850s. In the 1950s, much of the land was converted to salt production by the solar evaporation of San Pablo Bay water. Today, approximately 36% of this acreage remains classified as wetland habitat, while 25% consists of inactive solar salt production ponds, 12% residential and 20% cropland and pasture. The salt production ponds, cropland and pasture are diked to prevent tidal and fluvial inundation under normal conditions. A majority of the remaining wetland areas are public lands and are under the management of the CDFG. Figure 3-1 shows the salt ponds of the North San Pablo Bay in relation to the extents of tidal marsh in San Francisco Bay, past and present.

3.1.1 Past Management of the Salt Ponds

The salt ponds were managed by the Leslie Salt Company until the late 1970s when the system was acquired by Cargill, Inc. Over the past 50 years the largely agricultural use of the project site was converted to solar salt production ponds. Salt production operations ceased in the late 1980s. The ponds were sold to the State in 1994. Earlier levees eliminated tidal action which resulted in subsidence due to dewatering of the soil and lack of fresh sediment deposits and to a lesser extent compression due to grazing and oxidation of the marsh surface. This subsidence has resulted in the pond elevations being lower than adjacent marsh elevations and is one of the reasons that allowed the ponds to be operated primarily by gravity. The pond system comprises 8 ponds (Figure 3-2). Ponds 1, 2, 6 and 7 are divided into two cells by narrow levees. Historically these ponds were managed by pumping water from two main sources:

- Pumped flows at 30,000 gallons/minute (1.9 m³/s) from Pond 1 in the south of the site via a siphon under South Slough into Pond 2. Water flowed into Pond 1 via a tidal culvert to San Pablo Bay.
- Pumped flows at 6,000 gallons/minute (0.4 m³/s) from the Napa River into Pond 8 or via a canal bypassing Pond 8 into Ponds 7, 7A or via canal to Pond 6A.

Several structures are used to convey water throughout the site and these with various other terms are defined as:

- *Bittern*: The bitter water solution of bromides; magnesium and calcium salts remaining after sodium chloride is crystallized out of seawater.
- *Brine*: Water saturated with or containing large amounts of a salt, especially sodium chloride. According to the U.S. Geological Survey (USGS) classification, water classified as brine contains more than 35ppt (parts per thousand) total dissolved solids (TDS) of salt.

- *Canal*: An artificial channel that can be used to convey saline water around the periphery of the ponds.
- *Donut*: A small circular water distribution pond (approximately 20m (65 feet) in diameter) at the outlet of the siphon structures. Water can be diverted into channels or directly into the salt ponds by a system of gates (Figure 3-3).
- *Gates*: A vertical gate (or metal sheet) that can be lowered or raised by a screw mechanism. In the lowered position, the gate will eliminate any flow on the downstream side (except for minor leakage around the gate seals), whereas in the raised position flow will occur unhindered. An intermediate setting of the gate can be used to regulate flows in one direction.
- *Saline Water*: Water containing dissolved solids; generally referring to solid contents in excess of 1ppt TDS. The USGS classifies the degree of salinity of water as follows:
 - Slightly saline = 1–3 ppt
 - Moderately saline = 3–10 ppt
 - Very saline = 10–35 ppt
 - Brine = more than 35 ppt
- *Siphon*: A steel conveyance structure or culvert that conveys saltwater underneath the slough channels by gravity. The overall dimensions are typically 4 to 6 feet diameter. The flow rate through the siphon depends primarily on the difference in water surface elevation and density at the inlet and outlet of the siphon.
- *Slough*: Natural tidal channels that are now artifacts of the once extensive tidal marsh system. Due to the loss of inter-tidal marsh, these main slough channels are smaller in cross sectional area than historic conditions, but their plan-form has shown little change since the 1850s (Figure 3-4).

Figure 3-4 shows a color photographic mosaic of the salt pond system with a digitized overlay of the historical slough channel mapping from the T-sheets of 1860. Figure 3-5 shows an aerial photograph of the site viewed from over San Pablo Bay looking north towards Pond 1, 1A, 2 and Cullinan Ranch. Also shown in this figure is a land based photograph of the pond system. Figure 3-3 shows two land-based photographs of Pond 7 and a donut structure (a water conveyance structure used to distribute water between adjacent ponds). Notice the deep red color of the water in Pond 7 due to the bittern content of the pond.

3.1.2 The Restoration Initiative

The recent public acquisition of the former salt pond system provides an opportunity to restore tidal salt marsh and related habitats on an unprecedented scale within the San Francisco Bay system. The Napa Marsh occupies a key position on the Pacific Flyway, a major migratory route used annually by waterfowl and other birds. The Napa Marsh also provides habitat for a number of threatened or endangered species, some of which are endemic to the region. The restoration of 6,370-acres of inactive salt ponds to productive wetland habitat and the resulting restoration of nearly 3,500-acres of associated remnant sloughs and wetlands, will be a project of national significance.

The USACE completed the Napa River Salt Marsh Restoration Reconnaissance Study in August 1997. The Reconnaissance Study concluded that there is potential Federal interest in the restoration of salt marsh and ecologically related habitat within the former salt pond complex. The study has now progressed into the Feasibility Phase. The Feasibility Study will involve an in-depth examination of alternatives for salinity reduction in the ponds in addition to the restoration of wetlands and associated habitats within the former salt pond complex on the west side of the lower Napa River. Figure 3-2 shows the project study site. Detailed evaluation of alternatives will be undertaken to identify a plan that will meet the salinity reduction and ecosystem restoration goals of the California State Coastal Conservancy and the CDFG, and the planning objectives of the San Francisco Bay Area Wetlands Ecosystem Goals Project (Goals Project, 1999).

Present salinities in the ponds vary greatly from around 25ppt up to 400ppt dependent upon the position of the pond in the system and the season of the year. Figure 3-6 shows the variation of the salinities in the ponds over recent years and gives a comparison of these salinities to San Pablo Bay and the Napa River. In addition certain ponds are bitter in nature, specifically Pond 7. It is uncertain what other chemical constituents are in the pond water at the time of this report, but a water quality study is scheduled for the future. However, the magnitude of the salinity problem is apparent. The ecology of the region has adapted and evolved at salinities in the range of 5 to 25ppt with perturbations to zero caused by flood flows in the Napa River and Sonoma Creek and higher salinities in drought conditions. Salinities of 400ppt could have a density impact on the local ecosystem if uncontrolled releases, via an accidental breach, were to occur along some of the pond levees. The condition of the levees varies constantly with some locations more prone to failure than others.

3.1.3 Current Management of the Salt Ponds

Presently the CDFG attempt to manage salinities in the ponds by pumping lower salinity water from San Pablo Bay in the south of the site and from the Napa River north of the site. The pumped water is moved through the system of salt ponds via siphons, canals, gates and flap valves using successive water surface elevation differentials between ponds generated through pumping operations. However, this management system is extremely problematic due to the ageing infrastructure (most of the siphons are over 50-years old), and by the fact that although the management strategy attempts to maintain lower salinities by counteracting evaporation through pumping, dissolved salts in suspension are continuously being added to the system. As there is currently no outlet to the system, the ponds function as a sink with all the salt fed into the system ultimately remaining in the system.

Historic reclamation of the area of the salt ponds for agricultural purposes resulted in significant subsidence of the area. Figure 3-6 shows the present average elevations in the salt ponds. Present elevations of the salt ponds indicate that in almost every instance the elevations are significantly lower than inter-tidal marsh elevations. Substantial additional sediment will be required to restore the area to inter-tidal salt marsh. Pond 2A, however, is evolving successfully toward tidal marsh in a matter of approximately 4 to 5 years, due to the fact that Pond 2A was at a higher elevation than the other ponds since it was reclaimed more recently than the other ponds. (The integration of Pond 2A habitats into the final mosaic of habitats throughout the entire complex will be discussed in the Phase 2 Feasibility

Report). In addition, the subsided condition results in potential tidal prisms significantly larger than a vegetated tidal marsh of similar area. The mobilization of the potential tidal prism is likely to cause hydraulic scour of existing channels. Wind wave activity could also be greater than a natural marsh owing to large expanses of open water, which could inhibit sediment deposition.

4. GOALS AND OBJECTIVES

This study characterizes the “existing” hydrodynamic conditions at the former salt ponds, slough channels, rivers and marsh plains that represent the Napa-Sonoma Salt Marsh. This is a complicated system due to the complex geometry of channels and shifts in the tidal forcing at the boundaries to the east (Napa River and Mare Island Strait) and west of the site (Sonoma Creek and San Pablo Bay). Specifically the objectives are:

- To assimilate all readily available data required for computational modeling, review, process and check for adequacy and reliability.
- To develop a hydrodynamic and advection-dispersion 1-D computational model capable of simulating the conveyance, sediment and salinity transport in the slough channels, and rivers of the Napa-Sonoma Salt Marsh. The model will be calibrated and validated using readily available field observations.
- To produce a hydrodynamic and advection-dispersion 2-D computational model capable of simulating the conveyance and salinity transport and mixing processes in the former salt ponds and associated marsh plains of the Napa-Sonoma Salt Marsh. The model will use available field data for calibration and validation.
- To produce a qualitative, single layer, cohesive sediment transport model suitable to approximate the physical processes of transport within the system.
- To dynamically link the 1-D model of the sloughs and rivers to the 2-D model of the former salt ponds where appropriate (under existing conditions for Pond 2A only).
- To undertake sensitivity analyses with the computational model where appropriate to ascertain the suitability of the data provided for modeling purposes or of the modeling assumptions used.
- To produce animations and results capable of demonstrating the hydrodynamic operation and the salinity transport of the former salt ponds and associated slough channels and rivers under the present, “without project”, existing conditions.
- To understand the relative significance of various mixing processes to allow the Modeling Technical advisory Group (MTAG) and the Project Team to decide the appropriate level of modeling (1-D, 2-D or 3-D and spatial extents) of future phases of the Feasibility Study.
- To use hydraulic geometry relationships to undertake a preliminary estimate of potential changes in the Napa slough channels in response to the tidal restoration of the marsh system.
- To provide a basis for the Phase 2 scope of work, which is focused on evaluation of alternatives to support the environmental review and engineering feasibility analyses.
- To describe existing conditions and without project conditions for use in the environmental review process.

5. DATA COLLECTION AND PROCESSING

5.1 DATA COLLECTION, PROCESSING, REVIEW AND QUALITY CONTROL

PWA has been collecting data relevant to the Feasibility Study since September 1998. Processing, review and quality control procedures have been undertaken on the data collected, described in the following sections.

5.1.1 Digital Terrain Model of Rivers, Slough Channels and Ponds

Towill, Inc. of San Francisco, California, were contracted by the USACE to undertake topographic and bathymetric surveying of the slough channels, salt ponds and the associated marsh plains at the site. The surveying was undertaken using a variety of techniques including aerial photogrammetry and land based surveying which are described in further detail in the project report by Towill, Inc. (Towill, Inc., 2001).

It is important to emphasize that a high accuracy of vertical and horizontal control was a key component of the survey task. This was especially important for a large site where settlement has occurred and poor benchmarks existed. Towill Inc.'s scope included a very accurate primary control level loop through the site which was connected to high confidence benchmarks outside the site. Eight existing Federal benchmarks were identified by Towill, Inc. and a control network comprising eighteen new benchmarks was established to encircle the site. Datum conversions were calculated and all the surveying projections are in North American Datum (NAD) '83 (CCS 83, 1991.35 Epoch, Zone 2) and North American Vertical Datum (NAVD) '88. Metric units were used for surveying. In addition, benchmarks were established to tie in existing and future work to these new survey datums. Benchmarks were also established at ten of the seventeen UC Davis/USGS monitoring platforms (see Figure 5-1) that the Towill survey crews could locate, as detailed by Table 5-1. All aerial photography was conducted at a scale of 1:24,000.

Table 5-1. Benchmarks Established at UC Davis/USGS Monitoring Sites

Point No.	Northing (m)	Easting (m)	Elevation (m)	UC Davis Descriptor
175	559,854.699	1967,168.543	3.253	HUDE
2932	560,393.690	1963,790.653	3.830	TNS
3306	557,943.102	1962,903.640	2.558	SOCR
3581	554,360.744	1964,374.771	3.132	PABLO-14
3582	554,362.730	1964,349.933	3.244	PABLO-13
4281	554,130.257	1973,315.733	2.797	CHINA
4246	552,731.142	1973,082.781	2.485	PIPE
4290	551,323.685	1973,473.128	2.987	DUTCH
9998	549,387.269	1975,925.467	3.956	MIC
9999	Leveled only		2.653	M14

PWA and the Project Team worked closely with Towill, Inc. at all stages of the survey, from initial identification of the locations of cross-sections in the slough channels and rivers, marsh plain transects and pond transects to be surveyed by Towill, Inc. which were subsequently used in conjunction with photogrammetric techniques to produce the final digital terrain model (DTM) of the site. Cross-sections in the slough channels were spaced a nominal distance of 300m apart. Twelve cross-sections were measured in the Napa River and between three and six transects were measured in each of the ponds dependent upon the size of the individual pond. Cross-sections in Sonoma Creek were not measured upstream of the monitoring station SOCR (UC Davis/USGS) in this Phase of the Project. Regular quality control meetings were undertaken between PWA and Towill, Inc. over a period that extended from October 1999 to April 2001. Figure 5-2 shows a diagram that represents the cross-sections, marsh and pond transects that were collected at the site.

Delivery of a preliminary DTM to PWA by Towill, Inc. occurred in October 2000 and the DTM was used in the hydrodynamic modeling described herein. The finalized DTM was completed in March 2001. However, discrepancies were observed in the elevations of Pond 2A between the DTM of Pond 2A and published monitoring transect elevations (MEC, 1999). It appears that these discrepancies are due to the inherent inaccuracies present in photogrammetric techniques that were used for developing the DTM of Pond 2A. Ground survey techniques were not used for Pond 2A. (These discrepancies are being investigated at the time of the production of this report and recommendations for rectification will be made.)

5.1.2 U.C. Davis / USGS Monitoring Data

A monitoring program was established at the Napa-Sonoma Marsh which was supported by the USGS Federal/State Cooperative and Placed-Based Programs, the CDFG, the CSCC and the US Fish and Wildlife Service's (USFWS) Coastal Program. The monitoring program involved an intensive data collection effort in the tidal slough network and the adjacent riverine channels (Napa River and Sonoma Creek) to determine the physical processes controlling circulation patterns of water and suspended sediment. Velocity, water level, conductivity, temperature and suspended sediment concentration were measured at 17 sites (Figure 5-1) from September 1997 to March 1998. The data analysis showed that the hydrodynamics of the system is controlled by the development of two types of convergence zones, one driven by barotropic pressure gradients (asymmetric tidal signals on the east and west sides of the site) and one created from converging baroclinic pressure gradients (salinity minima and maxima in Mare Island Strait). Further details of this research can be found in the doctoral thesis of J.C. Warner (Warner, 2000).

The monitoring data, collected by Warner and the UC Davis/USGS team, were delivered to PWA in its final form in August 2000, for use in calibration and validation of the slough and river channel 1-D model. The data were subsequently processed to convert water depth in the slough channels and rivers into water surface elevation at the ten sites that were incorporated into the Towill, Inc. survey. Further processing was required to convert the data into a form suitable for use by the 1-D hydrodynamic model. Quality control checks were made prior to using the data for calibration purposes. Several anomalies were identified that were clarified or rectified by Dr. Warner. Figure 5-3 shows a sample of the type of data

that was collected. Not all parameters were available for the complete time period at each monitoring station. Table 5-2 summarizes the data that were available at each location.

Table 5-2. Summary of UC Davis Data – September 1997 to March 1998

Name*	Temp	Salinity			Susp. Sed. Conc.			Temp			Water Surface Elevation	Velocity
		Ave.	Bot.	Top	Ave	Bot.	Top	Ave.	Bot.	Top		
Buchli	X	X			X			X				X
Can	X	X			X			X				X
Car	X		X	X		X	X		X	X	X	X
China	X	X			X			X			X	X
Devil	X	X						X				X
Dutch	X	X			X			X			X	X
Hude	X	X						X			X	X
M14	X	X			X			X				X
M5	X		X	X		X	X		X	X	X	X
Mic	X		X	X		X	X		X	X	X	X
Napa	X	X						X				X
Pablo	X	X			X			X			X	X
Pipe	X	X			X			X			X	X
Sns	X	X			X			X				X
Socr	X	X			X			X			X	X
Ss	X	X						X				X
Tns	X	X			X			X				X

* Location of monitoring stations is shown by Figure 5-1.

The data covers a period of low flows in the Napa River and Sonoma Creek from September to December 1997 followed by a large El Nino flow event from January to March 1998. The breaches of Pond 2A, at China Slough and South Slough, had both occurred by September 1997.

5.1.3 Sonoma Creek Geometry

During the Phase 1 modeling, the survey for the Napa-Sonoma Marsh extended to the UC Davis/USGS monitoring station SOCR only. Numerical experimentation and evaluation by the MTAG resulted in the hydrodynamic model extending to approximately the tidal limit of Sonoma Creek, just upstream of the State Highway 12 bridge in Schellville. A review of available existing geometry identified only sparse cross sectional data for Sonoma Creek. The cross sectional data were obtained from Southern Sonoma County Resource Conservation District (SSCRCD) (personal communication with Paul Schaeffer of SSCRCD). The cross-sections were surveyed in 1998 and covered a reach of Sonoma Creek that extended approximately from the State Highway 12 bridge to 1.5 km downstream of the bridge. However, no information is available for a distance of approximately 6 km between the most downstream cross-section of SSCRCD and monitoring station SOCR. The cross sectional geometry of this section of the reach was

estimated using interpolated cross-sections in the absence of measured data. The significance of this assumption is discussed in the PWA report Intermediate Scoping Phase 1B (PWA, 2001a). The summary of this investigation indicated that while the interpolated sections had a negligible effect upon the hydraulics of the sloughs immediately surrounding the salt ponds, future studies incorporating Skaggs Island would be affected by these cross-sections. Thus, for the purposes of the Phase 1 study, interpolated cross-sections were considered to be acceptable but it was recommended that additional measured cross-sections be obtained for the Phase 2 study.

5.1.4 Sonoma Creek Hydrology

The hydrodynamic model extends to approximately the tidal limit at Schellville and estimates of the discharge hydrology are required for a boundary condition at this point (see also Figure 6-1). PWA reviewed the available sources of data but surprisingly little is known of the hydrology for the Sonoma Creek watershed.

Sonoma Creek is an un-gauged drainage basin that drains the entire Sonoma Valley between the Napa and Petaluma River watersheds. The USGS monitored the flow in Sonoma Creek daily at Agua Caliente from 1975 to 1981 (USGS, 2001). The Agua Caliente station was located at Agua Caliente Road and drains approximately 58.4 square miles (or less than 60%) of the watershed. Figure 5-4 displays the daily discharge recorded by the USGS from 1975 to 1981. An estimate of the flow discharge in Sonoma Creek from September 1997 through March 1998 was needed for calibration of the model. To estimate a hydrograph in Sonoma Creek for an un-gauged period, a statistical correlation was initially developed between Sonoma Creek and a hydrologically similar watershed nearby for the period of record. The Napa River was used for correlation purposes and is located immediately to the east of Sonoma Creek. The stream gauging station at Oak Knoll Avenue, near Napa, drains 218 square miles from the Napa River drainage basin and has a record of daily flow from 1975 to 1981 (USGS, 2001) and a record of hourly flow from September 1997 through March 1998 (CDEC, 2001). Figure 5-5 displays the daily discharge recorded by the USGS from 1975 to 1981.

The approximation of this simplistic correlation is recognized, but comprehensive hydrologic analyses were considered unnecessary until the sensitivity of the model predictions at the points of interest to the boundary conditions was established. In addition, the SSCRCD plan to establish a stream gauge at this location (A. Hutzler, Personal Communication, Nov. 2001). To simplify the analysis, the flow in the Napa River was assumed to share a linear relationship with the flow in Sonoma Creek. Figure 5-6 shows the estimated hydrograph for Sonoma Creek at Agua Caliente and the recorded discharge at Napa River at Oak Knoll Avenue for the period of September 1997 through March 1998, the calibration period for the hydrodynamic model. The upstream boundary of the hydraulic model is at the State Highway 12/121 Bridge, near Schellville, draining 90.88 square miles of the Sonoma Creek basin. Weighting the flow by the drainage area generates a hydrograph representative of the flow in Sonoma Creek at the downstream point. Figure 5-7 shows the resulting discharge estimation in Sonoma Creek at the State Highway 12/121 Bridge.

During the period in question, an approximate 10-year storm event was recorded in the Napa River at Oak Knoll Avenue (10:00 AM, February 3, 1998). This significant event proved to be useful to verify the hydrologic prediction techniques used here. The FEMA Flood Insurance Study for Sonoma County Unincorporated areas (FEMA, 1997) and Napa County Unincorporated Areas (FEMA, 1990) reports peak discharges for Sonoma Creek and Napa River at stations along the rivers. The peak event in the Napa River (10:00 AM, February 3, 1998) is 14% greater than the reported 10-year flood event. The correlated peak event at the same time on Sonoma Creek at State Highway 12/121 is 12% greater than the FEMA predicted 10-year storm event, suggesting that the hydrologic predictive methods adopted here were valid. Table 5-3 delineates the results and although this table indicates the validity for peak flows, details on the differential timing of the peaks, or the shape of the hydrograph are not available.

Table 5-3. Peak Discharges at Sonoma Creek and Napa River

Location	Peak Discharge (cfs)	
	February 3, 1998	FEMA FIS, 10-yr Event
Napa River, near Napa	21,039 (recorded)	18,400
Sonoma Creek, Hwy 12/121	11,846 (estimated)	10,590

Due to the magnitude of the peak discharge, the synthetic hydrograph generated from linear regression and weighted by drainage area was assumed to be valid for the hydraulic model analyses for the period September 1997 through March 1998. For modeling of Future Project Alternatives it will be beneficial to use further hydrologic estimation techniques (rainfall run-off modeling) to estimate the hydrology of the catchment for Sonoma Creek.

5.1.5 Napa River Geometry

It was necessary to extend the 1-D model of the slough channels and rivers to the tidal limits, which included the Napa River to the City of Napa. Cross section geometry was available from previous studies and hydrodynamic models (PWA, 1996, 1997) of the Napa River for the Napa River Flood Protection Project (NRFPP) (USACE, 1998). The cross sectional information, obtained from the USACE, Sacramento District, extended from the Oak Knoll gauging station (north of the City of Napa) to Mare Island Strait. The cross sections were based on surveys conducted by the USACE and do not represent the post project conditions of the NRFPP.

5.1.6 Napa River Hydrology

Napa River flow gauging for the calibration period of September 1997 to March 1998 was obtained from CDEC for the Oak Knoll gauging station. (CDEC, 2001)

5.1.7 Huichica Creek Hydrology

Huichica Creek flows into the north of the salt pond complex and the peak flows are negligible in comparison to the Napa River, Sonoma Creek or total flows in the system. The discharge in Huichica Creek was set to zero in this study.

5.1.8 Pond Salinity Data

Few pond salinity data are available. CDFG regularly record sample salinities at various locations around the pond complex. However, these data were collected for the purposes of tracking gross changes in salinity and were taken at selected locations. The data records are not continuous time series data and are not accurate enough for calibration or validation purposes of the 2-D advection-dispersion model of the salt ponds. The USGS have undertaken average salinity monitoring in a selection of the ponds since February 1999 in addition to measuring temperature, turbidity, pH and dissolved oxygen. These data are contained in a USGS report (Takekawa *et al.*, 2001). General salinity ranges in the ponds recently are shown by Figure 3-6.

5.1.9 Evaporation, Precipitation and Wind Data

Evaporation, precipitation and wind data is particularly important data for incorporation into the 2-D hydraulic model of the salt pond system, since these parameters have a critical effect on salinity levels and mixing processes in the ponds.

5.1.9.1 *Evaporation*

Two main sources of evaporation data were identified: Cargill, Inc. (Cargill), the previous owners of the salt pond complex, and California Irrigation Management Information System (CIMIS). Although observed pan evaporation was available from personnel at Cargill, computed evaporation at a shorter time scale (i.e., hours or days rather than months) are required for modeling purposes. A data set of computed evaporation was developed using standard formulations for evaporation rate, and observations of solar radiation, wind speed, air temperature and relative humidity from a nearby CIMIS station at Carneros (adjacent to CDFG Napa field offices, Duhig Road) north of the site. These measurements were compared with historic records of observed pan evaporation from Cargill in order to determine if computed evaporation rates appropriately characterized evaporation at Napa Salt Ponds.

- Observations of Pan Evaporation from Cargill Records
Monthly evaporation data in hard copy format were obtained for the period of January 1955 to present from a Cargill owned/operated weather station at Napa salt ponds (Cargill, 2001). At this location, fresh water evaporation was measured directly with a Weather Bureau Class A evaporation pan. These data were reported in inches of evaporation, and were not adjusted with a pan coefficient. Class A pans, because of their relatively small size do not have the same evaporation rate as larger bodies of water such as the Salt Ponds. Pan evaporation is typically

adjusted to compensate for differences in evaporation rate between the pan and the water surface for which evaporation is to be estimated using a pan coefficient. Cargill evaporation measurements were adjusted by PWA with a pan coefficient as well as a salinity coefficient in order to better approximate evaporation from saline ponds.

Cargill operates all of its weather stations according to the same standards as the National Weather Service (NWS) cooperative station. However, air temperature is only measured at the cooperative station. Mean annual evaporation data from Cargill's Napa station were compared with data from a nearby evaporation station at Dutton's Landing (State of California, DWR, 1979). Mean annual evaporation reported at Cargill was within 0.1% of Dutton's Landing for the period of 1956-1979. Thus, the Cargill data appears to be accurate, and a good basis of comparison for hourly (CIMIS) evaporation data to be used in modeling the Napa salt ponds.

- **Computed Evaporation Rates**

Hourly weather data (electronic format) were obtained for the period of April 20, 1993-June 20, 2001 from nearby DWR-CIMIS station #109 at Carneros (CIMIS, 2001). Hourly evaporation was computed using standard methods (Chow et al., 1988) from measurements of net radiation, air temperature, wind speed, and relative humidity. This computed evaporation is summarized in Figure 5-8 showing the evaporation daily totals from 1993 to 2001. It should be noted that these calculations should be adjusted for varying levels of salinity in the Napa salt ponds. Results shown in Figure 5-8 are computed for zero salinity. The density of water is a function of both temperature and salinity. An increase in salinity produces a corresponding decrease in evaporation rate. Napa Salt Pond salinities range from approximately 30 to 400 parts per thousand (ppt). Such high salinities can affect evaporation rates. 'Salinity coefficients,' which can be applied to fresh water evaporation to compute evaporation of saline water, were derived for salinity levels of 0-790ppt using the relationship given by Chapra, 1997. The resulting salinity coefficients were tabulated relative to salinity level (ppt), so that evaporation at varying levels of salinity can be easily computed from fresh water evaporation data (Appendix 1).

Monthly freshwater evaporation averages were calculated during the period May 1993 to March 2001 from CIMIS total monthly evaporation data. These averages in combination with salinity coefficients provide an estimation of total pond evaporation for a given month for future forecasting/modeling efforts as shown by Figure 5-9.

- **Comparison of Measured to Calculated Evaporation**

In order to compare Cargill and CIMIS monthly freshwater evaporation data, Cargill data must be adjusted with appropriate pan coefficients. Determining pan coefficients is an inexact science, however, the overall range is between 0.6, and 0.9 in the state of California (DWR, 1979). In Figure 5-10, comparison of computed and pan evaporation, monthly averages for 1993-2001, and raw Cargill data are plotted against computed evaporation. It is clear from this figure that computed values are approximately 30% lower than pan evaporation recorded at Cargill. This implies that if a pan coefficient of approximately 0.7 were applied to Cargill observed

evaporation at Cargill, there would be an acceptable correlation between computed and pan evaporation.

5.1.9.2 *Precipitation*

Similarly, two main sources of precipitation data were identified: Cargill and CIMIS.

- **Cargill**
Monthly total precipitation was obtained from the Cargill Napa weather station (Cargill, 2001) for the period of January 1955 to present. Monthly precipitation is recorded in inches and measured with a 24" non-recording cylinder gauge.
- **CIMIS**
Hourly precipitation data was obtained from CIMIS station #109 in Carneros (CIMIS, 2001); for the period April 20, 1993-present. Available CIMIS precipitation data are summarized in Figure 5-11 shows precipitation daily totals from 1993 to 2001.
- **Comparison of Precipitation Sources**
Monthly total mean precipitation for the period of May 1993-May 2001 for both Carneros and CIMIS stations is plotted in Figure 5-12. There is a 8.87% difference in mean annual precipitation between the stations for this period. The difference between the Cargill observations and the CIMIS computed monthly total evaporation is plotted in Figure 5-13. Differences in monthly precipitation values for the two stations are most likely due to the nature of the precipitation gauges (NWS/DWR side-by-side hourly gauges frequently report different values), but over time, the differences in gauges decreases significantly. The CIMIS is probably the more accurate gauge, since this gauge is a tipping bucket, which is usually more reliable than a non-automatic recording cylinder gauge, and hence statistical data generated from the CIMIS raw data were used for modeling purposes.

5.1.9.3 *Wind*

Two main sources of wind data were identified from Cargill and CIMIS records.

- **Cargill**
Wind speed is measured at the Cargill weather station in miles/24 hours. Cargill uses an electronic totalizing anemometer manufactured by NovaLynx Corporation.
- **CIMIS**
Hourly wind sensors operated at CIMIS station #109 in Carneros are: wind speed, resultant wind speed, wind direction, standard deviation of wind direction. Wind rose vector data are available only at a daily time step. Available CIMIS wind speed data are summarized in Figure 5-14 showing wind speed daily totals during 1993-2001, and a strong seasonal pattern. In addition, it is known that during much of the year a strong diurnal trend exists with strong winds often observed

in the afternoons (Figure 5-15). This strong wind is an important factor in vertical mixing (Section 5.3). Figure 5-16 shows the monthly average wind speeds from 1993 to 2001. Monthly average wind speeds are greatest between April and July. Figures 5-17 to 5-19 show wind roses for 1998 to 2000 respectively. These figures show that lower wind speeds (0.5-1.0 m/s) most frequently blow from the north in the mornings. However the higher wind speeds (1.0-3.0 m/s) most frequently blow from the west in the afternoons.

- Comparison of Wind Sources

The Cargill wind data was not available at the time of analysis and a comparison could not be undertaken.

5.1.10 Siphon Data Collection

PWA was contracted by the USACE in May 2000 to undertake fieldwork at the Napa-Sonoma Salt Marsh complex, to gain some insight to the operation and hydraulic efficiency of the siphons. The purpose of the fieldwork was to collect flow data through a sample of the siphons at the site. PWA also measured salinity and water surface elevation data in the salt ponds adjacent to the siphons. These data were collected to enable actual stage-discharge relationships to be developed that would supplement theoretical stage-discharge relationships for the flow through the siphons connecting the salt ponds. The siphons were constructed in the mid-1950s; and it is likely that the physical conditions of the siphons are corroded and filled with sediment in addition to subsidence that may have occurred over time. Therefore, it is unlikely that theoretically derived stage-discharge relationships will be valid. The original approach devised in collaboration between PWA and the USACE was designed to provide data that would characterize flow through the siphons sufficiently for incorporation into the numerical modeling effort.

The primary purpose of this study was to address concerns regarding the ability of the model to accurately simulate the transfer of water between ponds. Specifically, the goal was to generate accurate differential head (or difference in water surface elevation at either end of the siphons), salinity and discharge data. The results of this study were unanticipated, but reinforced the critical nature of the hydraulic characteristics of the siphons. The study indicated:

1. The potential existence of wedges of high salinity water in the siphon that must be removed prior to a free flowing siphon flow—or governed by the differential water surface elevations and the density differences in the ponds at the inlet and outlet of the siphon.
2. The density difference between the ponds can be extreme and even a water surface elevation of two feet or more may be insufficient to initiate flow between ponds. Due to the large surface area of the ponds, increasing the water surface elevations can be a costly and slow process if the pumps must be used to increase the water surface elevation difference.

In fact, under some density conditions during some seasons it may be impossible to get any flow through the siphons without major pumping to increase the differential head. Failure to recognize this phenomenon could have resulted in serious management problems during the implementation phase of the project. A relationship has been defined to predict the hydraulic and salinity conditions that will result

in no flow in the siphons, even if there is a significant difference in water surface elevations. This relationship can also be used to calculate the head difference required for flow through the siphons and therefore could provide a useful management tool. Stage-discharge relations for the monitored siphons that did experience some flows during the study period have been developed. Appendix 2 shows a summary of the findings of the study and the full details are discussed further in an earlier report (PWA, 2001b).

5.2 IDENTIFICATION OF GAPS IN DATA OR INADEQUACIES OF AVAILABLE DATA

The following inadequacies in the data available were identified:

- Sonoma Creek Geometry.
- Sonoma Creek Hydrology.
- Boundary at San Pablo Bay.
- Boundary at Third Napa Slough (TNS).
- Water Quality, Salinity and Sediment Data in Salt Ponds.
- Field data to verify model results of initial mixing zone in the ponds, effects of wind driven circulation in the ponds and extent and patterns of initial mixing in the receiving waters (Lower Napa River or Tidal Slough Channels).

These are addressed further in the accompanying report to this report, Napa-Sonoma Marsh Restoration Feasibility Study, Interim Scoping Phase 1B (PWA, 2001a).

5.3 ANALYSIS OF THE ROLE OF STRATIFICATION

5.3.1 Stratification in Rivers and Sloughs

The use of a depth-averaged model such as MIKE 21 implicitly assumes that the water column is not sufficiently density-stratified to significantly inhibit vertical mixing or drive gravitational circulation. The widespread successful use of depth-averaged models for simulation of estuary hydrodynamics suggests that this assumption is often valid in shallow systems. However, if possible this assumption should be verified a priori. The extensive instrumentation deployed in the marsh complex allows for such an analysis.

Estuaries will typically develop a vertical gradient in salinity, and thus density, as inflowing fresh water remains near the surface. Density increasing with depth is typically termed stable stratification. Stable density stratification will decrease vertical turbulent mixing due to a buoyancy force which opposes mixing of lighter fluid downward into the heavier fluid. Strong density gradients can drive gravitational circulation and have a significant effect on vertical mixing, such that depth averaged models should not be used under these conditions (Fisher, et al., 1979).

A typical measure of the strength of density stratification is the Richardson number, which represents the ratio of the buoyancy effects to the amount of turbulence. If the density gradient is strong and there is

little turbulence, the Richardson number will be high and significant reductions in vertical mixing may be realized. On the other hand, high turbulent energy production can overcome a strong density gradient such that there is little effect on turbulent mixing. The primary source of turbulence in the sloughs and rivers is shear production, which can be characterized by the vertical velocity gradient. Thus two commonly used forms of the Richardson number for estuaries are the gradient Richardson number:

$$Ri_g = \frac{-g \, d\rho/dz}{\rho(du/dz)^2}$$

and the estuarine Richardson number:

$$Ri_e = \frac{\Delta\rho \, gh}{\rho \, U_t^2}$$

where ρ is density, u is streamwise velocity, h is depth, and U_t is root mean square tidal velocity. The gradient Richardson number is a local parameter (in the vertical), while the estuarine Richardson number is a bulk measure of stratification.

Calculation of the Richardson numbers requires velocity and density data in the vertical, which has been collected by the USGS at many locations in the slough network. Analysis of this data has indicated that salinity gradients, and thus stratification effects, are small in the sloughs (Warner et al., 1999). In the lower Napa River, however, significant vertical density gradients exist. Gravitational circulation and baroclinic convergence zones have been observed (Warner, 2000). Rather than initially resorting to 3-D modeling of this region, however, the following strategy will be employed. The importance of the stratification to the management of the ponds and salinity reduction alternatives depends on the selected location of discharge points and the magnitude and concentration of the releases. If alternatives are selected which require a detailed understanding of the salinity structure in the Lower Napa River then this will be investigated as a potential problem area. If it is determined that indeed stratification effects are strong and must be accounted for, potential strategies will be investigated, and modifications to the model will be made as needed. More detailed representation of the Lower Napa River is being pursued in Phase 2 (PWA, 2002).

5.3.2 Stratification in Ponds - Pond Monitoring

The salt ponds present a rather unique physical situation, particularly in their current state of isolation from tidal mixing. The choice of the depth-averaged model was made based primarily on the shallow depths of the ponds in general, and the experience of CDFG, USGS and UC Davis staff that have worked extensively in the ponds. However, no systematic monitoring of the salinity structure of the ponds has been undertaken. The objective of this section is a discussion of the physics of density stratification in the ponds, preliminary data analysis, and suggestions for an adaptive approach to the modeling.

The salt ponds in their existing condition present a combination of lake and estuary processes, since there are salinity gradients and mixing processes but no periodic tidal forcing. Both types of water bodies may

develop vertical density gradients for several reasons. Temperature differences may cause vertical density gradients. During daytime heating, the surface water will be warmer, and thus lighter, than the deeper water, resulting in stable density stratification. During the night, the surface water will cool, and potentially sink into deeper water, resulting in vertical mixing. This type of mixing typically results in a well-mixed epilimnion in lakes and estuaries. Thermal stratification can also be induced due to differences in the receiving waters and inflows. It is expected that the shallow depths in the ponds will result in a relatively well-mixed water column in terms of temperature, due to the rigorous mixing of surface waves. In estuaries, surface salinity is typically less than bottom salinity, as described in the previous section, resulting in stable density stratification (density increasing with depth).

It is difficult to assess the strength of density stratification in the ponds without comprehensive data and/or modeling analysis. Under the existing condition, the primary mixing mechanism in the ponds is wind. Wind can induce waves, which generate circulation patterns over the depth and in extreme conditions these waves can break producing turbulence. Wind induced surface shear causes down wind surface currents that can result in a wind set-up and seiche in shallow waters and vertical circulation especially in deeper waters. Upon restoration of tidal action to the ponds, additional turbulence will be generated by the tidal flows. Assessment of the strength of stratification in the ponds requires either: 1) data on the vertical structure of density and turbulent energy in the ponds, or 2) a model of this vertical structure. Both of these are beyond the scope of this preliminary analysis, however, some data are available on density gradients in several ponds which provide some insight. Unfortunately no data is available on the vertical or spatial flow structure in the ponds, which precludes calculation of Richardson numbers and a definitive assessment of the effects of stratification.

The USGS has collected water quality data at various locations in several ponds between February 1999 and July 2001 (Takekawa, *et al.*, 2000). At several locations in Ponds 1, 2, 2A, 3, 4 and 7 salinity and temperature measurements were made. In some instances the measurements were made at two points in the water column, near the bottom and near the surface. These measurements were analyzed ascertain if any trend were observed. Initially, the first 6 months of data were analyzed intensively at every measurement period and location as shown by Table 5-4. Subsequently analyses were undertaken only where measurements were made near the bed and near the surface over the next 6 months of data as shown by Table 5-5. Finally measured data were analyzed where measurements were made near the bed and near the surface at intervals of 3 months as shown by Table 5-6.

Table 5-4. Summary of Pond Stratification Measurements February 8 to August 11, 1999

Pond	Date	Time	Temp. Range (°C)	Sal. Range (ppt)	Temp. Diff. Surface/Bottom (°C)	Sal. Diff. Surface/Bottom (ppt)	Wind / Waves?
1	2/8/99	11.00–12.00	0.8	0.9	N/A	N/A	Negligible
2	2/8/99	9.30–10.00	0.1	0.5	0	0.1	Negligible
3	2/11/99	12.00- 13.30	5	5.5	N/A	N/A	0.5' waves
4	2/11/99	9.20-13.10	2.6	38.9	N/A	N/A	1.0' waves
7	2/8/99	13.20-15.20	1.4	82	N/A	N/A	Negligible

Pond	Date	Time	Temp. Range (°C)	Sal. Range (ppt)	Temp. Diff. Surface/Bottom (°C)	Sal. Diff. Surface/Bottom (ppt)	Wind / Waves?
1	3/1/99	12.55-13.12	1.1	2.7	N/A	N/A	Negligible
2	3/1/99	10.35-11.30	1.2	1.19	1.0	0.4	Negligible
3	3/2/99	11.16-12.46	3.8	1.4	N/A	N/A	Negligible
4	3/2/99	10.30-13.16	1.3	0.7	N/A	N/A	Negligible
7	3/2/99	14.37-16.02	7	1	N/A	N/A	Negligible
1	4/12/99	14.45-13.05	1.4	0.7	N/A	N/A	1-1.5' waves
2	4/14/99	13.24-14.45	1.6	0.6	N/A	N/A	Negligible
3	4/14/99	10.21-12.44	9.1	0.5	N/A	N/A	N/A
4	4/14/99	9.12-14.49	3.5	2.0	N/A	N/A	N/A
7	4/12/99	10.14-12.24	4.0	2.0	N/A	N/A	N/A
1	5/13/99	15.16-16.00	2.8	0.1	N/A	N/A	20-25mph wind
2	5/13/99	13.37-14.23	2.6	0.8	0.4	0.2	Negligible
3	5/12/99	9.31-11.55	2.5	2.0	N/A	N/A	1-1.5' waves
4	5/12/99	8.50-12.31	3.1	2.0	N/A	N/A	5 mph wind
7	5/13/99	8.43-10.43	2.5	2.0	N/A	N/A	N/A
1	6/10/99	11.13-12.18	4.0	1.1	N/A	N/A	N/A
2	6/10/99	10.20-10.47	1.5	0.1	0.4	0.04	N/A
3	6/11/99	10.18-12.16	4.2	8.1	N/A	N/A	N/A
4	6/11/99	9.25-12.52	3.0	3.0	N/A	N/A	Windy
7	6/10/99	14.14-14.40	4.3	48.0	N/A	N/A	Windy
1	7/19/99	11.57-13.00	1.9	2.1	N/A	N/A	Windy
2	7/19/99	10.30-11.30	0.6	2.2	0.1	0.8	0.5-1.0' waves
3	7/21/99	8.45-10.10	1.6	9.1	N/A	N/A	Calm
4	7/20/99	8.05-13.20	5.0	3.0	N/A	N/A	Windy
7	7/19/99	14.38-15.35	2.9	22.0	N/A	N/A	V. Windy
1	8/11/99	10.20-11.28	1.5	3.9	N/A	N/A	Breeze
2	8/11/99	9.10-9.47	0.1	1.0	0.1	0.3	Breeze
3	8/12/99	9.50-11.54	4.2	6.1	N/A	N/A	Calm
4	8/12/99	12.39-14.02	5.5	3.0	N/A	N/A	Breeze
7	8/11/99	13.05-14.53	3.4	0	N/A	N/A	Windy

Table 5-5. Summary of Pond Stratification Measurements September 8, 1999 to March 31, 2000

Pond	Date	Time	Temp. Range (°C)	Sal. Range (ppt)	Max. Temp. Diff. Surf./Bott. (°C)	Max. Sal. Diff. Surface/Bottom (ppt)	Wind / Waves?
1	9/8/99	10.10-11.29	0.4	4.1	0	0.2	Breeze
2	9/8/99	8.55-9.34	1.2	0.1	0.1	0.1	N/A
1	10/14/99	9.44-11.00	3.6	3.1	0.2	0.8	Calm
2	10/14/99	8.20-9.05	1.7	0.1	0.3	0.1	Calm
2	11/12/99	9.36-10.29	1.3	0.2	0.7	0.2	Calm
1	12/30/99	10.30-11.52	3.3	4.2	0.1	0.1	Calm
2	12/30/99	9.00-9.38	1.1	0.1	0.1	0.1	N/A
1	1/26/00	10.25-11.48	1.7	4.5	0.1	1.9	Calm at Inlet
2	1/26/00	9.24-10.00	0.7	0.4	0.1	0.2	N/A
3	1/27/00	10.02-14.01	5.2	4.8	2.2	4.2	Calm
7	1/26/00	13.14-14.54	4.1	52	0.4	N/A	Breeze
1	2/28/00	14.02-14.54	2.0	2.4	0.1	0.9	Breeze
2	2/28/00	13.12-13.38	1.0	0.7	0.3	0.1	Breeze
3	2/29/00	13.08-14.50	1.9	0.2	0.8	0.2	N/A
1	3/30/00	14.35-15.16	4.8	1.0	0.1	0.1	Breeze
2	3/30/00	13.20-13.54	1.6	0.6	0.7	0.2	Breeze
3	3/31/00	9.22-11.00	0.6	0.3	0	0.3	0.5' wave

Table 5-6. Summary of Pond Stratification Measurements June 16, 2000 to March 27, 2001

Pond	Date	Time	Temp. Range (°C)	Sal. Range (ppt)	Max. Temp. Diff. Surf./Bott. (°C)	Max. Sal. Diff. Surface/Bottom (ppt)	Wind / Waves?
1	6/16/00	10.50-11.42	4.9	1.4	0.9	0.5	Windy
2	6/16/00	9.30-10.06	2.5	0.7	0.4	0.6	Breeze
1	9/29/00	14.48-16.14	6.2	9.3	0	0.2	Windy
1	12/28/00	13.04-14.10	3.3	1.5	1.9	0.6	Calm
2	12/28/00	11.50-12.24	1.4	1.7	0.7	0.2	Calm
2	3/28/01	12.18-12.48	1.1	0.4	0.1	0.2	Breeze
3	3/27/01	10.08-13.42	2.9	0.5	2.0	0	Windy

The tables show the date and time at which the measurements were taken in addition to the pond location. This may be an important factor in the analyses since wind velocity is usually observed to be greatest in the afternoons and, hence, more wind induced vertical mixing may be observed in the afternoon periods. The columns showing the total temperature and salinity ranges represent the horizontal spatial variation of these parameters. The temperature and salinity difference between the bottom and the surface of the pond

at specific measurement positions is also shown by the tables and a N/A entry indicates that only a surface measurement was taken. Finally, an observation was made at the time of the measurements to indicate the strength of the wind or an approximate wind induced wave height.

It can be observed from the data presented here that although there appears to be a horizontal spatial variation in the ponds measured of up to 6.2°C for temperature and up to 52ppt for salinity, vertical variations in both temperature and salinity are generally less than 1°C and 1 ppt, respectively. In certain instances the salinity and temperature difference in the vertical exceeded this threshold but this occurred near to inlets to the ponds.

It is likely that stratification may occur infrequently and for a short duration of time away from the inlet locations under some conditions. Stratification in the ponds may be investigated further if it is considered necessary in the future but based on the available data at this time, it is implied that stratification is negligible, except close to the inlet locations (the zone of initial dilution).

5.4 TESTING OF ALGORITHMS FOR CULVERTS AND OTHER HYDRAULIC STRUCTURES AT THE SITE

The siphons in the model are represented by circular culverts having the same length and diameter as the physical siphons. The siphons are assumed to be flowing full through the entire length of the siphon and the total loss through such a siphon consists of a friction loss (depends on the length, diameter and the resistance number of the culvert) and contraction and expansion losses, all of which are simulated by the culvert description in MIKE 11. As long as critical flow is not encountered in the siphon the culvert description is an accurate description of the theoretical hydraulics of the system. However, the actual roughness and culvert geometries are not known but are expected to vary depending on the degree of deterioration. Also, accumulation of high density saline and particulate plugs are not modeled but known to occur.

6. MODEL DEVELOPMENT, CALIBRATION AND VALIDATION

6.1 RIVER, CREEK AND SLOUGH CHANNEL MODEL

It was considered that, for the purposes of this study, the flow in the sloughs, creeks and rivers could be considered predominantly 1-D as described Section 5.3. Recent research by Warner (Warner, 2000) has shown that the flow in the Lower Napa River and Mare Island Strait exhibits complex 3-D behavior with the formation of a baroclinic convergence zone. However, the significance of the circulation pattern depends on the management alternative selected and until further guidance is obtained on the regulatory requirements that will be imposed by authorities such as the Regional Water Quality Control Board (RWQCB) for discharge purposes, it is considered that the flow can be assumed to be 1-D in this area for future salinity reduction alternative screening purposes. This approach will be reviewed in Phase 2 of the Feasibility Study.

6.1.1 1-D Numerical Model Description, MIKE 11

6.1.1.1 *Hydrodynamic (HD) Module*

MIKE 11 is a software package for the simulation of flows, water quality and sediment transport in estuaries, rivers, irrigation systems, channels and other water bodies (www.dhisoftware.com). It is a dynamic, 1-D modeling tool for the detailed design, management and operation of both simple and complex river and channel systems. Since MIKE 11 is a 1-D model, it is capable of simulating the flow in a network of branches in which the flow direction mainly follows the direction of the branches and stratification is negligible.

The hydrodynamic module of MIKE 11 HD solves the vertically integrated equations of conservation of continuity and momentum (the ‘Saint Venant’ equations (Chanson, 1999)), based on the following assumptions:

- the water is incompressible and homogeneous;
- the bottom-slope is small;
- the wave lengths are large compared to the water depth. This ensures that vertical accelerations can be neglected and a hydrostatic pressure gradient may be assumed; and
- the flow is sub-critical (super-critical flow is modeled, but more restrictive conditions must be applied).

MIKE 11 solves the continuity and momentum equations in the following form:

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = q$$

$$\frac{\partial Q}{\partial t} + \frac{\partial \left(\alpha \frac{Q^2}{A} \right)}{\partial x} + gA \frac{\partial h}{\partial x} + \frac{gQ|Q|}{C^2 AR} = 0$$

Where,

t	time
x	distance along branch
Q	discharge
A	flow area
q	lateral inflow
h	water surface elevation above datum
C	Chezy resistance coefficient (approximately related to Manning's "n" via $C = R^{1/6} / n$, where R is the hydraulic radius)
R	hydraulic or resistance radius
α	momentum distribution coefficient

The solution of the equations of continuity and momentum is based on an 6-point implicit finite difference scheme developed by Abbott and Ionescu (Chanson, 1999). The scheme is structured in order to be independent of the wave description specified (e.g. kinematic, diffusive or dynamic).

A computational grid of alternating Q (discharge) and h (water level) points is used as illustrated in Figure 6-1. The computational grid is automatically generated on the basis of the user requirements. Q-points are placed midway between neighboring h-points and at structures, while h-points are located at cross sections, or at equidistant intervals in between if the distance between cross sections is greater than a maximum spacing required for numerical stability. Further details of the numerical scheme may be found in the MIKE 11 Reference Manual (DHI, 2001).

6.1.1.2 Advection Dispersion Module

The advection-dispersion (AD) module is based on the one-dimensional equation of conservation of mass of a dissolved or suspended material, i.e. the advection-dispersion equation. The module requires output from the hydrodynamic module, in time and space, in terms of discharge and water level, cross sectional area and hydraulic radius.

The Advection Dispersion Equation is solved numerically using an implicit finite difference scheme which, in principle, is unconditionally stable and has negligible numerical dispersion. A correction term has been introduced in order to reduce the third order truncation error. This correction term makes it possible to simulate advection-dispersion of concentration profiles with very steep fronts.

The one-dimensional (vertically and laterally integrated) equation for the conservation of mass of a substance in a solution is given and described in the following section. The equation reflects two transport mechanisms, advective (or convective) transport with the mean flow and dispersive transport due to concentrations gradients. The main assumptions underlying the advection-dispersion equation are:

- The considered substance is completely mixed over the cross section, implying that a source/sink term is considered to mix instantaneously over the cross section.
- The substance is conservative or subject to a first order reaction (linear decay).
- Fick's diffusion law (Chanson, 1999) applies, i.e. the dispersive transport is proportional to the concentration gradient.

6.1.1.3 Cohesive Sediment Transport (CST) Module

The transport of suspended sediment is modeled with the advection-dispersion equation due to the low settling velocities of fine sediments. The processes of erosion and deposition are modeled as source and sink terms, respectively, in the advection-dispersion equation. The erosion rate depends on local hydraulic conditions whereas the deposition rate additionally depends on the concentration of suspended sediment. The 1-D advection-dispersion equation reads:

$$\frac{\partial AC}{\partial t} + \frac{\partial QC}{\partial x} - \frac{\partial}{\partial x} \left(AD \frac{\partial C}{\partial x} \right) = -AKC + C_2q$$

Where

C	Concentration (g/l)
D	Dispersion coefficient (m ² /s)
A	Cross sectional area (m ²)
K	Linear decay coefficient (s ⁻¹)
S	Source/sink concentration (g/l)
q	Lateral inflow (m ³ /s)
x	Space coordinate (m)
t	Time coordinate (s).

The single cohesive layer model for deposition is:

$$S_d = \frac{\omega_0 C}{h^*} \left(1 - \frac{\tau}{\tau_{cd}} \right) \text{ for } \tau \leq \tau_{cd}$$

Where

S _d	Sink term in the advection-dispersion equation
ω ₀	Mean settling velocity of the suspended particles and sediment flocs (m/s)
C	Suspended sediment concentration (kg/m ³)
h [*]	Average depth through which the particles must settle (m)

τ Critical shear stress for deposition (N/m²).

The bed shear stress can be given as:

$$\tau = \rho g \frac{U^2}{M^2 h^{1/3}}$$

Where

ρ Fluid density (kg/m³)
 g Acceleration due to gravity (m/s²)
 U Cross section averaged flow velocity (m/s)
 M Manning number (m^{1/3}/s)
 h Flow depth (m).

The single cohesive layer model for erosion is:

$$S_e = \frac{M^*}{h} \left(\frac{\tau}{\tau_{ce}} - 1 \right)^n \text{ for } \tau \geq \tau_{ce}$$

Where

S_e Source term in the advection-dispersion equation
 M^* Erodibility of the bed (g/m²-s)
 h Flow depth (m),
 τ Critical shear stress for erosion (N/m²)
 n Erosion exponent.

Furthermore, the model requires that the sediment bed be characterized by a set number of physical parameters. Those parameters include the bulk wet density, specific gravity (SG), and porosity. Also, the volume of sediment per unit reach length must be specified.

Single layer CST modeling can be considered a first approximation for sediment transport modeling since the cross section averaged flow velocity is used to compute the shear stress and the processes of erosion and deposition are sensitive to local flow velocities.

6.1.2 Model Schematization

6.1.2.1 *Branches*

The branch schematization of the MIKE 11 model includes the following:

- Napa River from Oak Knoll Ave to the confluence with Carquinez Straits.
- Sonoma Creek from Hwy 12/121 Bridge to the outlet to San Pablo Bay.

- All major tidal slough channels in-between Napa River and Sonoma Creek including:
 - Napa Slough
 - Second Napa Slough
 - Third Napa Slough
 - Hudeman Slough
 - South Slough
 - China Slough
 - Devils Slough
 - Dutchman Slough
 - Mud Slough
- The interconnected system of canals, siphons, weirs and gates that are located inside or adjacent to the ponds were constructed with the purpose to facilitate moving water under operation as a evaporation system for the production of salt. The branch schematization extends from the pumping station east of Pond 8 to the tidal inlet from San Pablo Bay into Pond 1.

The length and chainages (stationing) of the branches have been calculated based on on-screen digitization of the branch network. The digitization was undertaken with an overlay of digital USGS 7.5” Quad sheets. Figure 6-1 shows details of the schematization and linkage between the 1- and 2-D models, and the overall channel schematization is shown in Figure 6-2. In total, the model includes 33 branches.

6.1.2.2 Cross Sections

The cross sections used in the MIKE 11 model were supplied from a number of different sources as shown by Table 6-1. The model primarily used surveyed cross sections, augmented with data extracted from a digital terrain model (DTM).

Table 6-1. Cross Section Sources

Branches	Source
<ul style="list-style-type: none"> • All tidal slough channels • Napa River from the mouth to Fagan Slough. • Sonoma Creek from the mouth to the app. 700 meter upstream of the confluence with Second Napa Slough [extent of Towill Survey (Towill, 2001)]. 	Extracted from the digital terrain model supplied by Towill at distances of 250 meter or less.
Napa River from Oak Knoll Ave to Fagon Slough.	Copied from a Napa River MIKE 11 model which was developed as part of a previous study for the Napa River (PWA, 1997).
Sonoma Creek from Hwy 12/121 crossing to app. 1,500m downstream.	Based on survey data from 1998 made available by Southern Sonoma County RCD
Sonoma Creek 1,500m downstream of Hwy 12/121 bridge to the extent of Towill survey (700m upstream from the confluence with Second Napa Slough).	Cross sections surveyed by PWA in October 2001 under Phase 2 of the Feasibility Study.
The interconnected system of canals, siphons, weirs and gates.	Canal cross sections and elevations estimated from DTM. Siphons, weirs and gate sizes supplied by CDFG.

6.1.2.3 Hydraulic Structures

There are no hydraulic structures included in the MIKE 11 model of the Napa River or Sonoma Creek. The hydraulic structures that are included in the model are:

- The “donuts” that are small basins that form junctions between two or more internal channels meets. These are typically equipped with hydraulic structures such as moveable weirs or gates that allow for controlling flow to/from the donut. These weirs and gates are included in the MIKE 11 model as control structures.
- In the internal channels a number of siphons are present allowing water to cross under a tidal slough channel. All siphons are modeled as culverts.
- The levee surrounding pond 2A has been breached at two locations; on China Slough and South Slough. The flows through these are modeled with a MIKE 11 branch through which the flow is calculated using an equation describing flow across a broad crested weir.
- A weir and a culvert connecting Pond 2 to China and South Slough respectively.

A summary of the hydraulic structures that have been included in the MIKE 11 model are shown by Figures 6-3 to 6-8.

6.1.3 Boundary Conditions

The boundary conditions applied to the 1-D river and slough model are summarized in Table 6-2. For the purposes of defining the flows in Milliken, Napa, Tulucay and Carneros Creeks, synthetic hydrographs were generated by scaling the Oak Knoll hydrograph by the percentage shown in Table 6-2, where this percentage represents the ratio of each tributary’s peak discharge to the one at Oak Knoll. The timing of each tributary’s hydrograph was based on the size of the sub-catchment area since smaller watersheds typically respond more quickly than larger ones. Therefore, the tributary hydrographs were advanced in time by the amount indicated in Table 6-2. More details of the methods and assumptions used for generating these synthetic hydrographs are given by Mr. B. Zlomke (Zlomke, 1996). These data sources represent those used for the purposes of calibration and validation.

Table 6-2. Summary of Boundary Conditions

Location	MIKE 11 Branch	Chainage	Data Source
Hydrograph at Sonoma Creek at Hwy 12/121 crossing.	Sonoma-Creek-U/S	-282	Estimated. See section 5.1.4.
Tidal water level at San Pablo Bay.	Sonoma-Creek	6,080	Measured data from UCD/USGS. (See Section 5.1.2.)
Upstream end	Third Napa Slough	0	Zero inflow
Upstream end	Hudeman-Slough	0	Zero inflow
Upstream end	Mud-Slough	0	Zero inflow

Location	MIKE 11 Branch	Chainage	Data Source
Tidal water level at confluence with Carquinez Strait.	Napa-River	26,000	Measured data from UCD/USGS. (See Section 5.1.2.)
Hydrograph at Oak Knoll Ave.	Napa-River-U/S	45,330	USGS gauging station.
Milliken Creek.	Napa-River-U/S	52,500	Estimated as 19.4% of the flow at Oak Knoll Ave. Time lag = +3.0 hours.
Napa Creek.	Napa-River-U/S	55,580	Estimated as 10.0% of the flow at Oak Knoll Ave. Time lag = +3.6 hours.
Tuluca Creek.	Napa-River-U/S	57,070	Estimated as 9.0% of the flow at Oak Knoll Ave. Time lag = +3.7 hours.
Carneros Creek.	Napa-River-U/S	66,220	Estimated as 6.0% of the flow at Oak Knoll Ave. Time lag = +4.0 hours.

6.1.4 Model Calibration and Validation

6.1.4.1 *Hydrodynamic Model*

Water level measurements are available for calibration and verification at the following stations: China, Dutch, Pablo, Pipe, Socr, TNS, Hude, MIC (Figure 5-1). For most of the stations, data are available continuously at 10-minute intervals for the period September 5th 1997 to March 12th 1998. The hydrodynamic model has been calibrated mainly by adjusting the local Manning's resistance coefficient. Where changes had to be made that resulted in unrealistic or inconsistent values, the bathymetric representation was checked and adjusted if necessary to give a more realistic description. Thus the model was calibrated using the resistance coefficient but also by applying slight modifications to the bathymetry.

The data has been split into two sets, one for calibration (prior to January 1st 1998) and one for validation (after January 1st 1998). However as this gives a dry period for calibration and wet period for validation, which is not ideal, minor modifications to the model parameters have been done after the validation runs.

Water surface elevations from each of the sites were examined and suggest that data from some of the sites may be questionable. In general, water surface elevations are largely controlled by conditions at the boundaries (such as M5 and Pablo). This is especially true during dry periods. However, the measured water surface elevations at Pipe are typically 0.2 to 0.7 meters below the measured elevations taken from M5 and Pablo. This is also the case at Dutch and Hude, although to a lesser extent. Inspection of the longitudinal water surface profiles that would result if these gage elevations were all correct, indicated that it was highly unlikely that any combination of standing or progressive waves could account for this shift in elevation.

In order to identify which sites may contain problematic data, a 75-hour moving average was calculated for each of the 9 measurement stations where water levels are available. The average water surface

elevations are shown in Figure 6-9 and 6-10 for the calibration period and in Figures 6-11 and 6-12 for the validation period. Findings from these plots include:

- The moving averages indicate that there are some datum shifts between the different measurement stations. This is especially true for stations M5 and MIC, where the difference in mean water level increases over the calibration period until settling in the range of 30 cm. These two stations are only 3700 meters apart and located near the mouth of Napa River. This wide and large section of Napa River is strongly influenced by the tide, and a 30 cm difference in mean water level does not appear to be reasonable. To further visualize this discrepancy the mean water level difference between M5 and MIC is plotted in Figure 6-13.
- The measured water levels at China likely have a datum error. This is evident from both the calibration and the validation period. The mean water levels at China are much higher than expected when comparing this data to the nearby measurement site Pipe.
- Considerable drift in the mean water level is also evident at Pipe during the validation period, indicating possible problems with these data.

It is clear from the analysis of the measured data that the measured water surface elevation at M5 is questionable. These data are particularly important since M5 represents the downstream boundary of the model in the Lower Napa River and this is one of the driving boundaries of the model. It appears that the problematic issues appear to be datum shifts in the measured data. This may be due to survey errors between the establishment of temporary benchmarks and the measurement equipment platforms or survey errors between the temporary benchmarks and the project survey control network. Nevertheless, the calibration can still be carried out with the focus being more on the shape of the water surface profile and the timing of peaks and troughs than on forcing the model to coincide with water levels having datum shifts. If the model were to coincide with the measured data at all stations unphysical values of the bed resistance would have to be applied. This, however, would affect the timing of the peaks and the troughs.

In undertaking the calibration of the model, several characteristics of the tidal simulation are considered, including:

- Time of the peak high water
- Time of the low water
- The tidal range
- The elevation of high water
- The elevation of low water
- The shape of the tidal signal at a monitoring station.

The two main model parameters that could be varied are the resistance coefficient (Manning's n) and bathymetric representation, although bathymetry was only altered to give extra interpolated sections in complex flow regions. Sensitivity of these six parameters to model parameters such as bed resistance and bathymetry provides a significant test of the model. For example, if the roughness is altered to match a high flow, then there will also be an influence on the speed of wave propagation and timing of peak flows through the system. Comparison of the timings of the peaks throughout the system and the shape of the tidal fluctuation give additional checks that the selected value of the resistance is reasonable.

As an example, consider the results from Dutchman's slough. Of these six parameters, only the absolute elevations show considerable error. The shape of the simulated tidal response, tide range and timings of high and low water show a good match (Figure 6.14), but the absolute values of the simulated and observed high and low tides are 0.3 m different. Since the tide range is within 5%, this would imply that there is a systematic bias in the absolute elevation at one or more of the monitoring station. To further visualize this shift in the datum a scatter plot was produced for the corresponding period (Figure 6-15), and a linear regression and correlation were estimated. The correlation is 0.9937 and the slope of the linear regression 0.978. Both values indicating that the model is good at reproducing the dynamic response. The vertical axis intercept is 0.177, which indicates a vertical offset, or datum shift, of 0.177 meters.

Figures 6-16 to 6-22 show the measured and simulated water level during a 14-day and a 2-day period within the calibration period. Figures 6-23 to 6-29 show the measured and simulated water level during a 14-day and a 2-day period within the validation period. Figures 6-30 to 6-41 show scatter plots for the same 14-day periods for all measurement instants. Please note that upstream of TNS the boundary used is a zero flow condition, though Third Napa Slough is interconnected with Sonoma Creek. This explains the discrepancies at TNS.

Note that no data was available for TNS for the calibration period, or for Hudeman slough during the validation period. The linear regression is shown for all of the scatter plots. The linear regression indicates if any serious phase problems are present and may also expose a general trend in the errors such as a shift of the datum. If a phase problem is present (e.g., due to incorrect reach lengths) the slope of the linear regression deviates strongly from unity. With the exception of SOCR and TNS during the validation period all the linear regressions have a slope close to unity, indicating that the reach lengths used reflect the actual length in the channels and sloughs. If the slope is close to unity one may inspect the zero order term (vertical axis intercept) of the linear regression for any serious shifts in the datum. For most of the scatter plots the zero order term is of the order of 15 ~20 cm, indicating that the simulated data is generally above the measured data. The one anomaly is at China slough, where the simulated water level is typically below the measured data.

Figs 6-42 to 6-55 show scatter plots for the full calibration and validation periods in which only the peaks and the troughs are considered. Except for SOCR and TNS during the validation period these scatter plot indicate that the timing of the peak and the phase is reproduced well. The problems with SOCR and TNS are due to high flows during the validation period and also the lack of data for the upstream end of Third Napa Slough. During validation period, the modeled levees are overtopped and the simulated results are not realistic. Due to the limitation of the project (no floodplain analysis), flow over the floodplains was not taken into account, and thus the model is not adequate for very high flows such as was the case in 1998. This does not render the model as being invalid for the task of carrying out scenario simulations during non-extreme events. Additionally, it should be noted that the problems associated with high flow events seem to be confined to the area in the vicinity of SOCR and TNS. Further note that the upstream boundary of Sonoma Creek is based on an estimated hydrograph since there was no gauging station operational on Sonoma Creek during the calibration and validation periods. Thus, the discrepancies at

TNS and SOCR are not unexpected. Based on earlier recommendations from the MTAG, agencies and activities related to a watershed study on the Sonoma Valley (K. Rippey, USACE, personal communication, 2001) a gauging station is to be located on the lower Sonoma Creek, probably near Schellville. As an illustration of the uncertainties of the estimated hydrograph Figure 6-56 shows a close up of the simulated water level, measured water level and the hydrograph applied at the upstream end of Sonoma Creek. As is evident here the main deviations from the measured data are caused by the estimated hydrograph. This uncertainty carries through the calibration resulting in large deviations from measured data whenever the hydrograph shows any significant flow. Therefore the calibration results in Sonoma Creek should be considered with this uncertainty in mind.

Additionally a number of statistical quantities were calculated for the simulated data and measured data. This is displayed below in Tables 6-3 and 6-4 considering only peaks and troughs. Please note that for DUTCH, MIC and PIPE all measurements seem to be off by approximately 30 cm during the calibration period. All three measurements are located in the vicinity of M5, suggesting possible datum errors with the measured data (used as a boundary condition at the Napa River inlet). Furthermore, note that the absolute value of the average error is close to the root mean square error indicating that the errors are biased in the same direction which occurs if there is a datum shift.

Table 6-3. Statistical summary of measured data for calibration period.

(RMSE: root mean square error, N number of elements in data sets.)

Measurement Station	Average error (m)	Observed mean (m)	Simulated mean (m)	RMSE	Correlation	N
DUTCH	-0.3249	0.8084	1.1333	0.3486	0.8537	177
HUDE	-0.2875	0.8717	1.1591	0.3366	0.917	176
MIC	-0.3162	0.8932	1.2094	0.3571	0.9865	353
PIPE	-0.3264	0.803	1.1294	0.3594	0.8097	237
SOCR	-0.1395	1.0307	1.1702	0.205	0.8649	234
CHINA	0.298	1.4836	1.1856	0.3271	0.9781	471
NAPA3RD-SLOUGH	-0.3806	0.8842	1.2647	0.3915	0.8683	265

Table 6-4. Statistical summary of measured data for validation period.

Measurement Station	Average error (m)	Observed mean (m)	Simulated mean (m)	RMSE	Correlation	N
DUTCH	0.043	1.3998	1.3568	0.1145	0.9895	94
HUDE	-0.0451	1.535	1.5801	0.059	0.9997	10
MIC	-0.3336	1.0566	1.3902	0.3393	0.9971	275
PIPE	-0.1187	1.2539	1.3727	0.1826	0.9875	254
SOCR	-0.5048	1.4443	1.9492	0.7139	0.7256	306
CHINA	0.1876	1.5667	1.3792	0.199	0.9977	253
NAPA3RD-SLOUGH	-0.0403	1.4285	1.4688	0.2321	0.9505	275

By taking the previously mentioned datum shifts into consideration the model results are in as close agreement with the measurements as can be expected, considering the uncertainties of the measured data. Finally, note that the discrepancies between measured and simulated data are in excess of what can be accomplished by varying Manning's roughness coefficient.

In addition to calibration of water level, the MIKE 11 HD model has been calibrated with velocity. Figures 6-57 to 6-60 show the measured and simulated velocities during a 14-day and a 2-day period within the calibration period. Figures 6-61 and 6-62 show the measured and simulated velocities during a 14-day and a 2-day period within the validation period. Whereas the simulated water levels are directly comparable to the measurements, the simulated velocity is the average velocity (discharge divided by flow area) and thus not directly comparable to the measurements which is the velocity at a point. In general we might expect the point velocity to be 10 to 30% higher than the average velocity. However, comparing the velocities is very useful when evaluating if the model is predicting the right flow direction and that the change of flow occurs at the right time.

The velocity measurements are in general not available for the complete periods used for the water level calibration (see Table 5-2). Station Pipe is the only one that has velocity available for the full calibration and validation period. The other stations typically have velocity available for a one or two month period. Except for Hude all velocity stations show a similar pattern; the magnitude of the velocities differ due to the differences between the measured (point) data and simulated cross sectionally averaged velocity, but the flow direction and change in flow direction show reasonable correlation. Station Hude does not show the same pattern. This station is located close to where the tidal signal through Napa River meets with the signal through Sonoma Creek. The location of the zero-flow (or null) point in the model is very sensitive to the calibrated resistance numbers as these influence the travel time for the tidal signal. This sensitivity is representative of the delicate balance of the physical processes in the field, and wind stress and slight natural changes in the boundary forcing can be expected to alter the null point in the field. Based on the water level and velocity measurements the location of the zero-flow point was studied by Warner (2001), and identified to be close to Hude which correspond well to the MIKE 11 model predictions.

In summary, the discrepancies between the measured and the simulated data appear to be primarily attributable to datum errors in the measured data at some of the stations. The datum errors at gauging stations used as boundary conditions are of particular concern as the boundaries drive the modeled conditions throughout the project area under existing conditions. (For future conditions modeling and salinity reduction this is not of such great concern since Phase 2 of the Feasibility Study will allow for connection to a Bay wide model to supply boundary conditions.)

Limitations of the model along Sonoma Creek and Third Napa Slough during high flow events were demonstrated, although uncertainties with the estimated hydrograph at the upstream end of Sonoma Creek cannot be neglected. The uncertainties of this hydrograph give rise to large error bounds on the water level at SOCR. These error bounds are wider than a variation in bed resistance can compensate for.

Additional sources of errors that may affect the calibration and accuracy of future predictions include:

- Local wind stresses in the model domain.
- Bathymetric inaccuracies.
- Levee overtopping. This study is not intended to be a flood inundation analyses and as such the model does not account for flood attenuation storage areas.
- Stratified flow. Water elevations inferred from measurements taken by pressure transducers have additional uncertainties when significant stratification is present. This is the case in the present study, where water levels in the lower Napa River were estimated from pressure measurements. As an example of this uncertainty, the effect of stratification was estimated by considering a simple situation where the water column is assumed to have freshwater in the top half and saline water in the lower half of the water column. The deviation from hydrostatic pressure using a uniform density may be evaluated (for $x > h$) as:

$$\Delta p = -g(h_1 - x)(\rho_{\text{freshwater}} - \rho_{\text{saltwater}})$$

where

- g acceleration due to gravity
- h_1 water depth at which the density changes
- x water depth at which the pressure is evaluated
- $\rho_{\text{freshwater}}$ density of fresh water
- $\rho_{\text{saltwater}}$ density of saline water

The deviation from the depth is then given as:

$$\Delta h = \frac{\Delta p}{\rho_{\text{freshwater}} g} = (h_1 - x) \left(\frac{\rho_{\text{saltwater}}}{\rho_{\text{freshwater}}} - 1 \right)$$

This expression may be evaluated using the following typical values and setting $x = 0$:

- $h_1 = 5$ meters (half of the typical depth at M5)
- $\rho_{\text{freshwater}} = 1000 \text{ kg/m}^3$
- $\rho_{\text{saltwater}} = 1020 \text{ kg/m}^3$ (approx 3% salt contents)

which gives a difference in depth (and water level) of 10 cm.

- Temperature variations. Temperature difference between surface and bottom water will produce a density gradient and add to the uncertainties of the derived water levels.

6.1.4.2 Advection Dispersion Model

- **Boundary Conditions Salinities**

Boundary data were required at the open boundaries of the numerical model. Measured salinity from Pablo and M5 were applied at the downstream extents of Sonoma Creek and Napa River, respectively,

while freshwater was assumed at the upstream extents. At the downstream boundary of Napa River, the average of the bottom and top salinities were applied as boundary data.

The assumption of no stratification inherent in the MIKE 11 1-D model cannot be completely fulfilled at the downstream end of Napa River. Measurements of top and bottom salinity (Warner, 2001) indicate that the average top and bottom salinity during the period September 1997 to March 1998 were 12.9ppt and 15.2ppt, respectively. Obviously, this covers periods with large variations. Figure 6-63 shows two selected time periods of salinity at M5 (in Mare Island Strait) with relatively small and large differences. As expected there are small differences between bed and surface salinity during the dry period in September where the fresh water runoff from Napa River is less. At station Pablo only the depth averaged salinity was measured.

- **Calibration**

The MIKE 11 AD model was calibrated by setting the dispersion coefficients to obtain the best match between simulated and measured salinities, which led to a value of 50 m²/s. Figures 6-64 to 6-66 show time series of salinity at six stations for which salinity data is available throughout the calibration period.

The simulated salinities correspond well to the measurements when viewed at a seasonal scale as in Figures 6-64 to 6-66. These figures reflect that the AD model is primarily driven by the boundary conditions and thus by convective transport of the salinity coming into the model through the downstream boundaries. The ability of the model to simulate dispersive transport is emphasized when looking at a short time scale, as shown in Figures 6-67 to 6-68. In general the AD model is simulating salinities which compare well to the measurements, in particular when taking into account:

- The stratification in the downstream part of Napa River and the fact that the model is primarily driven by downstream boundary conditions.
- The quality of the advection dispersion model relies on the hydrodynamic model. In other words, any errors in the HD model solution will be reflected in the AD simulation.

To further illustrate the AD calibration, scatter plots of salinity peaks are presented in Figures 6-69 to 6-74. These scatter plots illustrate that the value of the dispersion coefficient is well chosen. All scatter plots have a slope close to unity and further the intersection with the y-axis are all within 2ppt.

6.1.4.3 *Sediment Transport Model*

As described in Section 6.1.1.3, the following model parameters need to be estimated: ω_0 , τ_{cd} , τ_{ce} , M , n , bulk wet density, SG , porosity, and volume of sediment per unit reach length.

The apparent settling velocity, ω_0 , can be estimated in the laboratory or in the field. In the laboratory, measurements are typically made in quiescent water whereby flocculation occurs due to different settling rates of different sized aggregates. In the field, measurements are made during fluid flow whereby flocculation occurs due to internal shearing. The latter cause of flocculation is thought to be of greater

importance under shearing conditions (DeVries, 1990). Figure 6-75 shows laboratory (van Rijn, 1989) and field (DeVries, 1990 & 1992) measurements of apparent settling velocity as a function of suspended sediment concentration (SSC) for Mare Island sediments where the salinity is greater than 10ppt. For the calibration period, the average SSC within the model branch network ranges from 30 mg/L during the dry season to over 200 mg/L during the wet season. Assuming a SSC of 200 mg/L, laboratory measurements from Figure 6-75 would predict a settling velocity of approximately 0.01 mm/s. From the field data that was collected by John Warner of UC Davis and the USGS (Warner, 2000) at the end of September 1987, a near-bed SSC around 100 mg/L produced a settling velocity an order of magnitude greater at approximately 0.55 mm/s. A value of 0.05 mm/s was selected for the 1-D slough network to approximate a depth-averaged settling velocity; a value of 0.01 mm/s would be more appropriate for the 2-D pond model due to wind hindered settling/re-suspension effects.

The critical shear stress for deposition, τ_{cd} , refers to a bed shear stress at which all of the suspended sediment load would eventually fall out of suspension. While determination of this parameter is favorable in a laboratory environment, it is a difficult parameter to measure in the field as there is no control over the shear stresses (DeVries, 1990). DHI (2001) suggest a typical range from 0.03 to 1.00 N/m². A value of 0.05 N/m² was selected with a time centering of 0.8.

The critical shear stress for erosion, τ_{ce} , involves the resuspension of material from the sediment bed either through surface erosion or mass erosion. Surface erosion typically dominates in a quasi-steady flow environment (DeVries, 1990). The most commonly used equation to represent surface erosion is:

$$E = M \left(\frac{\tau}{\tau_{ce}} - 1 \right)^n \text{ for } \tau \geq \tau_{ce}$$

Where

- E Rate of surface erosion (g/m²-s)
- M Erosion rate coefficient (g/m²-s)
- τ Bed shear stress (N/m²)
- n Exponent that describes the degree of non-linearity.

Field measurements conducted in Mare Island Strait at a shipyard berthing area resulted in 0.144 N/m² for τ_{ce} , 0.00625 g/m²-s for M, and 1 for n (DeVries, 1990 & 1992). DHI Inc. (2001) suggest the following typical range for the above parameters:

- 0.05 to 0.10 N/m² for τ_{ce}
- 0.20 to 0.50 g/m²-s for M
- 1 to 4 for n

Note that M and M* have the same units. The values selected are those of the field measurements.

The remaining parameters describe the sediment bed. Laboratory measurements conducted by *Cooper Testing Labs, Inc.* on a sediment core from Napa Slough resulted in a bulk wet density of 1300 kg/m³, a specific gravity of 2.70, and a porosity of 79.4 %. For comparison, additional measurements of bulk wet density, specific gravity, and porosity were taken on cores from the Cargill salt ponds, Sonoma Baylands, and Cooley Marsh; average values are 1390 kg/m³, 2.64, 75.3%; 1400 kg/m³, 2.77, 77.0%; and 1295 kg/m³, 2.66, and 81.5%, respectively. The values selected to characterize the sediment bed are those for the Napa Slough, which are within the range of values for the Cargill salt ponds. Additionally, assuming a bed thickness on the order of 1 m and a typical cross section width of 40 m, the volume of sediment per reach length was estimated at 40 m³/m; for Dutchman Slough and the Napa River, the volume of sediment per reach length was estimated at 80 and 140 m³/m, respectively.

Results for the selected values are displayed in Figures 6-76 through 6-81. Due to uncertainty in the hydrodynamic calibration and the recognized inherent difficulty in developing a calibrated single layer cohesive sediment transport model, the single layer model is not in a calibrated state as presented here. Figures 6-82 through 6-87 demonstrate that cyclical processes in SSC are just as difficult to simulate. The simulations do show trends and could be used for estimating relative differences in deposition rates and concentrations given different management alternatives, but more accurate simulations will require additional field data, ideally at multiple levels through the flow, to reduce the uncertainty in the parameters describing sedimentation processes.

6.2 2-D MODEL OF SALT PONDS

It was considered that, for the purposes of this study, circulatory flows in the salt ponds were predominantly 2-D in nature, i.e., mixing processes were assumed to be depth averaged. (For further details, see Section 5.3 Analysis of the Role of Stratification).

6.2.1 2-D Numerical Model, MIKE 21

The MIKE 21 hydrodynamic flow model is the basic module of the entire MIKE 21 system. It provides the hydrodynamic basis for the computations performed in most of the other modules, for example the Advection-Dispersion module.

MIKE 21 HD simulates the water level variations and flow response to a variety of forcing effects in lakes, estuaries, bays and coastal areas. The water levels and flow velocities are resolved on a rectangular grid covering the area of interest. The inputs to the model consist of:

- Bathymetry,
- bed resistance coefficients,
- wind field data,
- hydrographic boundary data (usually water surface elevations, velocities or flux),
- inflows of streams or rivers, and
- barometric pressure (considered insignificant at the scale of this simulation).

The advection-dispersion (AD) module of MIKE 21 simulates the spreading of a dissolved or suspended substance in an aquatic environment under the influence of the fluid transport and associated natural dispersion process. The substance may be a pollutant, which may be treated as conservative or non-conservative substance. Like the HD module, the concentration of the substance is calculated in each point of a rectangular grid covering the area of interest. The HD module provides information on the transport, (i.e., currents and water depths) in each grid point. Other data required include tracer concentrations and discharge quantities at outfalls, together with concentrations at the boundaries.

The system solves the equation of conservation of mass for a dissolved or suspended substance using a 2-D form of the QUICKEST finite difference scheme (this is a derivation of the QUICK scheme, a quadratic discretization interpolation scheme for convective kinetics after Leonard, 1979).

6.2.2 Model Schematization / Discretization

The hydrodynamic module of MIKE 21 solves the vertically integrated equations of mass and momentum conservation in two horizontal directions given by:

$$\begin{aligned} \frac{\partial \zeta}{\partial t} + \frac{\partial p}{\partial x} + \frac{\partial q}{\partial y} &= 0 \\ \frac{\partial p}{\partial t} + \frac{\partial}{\partial x} \left(\frac{p^2}{h} \right) + \frac{\partial}{\partial y} \left(\frac{pq}{h} \right) + gh \frac{\partial \zeta}{\partial x} + \frac{gp\sqrt{p^2 + q^2}}{C^2 h^2} \\ - \frac{1}{\rho_w} \left[\frac{\partial}{\partial x} (h\tau_{xx}) + \frac{\partial}{\partial y} (h\tau_{xy}) \right] - \Omega p - fVV_x + \frac{h}{\rho_w} \frac{\partial}{\partial x} (p_a) &= 0 \\ \frac{\partial q}{\partial t} + \frac{\partial}{\partial y} \left(\frac{q^2}{h} \right) + \frac{\partial}{\partial x} \left(\frac{pq}{h} \right) + gh \frac{\partial \zeta}{\partial y} + \frac{gq\sqrt{p^2 + q^2}}{C^2 h^2} \\ - \frac{1}{\rho_w} \left[\frac{\partial}{\partial y} (h\tau_{yy}) + \frac{\partial}{\partial x} (h\tau_{xy}) \right] - \Omega p - fVV_y + \frac{h}{\rho_w} \frac{\partial}{\partial y} (p_a) &= 0 \end{aligned}$$

Where

h	water depth
ζ	surface elevation
p,q	flux densities in x- and y-directions
C	Chezy resistance (see section 6.1.1 for relationship to Manning's "n")
g	acceleration due to gravity
f	wind friction factor
V, V _x , V _y	wind speed and components in x- and y-direction
Ω	Coriolis parameter

p_a	atmospheric pressure
ρ_w	density of water
x, y	space coordinates
t	time
$\tau_{xx}, \tau_{xy}, \tau_{yy}$	components of effective shear stress

The following effects may be accounted for:

1. Convective and cross momentum
2. Wind shear stress at the surface
3. Barometric pressure gradients
4. Radiation stresses due to the presence of a wave field
5. Coriolis forces
6. Momentum dispersion
7. Sources and sinks for mass and momentum
8. Evaporation
9. Pier resistance

Sources and sinks are used for coupling with the rivers and sloughs modeled by MIKE 11. MIKE 21 AD solves the advection-dispersion equation for dissolved or suspended substances in two dimensions. Discharge quantities and compound concentrations at source and sink points are included together with a decay rate. The advection-dispersion equation reads:

$$\frac{\partial}{\partial t}(hC) + \frac{\partial}{\partial x}(uhC) + \frac{\partial}{\partial y}(vhC) = \frac{\partial}{\partial x}\left(h D_x \frac{\partial C}{\partial x}\right) + \frac{\partial}{\partial y}\left(h D_y \frac{\partial C}{\partial y}\right) - F h C + Q_s (C_s - C)$$

Where

C	compound concentration (arbitrary units)
u, v	horizontal velocity components in the x, y directions
h	water depth
D_x, D_y	dispersion coefficients in the x, y directions
F	linear decay coefficient
Q_s	source/sink discharge per unit horizontal area
C_s	concentration of compound in the source/sink discharge

6.2.3 Boundary Conditions

At all open boundaries of the model area a set of boundary conditions must be supplied. The set of data constituting the boundary condition must consist of either:

- Surface levels and flux densities parallel to the boundary, or
- Flux densities both perpendicular and parallel to the boundary.

In the present study for the existing conditions, no open boundaries are present in the MIKE 21 set-up. The only boundary conditions for the 2-D model of the salt ponds are the inflow and outflow from the ponds. For future phases of the study, the 2-dimensional model will be modified for open boundaries to represent tidal breaching of the pond levees and connection to San Pablo Bay or the surrounding sloughs and rivers.

Since no open boundaries are present in the current 2-D model, the driving forces of the two dimensional dynamics are the source/sink terms describing the linkage with MIKE 11. The MIKE 11/ MIKE 21 linkage not only describes interaction of the ponds with the slough channels but also the routing of water through the channels located along the edges of the ponds.

6.2.4 Validation

Recently, some validation data has become available for the salt ponds. These data were collected between February 1999 and January 2001 by the USGS (Takekawa *et al.*, 2000). Specifically water temperature, pH, turbidity, dissolved oxygen and salinity (conductivity) were measured in Ponds 1, 2, 2A, 3, 4 and 7. In addition, depths in the ponds were measured approximately by Tom Huffman of CDFG, and these were subsequently converted to water surface elevation by PWA for validation purposes.

Figures 6-88 to 6-91 show validation plots of salinity in Ponds 1, 2, 2A and 3. The simulated 2-dimensional pond model used a time series of the operation of the pump between Pond 1 and 2 based on the operating records of Tom Huffman of CDFG. Since it is unclear of the conveyance capacity of Siphon 3 (between Ponds 3 and 4—refer to Figure 3-5) and Siphon 4, simulated results were not considered in Ponds 4, 5, 6 or 6A. It has been assumed for the purposes of this validation, that Siphons 1 and 2 probably convey water at close to their design capacity since these operate the most frequently. In addition, it was not possible to construct a time series based on the records provided of the operation of the pump in the north of the site (adjacent to Pond 8) with sufficient accuracy for validation purposes. Therefore, only sufficient data for the validation of Ponds 1, 2, 2A and 3 was available.

It should be noted that there is a 2-3ppt variation in measured salinity dependent on the measurement location within each pond, as shown by the figures. The key in each figure corresponds to the nomenclature defining the position in the ponds and is not important for validation purposes. Simply note that the solid lines correspond to simulated data and the symbols correspond to measured data. It can be observed from the figures that the 2-dimensional pond model predicts the salinities in Ponds 1, 2, 2A and 3 to within 3ppt which is acceptable to the tolerance of the measured data.

Figure 6-92 shows the validation of the water surface elevation in Pond 3. Again reasonable validation was achieved with the results showing less than 50mm difference between measured and simulated water surface elevation in the pond.

6.3 INTEGRATED 1-D AND 2-D MODELING

6.3.1 Model Coupling

In the present study, a coupled version of the MIKE 11 and MIKE 21 flow models has been utilized. These two models have been dynamically coupled to form a model capable of integrating the 1-D flows of narrow tidal channels and the 2-D circulation of the ponds.

MIKE 11 and MIKE 21 can be coupled for hydrodynamic (HD) and/or advection-dispersion (AD) simulations. Coupling takes place via

- MIKE 11 boundaries (H or Q boundaries).
- MIKE 21 boundaries (Q or H boundaries) and/or MIKE 21 source/sink points.

Thus, a MIKE 11 boundary can be coupled to a MIKE 21 boundary or to a MIKE 21 source/sink point. The latter is required in order to simulate flow from the slough system or hydraulic structures into the ponds.

MIKE 21 is the “governing” engine, meaning that MIKE 21 controls the coupling and executes the MIKE 11 model when required for exchanging data between the two engines. Specification of the coupling to MIKE 11 is defined through user-defined parameters for MIKE 21.

The MIKE 11/MIKE 21 coupling is an explicit coupling meaning that the MIKE 11 time step and the MIKE 21 time step are solved separately. This explicit nature of the coupling scheme has a significant effect on the numerical stability of the scheme, and forces a decrease in the time step and an increase in the run-time of the coupled model compared to the stand alone MIKE 11 and MIKE 21 models.

6.3.2 Napa Salt Ponds Set-up

The bathymetry used for the 2-D modeling of the Napa salt ponds is based on a 25x25 meter grid which was derived from the Digital Terrain Model (DTM) of the salt ponds produced by Towill, Inc. in 2001 (Towill, 2001). In order to confine the 2-D flow to the salt ponds, all areas not directly connected to the ponds were omitted from the simulation. This was done by simply redefining all points outside the ponds as land points. The bathymetry used for the modeling of the salt ponds is shown by Figure 6-93.

The siphons connecting the different ponds were modeled by the use of culverts in MIKE 11. Thus the major part of the routing of water between the different ponds was modeled using MIKE 11. The only exceptions being the connection between Ponds 1 and 1A and the connection between Ponds 4 and 5 (see also Figures 6-3 to 6-8 for details of water control structures). Both of these were simulated using the 2-D bathymetry.

Note the model has the capability to allow flow into all the ponds via various water control structures, though due to the gate settings in this preliminary set-up no flow occurred in Ponds 6, 6A, 7, 7A and 8.

The water was simply routed from the pump at Pond 8 through the internal channels through to the siphon between Pond 5 and 6. From here the flow through Ponds 4 and 5 was modeled using MIKE 21. From Pond 4 to Pond 3 the siphon was modeled using a MIKE 11 branch with a culvert. The flow between Pond 3 and Pond 4 was modeled using a siphon and an open channel. The latter flow does not interact directly with the dynamics of Pond 2A, since this pond was restored to full tidal action with the surrounding slough channels between 1996 and 1998. Pond 1 is tidally influenced through a pipe control structure which runs under Highway 37 to a channel that runs into San Pablo Bay. This pipe was modeled through the use of a MIKE 11 branch consisting of closed circular cross sections.

At the north end of Pond 1 and Pond 1A the interaction with Pond 2 is modeled through a siphon (MIKE 11 culvert) connected to a channel from Pond 1A and a pump to Pond 1. To prevent the pump from generating a flow back into Pond 1A an artificial flap gate was inserted. This flap only allows flow in the positive direction, i.e., out of Pond 1A. This represents the existing conditions and can be modified for future restoration alternatives. Figure 6-94 shows a scenario of the flow between Pond 1 and Pond 2. For purposes of conceptualization, the donut structure connecting the siphon from Pond 2 to Pond 1 is not shown, nor is the pump located in Pond 1. The flow through the siphon at this location however, is governed by the flow produced by the pump ($1.89\text{m}^3/\text{s} = 30,000$ gallons/minute) Note the interaction of Pond 2 with the slough channel through the flap gate operated pipe at the west end of Pond 2.

Examples of the HD and AD coupling are shown in Figures 6-95 and 6-96. Figure 6-95 shows the hydrodynamic conditions for Figure 6-96, which shows a close up of the mixing taking place between Pond 4 and Pond 3. The flow is in this case driven by a water level difference between Pond 3 and Pond 4. As there is no coupling from the AD to the HD the flow is solely driven by the head difference in the mathematical model, assuming uniform density throughout the system.

6.3.3 Calibration and validation

Since no data is available for calibration or validation such an analysis has not been carried out for the coupled model.

7. SENSITIVITY ANALYSES

7.1 RIVER, CREEK AND SLOUGH CHANNEL MODEL

In order to highlight the sensitivity of the model to changing model parameters a number of sensitivity analyses have been carried out. If the model results fluctuate dramatically as the parameters are varied according to the expected uncertainty, then the confidence in the predictive capability of the model is impaired. However, if the expected uncertainty in the input parameters make little difference to the results then the model is robust. Thus the sensitivity analysis identifies which uncertainties in the model are critical and if additional data collection or analyses are required.

7.1.1 Hydrodynamics

The sensitivity of the hydrodynamic 1-D model has been investigated through running the model with modified boundary conditions. In the following the effects of changing the downstream tidal boundary condition and the upstream discharge boundary conditions are shown.

7.1.1.1 Change in Downstream Tidal Boundary Condition

Sensitivity analyses were conducted on the tidal boundary condition at the downstream model extent of the Napa River (with the confluence of Carquinez Strait) and Sonoma Creek (at the estuary with San Pablo Bay). This was undertaken to assess the implications to the 1-dimensional modeling of inaccuracies in not only the measured water surface elevations but also the measured bathymetry. Therefore, the effect of a 50mm increase and a 50mm decrease of the tidal water level boundaries have been investigated. In each case two simulations have been made. One simulation covered a 5-day period in the dry season (Oct 1-6, 1997) and the second simulation covered a 5-day period in the wet season (Feb 1-6, 1998). The runoff from Napa River and Sonoma Creek drainage areas is almost zero in the first simulation period, whereas the second simulation period covers the largest runoff event in the wet season 1997-1998. A dry and a wet period have been chosen as the model response to a changed tidal condition is quite different in a wet and a dry period.

The result of the tidal level increase/decrease is shown in Figures 7-1 and 7-2 as profiles of water level increment in Napa River and Sonoma Creek. At high flows, the effect of the change in downstream boundary conditions has a negligible effect. Under low flow conditions, the influence of the downstream boundary condition is significant and demonstrates the importance of downstream water surface elevations.

The validity of the water surface elevations at M5 and PABLO were further investigated. As mentioned earlier the observed water surface elevations at M5 and MIC show a large water surface gradient (with the slope in the upstream direction) in the lower reach of Napa River, indicating a possible error in the M5 boundary data.

Uncertainties in the measured data may be due to:

- Errors in surveying the temporary benchmarks.
- Errors in surveying the elevation of the pressure transducers.
- Inaccuracies of computing water depth from pressure sensors in a stratified flow (see Section 6.1.2.3).
- Errors in transforming the initial water depth to water levels.

A sensitivity study was carried out in order to quantify the effect of these uncertainties in the model solution. This sensitivity analysis consisted of performing hydrodynamic simulations in which the M5 water surface elevation was varied by +/- 45cm in increments of 15cm. The PABLO water surface elevation was varied over the same range in order to complete the sensitivity analysis.

The comparison between simulated and observed data is summarized through plots of statistical quantities. To visualize the effect of the shift, the average error between the measured and simulated data is plotted in Figure 7-3 for all seven measurement stations. This plot illustrates that Dutch, Pipe, MIC and China are all affected by shifts in the M5 boundary data. Hude is to some extent also influenced, though this station is situated at an intermediate location between Sonoma Creek and Napa River thus it is also affected by the boundary at Pablo. Figure 7-4 illustrates the intersection of the linear regression of the scatter plots as a function of the shift in the M5 datum. These results indicate that the system is strongly driven by the tidal boundary data, and that a 30cm shift in the M5 water levels may account for much of the differences between simulated and measured data. The improvement agreement between measured and simulated water levels with the -30cm shift at M5 is shown in Figures 7-7 through 7-13.

Results from the shifts applied at PABLO (Figure 7-5 and 7-6) indicate that the water surface elevations in the eastern half of the slough system are not significantly influenced by the tidal boundary data applied at Sonoma Creek. However, simulated water surface elevations at SOCR and TNS are sensitive to PABLO boundary data but also (as previously mentioned) to the estimated hydrograph which is used at the upstream end of Sonoma Creek.

7.1.1.2 Change in Upstream Discharge Boundary Condition

Sensitivity analyses were conducted on the flows passing down the Napa River and Sonoma Creek into the slough channel network in order to assess the implications to the 1-dimensional modeling of inaccuracies in the flow estimates. The boundary condition at the model's upstream end of the Napa River is gauged data at Oak Knoll. The boundary condition at the model's upstream end of Sonoma Creek is the flow estimate based on the methods described in Section 5.1.3. In order to highlight the sensitivity of the present model to the flow boundary condition at the Napa River and Sonoma Creek two simulations have been carried out. For the rainfall which caused significant runoff between February 1-6, 1998, the discharge boundaries at the Napa River and Sonoma Creek have been increased and decreased by 25%.

The simulated water levels and velocities along a profile of Sonoma Creek are shown in Figures 7-14 and 7-15, respectively. Figure 7-14 indicates a +/- 25% uncertainty in the flow estimates can have a significant effect on the water levels in Sonoma Creek between the confluence with Third Napa Slough and the upstream Sonoma Creek Boundary. However, the effect is insignificant downstream of the Third

Napa Slough confluence. Similar trends are observed for the velocity in the channel as shown by Figure 7-15.

The implications of these results are that inaccuracies of up to 25% in the flow estimate for Sonoma Creek will have the greatest influence to hydrodynamic analyses with the Skaggs Island region but will have negligible influence to the slough channel network surrounding the salt ponds.

The simulated water levels along a profile of the Napa River are shown in Figure 7-16. As expected the model's response to changes in flow are more significant closer to the upstream boundary. At the downstream boundary and in the vicinity of the salt ponds the influence is negligible.

Figure 7-17 shows the results of a sensitivity test on the velocity in Third Napa Slough. This profile of velocity demonstrates similar characteristics to the previous figures of sensitivity analyses for Sonoma Creek in that effects on the hydrodynamics surrounding Skaggs Island could be significant but they are likely to be insignificant for the slough network.

At present the MIKE 11 model has a zero-flow boundary conditions at the upstream end of its Third Napa Slough (TNS) branch. With known cross section, water level and velocity at TNS, the flow area can be calculated and subsequently the flow (Q) can be estimated as the product of velocity (V) and cross sectional area (A). Substituting the zero-flow boundary conditions with the estimated discharge allows a simulation in which the model is compensated for the missing tidal prism. This is not applicable for simulating future conditions, as the velocity at TNS is unknown for future conditions. However, simulation of historic conditions with the estimated discharge boundary provides an opportunity to check if the model needs to be extended to include all of the tidally influenced part of Third Napa Slough.

The simulated water levels change only insignificantly after substituting the Q boundary as the water levels are mainly controlled by the down stream boundary conditions. The simulated discharges and velocities are significantly influenced. Figure 7-17 shows the minimum and maximum simulated velocity with $Q=0$ and with $Q=V*A$.

Based on the above analysis it seems important to include all of the tidally influenced part of Third Napa Slough. It has been recommended (NSMR Feasibility Study Phase 1B, PWA 2001) to ideally carry out a cross section survey in Third Napa Slough in the area between the limit of the current surveys and the limit of the tidal influence. However, a reasonable approximation of the tidal prism contributed by Third Napa Slough can be obtained using the method described previously. Additional cross section geometry is being collected as part of the NSMR Feasibility Study Phase 2 Stage 1 (PWA, *in progress*).

7.1.1.3 Change in Tidal Prism in Pond 2A

Pond 2A was not surveyed with ground surveys and cross sections since it is already restored to tidal action and therefore photogrammetry was used to map this area. This method of mapping is less accurate than ground surveys but in order to understand potential inaccuracies the sensitivity of the model to changes in the tidal prism two simulations have been carried out. These represent respectively a 25%

increase and a 25% decrease of the tidal prism of Pond 2A to investigate the effect on the hydrodynamic of uncertainties in the measured bathymetry of Pond 2A. The results in terms of time series of inflow from South Slough to Pond 2A are shown in Figure 7-18, illustrating the influence is limited. However, it is worth mentioning that the modeled flow in South Slough is dominated by the presence of Pond 2A and the opening to the pond. During periods of rising tidal water level the modeled flow direction in the eastern part of South Slough (to the east on the opening to Pond 2A) is from Napa River towards the opening. To the west of the opening the flow direction is opposite. This means that in the model the water flowing into Pond 2A is supplied from each end of South Slough. In the model approximately 80% is supplied from east and 20% from west. Similarly, during periods of decreasing tidal water level, Pond 2A drains through both ends of South Slough. If the opening were closed the flow direction in South Slough would be the same throughout, and Pond 2A does as such have a significant influence to the flow pattern in South Slough.

7.1.1.4 *Changes in Manning's "n"*

To assess the impact of changes in Manning's n a series of simulations were carried out using different values for Manning's n. The resistance throughout the model was altered in a series of steps to investigate the models sensitivity to variations in Manning's n. The model was run with the resistance being altered by 30%, 20% and 10% both as an increase and a decrease. As a way of displaying the effect of varying Manning's n, the simulated mean at the measurement stations was determined and the result plotted as a function of the variation in n as shown by Figure 7-19. As is evident from this plot a variation of +/- 30% in bed resistance only results in a variation of the water level in the order of 1-3cms for all the interior slough channels. Only SOCR deviates from this pattern. Thus apart from the upstream part of Sonoma Creek the flow is controlled by the water level at the inlets and not the resistance in the rivers and sloughs. In other words the discrepancies between the measured and simulated data cannot be compensated by varying the bed resistance. This is even more evident in Figure 7-20 where the mean deviation from the measured water level is plotted. The slope of these lines indicate that to compensate for the problems with the datum would require unphysical values of the bed resistance which in turn would undermine all the other calibration objectives listed in Section 6.1.4.1.

7.1.2 Salinity

In MIKE 11 the transport of a solute such as salinity is calculated as the sum of two methods of transport:

1. The advective transport being a result of the solute moving with the water in the direction and at the velocity of the average flow. The advective transport is thus only a result of the hydrodynamic conditions and not directly influenced by any of the advection-dispersion parameters.
2. The dispersive transport, which is calculated as a dispersion coefficient multiplied by the longitudinal concentration gradient. In a 1-D model this coefficient is a lumped description encompassing all mixing mechanisms.

There are other processes in tidal flows. For example the ebb tide may put some water back where it started or may spread it out (tidal trapping, tidal pumping etc).

The dispersion coefficient is the only parameter that is used for calibration of an advection-dispersion model, and selecting the right value is an important part of calibrating any advection-dispersion model. The dispersion coefficient can be looked upon as a measure of the degree of potential mixing in the fluid. The effect of varying the dispersion coefficient D is sketched in Figure 7-21 where an idealized situation with no tidal exchange is considered. The upper half shows the physical system and the lower half shows the concentration profile for different choices of the dispersion coefficient. The figure illustrates three main types of mixing:

- $D=0$. no mixing takes place and thus a steep gradient would be present from the bay into the slough
- D intermediate. The mixing would take its course over some time.
- D is very high. The mixing takes place very rapidly.

In summary the dispersion coefficient is a measure of how quickly concentration gradients are smoothed out.

Whether advection or dispersion is the dominating process depends on the flow velocity and the selected dispersion coefficient. In general lower velocities make dispersion more important and higher velocities makes advection more important. The latter could for instance be in those parts of the slough channels where the velocity is constantly low because the tidal influence from Napa River and Sonoma Creek is equally strong. Advection typically dominates in Napa River and Sonoma Creek and in the downstream extents (i.e. close to the confluence with either Napa River or Sonoma Creek) of the slough channels.

Sensitivity analyses were undertaken using three different dispersion coefficients; 10, 50 and $200\text{m}^2/\text{s}$. Figure 7-22 shows a longitudinal profile of the maximum simulated salinity during Sep 97 - Mar 98. The profile starts at the down stream end of Napa River goes through Dutchman Slough, South Slough, Napa 1st Slough and ends at the down stream end of Sonoma Creek. Figure 7-23 shows the time series of simulated salinity at the confluence of South and China Sloughs. Here there is no output from the ponds, the salinity gradient is due entirely to tides and river inflows.

The fact that higher dispersion coefficients give higher dispersion is typically visible when a front of saline water moves into the rivers and slough channel as the tide in San Pablo Bay increases. Higher dispersion coefficients results in increased mixing of the water when the saline front comes in, and thus a less abrupt increase in the salinity and lower salinity levels at interior ponds and higher salinity levels in the peripheral sloughs close to the Bay. In general, this can be expressed such that high dispersion gives lower maximum salinity and higher minimum salinity. This can also be seen from the times series plot. However, in some areas in the slough channels close to the zero-velocity points, the advective transport of salinity is less than the dispersive transport and here the influence of increasing the dispersion coefficient is opposite.

7.1.3 Sediment Transport

To test the sensitivity of the 1-D sediment transport model to deviations from the selected values and potentially refine the calibration, a limited set of parameters were selected for further examination. First, assuming that τ_{cd} was reasonably approximated, ω_0 was evaluated at 0.01 and 0.1 mm/s. Second, τ_{ce} and M^* were changed concurrently to provide visual fits to DeVries field data; referring to Figure 7-24, with τ_{ce} at 0.10 and 0.05 N/m², M^* should concurrently be changed to 0.0035 and 0.0017 g/m²-s, respectively.

Table 7-1 below displays the sensitivity of SSC to adjustments in a select set of parameters. Most noticeably is the influence of decreasing the settling velocity on SSC in Run 2. In the Runs 4 and 5, there was not much of a decrease in SSC, as expected, since those parameter values are variations of a best fit to the observed erosion data in Figure 7-24. An overall decrease in SSC is achieved because at a given bed shear stress the rate of surface erosion is smaller.

Table 7-1. Cohesive Sediment Transport Parameter Sensitivity

Run	Sensitivity Scenario			Percent Change in SSC ¹
	ω_0	τ_c	M^*	
1	0.00005	0.144	0.00625	---
2	0.00001	0.144	0.00625	+300
3	0.00010	0.144	0.00625	-40
4	0.00005	0.100	0.00350	-40
5	0.00005	0.050	0.00170	-10

¹ Duration of selected results: low flow period 11/20/97 to 1/20/98 during which bed sediment has not been depleted

Additionally, it has been determined that the model is sensitive to specification of the volume of bed sediment per unit reach length; as the volume increases, it should be expected that the simulated SSC to increase. Problems were also encountered during the high flow period with the eventual depletion of the sediment bed in many locations. It is recommended that for future sediment transport modeling:

- A mass balance should be conducted within each reach and at the boundaries to determine why the bed sediment depletes over time.
- The range of simulated bed shear stresses be reviewed and compared to Figure 7-24 in order to determine if a non-linear relationship between bed shear stress and erosion rate is more appropriate (i.e. $n \neq 1$).
- Assess the influence of specifying the initial SSC versus allowing the model to spin-up from a near-zero SSC.
- Use multilayer sediment transport to better represent the physical processes involved in the re-suspension and transport of cohesive sediments.
- Develop a better understanding of cohesive sediment transport parameters through some additional field data collection and model calibration.

7.2 TWO DIMENSIONAL MODELING

7.2.1 Grid Dependency Investigation: Pond 2A

To evaluate the results' grid dependency, three simulations were carried out using three different grid sizes. The simulations were carried out on a submodel consisting of Pond 2A. The flow was driven by sinks and sources placed at the breach points. The discharge time-series used as input for these sink/sources were extracted from the fully coupled simulation.

The grid sizes used were:

- 10 m x 10 m.
- 15 m x 15 m.
- 25 m x 25 m.

Figure 7-25 and 7-26 show iso-surfaces of velocity plotted for each of the three simulations.

As expected the simulation using the finest grid gave rise to a higher degree of detail of the flow-field. The difference between the results using the highest resolution and the results using the 15 m x 15 m grid are more or less indistinguishable. The results using the coarse grid do not display as many detailed features of the flow but adequately predicts the general velocity field.

Further, the cost in terms of computational time is crucial for the choice of grid. Comparing the computational time using the finest grid (10 m x 10 m) with the computational time of the more coarse grid (25 m x 25 m) one finds that the factor is

$$(2.5)*(2.5)*2.5 = 15.625$$

where the last factor (2.5) is due to the decrease in the time-step caused by the finer mesh.

Thus given the overall flow picture using the coarse grid and the computational time involved justifies the use of the 25 m x 25 m grid to identify the dominant processes.

8. MODEL OF EXISTING “WITHOUT PROJECT” CONDITIONS

8.1 INTRODUCTION

To satisfy the requirements of an Environmental Impact Report and Statement (EIR/S), the operation of the ponds under the present management strategy (or “Without Project” conditions) must be defined. There are presently three main situations that could occur at the site:

1. A catastrophic breach may occur under either a Napa River Flood scenario (as is most likely) or at low Napa River flows due to wind wave erosion. This scenario is termed the “Levee Breach”.
2. A situation could occur where the water conveyance structures at the site do not operate such as blockage of siphons due to a build up of a saline wedge in the pipe invert. This situation has occurred at the site and is presently preventing flow of water between Ponds 3 and 4 and Ponds 5 and 6. This scenario is termed the Non-Operation Scenario.
3. The situation that has been used for the last six years since the CDFG has owned and operated the ponds consists of two pumping options to move water through the system of ponds, siphons and canals from the north and the south of the site in an attempt to manage and maintain lower salinity levels in the ponds. This scenario is termed the On-going Maintenance Scenario.

8.2 LEVEE BREACH SCENARIO

Many sections of the levees surrounding the existing salt ponds have become weakened due to age and the effects of wind-wave erosion. Several sections have become critically weak as surveyed by the USACE (Ken Harrington, USACE, personal communication). One example of a critically weakened section was identified on the East levee of Pond 4.

For the purposes of this scenario, the 1-D model of the rivers and slough channels was coupled with a 2-D model of Ponds 4 and 5. A 45 m (150 feet) breach was represented between Pond 4 and the Napa River by removing land computational cells in the 2-D model domain and dynamically linking the adjacent computational cells to the 1-D river model. The breach elevation was fixed at 0.3 m (1 foot) below the average bed elevation of Pond 4 at the breach location to represent a degree of scour. The coupled model was run for a two-month simulation time period; September 22, 1997 to November 21, 1997.

8.2.1 Levee Breach Scenario Results

Time series of the flows passing through the breach in comparison to the flows in the Napa River are shown by Figure 8-1 and 8-2. A root-mean-square (RMS) analysis of the flows passing through the breach in Pond 4 and in the Napa River indicated the following ratio:

$$\begin{array}{lcl} \text{RMS Flow Napa River (RMS } Q_{NR}) & : & \text{RMS Flow (RMS } Q_{BREACH P4}) \\ \text{RMS } Q_{NR} = 622 \text{ m}^3/\text{s} & : & \text{RMS } Q_{BREACH P4} = 47 \text{ m}^3/\text{s} \\ \text{RMS } Q_{NR} : \text{RMS } Q_{BREACH P4} & = & 13 : 1 \end{array}$$

The results of the salinity transport modeling are shown by Figures 8-3 to 8-8. Figure 8-7 shows that the salinity in Pond 5 reduces gradually from approximately 175ppt to approximately 43ppt over a two-month period. The salinity in Pond 4 reduces more rapidly from approximately 165ppt to approximately 30ppt over a 10-day period. This is because of the efficient mixing processes that occur in Pond 4 due to the proximity of the levee breach and the limited mixing between Ponds 4 and 5 due to the 12-ft breach in the dividing levee. In addition the diurnal variability of the salinity in Pond 4 is significantly greater than in Pond 5 due to the attenuation of the tidal signal that occurs between Pond 4 and Pond 5. Figures 8-3 to 8-6 show more clearly the rate and process of mixing.

It is also significant to observe the effects of the discharge of higher salinity water from Pond 4 on the Napa River as shown by Figure 8-8. Notice that the background salinity in the Napa River has a diurnal variation due to tidal fluctuations. During an ebb tide the salinity is reduced (or a diurnal minimum) in the Napa River; during a flood tide the salinity is increased (or a diurnal maximum) in the Napa River. This variation is typically in the order of 2 to 4ppt during the period of interest. In contrast notice the Napa River salinity near the breach. During an ebb tide the salinity near the breach is at or approaching a diurnal maximum in opposition to the background salinity of the Napa River which is at or approaching a diurnal minimum. This is because flows out of Pond 4 during an ebb tide are at or approaching a maximum and hence the concentration of high salinity water mixing with lower salinity Napa River water is at or approaching a maximum. During a flood tide the opposite effects in concentrations of salinity are observed.

The fluctuation in the salinity near the breach is greatest just after the initial breaching of Pond 4 and reduces as the salinity of Pond 4 reduces. Differences in Napa River salinity near the breach and background salinity typically range from 12 to 3ppt over the duration of the simulation.

Figures 8-9 and 8-10 show longitudinal profiles of salinity in the Napa River, 6 hours, 3 days, 1 week and 1 month after the initial breach respectively. The maximum salinity can be observed to occur near the breach shown by the localized spike in the salinity profile as expected but reduces with increasing distance away from the breach. In addition, as releases continue from the Pond 4 breach with respect to time, the zone of influence of discharge of higher salinity water into the Napa River increases upstream and downstream.

It should be noted that the salinities in the Lower Napa River will show a significant lateral gradient and the average value near the breach is not representative of what may be observed. Concentrations could be significantly higher close to the breach.

8.2.2 Levee Breach Scenario Implications

Salinity maxima in the Napa River could be higher than predicted due to different breach conditions and/or over estimation of the mixing due to 1-D modeling of the Napa River. The implications of the results previously described are significant to the localized salinity levels in the Lower Napa River. During the low flow period in the Napa River analyzed here, increases to the salinity could be considered

to be excessive and unacceptable. However it is likely that during higher winter flows in the Napa River, the implications to salinity levels would be less severe. Therefore the implication that can be drawn from the analyses shown here are that accidental levee breaches of ponds in excess of 150ppt should be avoided with strengthening of weakened levees until salinity reduction strategies have been implemented and are successful. The biological implications of such a breach are not the focus of this study.

8.3 NON- OPERATION SCENARIO

For the purposes of this scenario, a simple spreadsheet analysis for Ponds 1 to 8 was formulated. The spreadsheet analysis simulated the time taken for each pond to evaporate (although minimal ponding may occur) assuming no flow passing between the ponds via pumped flow. The spreadsheet analysis accounted for average daily precipitation and evaporation rates were adjusted dependent upon pond salinity (as described in Section 5.1.9). Details of the spreadsheet analysis are described as follows.

Inputs into the model:

- Monthly precipitation and evaporation values developed as part of this study (Section 5.1.9).
- Water surface elevation-volume-area tables for each pond as computed by the USGS from Towill AutoCAD surface model (April 11, 2001).
- Initial pond depth and salinity measurements by CDFG (Tom Huffman during February 1998 and March 2001) as shown by Table 8-1.
- Salinity coefficients for evaporation, as computed as part of this study (Section 5.1.9 and Appendix 1).

Output from the model:

- Total time for a specified pond to dry out when pumps stop operating (due to pump failure or siphon blockages). When the pumps are not operating the only pond input/output is precipitation/evaporation. Each pond is analyzed under two different initial conditions: February 1998 and March 2001. The spreadsheet model yields time, in months, for water surface elevation to drop below mean bed elevation. Thus, the ponds will still have disconnected puddles at this point, with highly saline water. Also, should precipitation or seepage occur through levees, it can be expected that the water level could rise and fluctuate in future months if the simulation period was extended. Levee seepage will increase as the water surface elevation between the sloughs and the river increases.

Table 8-1. Starting Water Surface Elevations and Salinities

Pond	Water surface elevation Feb. 98 (NAVD88 m)	Salinity Feb. 98 (ppt)	Water surface elevation Mar. 01 (NAVD88 m)	Salinity Mar. 01 (ppt)
1	2.034	5	1.820	9
1A	2.034	17	1.820	10
2	2.308	17	1.790	6
3	2.278	23	1.942	29
4	2.247	62	1.363	166
5	2.247	67	1.363	173
6	2.308	27	1.577	65
6A	2.278	27	1.546	55
7	2.064	135	1.455	323
7A	2.125	46	1.698	48
8	1.973	86	1.424	226

8.3.1 Non-Operation Scenario Results

Tables 8-2 and 8-3 show summary tables of the time for Ponds 1-8 to fall below mean bed elevation during a pump failure starting in February 1998 and March 2001 respectively.

Table 8-2. Existing Conditions: Non-Operation - February 1998 Starting Conditions

Pond	Time (months) for water surface to dry to below mean bed elevation (minor/disconnected ponding)*
1	19
1A	20
2	31
2A	Tidal
3	20
4	33
5	44
6 & 6A	31
7	57
7A	32
8	93

* Assuming mean monthly precipitation and neglecting seepage through levees as the water level in the ponds drops

Table 8-3. Existing Conditions: Non-Operation–March 2001 Starting Conditions

Pond	Time (months) for water surface to dry to below mean bed elevation (minor/disconnected ponding)
1	8
1A	7
2	18
2A	Tidal
3	18
4	7
5	8
6 & 6A	7
7	44
7A	18
8	Never dries out

Figure 8-11 shows graphs of the evaporation histograms for the ponds to empty.

8.3.2 Non-Operation Scenario Assumptions and Accuracy

The model provides an estimate of the “worse case scenario” of pump/siphon failure, based on monthly evaporation and precipitation for an average water year. The ponds are not actually dry when the water surface elevation falls below the mean pond bed elevation but most likely there are discontinuous, highly saline “puddles” at that point (i.e. ponding). The results are intended to guide broad-scale pond management. This model does not attempt to predict future conditions or simulate real events.

8.3.3 Non-Operation Scenario Implications

The results shown by the scenario described here imply that maintenance of the siphons and pumps of the pond system is vital for habitat and salinity management in the interim period until the ponds are restored. The water levels and salinities in February 1998 could sustain pond habitat for significant periods of time, in excess of two years in most cases. However, recent water levels and salinities as measured in March 2001, represent critical levels of maintenance with at least five of the ponds highly likely to dry out within one year if pumps or siphons fail. These results are reinforced with reference to recent actual events. CDFG experienced management problems in the early months of 2001 with blocked siphons between Ponds 3 and 4 and Ponds 5 and 6. This prevented CDFG from moving water into Ponds 4 and 5 from Ponds 3 and 6 respectively. By September 2001 Ponds 4 and 5 were completely empty with only a precipitated salt crust on the bed of the ponds. This result would indicate that there is minimal seepage through the levees with these hydrologic conditions.

8.4 ONGOING MAINTENANCE SCENARIO

The ongoing maintenance scenario involved the use of the full 2-D model of all the Ponds 1 to 8, to simulate the present maintenance regime for the salt ponds. The results give indications as to the effectiveness of the maintenance regime to reduce (or dilute) salinities within the ponds. Three simulations were undertaken:

- Flows pumped at 1.89 m³/s (30,000 gallons/minute) from Pond 1 via a donut structure and a siphon under South Slough into Pond 2. *South O/M* simulation.
- Flows pumped at 0.38 m³/s (6,000 gallons/minute) from the Napa River via a canal running to the north of Pond 8, through a siphon under Mud Slough, into a canal bypassing Ponds 7 and 7A and through a siphon under Napa Slough into Pond 6A. *North O/M* simulation.
- Flows pumped from both North and South pumping sites. *North & South O/M* simulation.

8.4.1 Ongoing Maintenance Scenario Results

Figures 8-12 to 8-23 show the variation in water surface elevation and salinities in the ponds with respect to the time of the pumps operating in the north and south of the site and when operating simultaneously. The results shown in the Figures can be summarized in Table 8-4.

Table 8-4. Ongoing Maintenance Results Summary

Pond	Season	Pump Operation	Water Surface Elevation	Salinity	Comments
2	Summer	(N)orth	Drops rapidly initially then tidally influenced	Rises gradually from 10 to 25 ppt	Negligible influence from North. Evaporation rate marginally > pump + precipitation
		(S)outh	As N	As N	As N. Greatest influence from South.
		N & S	As N	As N	As N
2	Winter	N	Drops rapidly initially then tidally influenced	Rises gradually from 8 to 18 ppt	Evaporation rate marginally < pump + precipitation
		S	As N	Rises gradually from 8 to 16 ppt	As N
		N & S	As S	As S	As N
3	Summer	N	Drops gradually	Rises parabolically from 25 to 250ppt	Evaporation rate >> pump + precipitation

Pond	Season	Pump Operation	Water Surface Elevation	Salinity	Comments
		S	As N	As N	As N
		N & S	As S	As S	As N
3	Winter	N	Drops initially then rises gradually at a higher rate than S	Rises gradually from 30 to 70 ppt	Higher salinity water moving from 4 into 3
		S	Drops initially then rises gradually at a lower rate than N	Rises only marginally from 30 to 35 ppt	Evaporation rate < pump + precipitation
		N & S	As N	As N	Higher salinity water moving from 4 into 3
4	Summer	N	Drops gradually	Rises gradually from 165 to 350 ppt	Evaporation > pump + precipitation
		S	As N	As N	As N
		N & S	As N	As N	As N. N & S both influence 4.
4	Winter	N	Drops initially then rises gradually	Rises initially from 165 ppt then drops steadily to approximately 175 ppt	N has slightly greater influence than S. Evaporation < pump + precipitation
		S	Drops for significantly greater extent than N then rises marginally	Similar to N but slightly less influence	As N
		N & S	Drops initially then rises gradually at a faster rate than N alone	As N	As N
5	Summer	N	Drops gradually at approximately constant rate	Rises gradually from 175 to 300 ppt	N has greatest influence on 5. Evaporation > pump + precipitation
		S	Drops at a greater rate than N	Rises at an increased rate to N from 175 ppt to approximately 350 ppt	As N. S has negligible influence on 5.
		N & S	As N	As N	As N

Pond	Season	Pump Operation	Water Surface Elevation	Salinity	Comments
5	Winter	N	Drops initially then rises gradually	Rises initially from 175 ppt then drops steadily to approximately 140 ppt	Significant influence from N. Evaporation rate < pump + precipitation
		S	Drops for significantly greater extent than N then rises marginally	Rises gradually from 175 ppt then drops slightly to 190 ppt	Negligible influence from S
		N & S	Drops initially then rises gradually at a marginally faster rate than N alone	Rises initially from 175 ppt then drops steadily to approximately 140 ppt	Marginal additional influence from S
6	Summer	N	Drops gradually	Rises gradually from 65 to 155 ppt	Significant influence from N. Evaporation rate > pump + precipitation
		S	Drops at a greater rate than N	Rises at an increased rate to N from 65 ppt to approximately 250 ppt	Negligible influence from S
		N & S	As N	As N	As N
6	Winter	N	Drops initially then rises gradually	Rises initially from 65 ppt then drops gradually to approximately 63 ppt	Significant influence from N. Evaporation rate < pump + precipitation
		S	Drops gradually with a slight rise latterly	Rises gradually from 65 to 73 ppt	Negligible influence from S
		N & S	As N but slightly higher elevation due to S	As N	As N
6A	Summer	N	Drops gradually	Rises gradually from 55 to 110 ppt	Significant influence from N (greater than 6). Evaporation rate > pump + precipitation

Pond	Season	Pump Operation	Water Surface Elevation	Salinity	Comments
		S	Drops at a greater rate than N	Rises at an increased rate to N from 55 ppt to approximately 250 ppt	Negligible influence from S
		N & S	As N	As N	As N
6A	Winter	N	Drops initially then rises gradually	Rises initially from 55 ppt then drops gradually to approximately 58 ppt	Significant influence from N. Evaporation rate < pump + precipitation
		S	Drops gradually with a slight rise latterly	Rises gradually from 65 to 70 ppt	Negligible influence from S
		N & S	As N but slightly higher elevation due to S	As N	As N

Note: Summer = July and August, 2001, Winter = October and November, 2000

8.4.2 Ongoing Maintenance Scenario Implications

The implications of the ongoing maintenance scenario can be most efficiently described with comparison to Table 8-5. Table 8-5 shows results derived from the spreadsheet model described in Section 8.3. Approximate net flow rates in or out of the ponds due to the balance between evaporation and precipitation were calculated assuming no pump induced flows between ponds.

Comparisons of the results shown by Table 8-5 with Table 8-4 and Figures 8-12 to 8-23 indicate qualitatively the benefit that operation of the pumps in the North and South has on the maintenance of water levels and salinity. It can be observed that during the summer months simulated here, it is unlikely that operation of the pumps can counterbalance evaporation and reduce salinities. The net outflow of water due to evaporation and precipitation from the Ponds simulated during July and August 2001 was 1.2 m³/s compared to the theoretical maximum combined rated flow generated by the pumps in the North and South of 2.1 m³/s. Losses due to leaking siphons, pump inefficiency and losses between ponds, canals and other water control structures probably significantly reduce this theoretical maximum flow and hence management attempts in the summer months to reduce salinities and maintain water levels are less successful.

Table 8-5. Net Flow Rates into / out of Ponds due to Evaporation and Precipitation

Pond	Season	Water Surface Elevation Change (mm)	Net Flow Rate (-'ve out of pond) (m³/s)	Change in Salinity (ppt)
2	Summer	-323	-0.190	13 to 24
	Winter	-43	-0.025	No significant change
3	Summer	-316	-0.315	29 to 51
	Winter	-41	-0.041	No significant change
4	Summer	-281	-0.201	166 to 438
	Winter	-29	-0.021	No significant change
5	Summer	-280	-0.168	173 to 444
	Winter	-28	-0.017	No significant change
6	Summer	-307	-0.104	65 to 157
	Winter	-37	-0.013	No significant change
6A	Summer	-307	-0.173	55 to 142
	Winter	-38	-0.022	No significant change

Note: Summer = July and August, 2001, Winter = October and November, 2000

However, management attempts to reduce salinities and maintain water levels in the ponds are significantly assisted by increased precipitation during the winter months. Water levels in the ponds do not reduce to the same extent under non-operation of the pumps although there still exist a net outflow of water by evaporation at a reduced rate and water levels increase with various combinations of the pump operations. In addition, salinities in the ponds can be actively reduced during the winter months with pump operation, since precipitation alone largely maintains salinities to a constant level during these months.

In terms of the relative influence of the pumps on salinity and water levels in the ponds, clearly the South pump influences the southerly ponds to the greatest extent, particularly Ponds 2 and 3 and to a lesser extent 4. The North pump influences Ponds 6 and 6A to the greatest extent and to a lesser extent Pond 5. The North pump has a progressively negligible to zero effect on Ponds 4, 3 and 2, respectively.

In summary the management of salinities and water levels in the ponds is largely dependent upon the seasonal balance of evaporation and precipitation. Operation of the pumps in the North and South of the site is not sufficient in isolation to control salinities and water levels. The effects of the pumps merely

reduce the rate at which pond elevations drop or salinities increase and during the summer months the effect of pumping is negligible. In addition, in summarizing the results of this analysis it is assumed that the pumps in the North and South run constantly. In practice this is not the case since CDFG can only afford to operate the pumps at certain times of the year when funding permits and this further minimizes the effect of pumping.

9. GEOMORPHIC ANALYSIS AND EMPIRICAL PREDICTORS

Hydraulic geometry relationships developed by PWA for the San Francisco Bay area were used to predict potential changes in the Napa slough channels in response to restoration. These relationships and the assessment of geomorphic evolution of the slough channels are described below.

9.1 HYDRAULIC GEOMETRY

Hydraulic geometry relationships are empirical relationships between tidal flow and channel dimensions, such as depth, top width, and cross sectional area. For this assessment, tidal flow is represented by “tidal prism,” the volume of water stored between MHHW and MLLW upstream of a given location. In this definition, the tidal prism is defined as “potential” and assumes the water upstream of the location will rise and fall to MHHW and MLLW at the downstream end of the slough channel. Hydraulic geometry has been used as a geomorphic design tool in numerous San Francisco Bay salt marsh restoration plans. The application of these data is regional.

The hydraulic geometry regression relationships are shown on Figure 9-1. The database used to create the relationships includes channel cross sections and tidal prism for six existing and six historic San Francisco Bay marshes. For this study, an existing hydraulic geometry database was expanded to include data from historic channels in the Napa system (SFEI, 2000), as well as other historic San Francisco Bay marsh channels. In this way, the database was expanded to include larger marshes.

Channel cross section data for the larger historic marsh channels were obtained from nineteenth century U.S. Coast and Geodetic Survey charts. Channel top width was measured from the topographic maps and channel depth calculated from the bathymetric maps. In cases where the historical survey shows only one channel depth (mid-channel), we assumed that the thalweg was located mid-channel and we calculated cross sectional area assuming an elliptical cross sectional shape. Historical mapping was not sufficiently detailed to estimate tidal prism based on bathymetry. Instead, tidal prism was extrapolated based on a correlation of marsh area and potential tidal prism (Figure 9-2) using the existing ancient marsh channel data and historical tidal flow data from Ravenswood Slough (Gilbert, 1917). Tidal prism was calculated as $935 \times (\text{marsh area})^{1.17}$, where marsh area is in hectares and tidal prism in cubic meters.

9.2 ESTIMATION OF THE GEOMORPHIC EVOLUTION OF SLOUGH CHANNELS

Hydraulic geometry relations characterize the morphology of two representative slough channels (Dutchman Slough and South Slough downstream of the China Slough confluence) for both post-restoration and long-term future conditions. A simplified restoration scenario in which Ponds 3, 4, 5, 6, 6A, and Cullinan Ranch are all breached at the same time (i.e., a “No-Phasing” Alternative) at historic channel locations (rather than at the Napa River or San Pablo Bay) was evaluated.

9.2.1 Short-term / Post-restoration Morphology

Estimates of potential diurnal tidal prism following restoration were made for each slough. PWA calculated the volume between MLLW and MHHW in each pond from an analysis of hypsometric data (pond area versus elevation, see Figure 9-3) developed from Towill survey and DTM data. The resulting tidal prisms are listed in Table 9-1. The existing tidal prisms in Dutchman and South Sloughs were calculated by integrating discharge output from a MIKE 11 hydrodynamic model of existing conditions over several representative tide cycles, giving an average existing tidal prism of 2.5 million m³ for South Slough and 1.3 million m³ for Dutchman Slough.

Table 9-1. Estimates of Post-Restoration Potential Diurnal Tidal Prism for Each Restored Pond

Restored Pond	Potential Diurnal Tidal Prism (m³)
Pond 3	3,800,000
Pond 4	3,500,000
Pond 5	2,980,000
Pond 6	2,620,000
Pond 6A	1,540,000
Cullinan Ranch	9,350,000

The total post-restoration potential diurnal tidal prism was estimated by portioning the tidal prism of the ponds based on an analysis of existing slough channel hydrodynamics and adding this portion of tidal prism to the existing tidal prism in the sloughs. As a rough approximation, the portioning of potential diurnal tidal prism within the ponds was done as follows:

- 100% of the tidal prism from Ponds 4 and 5; 50% of the tidal prism from Ponds 3, 6, and 6A; and 10% of the Cullinan Ranch tidal prism flow through South Slough (and via China Slough).
- 90% of the Cullinan Ranch tidal prism and 50% of the Pond 3 tidal prism flow through Dutchman Slough.

The resulting potential diurnal tidal prism following restoration is 13.1 million m³ for South Slough and 11.6 million m³ for Dutchman Slough.

These tidal prism volumes were input to the hydraulic geometry relations between channel depth, width, and cross sectional area versus tidal prism (Figure 9-1) to calculate the following estimated short-term slough channel geometry:

- South Slough: 280 m width at MHHW, 6.9 m depth below MHHW, and 1,200 m² cross sectional area below MHHW.
- Dutchman Slough: 270 m width at MHHW, 6.8 m depth below MHHW, and 1,100 m² cross sectional area below MHHW.

This geometry is graphically shown along with predicted long-term future geometry in Figures 9-4 and 9-5. Figures 9-4 and 9-5 indicate the predicted channel cross sections are inconsistent with existing levee locations (see discussion below). This condition was not considered when apportioning flow.

9.2.2 Long-term Future Morphology

Assuming that the diked ponds are restored by breaching along the internal slough channel network at the locations of historic channels, the long-term future morphology or evolved condition is expected to be similar to historic slough morphology. Thus, the best estimate of long-term future morphology is made from historic morphology, presuming breaching at historic locations. A comparison of existing and historic slough channel geometry was conducted.

Slough channel cross section surveyed by Towill and digital elevation model (DEM) data in conjunction with a GIS database based on a San Francisco Estuary Institute overlay of the historic map of the Napa salt marsh (SFEI, 2000 and USCGS, 1860) over recent aerial photography (Figure 9-6) were used and the PWA hydraulic geometry database to perform this comparison. Existing slough channel cross sections were taken from the Towill data at South Slough just downstream of the confluence with China Slough and just upstream of the mouth of Dutchman Slough. The historic widths (at MHHW and MLLW) of the sloughs at these locations were digitized from the historic map in the GIS database. The hydraulic geometry relationships between equilibrium tidal prism and both channel width and depth were manipulated to yield the following relationship for equilibrium channel depth as a function of width:

$$0.807 \times (\text{channel top width})^{0.382}, \text{ where both channel depth and width are in meters.}$$

This relation was used to estimate the depth of the historic channel from the measured historic channel width. Figures 9-4 and 9-5 show a graphical comparison of existing and historic channel cross sections and estimation of long-term future channel morphology and evolution.

9.3 DISCUSSION

The results of this analysis suggest that the slough channels are significantly undersized compared to potential tidal flows. Therefore, the potential exists for a combination of significant channel erosion and tidal damping for the simplified restoration scenario evaluated. If the ponds are breached, the change in the slough channel cross section may be such as to significantly increase the risk of levee failure.

The magnitude of the increase in potential diurnal tidal prism and predicted channel geometry, especially in the short-term, indicate that the evolving morphology of the interior slough channel network will be highly dynamic. For this reason, the accuracy of morphological predictions based on the empirical relations for equilibrium channels may be limited. Restoration experience at Sonoma Baylands and Warm Springs has shown that slough channels responding to a large increase in upstream tidal prism tend to down-cut and deepen first and then widen by the slump-block erosion of the channel edge (Williams *et al*, in press). Therefore, it may be that the evolving slough channels never reach the predicted short-term width and deepen below the predicted depth to achieve an increase in cross sectional area. Erosion is also

likely to occur preferentially along the side of the channel where new marsh has formed over the last 150 years because this young marsh is less consolidated than the side of the channel adjacent to the more compacted levee. However, results do indicate that there is the potential for levee erosion and failure.

Tidal muting caused by friction in the undersized existing channels is expected to limit the actual post-restoration tidal prism to a volume less than the predicted potential diurnal tidal prism. Predicted short-term channel geometry may therefore be over-estimated. Also, the empirically based hydraulic geometry estimates may not accurately capture the complex hydrodynamics within the looped Napa salt marsh slough channel network because the hydraulic geometry database consists of channel dimensions for enclosed tidal drainage areas only. Furthermore, the changes in slough channel hydrodynamics following restoration will depend on the elements of the restoration design such as breach phasing and location. For these reasons, the evolution of slough channel morphology may be more accurately evaluated by geomorphic evaluations and numerical modeling in future efforts.

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APPENDIX 1

Evaporation Rates and Salinity Coefficients

Look up table for converting fresh water (salinity=0) evaporation values (mm) to evaporation for saline water, where all other environmental factors (temperature, humidity, radiation, wind, etc) are constant.

Note: these conversion factors will yield theoretical estimates only.

Equations used to derive this table are from:

Applied Hydrology, Chos/Maidment/Mays, copyright 1988 McGraw-Hill

Millero and Poisson, 1981 found in Surface Water Quality Modeling, copyright 1997 McGraw-Hill

Salinity (ppt)	Conversion factor for fresh water evaporation	Salinity (ppt)	Conversion factor for fresh water evaporation	Salinity (ppt)	Conversion factor for fresh water evaporation
0	1.00	265	0.82	530	0.67
5	1.00	270	0.81	535	0.67
65	0.95	330	0.78	595	0.64
70	0.95	335	0.78	600	0.63
75	0.94	340	0.77	605	0.63
80	0.94	345	0.77	610	0.63
85	0.94	350	0.77	615	0.63
90	0.93	355	0.76	620	0.62
95	0.93	360	0.76	625	0.62
100	0.93	365	0.76	630	0.62
105	0.92	370	0.75	635	0.62
110	0.92	375	0.75	640	0.62
115	0.92	380	0.75	645	0.61
120	0.91	385	0.75	650	0.61
125	0.91	390	0.74	655	0.61
130	0.91	395	0.74	660	0.61
135	0.90	400	0.74	665	0.60
140	0.90	405	0.74	670	0.60
145	0.90	410	0.73	675	0.60
150	0.89	415	0.73	680	0.60
155	0.89	420	0.73	685	0.59
160	0.89	425	0.72	690	0.59
165	0.88	430	0.72	695	0.59
170	0.88	435	0.72	700	0.59
175	0.88	440	0.72	705	0.59
180	0.87	445	0.71	710	0.58
185	0.87	450	0.71	715	0.58
190	0.87	455	0.71	720	0.58
195	0.86	460	0.70	725	0.58
200	0.86	465	0.70	730	0.57
205	0.86	470	0.70	735	0.57
210	0.85	475	0.70	740	0.57
215	0.85	480	0.69	745	0.57

220	0.85	485	0.69	750	0.57
225	0.84	490	0.69	755	0.56
230	0.84	495	0.69	760	0.56
235	0.84	500	0.68	765	0.56
240	0.83	505	0.68	770	0.56
245	0.83	510	0.68	775	0.56
250	0.83	515	0.68	780	0.55
255	0.82	520	0.67	785	0.55
260	0.82	525	0.67	790	0.55

APPENDIX 2

Summary of Findings – Siphon Data Collection

Siphon / Location	Type / Parameter	Condition / Findings
Siphon 6	Canal to canal siphon	<ul style="list-style-type: none"> ▪ Controlled by pump rate at inlet. Stage-discharge not important provided current pump system is used. ▪ Siphon is probably leaking. ▪ Salinity level approximately same either side of siphon. ▪ Differential stage-discharge relation established at start up of pump.
Siphon 5	Canal to pond (donut) siphon	<ul style="list-style-type: none"> ▪ Controlled by pump rate at inlet of Siphon 6. Stage-discharge not important provided current pump system is used. ▪ Siphon is probably leaking. ▪ Differential stage-discharge relation established at start up of pump.
Siphon 3	Pond to pond siphon	<ul style="list-style-type: none"> ▪ No flow in Fall due to exceptionally high density gradient. Flow was observed in Spring prior to monitoring study when salinity gradient was much lower. ▪ Demonstrates that timing of circulation between ponds is critical in the early stages of the rehabilitation of the ponds.
Pond 6A	Salinity	<ul style="list-style-type: none"> ▪ Salinity reduction success due to pumping through Siphons 6 and 5 is marginal. ▪ Risk of increasing salinities exists – pumping saline solution into a closed system.
Pond 6A	Wind and seiche observation	<ul style="list-style-type: none"> ▪ Diurnal pattern to water surface elevation assisting in circulation in pond. ▪ Diurnal pattern is seasonal
Pond 3 and 4	Salinity	<ul style="list-style-type: none"> ▪ Salinity reduction technique ineffective due to seasonal blockage of siphon. ▪ Risk of increasing salinities exists – pumping saline solution into a closed system.
Pond 3 and 4	Wind and seiche observation	<ul style="list-style-type: none"> ▪ Diurnal winds create seiche pattern to water surface elevation assisting in circulation in pond. ▪ Diurnal pattern is seasonal
Pond 2	Wind and velocity observations	<ul style="list-style-type: none"> ▪ Wind induced velocities have a significant effect in internal circulation in relatively shallow pond (5-6 ft in depth).
Pond 2	Stratification	<ul style="list-style-type: none"> ▪ Mixing of flows over the depth is confined to a near-field region within 200 ft of the siphon. ▪ Far-field regions are well mixed.