RESOURCE
CONSERVATION DISTRICT
OF SANTA CRUZ COUNTY

Watsonville Sloughs Hydrology Study

Prepared For: **Santa Cruz Resource Conservation District**

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SUMMARY

The Project team has completed work on the Watsonville Sloughs Hydrology Study, a combined hydrologic monitoring and modeling project designed to produce data and modeling tools to better understand the hydrology and hydraulics of the Watsonville Sloughs watershed. The work plan included three core components: data collection, hydrologic modeling, and hydraulic modeling. The tools developed from the core work were then used in two specific applications: revision of the Slough system water balance and modeling of a range of selected management scenarios intended to demonstrate the utility of the tools.

Completed tasks and principal findings of the Study include the following:

Data Collection Program

- *Topographic survey*. Environmental Data Solutions (part of the project team) completed detailed survey work in the spring of 2012. This work included survey of 30 channel cross-sections, long profiles of the main Watsonville Slough channel from Highway 1 down to Beach Road on two separate dates, survey of all monitoring sites to the North American Vertical datum, surveys of ten stream crossings, and establishment of four benchmarks to aid future work.

- *Installation of monitoring equipment*. The Project team, assisted by staff from the Pajaro Valley Water Management Agency (PVWMA) installed two tip-bucket rain gages, three automated stage recorders, six stream gaging stations, and four shallow piezometers (for groundwater levels). Two of these sites (Shell Road at the downstream end of the system and Watsonville Slough at Highway 1 at the upstream end) were telemetered to provide near real-time data transmission. The team collected auxiliary hydrologic data (such as temperature and salinity) whenever practical. Data from on precipitation station was lost due to persistent equipment issues and one stream gaging station was lost due to vandalism.

- *Existing monitoring equipment*. The team, again assisted by PVWMA, monitored six additional sites previously instrumented by PVWMA. One of these sites (Watsonville Slough at San Andreas Road) was adapted to provide an important record of flow used in model calibrations and revisions to the Slough water balance.

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- *Field response*. The project team completed field visits on a total of 26 days over the course of the Study, including response during storm events and routine dry-weather monitoring/maintenance.

- *Channel long profile and sediment dams*. The main Watsonville Slough channel from Shell Road up to Highway 1 is frequently obstructed by sediment "dams" and dense aquatic vegetation the create major barriers to flow. The dams appear to form from sediment brought into the main channel by side drains or bank failures since. The lower Slough system is segmented into a series of pools controlled by the elevation of the sediment dams. The obstructions are responsible for the expanding inundation of the bottomlands in the Slough system and have likely persisted due to the curtailment of regular channel maintenance activities. In a number of locations the channel obstructions have led to an increase in upstream water surface elevations of two to three feet.

- *Precipitation during the Study period*. Excellent long-term precipitation data is available from the Watsonville Water Treatment Works (WTW) gage site. Mean annual rainfall at that location for the 30-year period from WY1981 to WY2010 is 23.3 inches. Both of the years of the data collection program (WY2012 and WY2013) were very dry, with annual rainfall totals of 15.0 and 13.9 inches respectively. All findings based on field data are therefore strictly only applicable to periods of lower than normal precipitation, although the Project team used accepted methodologies to extend the applicability of the findings wherever practical.

- *Downstream tailwater conditions and beach overtopping*. Continuous recording of water levels in the tidal reach of the Slough system west of Shell Road proved to be very important for understanding the dynamics at the downstream end of the system. The Slough system ends at the Pajaro Lagoon, which can be open or closed to tidal action depending on the configuration of its barrier beach and river flow. Information on water surface elevations and lagoon closure collected during the Study period was correlated to tidal gage data collected by the National Oceanic and Atmospheric Administration (NOAA) in Monterey Bay and Pajaro River flow data collected by the United States Geologic Survey (USGS).

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- *Beach overtopping events and seawater incursions*. The monitoring equipment recorded the timing and magnitude of beach overtopping events in January 2012 and November 2012 that resulted in flow reversal in the lower Slough system, with the former event propagating east to impact Harkins Slough. In both occasions significant flow bypassed the flapgates on the Shell Road culverts through vent slots located at an elevation of 7.5 feet. The January event was large enough to also overtop Shell Road (approximate elevation of 8.4 feet), and gaging records indicate that approximately 270 acre-feet of seawater flowed upchannel into Harkins Slough. The increase in ocean wave energy in the fall of each year coincides with the annual water elevations in the lower Slough system and leaves it susceptible to seawater incursions during large overtopping events. The same channel obstructions that restrict channel conveyance in the downstream direction function to limit upchannel flow susceptibility in the Watsonville-Struve branch of the system.

- *Flow patterns at the confluence of the Watsonville Slough channel and Harkins Slough*. Collection of water surface elevation data on a consistent datum (NAVD 88) provided insight into flow patterns at the confluence where the Harkins Slough branch of the system joins the main Watsonville Slough channel. Key findings include the fact that water surface elevations were almost always higher in the Watsonville Slough channel than in Harkins Slough, leading to persistent inflow to Harkins from leaking through and/or overtopping of the weir located at the confluence. Additionally, sediment dams and vegetation in the channel downstream of the Railroad crossing led to frequent and persistent overflow from the channel across the Knox Property and into Harkins Slough. The latter flow path accounted for as much as 500 acre-feet of runoff into Harkins from the Watsonville-Struve branch of the system. These findings are consistent with very low runoff rates in the Harkins Slough watershed, especially when compared to the Watsonville-Struve branch.

- *Stream gaging*. The data collection program compiled flow records at a total of six locations in the watershed, with five of the gages measuring inflow to the Study area from approximately 56% of the total watershed area. The sixth gage was near the downstream end of the system (San Andreas Road) and recorded outflow from roughly 95% of the watershed area. Though each year of the Study

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was dry, equipment was in place to measure runoff in the short-duration relatively wet periods in late spring and early winter of 2012.

- The flow record compiled at the two rural area upland gaging sites, Harkins Slough at Buena Vista Road and Gallaghan Slough below the landfill, showed low peak flow rates and very low runoff volumes. Runoff coefficients were 0.04 and 0.07 respectively, meaning only four and seven percent of the rainfall in each contributing watershed was converted into runoff.

- The record at one urbanized upland site, West Struve Slough at the Pajaro Valley High School, confirmed much higher peak flow rates and total runoff volumes, with significant and rapid response to even small rain events. This behavior is consistent with the much higher impervious cover in the contributing watershed. The runoff coefficient was 0.55, roughly an order of magnitude higher than the rural gages.

- The flow record compiled for the East Struve Slough at Main Street site showed atypically low peak flow rates and runoff volumes given the very high level of impervious cover upstream. The team hypothesizes that runoff is percolating upstream of the gage site and passing under Main Street as shallow groundwater flow. Future verification of runoff at this site is needed as the overall water balance for the Slough system cannot be closed using only the runoff volumes observed there.

- The flow record collected at Watsonville Slough at Highway 1 also showed lower than expected runoff rates given the upstream impervious cover. The runoff coefficient was 0.24 or roughly half that at the West Struve site. Further verification is needed at this site as well, since the observed rates are anomalously low.

- *Groundwater monitoring*. The Project team compiled shallow groundwater level records from the four piezometers installed as part of the Study. Measured levels were consistent with a shallow groundwater gradient toward the Sloughs in the Pajaro River floodplain. Signals associated with rain events were clear and drawdown during the dry season was generally steady, although increased groundwater levels from local irrigation applications were observed. The team

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concluded that direct interactions between the Slough channels and the local shallow groundwater table were likely not large enough to include in the hydraulic model, but would be accounted for in the hydrologic model's groundwater component.

- *Measurement of Shell Road pump capacity*. The team coordinated with staff from the County of Santa Cruz to complete measurements of the capacity of the two pumps at the Shell Road Pump Station at the downstream end of the system. The measured discharge capacity with both pumps running was 5,300 gallons per minute (11.8 cubic feet per second). This value is well below that previously estimated and constrains the quantity of outflow when gravity flow through the Shell Road culverts is impossible due to high tides or when the lagoon is closed.

Hydrologic modeling

- *Model type and methodology*. The Project team, in consultation with the Technical Advisory Committee (TAC), selected the U.S. Army Corps of Engineers' HEC-HMS software for the hydrologic modeling component of the work as it is available free of charge, is well-supported by the Hydrologic Engineering Center, and is fully compatible with the hydraulic model used. Model runs used the Soil Moisture Accounting methodology (SMA) to account for rainfall losses and the Clark unit hydrograph as the rainfall-runoff transform.

- *Sub-watersheds*. Model preparation included GIS work to delineate and compile hydrologic parameters for 13 sub-watersheds in the 11,867 acre overall watershed area. Each sub-watershed was further divided into basins on the basis of recharge potential and land use type. Data assembly followed a hierarchy proceeding from watershed delineation to characterization of groundwater recharge potential to identification of land use type to extraction of soil parameters and quantification of impervious cover. Methodologies and data sources are summarized in the Watsonville Sloughs Modeling Tools Users' Guide document so that future users can readily update and/or modify the model in a consistent manner.

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- *Model simulation period*. The Project scope called for continuous simulation modeling over an extended period of time to capture both the intra- and inter-annual variability in the hydrologic behavior of the Slough system. The team compiled and reviewed available precipitation data and selected the ten-year period from WY2003 to WY2012 as the most appropriate simulation period. This led to selection of the hourly rainfall and climate data collected at the California Irrigation Management Information System (CIMIS) gage 129 (located in the community of Pajaro) as the climate input record for the model. Applied irrigation was accounted for using irrigation records provided by the PVWMA.

- *Hydrologic model calibration*. The primary overall objective of the hydrologic modeling was to produce simulated runoff records for use in the hydraulic model. The Project team calibrated the model against observed runoff volumes, since instantaneous peak flows are less important given the extremely large storage volumes in the Sloughs. Calibration results based on overall discharge at San Andreas Road were very good. However, agreement between modeled and observed runoff volumes for the individual upland sites was varied. Detailed calibration for the upland sites should be revisited when data from a wider range of hydrologic conditions (e.g. average and/or wetter than average conditions) are available.

- *Hydrologic model results*. Important findings of the 10-year continuous simulation runs include:

    - Total runoff from land areas (e.g. non-ponded) of the watershed averages 4,260 acre-feet per year, with the median value markedly lower at 3,680 acre-feet.

    - Total annual runoff ranges from a low of 2,070 acre-feet (WY2007) to a high of 7,010 acre-feet (WY2010). Runoff in WY2012 was the second lowest in the 10-year period at 2,470 acre-feet and correlated very well with the field monitoring findings.

    - The Harkins Slough branch of the system comprises 59% of the total watershed area, but produces only 26% of the total watershed runoff on average.

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- The hydrologic model generates 58% of the total annual runoff from the three most heavily urbanized sub-watersheds, all of which drain to the Watsonville-Struve branch of the system.

- Annual runoff rates as a function of rainfall for each branch of the system fit very well to linear curves, but the curves are markedly different for the two branches. Runoff rates increase much faster with increasing rainfall in the urbanized Watsonville-Struve branch of the system.

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1 OBJECTIVES AND TECHNICAL APPROACH

1.1 Format of this Report

The overall structure of this report is intended to document the progression of the work completed to study and assess the hydrology of the Watsonville Slough System and develop tools to model potential watershed management techniques. Chapter 1 provides an overview of the study objectives and the specific work plan that was implemented in light of those identified goals. Chapter 2 briefly reviews the setting of the study area, focusing solely on those aspects germane to the work conducted. The remaining chapters (Chapters 3-5, and 7) successively address the three core components of the work plan. Additional chapters (6 and 8) are dedicated to discussions of the first direct applications of the tools that have been developed.

1.2 Project Funding

This study was funded in part by the California Department of Water Resources' Integrated Regional Water Management (IRWM) program through the IRWM Planning Grant awarded to the Regional Water Management Foundation on behalf of the Santa Cruz IRWM Region (Agreement N0. 4600009400). The study was funded also funded in part by the State Coastal Conservancy through a grant to the Resource Conservation District of Santa Cruz County (Agreement No. 11-076). The Project Team gratefully acknowledges this funding support in enabling this study to be completed.

1.3 Study Objectives

The Watsonville Sloughs Hydrology Study (herein "Study" or "Project") is the direct result of a long-perceived need, on the part of resource managers and stakeholders, for a set of specific tools to assist with management of the Sloughs watershed. Those working in and around the Sloughs have witnessed significant changes over the last several decades. Some of these changes have been dramatic. Examples include:

- Markedly higher water surface elevations than experienced in the past, including seasonally higher maximum, average, and minimum water levels.
- Inundation of extensive areas in the slough bottomlands as a direct result of higher water.
- Perceived increases in erosion rates evidenced by sediment accumulations both adjacent to and within slough channels.

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Concurrently, community perception of the Watsonville Sloughs has shifted, now seeing these waterways as a drainage, water-supply, and habitat resource. This perception is significantly different from that of the Sloughs three or four decades ago when their primary purpose was viewed as agricultural drainage and flood control.

Decision makers need a more informed understanding of the hydrologic processes that drive the inflow and outflow of water from the Slough system and the hydraulic factors that control movement of that water within the system itself, in order to manage the uses and treatment of the Sloughs.

The primary objectives of this study were two-fold. First, review ongoing data collection efforts and pursue additional data gathering as needed to comprehensively understand the hydrologic and hydraulic behavior of the Sloughs. This was done by installing a series of gages and measuring devices in different critical areas of the Slough System and carrying out field monitoring over a period of 16 months. The second objective was to apply the understanding of the Slough system to provide two important tools in the form of models to simulate the hydrology and hydraulics of the Sloughs.

Study specifications called for models that:

- accurately represent existing conditions;
- are capable of carrying out long-term, continuous simulations needed to represent the inter-annual climate variability that characterizes coastal California;
- can be easily adapted to assess scenarios related to management actions or other short- to long-term changes within the study area;
- use platforms that are readily available to all potential, interested users; and
- are transparent in their data sources and structure so that opportunities for enhancing model performance are maximized.

After reviewing available modeling platforms, and in consultation with the Technical Advisory Committee (TAC), the project team selected the Hydrologic Modeling System for hydrology and the River Analysis System for hydraulics, both of which were developed by the U.S. Army Corps of Engineers' Hydraulic Engineering Center (USACE

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HEC-HMS and HEC-RAS respectively). These models are publically available free of charge, have extensive simulation capabilities without being overly complex, and are actively supported in terms of documentation and upgrades on an ongoing basis by the USACE HEC.

1.4 Work Plan

The Project team developed a work plan structured around three core components to achieve the overall study objectives. The core components were:

1. *Data collection*. The Project team compiled a Data Collection Program (DCP) document as an early element of the work plan. The DCP identified existing sources of pertinent hydrologic data as well as the locations of, and equipment to be deployed at, additional monitoring locations. Fieldwork activities began immediately upon approval, in order to collect data from as much of two consecutive water years as possible, in this case WY2012 and WY2013.[1] Collected data were post-processed to standardize parameters across the locations monitored and archived for use in the remainder of the project work. Additionally, the Project team completed a comprehensive land survey of the lower Slough system channels and all monitoring locations so that collected data could be tied into a common datum, in this case the North American Vertical Datum of 1988 (NAVD).[2]

2. *Hydrologic modeling*. The Project team began its hydrologic modeling work with extensive efforts to compile pertinent current spatial data, which was used to construct the GIS database for the study. This data was processed using GIS tools to provide input data needed to parameterize the HEC-HMS model build. A key technical point was the selection of the Soil Accounting Method (SMA) routing within HMS to provide the continuous simulation capabilities called for by

1 A water year in California is defined as the consecutive time period of October 1 through September 30 of the named year. For instance, WY2012 encompassed hydrologic conditions from October 1, 2011 through September 30, 2012.

2 All elevations reported herein will reference NAVD unless otherwise noted. For the lower slough system, elevations in NAVD can be converted to the former National Geodetic Vertical Datum of 1929 (NGVD) by subtracting approximately 2.8 feet from an NAVD value, although actual precise conversions vary with specific location.

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the project goals. The model build included calibration using flow rates measured in the data collection phase. This resulted in a final hydrologic model output dataset that explicitly includes essentially all pertinent components of the surface water cycle for the selected 10-year base period of analysis (WY2003 through WY2012).

3. *Hydraulic modeling*. To build the hydraulic model, the Project team initially compiled existing topographic information along with the extensive site-specific data collected as part of the project survey. The combined dataset was used to build the model geometry, which includes the main channel and open water portions of the lower Slough system. The 10-year dataset from the hydrologic modeling was imported into the hydraulic model, supplemented by adjunct data (e.g. tidal tailwater, pumping for water extractions). The model was then run in continuous simulation mode and calibrated to water level information collected through the fieldwork activities in order to create the 10-year base period hydraulic model output.

Following the completion of the three core components detailed above, the Project team applied the modeling tools with the dual intent of assessing their utility for the intended purposes and to provide information related to specific concerns previously identified by managers and stakeholders. The specific applications included:

1. *Water balance update*. Previous studies had prepared water balances for the overall slough system as well as for the Harkins Slough sub-watershed. The previous analyses were updated in two steps. First, the Project team recalculated the prior water-balance using the same methodology but with more recent land-use and water-extraction data. Secondly, the Project team utilized an entirely new framework that extracted information for the water balance based on statistical analysis of the 10-year base period dataset derived from the study's HEC-HMS hydrologic model.

2. *Management scenarios*. The Project Team and the TAC identified two distinct suites of management scenarios for analysis using the HEC-RAS hydraulic modeling tool. The first of these consisted of an updated baseline model to include wetland restoration work and changes to water extractions that are anticipated in the near-term. The second scenario included the previous changes, but with revised channel geometry data to simulate Slough behavior

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under an active channel maintenance program. Both of the scenarios included additional model runs to assess predicted conditions in light of mid- and long-term estimates of sea level rise (SLR) and to demonstrate that the modeling tools can be applied to climate change evaluations.

1.5 Acknowledgments

In addition to the funding support noted previously, the Project Team would also like to acknowledge the extensive assistance provided by others during the course of the study. This includes the Pajaro Valley Water Management Agency (especially Brian Lockwood and Casey Meusel), which provided staff time, historical data, and access to their extensive gaging network, all of which were vital in meeting the study objectives. Additional credit goes to all members of the Technical Advisory Committee for their input, feedback, and comments that were so instrumental in framing the work plan and improving the final presentation of the findings. This includes Jim Van Houten who has long been an advocate for developing a comprehensive hydrologic study of the Slough system. And, of course, particular note should be made of the efforts of Susan Pearce and the rest of the staff at the Resource Conservation District of Santa Cruz County for overall coordination and outreach related to the study.

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2 PROJECT SETTING

The physical setting of the Watsonville Sloughs watershed has been described in depth in previous studies and the reader is referred therein for a more detailed description of the physical setting (see References in Chapter 10). This chapter focuses on those aspects of the setting that are particularly pertinent to understanding the scope and results of the project.

2.1 Location and Study Limits

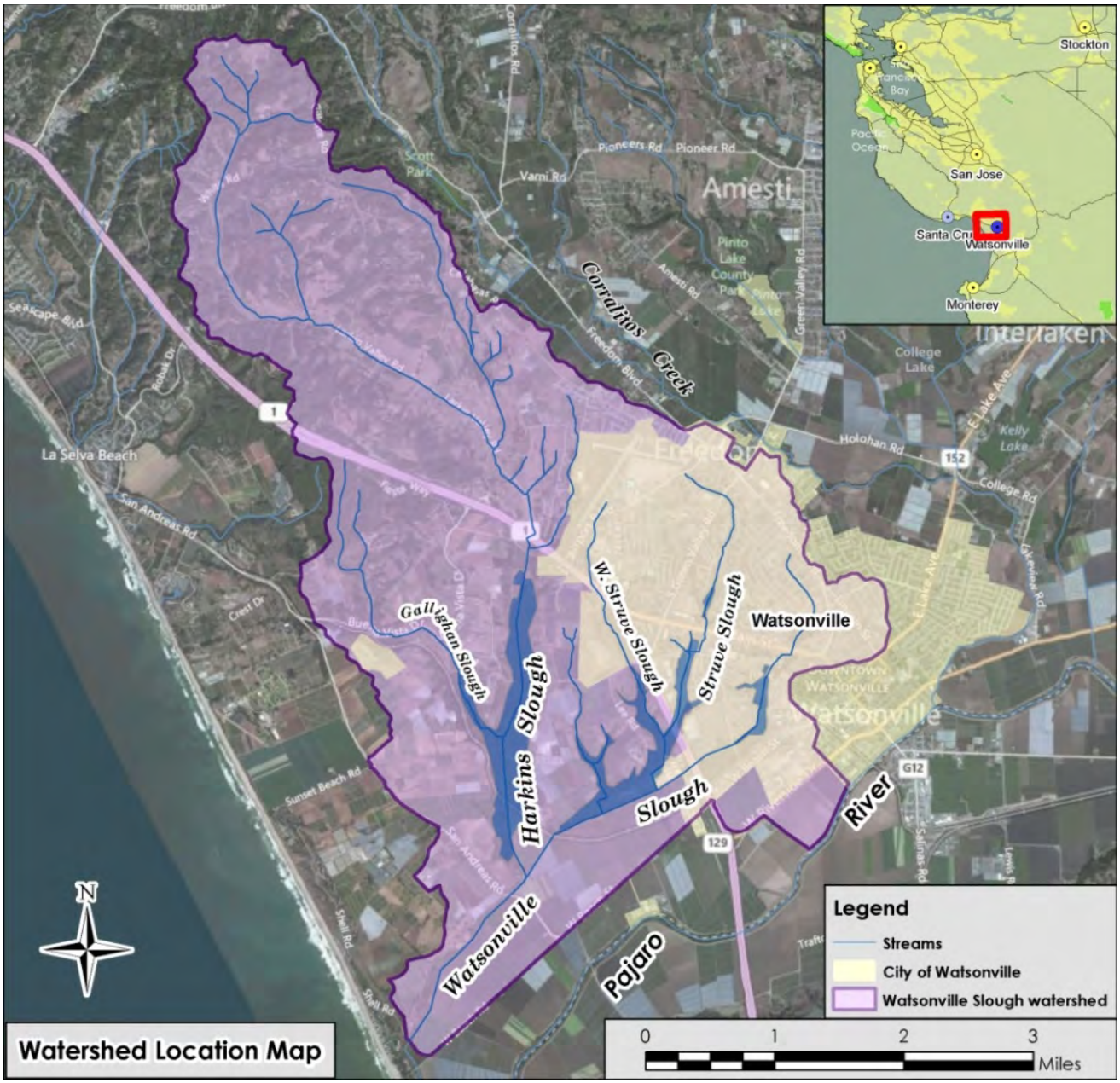
The study area is located in Santa Cruz County and is shown in Figure 2-1. It encompasses the entire Watsonville Sloughs watershed downstream to the West Beach Street road crossing, located approximately one mile north of the Sloughs' outlet at the mouth of the Pajaro River.[3]

The overall Sloughs watershed as mapped for this study has an area of 11,867 acres (18.5 square miles). Land use varies across the watershed, with the eastern portion (including roughly two-thirds of the City of Watsonville) accounting for almost all of the highly urbanized areas. However, in the watershed as a whole, land use is almost equally divided between agricultural lands (3,715 acres or 33%), developed areas (3,619 acres or 32%), and open space (3,984 acres or 35%)[4].

3 Watershed parameterization and hydrologic modeling included the area down to the important Shell Road crossing (approximately 0.25 miles upstream of West Beach Street). However, the hydraulic modeling domain extends all the way to West Beach Street.

4 The open space designation is used for lands that are not classified in the other two categories and does not imply a formal designation. Totals cited above do not include the 548 acres (4.6% of the watershed area) of the Sloughs proper which were perennially ponded at the time of study.

Figure 2-1 Location Map of the Watsonville Sloughs Hydrology Study Area



2.2 Climate

The Sloughs watershed lies within the Mediterranean climate zone that typifies central, coastal California. The area experiences cool, wet winters, with essentially all precipitation falling in the period from October to April. Summers are generally dry, with

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temperatures significantly moderated by proximity to the Pacific Ocean and frequent coastal fog.

Records of annual rainfall totals show wide variability, reflecting a number of climatic factors including those associated with global cycles such as the El Niño Southern Oscillation (ENSO or "El Niño"). Additionally, the watershed is located entirely on relatively low elevation coastal terraces and valley bottomlands, separated from the coastal mountains by the Corralitos Creek watershed. The low elevation setting limits the impact of orographic effects and results in precipitation totals and rates that are much lower than other locations in Santa Cruz County. Practically all precipitation falls as rain, with snowfall events being rare and not a significant factor in total precipitation. Watershed area-weighted mean annual precipitation (MAP) for the 30-year period from WY1981 to WY2010 is 23.9 inches, as calculated using data provided by the PRISM Climate Group at Oregon State University. This MAP value is very close to the value of 23.3 inches recorded for the same period at the Watsonville Waterworks gage (NOAA station USC00049473, identified as "WTW" in monitoring station Figure 3-2). Inter-annual variability for the 30-year period at the WTW gage is exemplified by the maximum value of 46.3 inches in WY1998 and the minimum value of 13.2 inches in WY1987. It is also important to note that the rainfall data show a distinct skew in part due to the few very wettest years. Median annual precipitation for the same 30-year period is only 19.7 inches.

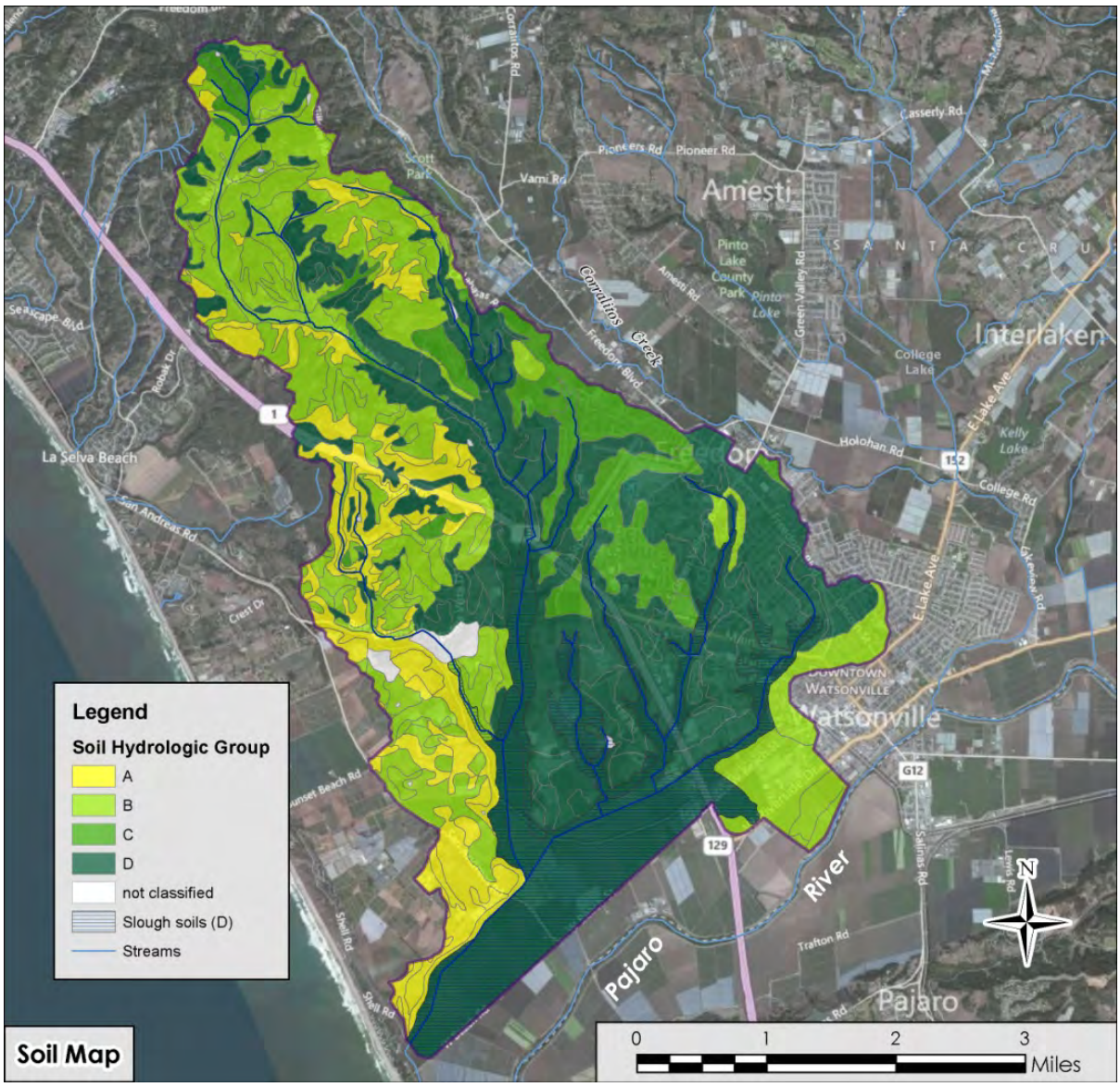
Evapotranspiration is another key component of the watershed hydrologic cycle and, in fact, is typically much higher than annual precipitation. Representative mean reference evapotranspiration (ET_o) is 39.2 inches per year as calculated using data collected at the California Irrigation Management Information (CIMIS) Station 129, located less than a mile to the east of the project area. Evapotranspiration values vary much less from year to year than rainfall totals and are generally lowest in years with the highest rainfall.

2.3 Soils

Soil characteristics are an important input for the hydrologic modeling component of the work plan. The watershed includes 38 distinct mapped soil units, although many occupy only small areas. The three most prevalent soils cover roughly 36% of the land area and include: Clear Lake clay (Hydrologic Soil Group, HSG "D", 1,424 acres or 13%), Watsonville loam (HSG D, 1,392 acres or 12%), and the Tierra-Watsonville complex (HSG

D, 1,196 acres or 11%). A useful general classification of the various soil types is by Hydrologic Soil Group (HSG) as illustrated in Figure 2-2.

Figure 2-2 Distribution of Hydrologic Soil Groups in the Study Area



The HSG classification system represents the tendency for runoff, with A soils having little runoff potential and D soils typically having relatively high runoff potential. As shown in Figure 2-2, D soils are the predominant grouping, occupying most of the central and eastern portions of the area. Group A and B soils are generally confined to the western portions of the watershed.

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2.4 Drainage Features and Patterns

Several drainage features and associated flowpaths are important in the context of the study and merit specific mention. Many of these features are shown on Figure 2-1, and others are illustrated in detail on the Hydraulic Model Work Map included in Appendix A. The watershed includes a number of named sloughs that in most if not all cases bear the names of early owners of the low-land areas. The distinctions and boundaries of these named sloughs have become somewhat less meaningful as perennial inundation has become the norm over the last few decades. The intermittent and perennial creeks that feed into the ponded areas generally do not have distinct names, being referred to by the low-land slough they drain into. The overall watershed can be divided into three distinct drainage areas based on flowpaths. These areas are not necessarily distinct from a land use or geologic perspective, but are important in understanding movement of water through the system. Important aspects related to watershed drainage include the following:

- The largest of the drainage areas will be referred to herein as the Harkins Slough branch and includes most of the northern and western portions of the overall watershed. It includes all upper watershed area above Harkins Slough, Harkins Slough proper, and Gallighan Slough, now a northwesterly-extending inundated area off of the larger Harkins Slough ponded area. With a total drainage area of 6,967 acres, the Harkins Slough branch comprises roughly 59% of the entire watershed. It includes approximately 304 acres of perennially ponded area, representing about 55% of the ponded area in the Sloughs system.
    - The Harkins Slough branch receives runoff almost exclusively from coastal terrace areas that are predominantly agricultural and open space land uses. Outside of the ponded areas, impervious cover in the Harkins Slough branch is only 5% (i.e. highly non-urbanized). Additionally, the vast majority of HSG A, B, and C soils in the overall watershed are found in the Harkins Slough branch (i.e. lower runoff potential).
- A second, large area drains the eastern part of the watershed and will be referred to herein as the Watsonville-Struve branch. It includes Watsonville Slough proper as well as Hanson Slough, West Struve Slough, and East Struve Slough. This drainage area includes 3,732 acres or roughly 34% of the system

total. Ponded areas constitute approximately 245 acres of this area, or about 45% of the ponded area in the overall watershed.

- The Watsonville-Struve branch receives runoff from the coastal terrace and from valley bottom areas. The most salient characteristic is the much higher level of urbanization as contrasted with Harkins Slough, with essentially all urbanized lands lying within this drainage area. Impervious surfaces cover 40% of the area, and it is underlain to a very large extent by HSG D soils.

- The Harkins and Watsonville-Struve branches join at a point approximately 1,000 feet upstream of San Andreas Road. The slough channels leading to the confluence point bound a triangular area known as the Knox property, where a wetland restoration project is currently planned. The very end of the Harkins Slough channel is separated from the Watsonville Slough channel by a segmented block weir structure (Figure 2-3). Immediately upstream (on the Harkins side) of this structure, the Pajaro Valley Water Management Agency (PVWMA) operates a pumping facility. Formerly the pumping operations were used to help maintain low water elevations in Harkins Slough by lifting runoff over the weir. After facility upgrades, the pump station is now used to divert water from Harkins Slough to groundwater recharge facilities located in the sandy areas to the west. PVWMA currently has permits to divert up to 1,000 acre-feet/year of water from this location, subject to certain restrictions.

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Figure 2-3 Flow control weir, maintenance crossing, and pumphouse at the downstream end of Harkins Slough and at the confluence with Watsonville Slough. Harkins Slough has open water compared to the overgrown channel in Watsonville Slough. The weir and the weir notch are outlined by the aquatic vegetation seen under the wooden deck

- The last drainage area includes those portions of the watershed that drain to the Watsonville Slough channel downstream of the confluence with Harkins Slough. Much smaller in size than the upper branches, this area is approximately 923 acres in size (roughly 8% of the total area). Runoff in this drainage area originates from coastal terraces and valley bottom areas, almost all of which are dedicated to agricultural uses. Impervious cover is quite low (about 5%), while the underlying soils are a mix of sandy soils on the terraces and clay soils in the valley bottomland.

The most important drainage control features in the watershed are located at the Shell Road crossing. The crossing includes a pump station operated by the County of Santa Cruz, an array of eight 48-inch diameter reinforced concrete pipe culverts passing under the road, and an old flow control weir that is no longer functional or pertinent to current drainage conditions (Figure 2-4). The culverts at this location are equipped with

flapgates on their downstream ends to prevent upchannel flow of ocean water, as Shell Road is intended to be the demarcation point between the freshwater channels and sloughs upstream and the tidally-influenced environment that characterizes the remaining run of the system to its ultimate terminus at the mouth of the Pajaro River.

Figure 2-4 Shell Road crossing showing culverts and pump station operated by the County of Santa Cruz. This site was also equipped with telemetered monitoring equipment to transmit near real-time information on precipitation, water surface elevations, specific conductance, and temperature (photo January 18, 2012).

The slough channel downstream of Shell Road is characterized by brackish and salt marsh habitats that reflect the intermittent, but often long-duration, direct tidal connection to Monterey Bay (see Chapter 4). The tidal channel runs roughly 7,800 feet in an east south-easterly direction immediately behind the Pajaro dune field to join the Pajaro Lagoon directly upstream of the barrier beach (Figure 2-5). The lagoon is a bar-built estuary that forms at the mouth of the Pajaro River, typically open to tidal effects during winter and spring as long as upland runoff is enough to overcome wave forces tending to build the beach bar. Beach Road crosses the channel approximately 0.25 miles downstream of Shell Road and includes an array of six 48-inch diameter reinforced concrete pipe culverts that do not have flapgates. Field observations and

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modeling show that this crossing rarely, if ever, controls flow rates in the system and, in fact, is frequently inundated during periods of extended rainfall or high winter tides.

Figure 2-5 Watsonville Slough channel from the west meeting the Pajaro River just inland from the river mouth (photo courtesy of the California Coastal Records Project).

A number of other crossings are located within the watershed. However, few of these actually play a significant role in the hydrodynamics of the system, as of 2012. Those that do include dual corrugated metal pipe (CMP) culverts at the former Southern Pacific Railroad crossing of Watsonville Slough, and dual CMP culverts at the Lee Road crossing of Watsonville Slough. Other crossing points include the San Andreas Road bridge on Watsonville Slough, the farm road upstream of the PVWMA pump station on Harkins Slough, the Harkins Slough railroad trestle, and the State Highway 1 bridge over Watsonville Slough. Each of these has sufficient conveyance capacity that they do not significantly restrict or control water movement. Additional culvert crossings of Harkins Slough at Harkins Slough Road and Struve Slough at Lee Road formerly were significant control points (at least seasonally), but have recently become immaterial as inundation levels now permanently top the roadways at these locations, allowing practically unlimited conveyance between the slough bodies on either side. The latter two crossings could again play an active role if management actions are taken that lower water surface elevations.

There have been relatively dramatic changes over the last several decades in the extent and duration of ponding within the Slough system. As recently as the 1990s, summer water levels were contained within relatively small channels along the bottomland areas and both Lee Road and Harkins Slough Road were passable for much of the year. Monitoring data presented in Chapter 4 confirms that hundreds of acres of bottomland are now flooded even at the end of the summer season following relatively dry years such as WY2012. Several factors have been postulated as contributing to these changed conditions, including increased runoff, channel blockages, and land subsidence.

Encroachment into the Slough channel network by aquatic vegetation is an ongoing issue as well, particularly with the spread of marsh pennywort in dense mats throughout the lower Slough system (Figure 2-6). The mats have been observed to be dense enough that they impede water flow in a significant manner, especially at low- to mid-flow ranges.

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Figure 2-6 Images representing typical slough channel geometry (just upstream of Shell Road) with and without encroachment by aquatic vegetation.

Channel with aquatic vegetation (November 17, 2011)

Channel without aquatic vegetation (March 20, 2012)

3 DATA COLLECTION PROGRAM

As discussed in Chapter 1, a significant portion of the work undertaken for the study consisted of data collection. The first deliverable under the work plan was the summary of the Data Collection Program (DCP). The Project team collected, compiled, and post-processed significant datasets, for a total of 13 sites within the study area. The PVWMA assisted with equipment access and staff time. This assistance was particularly instrumental in the success of the data collection efforts. There were a few difficulties with data collection, described in this section.

The Project team structured the DCP to provide new information in three main categories: detailed topographic survey, hydrologic data collection, and shallow groundwater monitoring. The objectives and details of each of these components are discussed below. Results and findings of the DCP are presented in Chapter 4.

3.1 Topographic Survey

Topographic survey work was carried out by Environmental Data Solutions (EDS) with the majority of the field work taking place in the late spring and early summer of 2012. Survey data collection included topographic data from a large area encompassing the lower Slough system (Figure 3-1).

Figure 3-1 Extent of survey information collected by Environmental Data Solutions (EDS).

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The objectives of the survey included data collection to support interpretation of the hydrologic and groundwater data gathered and, particularly, data collection to obtain detailed information on channel and structure geometries used to construct the existing conditions build of the project hydraulic model. Specific data collected in these surveys is discussed in Appendix B and included:

- A detailed survey, using differential GPS technology, of 30 channel cross-sections within the limits of the study hydraulic model.
- Two long profile surveys. The first encompassed the entire channel reach from Beach Road in the west, upstream to the Highway 1 crossing in April 2012 and the second included the same geographic extent in May 2012.
- Survey of the staff plates at all data collection locations the NAVD datum.
- Additional point surveys of key structures, including 10 stream crossing and flow controls.
- Establishment of four survey benchmarks as reference points for future studies and management activities.

3.2 Hydrologic Data Collection

The monitoring sites where hydrologic data were collected span a range of conditions from upland stream channels, through slough channels, to tidal marsh at the western, downstream end of the study area (Figure 3-2). The monitoring sites for hydrologic data can be grouped into four categories depending on the primary hydrologic parameter measured, although opportunities to measure multiple parameters were exploited wherever equipment costs and conditions allowed. A summary of monitoring site information is provided in Table 3-1 and observers' logs are included in Appendix C.

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Figure 3-2 Watsonville Sloughs Hydrology Study monitoring stations and other local hydrologic data sources

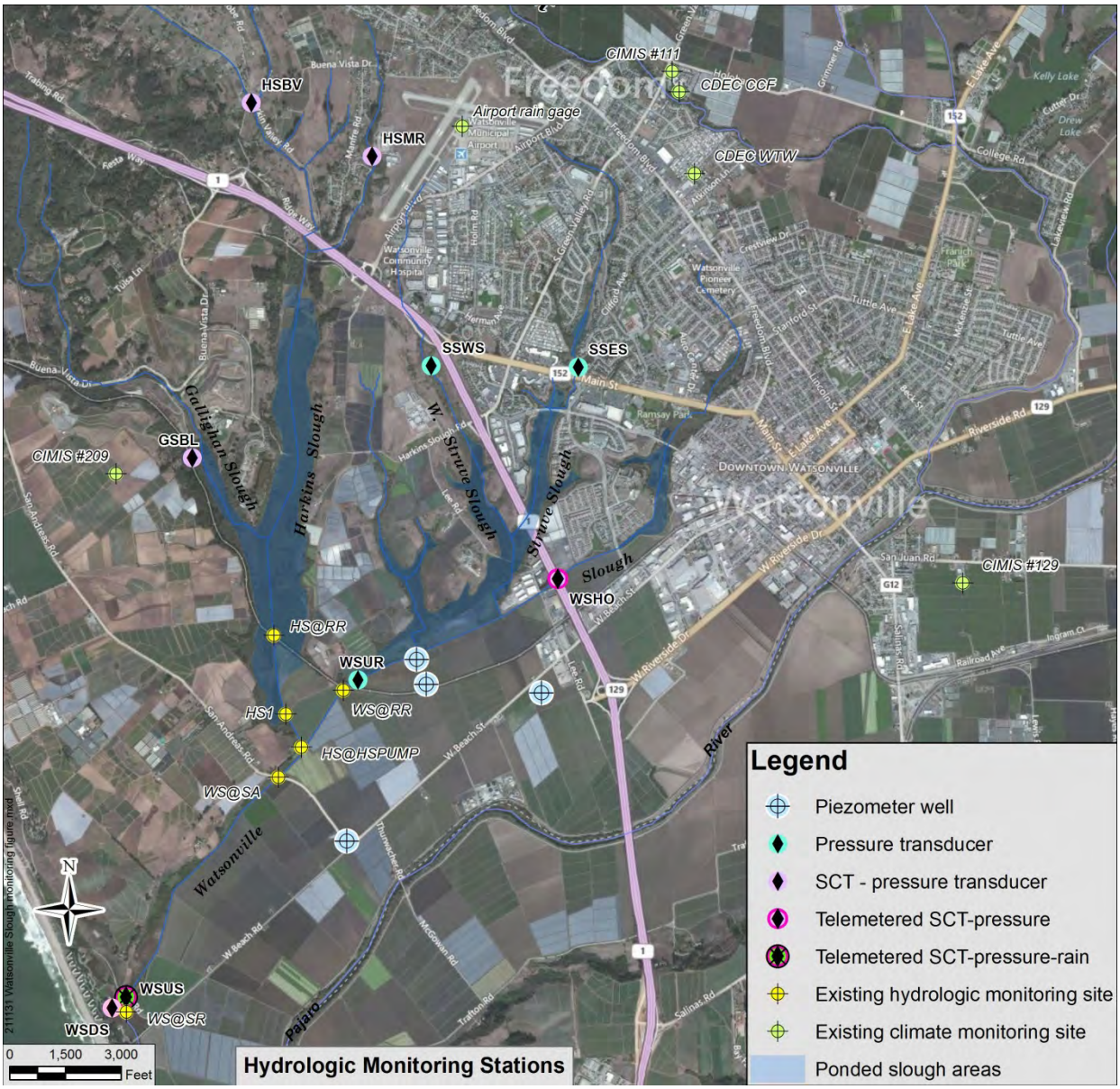

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Table 3-1. Watsonville Sloughs Hydrology Study monitoring stations and other local hydrologic data sources.

Installed Station Name[1]	Code	Latitude	Longitude	Objective[3]	Equipment[4]
East Struve Slough at Main Street	SSES	36.918	-121.778	Flow	PT
West Struve Slough at High School	SSWS	36.918	-121.791	Flow	PT
West Struve at Wetlands Watch	SSWW	36.990	-121.798	Rainfall	TB
Watsonville Slough at Highway One	WSHO	36.903	-121.779	Flow/Salinity	PT/SCT (Telemetered)
Watsonville Slough upstream of Railroad	WSUR	36.895	-121.797	WSE	PT
Watsonville Slough upstream of Shell Road	WSUS	36.871	-121.818	E/Salinity/Rain	PT/SCT,TB (Telemetered)
Watsonville Slough downstream of Shell Road	WSDS	36.871	-121.819	WSE/Salinity	PT/SCT
Harkins Slough at Buena Vista Road	HSBV	36.938	-121.808	Flow/Salinity	PT/SCT
Harkins Slough at Manfre Road	HSMR	36.934	-121.797	Flow/Salinity	PT/SCT
Gallighan Slough below Landfill	GSBL	36.911	-121.813	Flow/Salinity	PT/SCT
Land Trust at Watsonville Slough	PWWS	36.897	-121.792	WSE	PZ
Land Trust at Railroad	PWRR	36.895	-121.791	WSE	PZ
Beach Road west of Lee Road	PWBR	36.894	-121.781	WSE	PZ
San Andreas Road at Beach Road	PWSA	36.883	-121.798	WSE	PZ
Existing Station Name[2]					
Watsonville Slough at Railroad	WS@RR	36.894	-121.799	WSE/Salinity	PT/SCT
Watsonville Slough at San Andreas Road	WS@SA	36.888	-121.805	Flow/Salinity	PT/SCT
Watsonville Slough at Shell Road	WS@SR	36.871	-121.818	WSE/Salinity	PT/SCT
Harkins Slough at Pumsp Station	HS@HSPUMP	36.890	-121.803	WSE/Salinity	PT/SCT
Watsonville Slough at Railroad	WS@RR	36.894	-121.799	WSE/Salinity	PT/SCT
Harkins Slough at Railroad	HS@RR	36.898	-121.805	WSE/Salinity	PT/SCT

Notes:
1. Installed stations were those set up by Balance Hydrologics in support of project objectives.
2. Existing stations are part of the previously established PVWMA monitoring network.
3. Objective is the primary parameter measured for the purposes of this study. WSE = water surface elevation.
4. Equipment type as follows: PT = Pressure transducer records submerged pressure and is calibrated to water surface elevation via staff plate readings; PT/SCT = Pressure transducer instrument with an attached specific conductance/temperature probe to measure salinity; TB = Tip-bucket rain gage; PZ = Piezometer wells equipped with pressure transducer.

Two of the monitoring sites (Watsonville Slough upstream of Shell Road as shown in Figure 2-4 and Watsonville Slough at Highway 1) were equipped with full telemetry capabilities to transmit field data in near real-time to a central data server. Access to the telemetered data feeds was instrumental in coordinating the timing and frequency of field response activities for storm events and dry weather data collection, as well as routine and non-routine maintenance.

3.2.1 PRECIPITATION GAGING

Two tip-bucket rainfall gages were installed to supplement other precipitation gages operated by various entities in the vicinity of the study area. The most important of these from a geographic and project management perspective was that installed at the Shell Road Pump Station (WSUS). This site was selected to provide rainfall data from the lowest elevation and most westerly location in the Sloughs watershed, and was particularly useful in coordinating field response for storm events.

A second tip-bucket gage (SSWW) was installed at the Watsonville Wetlands Watch facility adjacent to the Pajaro Valley High School. Unfortunately, this station proved problematic, because the Project Team encountered ongoing equipment issues in downloading the stored data. Given the interrupted nature of the dataset, it is not presented as part of the study. Future operation of this site will require coordination with the equipment manufacturer. The Project Team recommends that the installation configuration be modified, as the site is prone to attracting roosting birds, and the team observed persistent fouling problems on nearly every site visit.

3.2.2 WATER SURFACE ELEVATION GAGING NETWORK

The gaging network included a large number of stations designed primarily to measure water surface elevation data to interpret Slough flow patterns and stored water volumes and for model calibration purposes. An important note in this regard is that staff plates were installed at each location and these were surveyed to the NAVD datum as part of the work described in Section 3.1. Therefore, water surface elevations can be compared between different points in the system with important implications for understanding system behavior. Unless otherwise noted, all water surface elevation loggers were sampled at a 15-minute interval.

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3.2.2.1 Previously Established Gaging Sites

As summarized in Table 3-1, five of the sites that primarily focused on water surface elevation data were previously installed by PVWMA, and one additional PVWMA station (WS@SA) was adapted for flow gaging purposes. Three of these gages are located on the lower Watsonville Slough channel spanning the area from the railroad crossing downstream to the Shell Road Pump Station. The remaining three are located in the lower reaches of the Harkins Slough branch, from the railroad crossing downstream to the PVWMA pump station.

3.2.2.2 New Gage Installations

New water surface elevation monitoring stations were installed at three additional locations as part of this study. The Watsonville Slough downstream of Shell Road (WSDS) installation was particularly important to project objectives. It was installed to provide a continuous record of tidal tailwater conditions and was critical in interpreting tidal influences, Pajaro River backwater effects, and when the Pajaro River mouth was open or closed over the monitoring period.

3.2.3 STREAM FLOW GAUGING NETWORK

A total of six stream flow gaging stations were established as part of this study. With the adaptation of the WS@SA station previously installed by PVWMA, this provided the study with a total of seven flow monitoring sites. Installations consisted of staff plates and stilling wells equipped with continuously logging pressure transducers to record water depth. Rating curves were developed for each installation based on manual flow measurements conducted over the monitoring period.

Unfortunately, one of the sites, Harkins Slough at Manfre Road (HSMR), was vandalized early in the monitoring period, resulting in the loss of the installed equipment. Due to the short period of record, data from this station is not included in this report.

3.2.4 GROUNDWATER MONITORING

A groundwater monitoring grid array was also set up to describe the nature and degree of connectivity between the lower Watsonville Slough channel and the local shallow groundwater system.

Four shallow groundwater monitoring wells (piezometers) were installed in the Pajaro River floodplain as shown in Figure 3-2. Two were located along Beach Road on

existing PVWMA easements. The first was installed near the PVWMA supplemental well #2 near Highway 1 (PWBR) and the second further west at the intersection of San Andreas Road and Beach Road (PWSA). These were installed to assess any changes in shallow groundwater dynamics along the long profile of lower Watsonville Slough. In addition, two more piezometers were installed upstream of the railroad tracks on property owned by the Santa Cruz County Land Trust, one approximately 950 feet from Watsonville Slough (PWRR) and another approximately 430 feet from Watsonville Slough (PWWS). These piezometers, in tandem with the others, were located to assess the lateral flow of near surface groundwater to and from the sloughs.

The well borings were logged (see Appendix D) and casings installed in April 2012, within a week of the last major storms of the 2012 winter. Casings were 2-inch diameter PVC, screened at the bottom with a pea-gravel annulus and sealed with at least one foot of bentonite. Material was mounded around the base of the PVC casing at the surface to prevent water from pooling and affecting the installations. Each piezometer was equipped with a continuously logging pressure transducer.

3.2.5 SALINITY AND TEMPERATURE

A total of twelve stations were equipped with specific conductance/temperature probes to collect data on salinity, as shown in Table 3-1. These installations proved particularly useful in tracking the timing, duration, and spatial extent of salinity intrusions from the downstream end of the system following wave overtoppings of the beach bar.

3.2.6 SHELL ROAD PUMP STATION CALIBRATION

An additional logging pressure transducer was installed on a temporary basis in the stilling well outlet of the Shell Road Pump Station. This installation was part of the work conducted in the spring of 2013 to measure the discharge rate of the two pumps that are operated at that location.

3.3 Field Visits and Measurements

Field measurements were carried out throughout the study period as needed to maintain and calibrate the installed equipment. In total, the field staff was deployed on 26 days. Rating curve development required field deployment during storm events to collect measurements at high flow levels. Observers' logs giving conditions noted at each site visit are provided in Appendix C.

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3.4 Data Reduction and Post-Processing

The collected field data was compiled and post-processed with special emphasis on calibration and correction for changing field conditions. Important results stemming from the DCP are presented in Chapter 4.

4 FIELD DATA DISCUSSION

The DCP generated a very large quantity of data directly related to understanding the behavior of the Watsonville Sloughs system. This chapter discusses the most important findings from review and analysis of the field data, noting that a full discussion of all the data is beyond the scope of this report. To the extent practical, more detailed compilations of the data are presented in the Appendices, and full digital versions of the datasets are available on request.

4.1 Topographic Survey Findings

Survey work was completed in April and May of 2012 and the summary report prepared by EDS is included as Appendix B. Almost all of the roughly 1,800 survey points collected were used as direct input to supplement regional LiDAR for the hydraulic model build (see Chapter 8). As part of its survey work, EDS completed two important objectives: setting survey control points and compiling a "long profile" of the Watsonville Slough Channel.

4.1.1 SURVEY CONTROL POINTS

The survey included the establishment of four separate control points as shown in Figure 4-1. Two National Geodetic Survey control points with a Class A stability rating were used as the basis of the survey.

The new points were established to provide geographic coordinate system controls for this study as well as for any survey work in the future. Of particular note are points CP6, CP7, and CP8 which included the installation of aluminum discs in locations suitable for monitoring potential subsidence trends. These three points were set at the lower end of the system (Shell Road), an important mid-point location (confluence of Watsonville and Harkins Sloughs), and an upper slough location (Harkins Slough Road) such that ongoing subsidence can be measured in the future and differential rates determined. For example, detection of faster rates of subsidence in upstream locations than at Shell Road would have definite management implications with respect to inundation depths and extents and would be a significant consideration in habitat enhancements or flow control facility upgrades.

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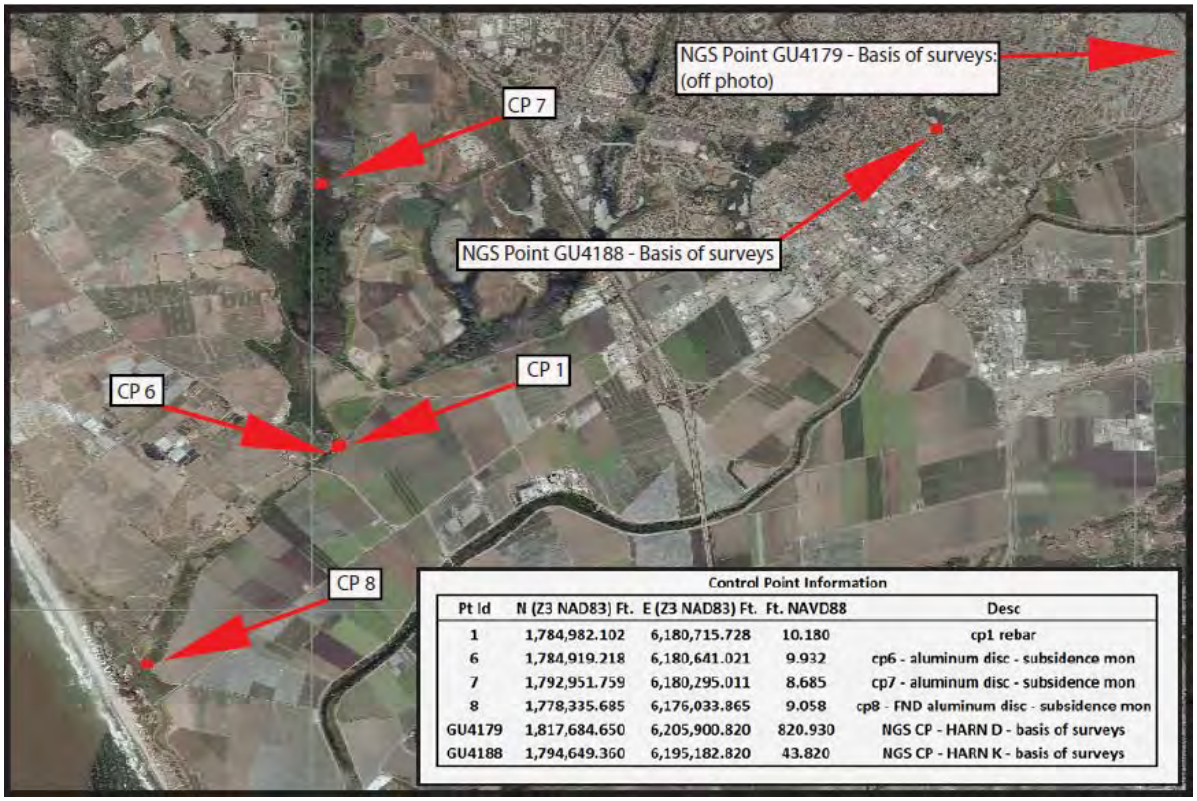
Figure 4-1 Basis of survey and control points established by Environmental Data Solutions for the topographic survey.



Control Point Information

Pt Id	N (Z3 NAD83) Ft.	E (Z3 NAD83) Ft.	Ft. NAVD88	Desc
1	1,784,982.102	6,180,715.728	10.180	cp1 rebar
6	1,784,919.218	6,180,641.021	9.932	cp6 - aluminum disc - subsidence mon
7	1,792,951.759	6,180,295.011	8.685	cp7 - aluminum disc - subsidence mon
8	1,778,335.685	6,176,033.865	9.058	cp8 - FND aluminum disc - subsidence mon
GU4179	1,817,684.650	6,205,900.820	820.930	NGS CP - HARN D - basis of surveys
GU4188	1,794,649.360	6,195,182.820	43.820	NGS CP - HARN K - basis of surveys

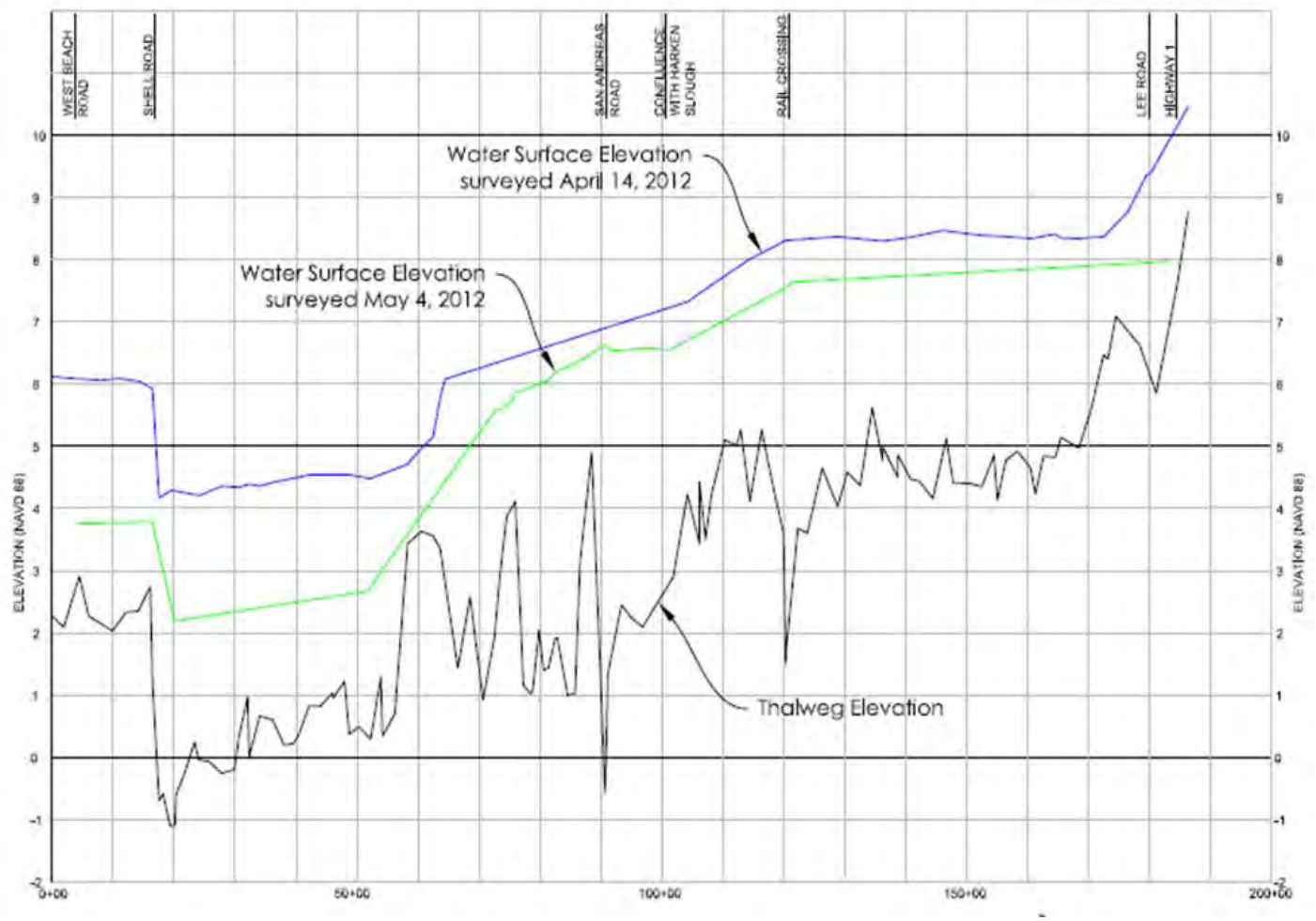
4.1.2 WATSONVILLE SLOUGH CHANNEL LONG PROFILE

An important element of the survey work was collection of point data including channel bed and water surface elevations needed to compile a long profile of the Watsonville Slough channel from Shell Road upstream to the Highway 1 crossing, a distance of just over 3.5 miles. Survey activities progressed in a manner that allowed full channel bed information to be developed along with reach-long water surface elevation profiles on two separate dates. The resulting long profile data is illustrated in Figure 4-2.

The water surface elevations recorded in the long profile proved particularly useful as calibration points for the hydraulic modeling work, both in the extent and density of the information and in the fact that two distinct flow conditions were captured. The first set of data collected on April 14, 2012 was obtained on the day after the peak of the late season rains that characterized WY2012 and included conditions with both Harkins Slough and Watsonville Slough at their respective annual peak stages. The data from

May 4, 2012 show how water levels change as the overall system drains with the post-rain spring drawdown.

Figure 4-2 Long profile of the Watsonville Slough channel from Shell Road upstream to State Highway 1 with observed water surface elevations on April 14 and May 4, 2012.



In addition to the water surface data, the long profile captures several very important aspects about the geometry of the main Watsonville Slough channel. These include:

- *Critical elevations*. Channel bed elevations are notably irregular (see thalweg elevation profile on Figure 4-2) with several locations of note. Base elevations at the downstream end of the system are set by the invert of the culvert array under Beach Road, with an elevation just under three feet, roughly at the mean tide elevation in Monterey Bay. At the Shell Road crossing, the culverts sit slightly

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lower at roughly 2.7 feet. However, the low point of the channel is just upstream, at the intake to the Shell Road Pump Station where the bed drops to approximately -1.0 feet, in other words, roughly four feet below mean sea level. The bottom elevation at confluence of the Harkins and Watsonville-Struve branches of the system is 2.5 feet, and bed elevations increase to just under six feet downstream of the Highway 1 bridge. Several deep pools were found, notably at the San Andreas and Railroad crossings. In other environments these pools might typically be scour holes, but the very low velocity environment of this channel indicates that these are more likely evidence of past dredging activities at these crossing points.

- *Channel slope.* Despite the irregularity of the channel bed profile, there is a relatively uniform overall slope. Measured from the low point at the Shell Road pumps up to Highway 1, there is a uniform, but very gentle slope of 0.04 percent (2.3 feet/mile). The uniformity of this slope likely represents the line of previous dredging activities used to construct and then maintain the slough channel in the past. Conveyance capacity at this channel slope is quite low, especially given the low elevation of the channel banks in many locations.

- *"Dams".* The long profile survey of the Watsonville Slough Channel confirms observations by others of numerous obstructions along the channel that effectively create "dams" that hold back water flow and effectively segment the system into distinct storage areas independent of other constructed flow control points. Particularly evident examples of these dams are seen at Stations 60+00, 75+00, 88+00, 115+00, and 135+00. The dams typically represent localized sediment aprons at inflow points draining off-channel erosion-prone areas and/or areas particularly encroached by aquatic vegetation, although these two factors are mutually supporting and often go hand-in-hand. The water surface profile captured during higher flow conditions on April 14, 2012 demonstrates how strong a control these dams exert. For example, the downstream dam at Station 60+00 sets an upstream limit on drawdown from the pump station at Shell Road, meaning that the pump station is only actively regulating water levels for roughly the first 4,000 feet upstream. The dams also serve to markedly reduce the effective channel slope over long reaches. An example is the very pronounced dam at Station 88+00 (just downstream of San Andreas Road) with an elevation of 4.9 feet. This high point reduces the

effective channel slope upstream to Highway 1 to only 0.01 percent (0.6 feet/mile), roughly one quarter of the overall uniform slope line. This markedly lower slope further reduces channel conveyance (causing more frequent overbank flows) and lowers velocities (leading to increased sedimentation potential).

- *Pool elevations.* As noted, the dams along the channel profile create "storage pools" upstream of their respective locations and effectively set base water surface elevations for those pools. The two branches of the overall system are controlled by different dams. The dam at Station 88+00, with an elevation of 4.9 feet, currently regulates water surface levels in Harkins Slough. Slough elevations below 4.9 feet in Harkins Slough can only result from losses due to evaporation or pumping. Without the dam, the pool elevation would quickly drop to that of the confluence point with the Watsonville Slough channel or roughly 2.5 feet. This difference in and of itself is enough to cause perennial flooding of the Harkins Slough Road crossing. Likewise, the dammed reach around Station 115+00 (just downstream of the Railroad crossing on Watsonville Slough) sets a base elevation of 5.2 feet for the system upstream of the crossing. Struve Slough is cut off from the main channel at an elevation of approximately 6.0 feet.

4.2 Hydrologic Data Findings

With data collected and processed from a total of 18 gaging stations, the study produced a large compendium of hydrologic information.

4.2.1 PRECIPITATION GAGING RESULTS

Rainfall gaging data, augmented by local, non-project gages, is especially important in characterizing the two water years in which data was collected. Table 4-1 gives monthly rainfall totals observed by the project gage at Shell Road (WSUS), as well as those for CIMIS Station 129 and for the gage at the Watsonville Waterworks (WTW) along with the respective longer-term mean value for the latter.

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Table 4-1 Rainfall totals by month for WY2012 and WY2013 in the vicinity of the Watsonville Sloughs Hydrology Study

	WTW (Waterworks)			CIMIS 129		WSUS (Shell Road)	
Month	WY2012	WY2013	30-yr Mean	WY2012	WY2013	WY2012	WY2013
October	1.87	0.41	1.05	1.78	0.42	n.d.	0.12
November	2.10	2.98	2.68	1.77	3.20	n.d.	2.80
December	0.04	7.90	4.11	0.04	6.24	n.d.	4.99
January	2.81	0.72	4.60	2.92	0.63	n.d.	0.43
February	0.65	0.54	4.67	0.66	0.85	0.48	0.64
March	4.13	0.53	3.61	3.29	0.68	2.89	0.58
April	3.13	0.80	1.65	1.99	0.44	2.04	0.31
May	0.01	0.00	0.59	0.01	0.07	0.02	n.d.
June	0.20	0.04	0.11	0.22	0.06	0.16	n.d.
July	0.04	0.01	0.01	0.02	0.00	0.00	n.d.
August	0.00	n.d.	0.02	0.00	n.d.	0.00	n.d.
September	0.00	n.d.	0.22	0.00	n.d.	0.00	n.d.
Totals	15.0	13.9	23.3	12.7	12.6	n.d.	n.d.

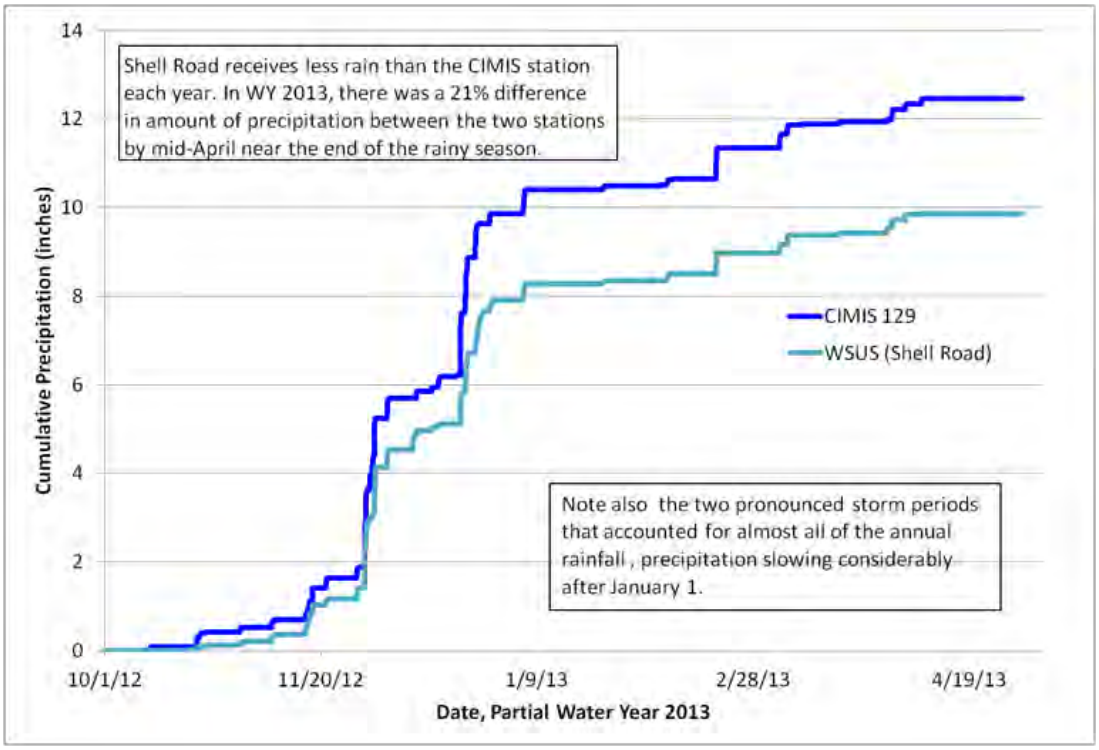
Notes:
Station WSUS (Shell Road) became operable on 01/18/2012.
30-year mean values for the WTW gage are for WY1981 through WY2010

As shown in Table 4-1, both water years of the study period were well below the long-term averages. In fact, WY2013 (to date) with a total of 13.9 inches is only 0.7 inches above the minimum recorded at the WTW gage in the last 30 years. Not only were the annual rainfall totals low, but the monthly distribution was notably skewed, toward the end of the season in WY2012 and toward the beginning of the season in WY2013. A total of 7.3 inches of rainfall was recorded in the months of March and April in WY2012, fully 48 percent of the total for that water year and significantly higher than the longer-term mean of 5.3 inches for those two months. Conversely, the months of November and December in WY2013 saw 10.9 inches of rain, accounting for 78% of the total in the water year, after which storm activity decreased dramatically, ending with one of the driest spring periods on record.

In addition to transmitting near real-time 15-minute data on rainfall, the tip-bucket rain gage at Station WSUS (Shell Road) provides a reference point for precipitation totals at the very low-elevation western end of the watershed. Monthly rainfall totals at WSUS are universally lower than those at the other gages summarized in Table 4-1. Figure 4-3

shows cumulative rainfall as observed at WSUS, plotted against that recorded at CIMIS Station129—hourly data that is the source of the precipitation distribution dataset used in the 10-year hydrologic model runs. Rainfall is typically registered at both stations every rain event, as would be expected given their close proximity. Nonetheless, precipitation is consistently lower at WSUS leading to a significant cumulative deficit by mid-year compared to CIMIS 129.

Figure 4-3 Cumulative rainfall recorded at Station WSUS (Shell Road) and CIMIS Station 129 for WY2013.



4.2.2 WATER SURFACE ELEVATIONS

Monitoring of water surface elevations yielded data leading to a number of fundamental findings. PVWMA has operated gaging installations at a number of key locations for many years, and the value of this approach to understanding the hydrodynamics of the slough system cannot be understated. The following Sections discuss the most important findings starting at the downstream end of the system.

4.2.2.1 Tidal Conditions and Pajaro River Interactions

As noted in Chapter 2, the Watsonville Sloughs watershed terminates at the Pajaro River Lagoon approximately 7,800 feet (1.5 miles) downstream of Shell Road. Gaging station

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WSDS was installed on the downstream side of Shell Road at the end of November 2011and logged 15-minute data on water surface elevations, specific conductance, and temperature until the logger device failed in May 2012. Fortunately, a temporary logging pressure transducer (Station WSSW) had been installed in the Shell Road pumps' outflow stilling well prior to May 2012. The combination of the two datasets provides a continuous record of the water surface elevations that are the principal downstream hydraulic control for the system.

Typical of other bar-built estuaries, the seasonal and inter-annual environment is a dynamic one, representing the interplay of factors such as riverine flow rates, ocean wave dynamics, and fluvial and coastal sediment transport. Aspects of this dynamism are illustrated in Figure 4-4. The barrier beach is frequently closed at the start of each water year. In this condition, water surface elevations downstream of Shell Road show little daily fluctuation. However, this is also often the period when average daily water surface elevations in the Slough downstream of Shell Road are at their highest, representing the balance between inflow from Pajaro River and Watsonville Sloughs baseflow and outflow as seepage through the beach. This seasonally-closed state is punctuated by periods of beach overtopping when higher energy waves wash over the barrier introducing highly saline seawater into the lagoon and potentially the entire Slough system. Notable examples of this periodic overtopping are seen in mid- and late-November 2012 in Figure 4-4.

Given the large size of the Pajaro River watershed (1,186 square miles at the USGS Chittenden gage), sufficient river flows typically develop during the course of the winter rainy season to raise lagoon levels high enough to breach and/or maintain an open beach. Breaches are also often initiated by large overtopping events followed by subsequent failure of the bar as tides later recede. Water surface levels in the lagoon can rise to quite high levels prior to the first seasonal opening (see discussion in the next section), and intervention by County crews is occasionally needed to initiate the first outflow to prevent prolonged flooding of upstream infrastructure such as Beach Road, Shell Road and adjacent private and State properties.

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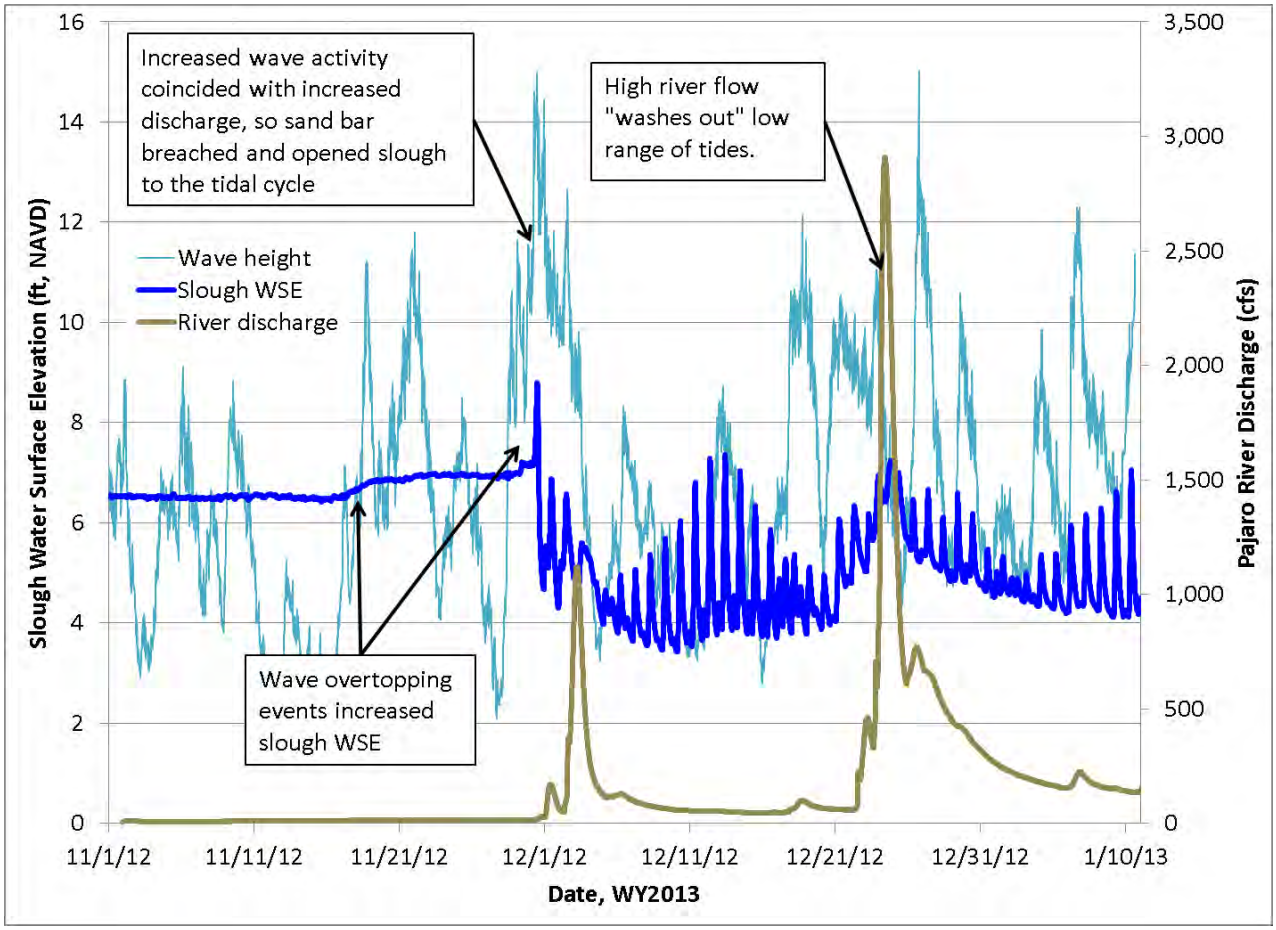
Figure 4-4 Water surface elevations at Station WSDS (Watsonville Slough downstream of Shell Road) and relation to Pajaro River flow and wave height in Monterey Bay. Pajaro River flow as measured at the USGS Chittenden Road gage (#11159000), wave height as measured at the Outer Monterey Canyon buoy (#46236). Note that wave height is just one measure of the wave energy and potential for overtopping.



Once the bar is open, water levels in the lagoon (and lower Watsonville Slough upstream to Shell Road) rise and fall with the tides in Monterey Bay. Outflow from the lagoon generally leads to a more muted cycling at higher elevations than the respective tide level (see Table 4-2 for Monterey Bay tidal datums). During periods of particularly high Pajaro River flow, much of the tidal cycle can be "washed out" as high discharge at the mouth leads to river stages much higher than the ocean tide resulting in backwatering of the lower slough channel (Figure 4-4).

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Table 4-2 Tidal datum information representative of Monterey Bay at the mouth of the Pajaro River

Tidal datum	*(ft, NAVD)*
Mean higher high water (MHHW)	5.48
Mean high water (MHW)	4.78
Mean tide level (MTL)	3.01
Mean low water (MLW)	1.24
Mean lower low water (MLLW)	0.14

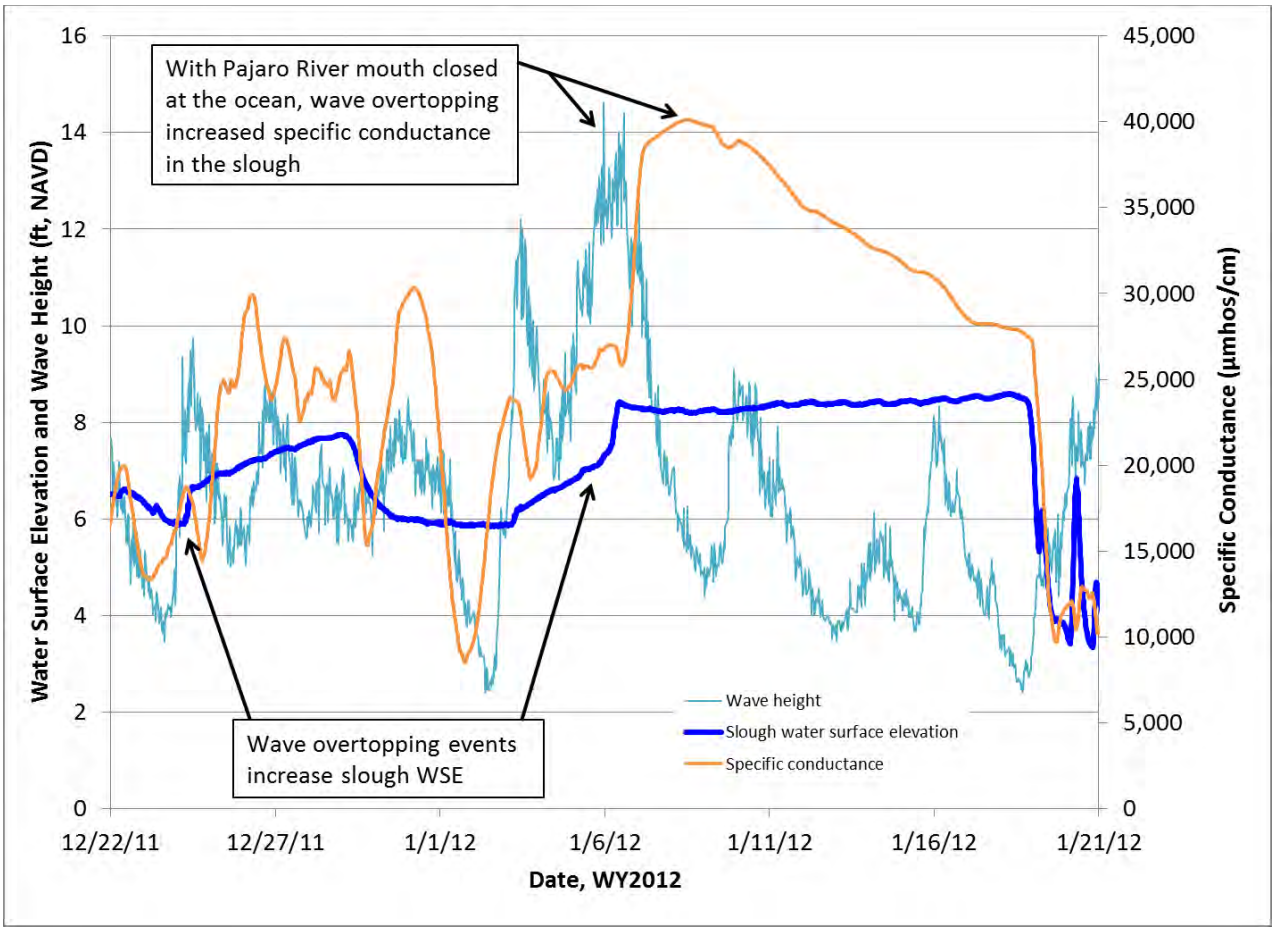
Note
Data from NOAA gage #9413450, Monterey Harbor
Datums are from current tidal epoch, 1983-2001

4.2.2.2 Barrier Beach Overtopping and Seawater Incursions

As discussed in the previous section, beach overtopping events play an important role in the Pajaro Lagoon and Watsonville Slough channel downstream of Shell Road, particularly as ocean wave energy increases going into the fall and winter season. Monitoring at the WSDS gage in WY2012 captured a particularly large beach overtopping event accompanied by extended high water levels in January 2012 (Figure 4-5).

As shown by the specific conductance values, persistent overwash events in early January culminated with a major overtopping on or about January 6, 2012, with enough seawater entering the lagoon to raise salinities in the slough at station WSDS to essentially those of the open ocean. Salinity impacts propagated upchannel in the sloughs as far as the railroad crossing on Watsonville Slough and above the railroad crossing on Harkins Slough. Salinity levels declined after the event as outflow from the Pajaro River diluted the water stored in the lagoon. However, the barrier beach did not breach after the overtopping and water surface elevations remained high for almost two weeks, until County crews mechanically breached the bar on January 18, 2012.

Figure 4-5 Slough water surface elevations at Station WSDS, specific conductance, and ocean wave height in the period around the large beach overtopping event in January 2012. Note that elevations higher than approximately 7.5 feet lead to intrusion of seawater through vent slots in the culvert array at Shell Road. Elevations greater than 8.4 feet overtop Shell Road at Beach Road and allow ditch flow upstream into the Slough system. Full-strength sea water has a specific conductance of about 53.000 µmhos/cm.



Although the culverts at Shell Road are equipped with flapgates (which have been observed to leak to a certain degree), prolonged high water levels can cause substantial flows up into the normally freshwater Slough system. The first significant route for upchannel flow is initiated at an approximate water surface elevation of 7.5 feet, at which point water can flow into the vent slots (eight total) on top of the downstream headwall of the Shell Road culverts. Once water surface elevations reach roughly 8.4 feet, water can overtop Shell Road near the intersection with Beach Road and flow into the Slough channel via the ditch on the east side of the road. Field visits during and after the event indicated that both flow paths were engaged at some point, with water levels high enough to flow through the vent slots for more than 12 days. Reference to Figure 4-4 shows that water surface elevations rose above 7.5 feet for approximately

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five hours on November 30, 2012 (WY2013) before the beach naturally breached. Flow upchannel in this latter event was particularly high because the Shell Road pump station was inoperable at the time and water was flowing back through the pump outlet pipes into the Slough channel.

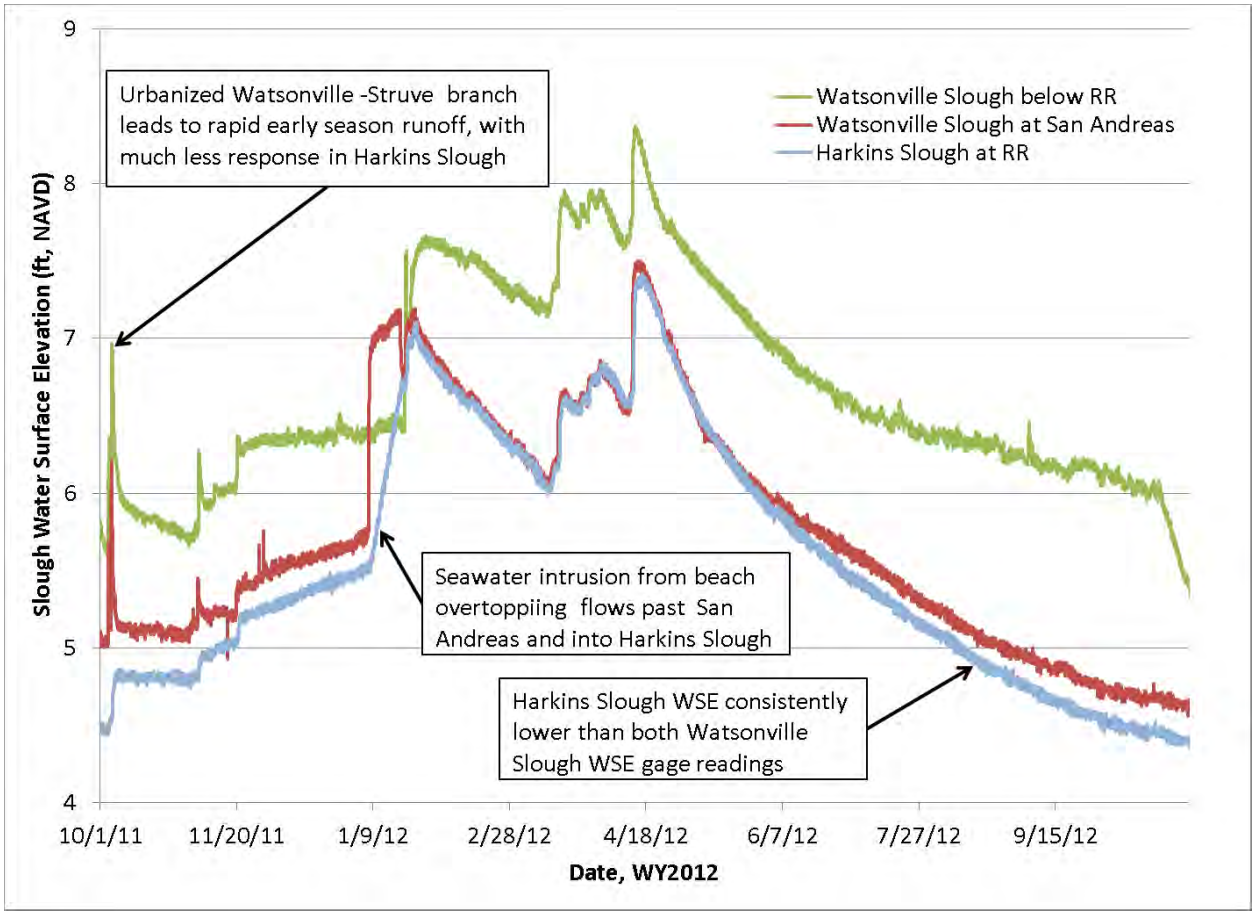
The occurrence of beach overtopping events, with a relatively high probability from mid-September into the winter season, can have a pronounced impact on the Sloughs, particularly when they occur early in the season before sufficient rain has fallen to generate runoff and raise slough water levels (as was the case in WY2012). It is reasonable to assume that beach overtopping risk will increase with increased sea level, and the higher incidence of extreme weather events expected with climate change. The potential impacts of more frequent upchannel seawater intrusions are examined in the model application scenarios discussed in Chapter 10.

4.2.2.3 Slough Interactions (a Complicated Confluence)

The dense array of monitoring stations tracking water surface elevations near the confluence of the Harkins and Watsonville-Struve branches of the system provided particular insight into flow interactions between these major parts of the watershed, and generated important data for validating the project hydrologic model.

The actual confluence point is well triangulated by three of the previously established monitoring stations installed by PVWMA. The downstream conditions are captured by the Watsonville Slough at San Andreas Road (WS@SA) gage and upstream conditions are recorded at Watsonville Slough below the Railroad (WS@RR) and at Harkins Slough at the Railroad (HS@RR). Although the WS@RR gage is located downstream of the twin 60-inch corrugated metal pipe culverts at the Railroad crossing, the measured water elevations at that point are indicative of upstream channel conditions as well, since the Project Team never observed flow rates high enough to create a significant sustained difference in elevation between the two ends of the culverts. Observations compiled over the course of an entire water year are particularly revealing and those for WY2012 are shown in Figure 4-6.

Figure 4-6 Water surface elevations measured during WY2012 in the vicinity of the Harkins Slough confluence with the Watsonville Slough channel as collected at previously established PVWMA stations.



The water elevation data for WY2012 show distinct flow regimes that are typical of each annual cycle and specific events that are less frequent but evident in this particular year. Significant findings include:

- *Relative water levels*. Due at least in part to the "dams" discussed previously, water levels in the Watsonville-Struve branch of the system are perennially higher than those in Harkins Slough. The difference measured in WY2012 was generally on the order of one foot, but increased during the summer season.

- *Response to storm events*. Water levels in the Watsonville Slough channel (both at the Railroad and at San Andreas Road) show a distinct transition from a rapidly changing regime in the earliest part of the water year to a more gradual response after November 14. Field observations indicate that the more peaked

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responses to the first storms (October 5 and November 6) reflect the fact that upstream ponded areas had not filled enough to reestablish direct outflow to the channel. For example, the extensively flooded bottomlands of Struve Slough (including Hansen Slough) are only significantly connected to the channel once they reach an elevation of roughly six feet. Therefore, the signals from the first storms represent runoff to the channel that is not attenuated by passing through ponded areas. Once enough runoff has occurred to raise Struve Slough (and the open water portions of Watsonville Slough upstream of Highway 1) to engage the channel, the large pool areas work to greatly smooth storm hydrographs, with runoff released downstream over a period of days and weeks rather than hours.

- *Seawater intrusion*. The upstream seawater encroachment from the January 2012 beach overtopping event is clearly evident with a sharp jump in stage at San Andreas Road beginning on the morning of January 7, 2012 (Figure 4-6). Water levels began to rise in Harkins Slough thereafter, however much more slowly given the large surface area of the Slough and the weir at the confluence that restricted flow. High inflow rates persisted into Harkins Slough until January 19, 2012, when the effects of the mechanical breaching of the beach bar were translated upstream. Based on the change in water surface elevation over this period, we estimate that approximately 270 acre-feet of ocean water moved upstream and into Harkins Slough. One direct implication of this event was that PVWMA reported that it was not able to run the recharge diversion pumps in WY2012 due to excessive salinity levels in Harkins Slough. Although unfortunate from a water- production perspective, the absence of pumping offered an excellent control situation for the hydrologic modeling and water balance portions of the project work. It is equally important to note that the WS@RR gage showed very little or no significant change in elevation during the seawater intrusion event. This implies little if any water moved up through the culverts at the RR, a fact that is almost certainly explained by the highly obstructed intervening channel reach forcing flow to take the easier path over the confluence weir into Harkins Slough.

- *Overflows into Harkins Slough*. Although not readily evident in Figure 4-6, field observations confirmed that flow regularly breaks out of the Watsonville Slough channel on the right bank just downstream of the Railroad crossing. The

breakout is the direct result of the "dammed" condition of the channel downstream and occurs at and above elevations of roughly seven feet as recorded at the WS@RR gage. The overbank flows proceed across the Knox property, with the majority conveyed under the intervening farm road in an 18-inch culvert and into Harkins Slough as overland flow (see path 2 on Figure 4-7). The remainder of the breakout flow runs along the farm road and into Harkins Slough just below the farm road crossing. The monitoring data shows that this overbank flow persisted for approximately four months, even in the dry conditions of WY2012. Average overflow rates during this period were on the order of several cfs, so total runoff volume directed to Harkins Slough by this path could be 500 acre-feet or more.

Figure 4-7 Seasonally varying flow paths in the vicinity of Watsonville Slough channel and Harkins Slough at and near their confluence.

- *Flow direction at the Harkins Slough weir*. A finding with important implications for the hydrologic modeling component of the project work relates to the duration of outflow from Harkins Slough. The previous discussion noted that water can move up into Harkins Slough after beach overtopping events of sufficient magnitude and duration (though infrequently) and from breakout

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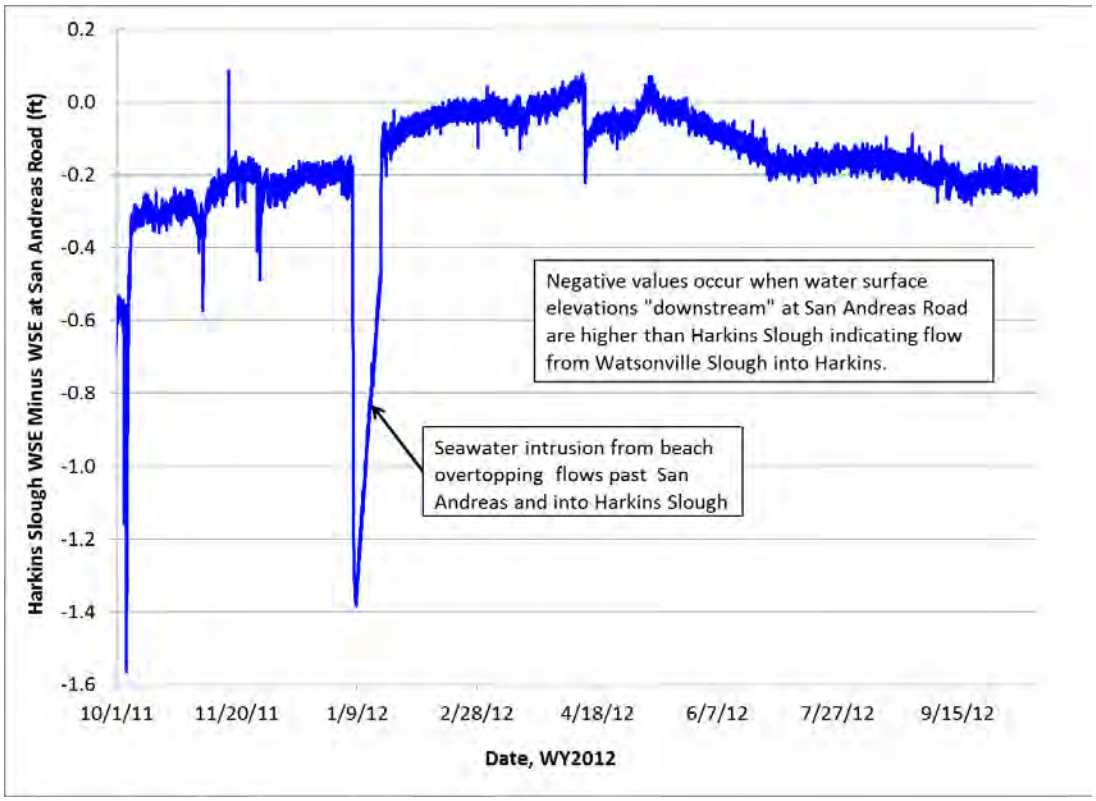
flows originating just downstream of the Railroad crossing (observed to be of relatively high flow and long duration in each year of the study). However, close inspection of the relative elevation of the water surface as measured in Harkins Slough and that at San Andreas Road shows that inflow from the Watsonville Slough channel into Harkins Slough is very persistent. A plot of the difference between the two water surface elevation records is provided in Figure 4-8.

Figure 4-8 Difference in measured water surface elevations between Harkins Slough at the Railroad (HS@RR) and Watsonville Slough at San Andreas Road (WS@SA). Note that negative values are indicative of flow into Harkins Slough passing over or leaking through the weir installed in the Harkins Slough channel at the confluence.



The data are consistent with field observations and show that outflow from Harkins Slough, even absent recharge diversion pumping, is of relatively short duration in the context of the entire water year. Coupled with the observation that some 500 acre-feet of the outflow actually originates in the Watsonville-Struve branch of the system as overflow to Harkins, the data is strong evidence of low overall winter runoff rates and

low dry season baseflows from the sub-watersheds that drain directly into Harkins Slough.

4.3 Stream Gaging Results

The stream gaging portion of the DCP identified an ambitious suite of monitoring stations intended to generate stream flow data for as representative an area and mix of land use types as practical. Although intended to cover two full water years, issues with contracting at the start and the need to have compiled stream flow records well before the end of the contract period led to curtailed datasets. Yet, the distribution of rainfall in the two dry years of the study period could not have been more propitious for project timing. Stream gage installations were generally in place (with one exception) before the concentrated spring rains of WY2012 and the early occurrence of rain in WY2013 allowed for final dataset processing essentially concurrent with the end of significant rainfall.

An overview of the stream gage results is provided in Table 4-2 below and the following sub-sections discuss particularly noteworthy findings from the various gage sites on a location-by-location basis. More complete data for each site is presented in Appendices B and C and is available digitally on request.

Table 4-3 Summary of stream gaging stations and parameters observed during the study period (WY2012 – WY2013)

	Gaging Station					
	HSBV	**GSBL**	**SSES**	**SSWS**	**WSHO**	**WS@SA**
Sub-watershed	L	J	E	F	D	n.a.
Date installed	11/30/2011	3/12/2012	12/15/2011	1/12/2012	4/11/2012	Existing
Last date compiled	1/18/2013	1/15/2013	1/18/2013	1/18/2013	4/30/2013	1/14/2013
Area at gage (*acres*)	2,721	1,244	845	378	1,469	11,332
% of total sub-watershed	73	81	74	62	100	95
Peak flow recorded (*cfs*)	96	43	19	81	29	15
Date of peak	4/13/2012	4/13/2012	4/13/2012	4/13/2012	12/23/2012	12/30/2012
Unit peak flow (cfs/sq mile)	23	22	14	137	13	0.8
Total runoff (ac-ft)	274	141	102	406	*426*	*1,407*
Effective runoff coefficient	0.04	0.07	0.06	0.55	*0.24*	*0.09*

Notes:
Total runoff and effective runoff coefficient for WSHO are for WY2013 only due large upstream storage areas and late gage installation in WY2012.
Total runoff and effective runoff coefficient for WS@SA are for WY2012 only due large upstream storage areas and incomplet e drawdown by 01/2013.
HSBV = Harkins Slough at Buena Vista GSBL = Gallighan Slough below Landfill SSES = East Struve Slough at Main Street
SSWS = West Struve Slough at High School WSHO = Watsonville Slough at Highway 1 WS@SA = Watsonville Slough at San Andreas Road
HSBV and SSWS both out of service mid-May to early August 2012, baseflow estimated for those records and included in total runoff.

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The upland stream gages (those except WS@SA) directly measured runoff from 6,657 acres or 56 percent of the entire watershed. Given its downstream location, the WS@SA gage measured runoff from fully 95 percent of the overall watershed. All stream gaging results should be viewed in the context of the dry conditions that prevailed during the study period.

4.3.1 HARKINS SLOUGH AT BUENA VISTA (HSBV)

Sub-watershed L is the largest sub-watershed (see Chapter 5) and the gage site at Buena Vista Road included the largest contributing area of all the upland stream gages at 2,721 acres. The gage installation was intended to provide stream flow data representative of the northern and predominantly undeveloped and sandy portions of the overall Watsonville Sloughs watershed. The gage location proved to be very good, with excellent downstream control provided by the armored channel bed under the Buena Vista Road bridge and low potential for backwater effects given the grade break at the end of the armored section.

Despite the large drainage area size and early equipment installation date, total runoff at this gage was quite low. The effective runoff coefficient for the period was calculated as 0.04. This means that only four percent of the rainfall volume over the gaging period actually resulted in stream flow runoff at the site, indicating high loss rates in the sub-watershed. The peak flow of 96 cfs was recorded during the storm of April 13, 2012. On a unit area basis, this peak flow works out to 23 cfs per square mile.[9] Late summer baseflow values were on the order of only 0.02 cfs and did not show any strong indication of being impacted by irrigation of upstream areas.

4.3.2 GALLIGHAN SLOUGH BELOW LANDFILL (GSBL)

The second stream flow gaging installation intended to characterize the undeveloped portions of the overall watershed was the GSBL site in Sub-watershed J located downstream of Buena Vista Road adjacent to the County landfill. The drainage area of 1,244 acres at the gage is roughly 46 percent of the size of that at the HSBV gage. The gage location proved to be fair to good, with the primary concern being potential

9 As one point of reference, the highest peak unit discharge in the adjacent 27.8-square mile Corralitos Creek watershed for the same period was 117 cfs/square mile recorded on December 23, 2012. (USGS gage data for Site 11159200, Corralitos Creek at Freedom)

rating curve shifts associated with intra- and inter-storm scour of the sand bed that characterizes the reach. The low-gradient channel also presents the possibility of backwater effects at high flow, though none were conclusively identified over the study period.

Total runoff volume over the gaging period was on the order of 140 acre-feet. Despite the high proportion of sandy soils in the contributing area, the effective runoff coefficient was 0.07. This value is nearly twice that observed at the HSBV gage and may reflect higher runoff volumes associated with improved drainage around the landfill or more extensive use of plastic observed in adjacent agriculture fields. Peak discharge at this site was also recorded on April 13, 2012 and measured at 43 cfs, equivalent to a unit peak discharge of 22 cfs/square mile. Base flow at this location at the end of summer was on the order of 0.02 cfs, or roughly the same as at HSBV despite the smaller watershed area. A two-month period of elevated baseflow (average of 0.07 cfs) was observed in June and July of 2012, although work was not carried out to identify the provenance of this additional flow that amounted to approximately six acre-feet of additional runoff over the lower baseflow conditions. A reasonable possibility is irrigation return flows from one or more agriculture operations.

4.3.3 EAST STRUVE SLOUGH AT MAIN STREET (SSES)

The first of two stream flow gages in the Struve Slough portion of the watershed was installed on the East Struve channel upstream of Main Street in December of 2011. The drainage area at the gage site is 845 acres or 74 percent of the total area of Sub-watershed E delineated for the project HMS model. The site was originally selected to measure runoff from a highly urbanized area and to take advantage of the excellent hydraulic control of the culvert under the large Main Street embankment. The latter hydraulic control was indeed effective, however, the gaging results are very difficult to explain in terms of anticipated runoff volumes and the choice of gaging site may have been inappropriate. The site should be flagged for further investigation in the future to better verify the amount and timing of runoff passing downstream from Main Street.

Despite the high level of urbanization in the contributing drainage area, total runoff volume was measured as only 106 acre-feet. This yields an effective runoff coefficient of 0.06, on par with that for the GBSL gage, but very difficult to reconcile with the high levels of impervious cover upstream (on the order of 47 percent, see Chapter 5). Measured peak flow recorded on April 13, 2012, was also low at 19 cfs. This is

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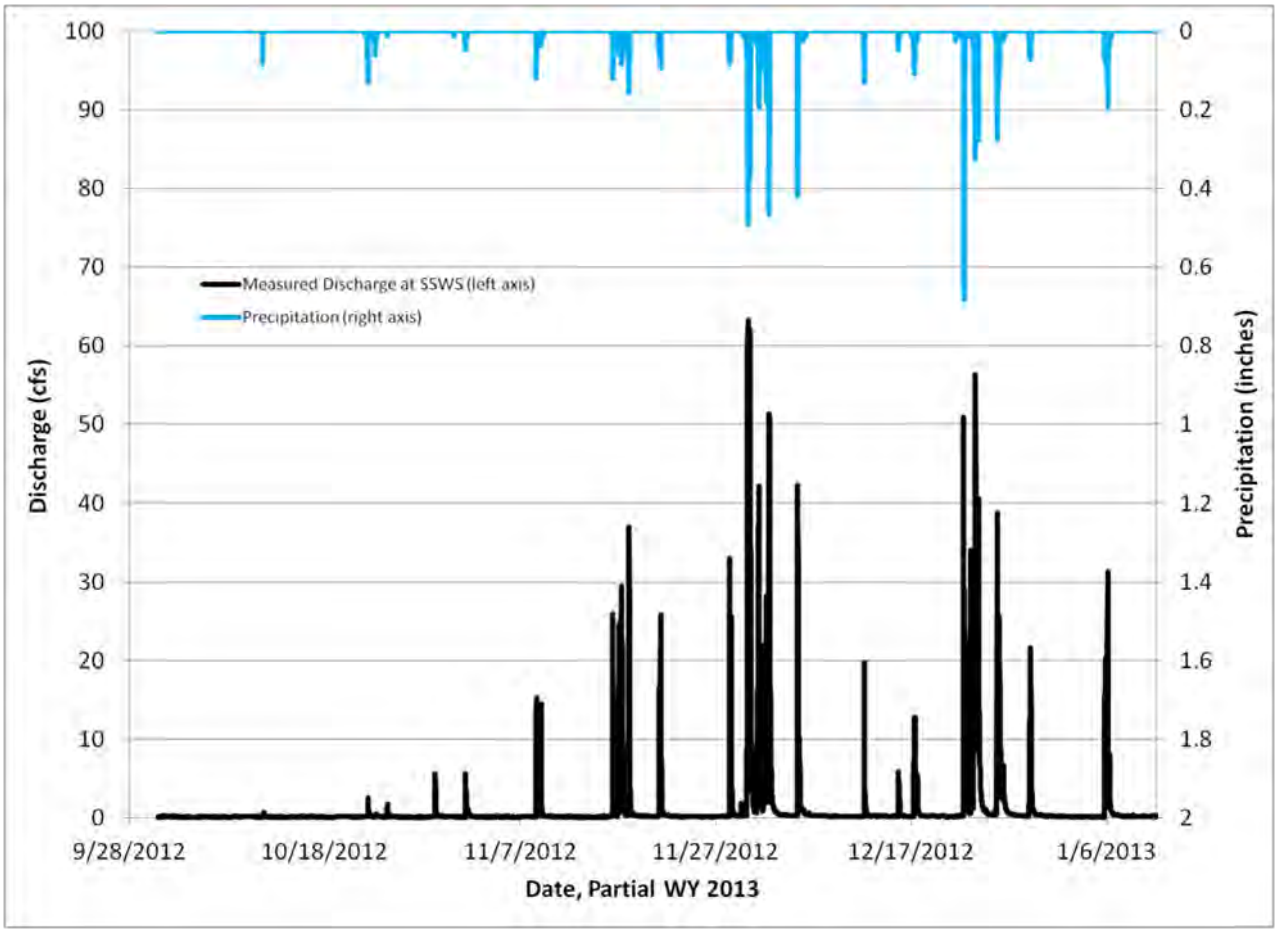
equivalent to a unit peak discharge of 14 cfs/square mile, which is extremely low for an urbanized setting, though the extensive available ponding areas upstream would be expected to attenuate peak discharge to a substantial degree. Discharge at this gage site was non-existent for extended periods of time, so there is effectively no baseflow. This also, is extremely hard to reconcile with an urban setting.

The gaging data for this site including metrics of total runoff volume, peak flow, and baseflow are all consistent with the explanation that the sump area created by the Main Street fill first ponds and then percolates the majority of runoff from the upstream area. The sump area lies four to five feet above the flooded portions of Struve Slough south of Main Street, so groundwater flow under the embankment is possible. However, work to ascertain the true quantities of runoff originating in the upstream watershed was beyond the scope of the project work. Given the large portion of the overall watershed impervious area above Main Street, the possibility that a significant part of the resulting runoff is delivered to Struve Slough by a pathway other than direct runoff has important implications for the calibration of both the project hydrologic and hydraulic models.

4.3.4 WEST STRUVE SLOUGH AT THE HIGH SCHOOL (SSWS)

The second stream flow gage in the Struve Slough portion of the watershed was installed on the West Struve channel below the high school in March of 2012. The watershed area at this gage site is 378 acres. The site was selected as another point to measure runoff from a highly urbanized drainage area. Overall the gaging location proved to be relatively good with a stable bed, only somewhat encumbered by the need to measure overbank flows on a highly vegetated lower floodplain terrace. The wide floodplain serves to limit the potential for backwater effects. Measured discharge for this gage site for WY2013 is illustrated in Figure 4-9 below.

Figure 4-9 Measured discharge and rainfall for the first part of WY2013 at the SSWS gage site (West Struve at High School).



Total runoff measured at this site was much higher than at the three previously discussed gage sites. Figure 4-9 shows that this site responds with a large discharge signal for almost all rain events, including those early in the rainfall season, a characteristic typical of drainage areas with a high level of impervious cover. A total of 406 acre-feet of runoff was measured, yielding an effective runoff coefficient of 0.55, nearly an order of magnitude higher than the coefficients for the other gage sites discussed above. A similar trend holds true with peak flow rates. The peak discharge (again on April 13, 2012) was roughly 80 cfs, giving a unit peak discharge of 137 cfs/square mile. This metric is also on the order of 10 times those for the sites discussed previously. Late season baseflow values were also elevated compared to the other gage sites, with late summer values on the order of 0.1 cfs, which at 0.17 cfs/square mile is not unusually high for dry-season flow rates from an urbanized drainage area.

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The measured stream flow at this gage site is strongly indicative that the anomalously low discharge values measured at the SSES and WSHO sites (the latter discussed below) can be explained by factors other than atypically low runoff rates.

4.3.5 WATSONVILLE SLOUGH AT HIGHWAY 1 (WSHO)

Stream flow in the upper reaches of the Watsonville Slough watershed was gaged at a telemetered site located at the State Highway 1 crossing. The watershed area at the gage site is 1,469 acres. The site was selected as a major point of concentration for the largest urbanized drainage area in the entire watershed. There is extensive flooded bottomland area upstream of the gage site (including previously constructed wetland restorations), and it was understood at the time of the gage installation that the site would be more appropriate for assessing total runoff volumes rather than instantaneous peak flow rates due to the upstream storage areas. The gage site proved to be additionally problematic due to sedimentation issues (silt and trash) as well as emergent vegetation and interruptions related to upstream wetland restoration construction. The overall rating curve is considered fair due to these factors and the potential for backwatering by the culvert crossing downstream at Lee Road.

Given the mid-April 2012 installation date for the gage and the high upstream storage volume, it is difficult to accurately assess runoff rates for WY2012. However, the total measured runoff for the partial WY2013 alone was 426 acre-feet, which is higher than any of the other upland gage sites. Though high, this represents an effective runoff coefficient of only 0.24, or roughly half of that observed at Station SSWS. This represents less of a departure from expectations than Station SSES, but is still difficult to explain given that the contributing drainage area is nearly 50 percent impervious. Peak discharge was expectedly low at 29 cfs, which is consistent with the upstream storage. Conversely, baseflow rates were higher than the other gage sites, showing a minimum at the end of the summer season on the order of 0.2 cfs.

As with the East Struve gage at Main Street, the Watsonville Slough at Highway 1 location should be flagged for further study to assess the true magnitude of runoff volume entering the upper extents of this branch of the system.

4.3.6 WATSONVILLE SLOUGH AT SAN ANDREAS (WS@SA)

This monitoring site was previously established as a water level and conductivity monitoring location by PVWMA. However, discharge measurements were completed during most field visits and sufficient data was collected to complete a rating curve for

this location near the downstream end of the overall watershed and downstream of all the major flooded bottomland areas. With a contributing drainage area of 11,332 acres, the site allows for direct measurement of the outflow from 95 percent of the total watershed area. For stream gaging purposes, the site is fair with a notable sandy sediment dam creating a control point. Shortcomings include the potential for erosion or maintenance removal of the bar.

Runoff patterns measured at this site, like those at WSHO, are heavily impacted by the very large storage volumes upstream. For this reason, the full WY2012 gaging record is a more appropriate metric than the partial record for WY2013. For the former water year, total outflow volume was measured as 1,430 acre-feet. However, it is important to recall that this value includes approximately 270 acre-feet of water that moved upchannel into the system during the beach overtopping event of January 2012. Therefore, the effective runoff coefficient should be based on a total watershed runoff of roughly 1,160 acre-feet, yielding a very low coefficient of 0.08. It is important to acknowledge that runoff at this point in the watershed is reduced by evaporation in the upstream open water areas. Nonetheless, the gage data supports the finding that only approximately six percent of the rainfall volume that falls on the overall watershed results in outflow past San Andreas Road. This is an important observation with respect to the hydrologic modeling and water balance updates presented in ensuing chapters.

4.4 Groundwater Monitoring Findings

Piezometer installations for monitoring groundwater levels were completed in mid-April 2012 and data has been collected and post-processed for the period through mid-January 2013. The compiled data is illustrated in Figure 4-10 presenting the time series of groundwater elevations, compared to water surface elevation data collected in Watsonville Slough upstream of the Railroad (Station WSUR) and rainfall measured at Shell Road (Station WSUS).

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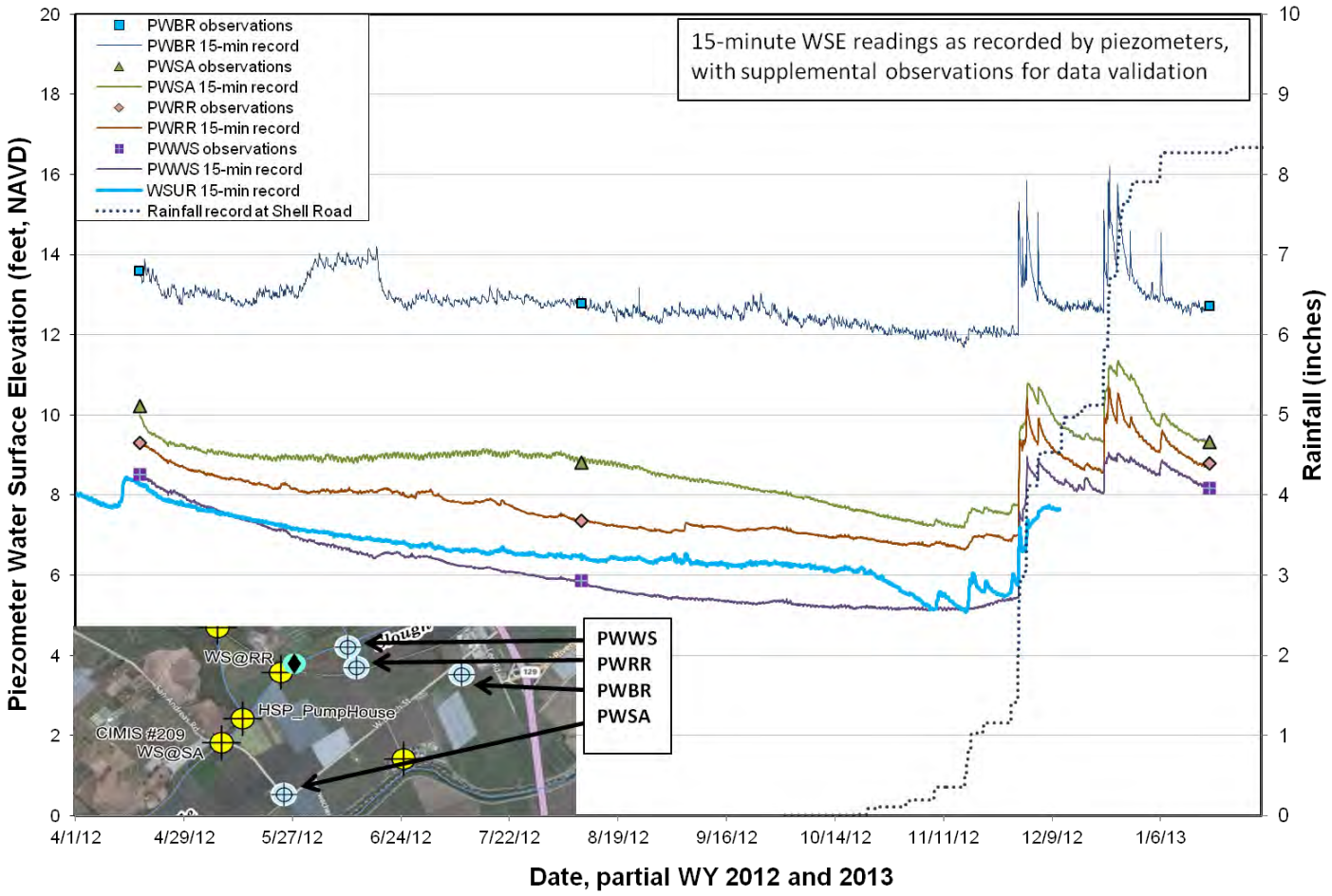
Figure 4-10 Groundwater monitoring data from piezometers installed along the Watsonville Slough – Pajaro River floodplain.



Review of the data yielded the following observations and findings:

- All water levels declined rapidly during the first one to two weeks after installation. We interpret this decline as natural drainage from the storm of April 13-14, 2012. By the end of April 2012, water levels in the piezometers stood at very similar levels to their mid-winter base levels in mid-January 2013, about two weeks following the end of the December 2012 rainfall. Measured as post-winter storm base levels, there was very little change in shallow groundwater levels during the monitoring period.

- With the exception of Piezometer PWWS adjacent to the Watsonville Slough channel, groundwater levels were always higher than in the sloughs. This

suggests a slope in water elevations toward the north, and thus presumably shallow groundwater flow in the area of the piezometer array is generally toward Watsonville Slough.

- Piezometer PWWS indicates that shallow groundwater levels directly adjacent to the slough channel are higher than the water surface of the slough in the winter and generally lower in the summer. This may imply that water levels in the slough are sustained at a higher level during summer from water emanating from: (a) areas upstream in the urbanized portions of the watershed and/or (b) lateral inflow from the valley floor (perhaps through tile drains), while evapotranspiration continues to lower water levels in the heavy clay soils along the slough. By mid-October 2012, additional inflows ceased, with water levels in the slough falling – relatively quickly – to the groundwater level characterized by Piezometer PWWS. It is possible that inflow to the Slough channel may have been cut off in mid-October 2012 due to restoration work upstream. Piezometers PWWR and PWWS appear to show many (but not all) of the same micro-fluctuations during the recession limb throughout much of the dry season of 2012, suggesting that the shallow groundwater zone may be partially confined.

- Piezometer PWSA water levels remained nearly constant between May and early August, 2012. We suspect this was due to irrigation of fields near this piezometer that may have diminished in early August 2012.

- Through late May and early June, 2012, Piezometer PWBR appeared to have an elevated water surface, not observed in the other piezometers. This may be a result of irrigation in the field surrounding the piezometer, or activities at the adjacent supplemental well. The piezometer responded rapidly and sensitively to all rainfall events, suggesting that the aquifer near Piezometer PWBR is closely linked to the surface water system, perhaps through a tile drain network.

- Piezometers PWRR, PWSA, and possibly PWBR responded to minor rains of early October, including rain events of a fraction of an inch, with all piezometers responding by the mid-November rain which brought cumulative seasonal rainfall to about 1.3 inches. The rapid and consistent response suggests that soils are kept nearly saturated through the summer.

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- Groundwater levels declined seasonally (beginning about May 1) by about 0.7 feet in Piezometer PWBR, with the sandiest soils, and about 2.7 feet in Piezometer PWWS, with the most clay-rich soils. Over the course of the summer, the groundwater gradient seems to steepen gradually toward the slough.

In summary, it appears that during winter months, water that has infiltrated into the lower Watsonville Slough/Pajaro River floodplain moves toward the slough. Groundwater connections with the slough may be very local during the dry season, depending perhaps on very local changes in slough water surface elevations, with some segments of the slough interacting with local groundwater, and some not. In part, the extent of these interactions reflects the extent and condition of the tile drain systems that underlie much of the area in the lower valley.

Based on these findings, the decision was made to not include off-channel storage areas as a means to account for groundwater exchange in the project hydraulic model, as had been originally considered. Shallow groundwater inflows can be accounted for adequately in HEC-HMS, thus, allowing for the hydraulic model domain to be limited to surface water dynamics only.

4.5 Additional Data Collected

Maintenance issues with the Shell Road Pump Station in the winter season of WY2013 meant that the pumps did not run for much of the rainy season, going off-line around November 1, 2012 and not coming back on-line until February 2013. From the perspective of the study, this offered an opportunity to observe how the infrastructure at Shell Road operates under gravity-flow conditions. However, work on the water balance and modeling components of the Project made clear that the discharge rate of the pumps are an important factor in understanding the hydrology and hydraulics of the watershed. Each pump outlet pipe is equipped with a magnetic flow meter, but the placement of the meters is such that it is not clear whether they are accurately measuring the actual discharge.

4.5.1 SHELL ROAD PUMP CALIBRATION

Based on the above observation, arrangements were made with County staff to carry out discharge measurements of the two pumps under controlled conditions once they were brought back on line in the spring of 2013. The measurements consisted of operation of each pump individually and both pumps together while velocity

measurements were taken at the outlet culvert on the downstream side of Shell Road. The resulting integrated point velocities were converted to flow rate based on detailed measurement of the geometry of the culvert outlet, including partial blockage by debris.

The measurements taken on May 21, 2013 found total discharge rates for each pump that were well below previous estimates (e.g. Questa Engineering, 1995). Operated individually, the discharge rate of the north pump was found to be roughly 2,300 gpm (5.1 cfs) and that for the south pump was measured as 2,800 gpm (6.3 cfs). Operated together, the combined flow rated was measured as 5,300 gpm (11.8 cfs) and thus confirms the general accuracy of the individual measurements.

Since previous water balance work was based on a markedly higher assumption of pump discharge rate, the findings of the supplemental pump measurements have critically important ramifications as a downstream check on the modeling work discussed in subsequent chapters.

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5 HYDROLOGIC MODELING

Building the hydrologic modeling tool for the study involved the collection and processing of pertinent data sources in GIS, importing the information into the HEC-HMS software platform, completing calibration runs based on field data collected in the DCP, and post-processing data for use in subsequent portions of the overall project work plan.

As mentioned in earlier chapters, a primary goal was to provide an accessible modeling tool that can be adopted for and adapted to a range of applications related to managing the resources inherent in the Watsonville Sloughs watershed. To this end, a separate Watsonville Sloughs Modeling Tools Users' Guide has been compiled. The reader is directed to that document for detailed information on parameterizing and running the model. This chapter provides a more general overview of the most important aspects of the hydrologic modeling and the results from the final model runs.

5.1 Model Structure

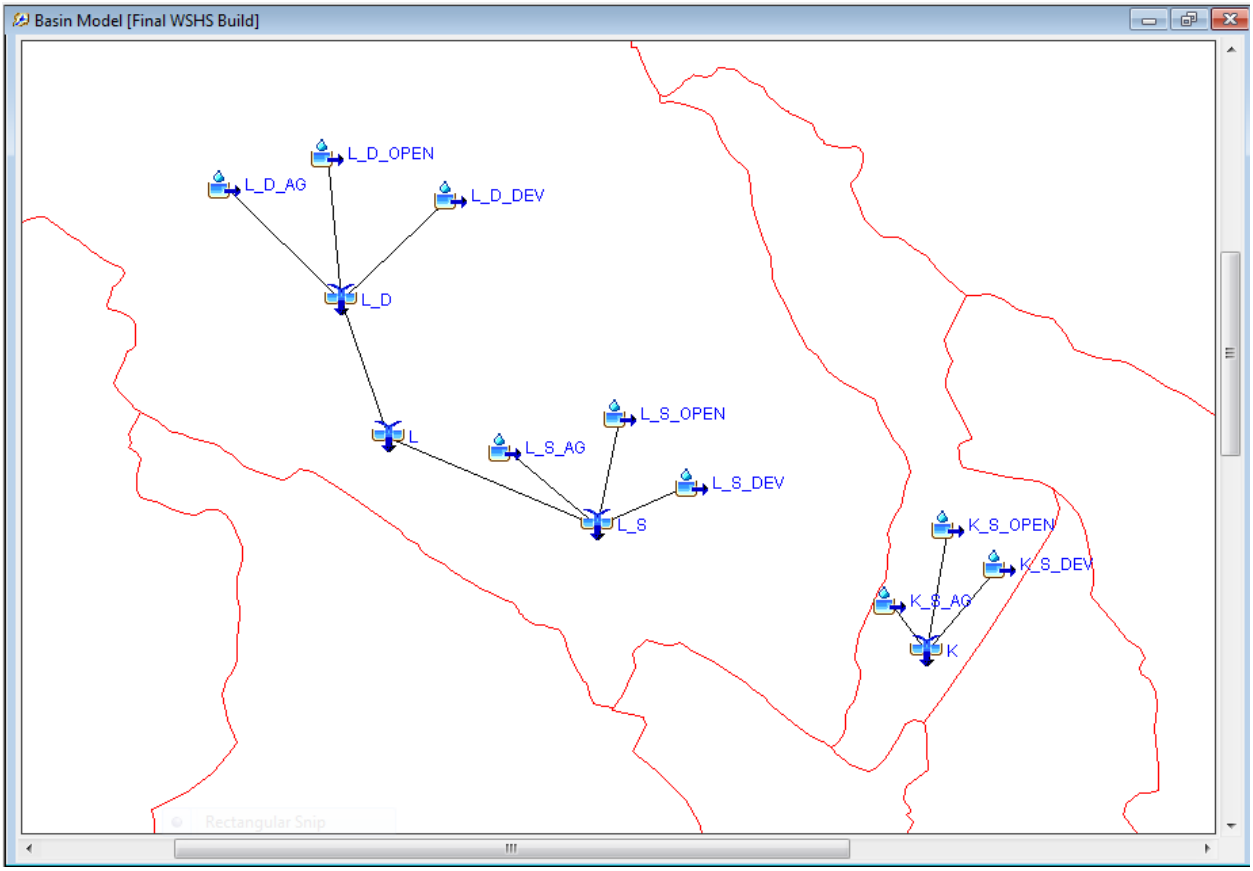
All hydrologic modeling was completed using Version 3.5 of the U.S. Army Corps of Engineers' HEC-HMS software package, a readily-available and well-supported platform available from the Hydrologic Engineering Center (HEC) free of charge. Given the overall goal of simulating water movement throughout the lower slough system over a real world suite of conditions, the decision was made from the onset to run the model in continuous simulation mode, rather than the more frequently used single-event format. The former mode of operation is particularly suited for the Watsonville Slough watershed given the very large storage volumes in the slough bottomlands. In such an environment, antecedent conditions and intra- and inter-annual variations in rainfall patterns are extremely important factors that are essentially impossible to capture in single-event models.

After pre-testing various complexities of potential model configurations, a 10-year continuous simulation period was selected as being the largest practically managed time frame. Ideally, a longer period would be selected, with 30 years often cited as best for capturing the full range of climate variability that characterizes coastal California environments. However, we feel that proper selection of an appropriate 10-year data series provides ample information on the response of the system to variability, while keeping run times and data management overhead within tractable limits.[15]

15 Project objectives required that output of the hydrologic model be used as input to a continuous simulation hydraulic model. Computational limitations in that model were actually more important in restricting the time period to 10 years.

The HMS model run in this manner is a "lumped-parameter" application. That is, the sub-watersheds are represented by discrete units that aggregate the actual physical properties of a diverse combination of soil, land use, and cover types. However, the project team recognized the potential interest in assessing the hydrologic responses, and spatial distributions of those responses, from more discrete units than simply one per sub-watershed. Therefore, the model build used multiple "basins" to represent combinations of two important characteristics: deep groundwater recharge potential and land use type. The combination of two recharge types and three land use types yielded a maximum of six discrete basins within each sub-watershed. Figure 5-1 shows the configuration of basins within the sub-watersheds in the northern extent of the geographic model domain.

Figure 5-1. Example of basins within two of the sub-watersheds (K and L) in the HEC-HMS model domain. Note that Sub-watershed K had no deep groundwater recharge areas and therefore had only three discrete basins.



Sub-watershed and basin combinations were labeled following a consistent naming convention, using the format "Sub-Watershed_Recharge Character_Land Use Type". Therefore, basin J_S_AG represents agricultural land use in a shallow groundwater recharge area (e.g. little or no potential for percolation to deeper groundwater basins)

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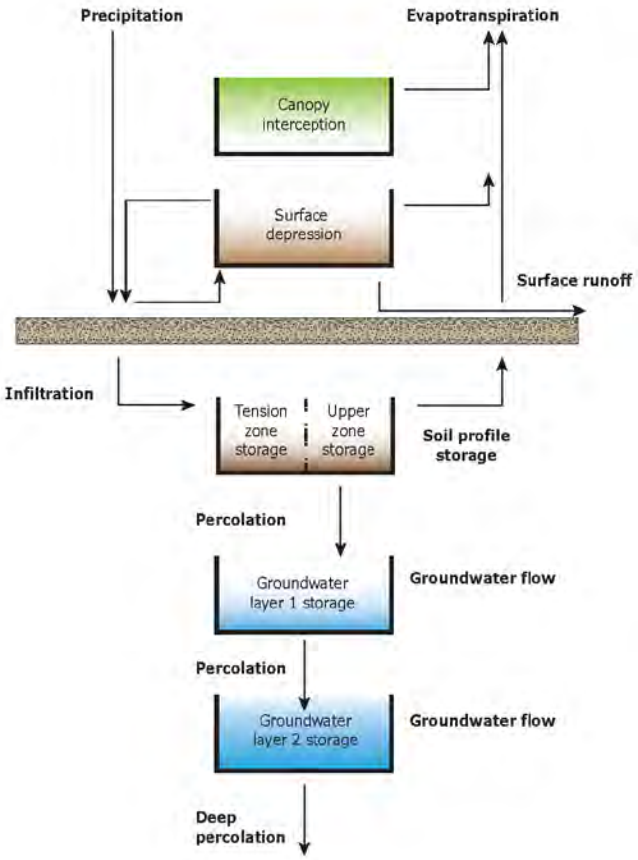
in the J sub-watershed. "D" in the second position indicates deep groundwater recharge potential and the other two land use types are "OPEN" for open space and "DEV" for developed (urbanized). Inundated areas of the watershed were represented by separate "ponded" basins within the model as essentially all rainfall becomes "runoff" with no immediate losses. These areas were identified using the naming convention "Sub-Watershed_Ponded".

It is important to note that the primary desired output of the model was a representative time-series of runoff volume. This again recognizes the importance of the storage volume in the open water portions of the slough system at the outlet of almost all the sub-watersheds, rendering peak flow magnitude of individual storms much less important to downstream hydrodynamics. Of course, extensive datasets on peak flow response were collected as discussed in Chapter 4 and would be especially useful for future modeling efforts of the upland stream network. Rainfall-runoff transformation in the model was accomplished using the Clark Unit hydrograph method. The Soil Moisture Accounting method (SMA) was selected as the loss method most appropriate to the long-term simulation objectives of the modeling effort.

The SMA method provides for a discretely partitioned accounting of losses within the modeled hydrologic cycle for each defined basin in the model. Unlike generalized loss methods, SMA explicitly tracks losses due to canopy storage, surface depression storage, soil profile storage (including tension and void space storage), and storage in up to two underlying groundwater layers as illustrated in Figure 5-2. Losses to evapotranspiration are calculated based on potential evaporation rates (calculated from solar radiation data) and water held in the canopy, surface depression, and soil profile storage areas. Stream flow can originate as surface runoff and (though not explicitly shown in Figure 5-2) as baseflow originating in either of the groundwater layers. Water that moves out as deep percolation is lost to the system. Although SMA can be run with only one of the two groundwater layers, it was applied with both layers to all basins in the project HMS model.

Figure 5-2. Diagram of water fluxes in the Soil Moisture Accounting loss method used in the HEC-HMS model.



5.2 Data Sources and Model Input

A number of publically-available sources were used to parameterize the hydrologic model, with a conscientious effort to set up transparent and easily-applied routines for collecting and manipulating source data in the model build. [16] Data sources were assembled following a tiered approach of transformations using GIS software. The process started with delineation of sub-watershed areas, which were combined with groundwater recharge potential to form the Tier 1 spatial data union designated WS_RC (see Figure 5-3). The WS_RC coverage was then combined with the land use type information to create the Tier 2 spatial data union designated WS_RC_LT. The Tier 2 coverage was then used to create the two Tier 3 coverages. The first of these

16 The interested reader is again directed to the Watsonville Slough Modeling Tools Users' Guide for detailed and step-by-step directions on how to alter the base dataset or assemble different datasets for use in future modeling activities.

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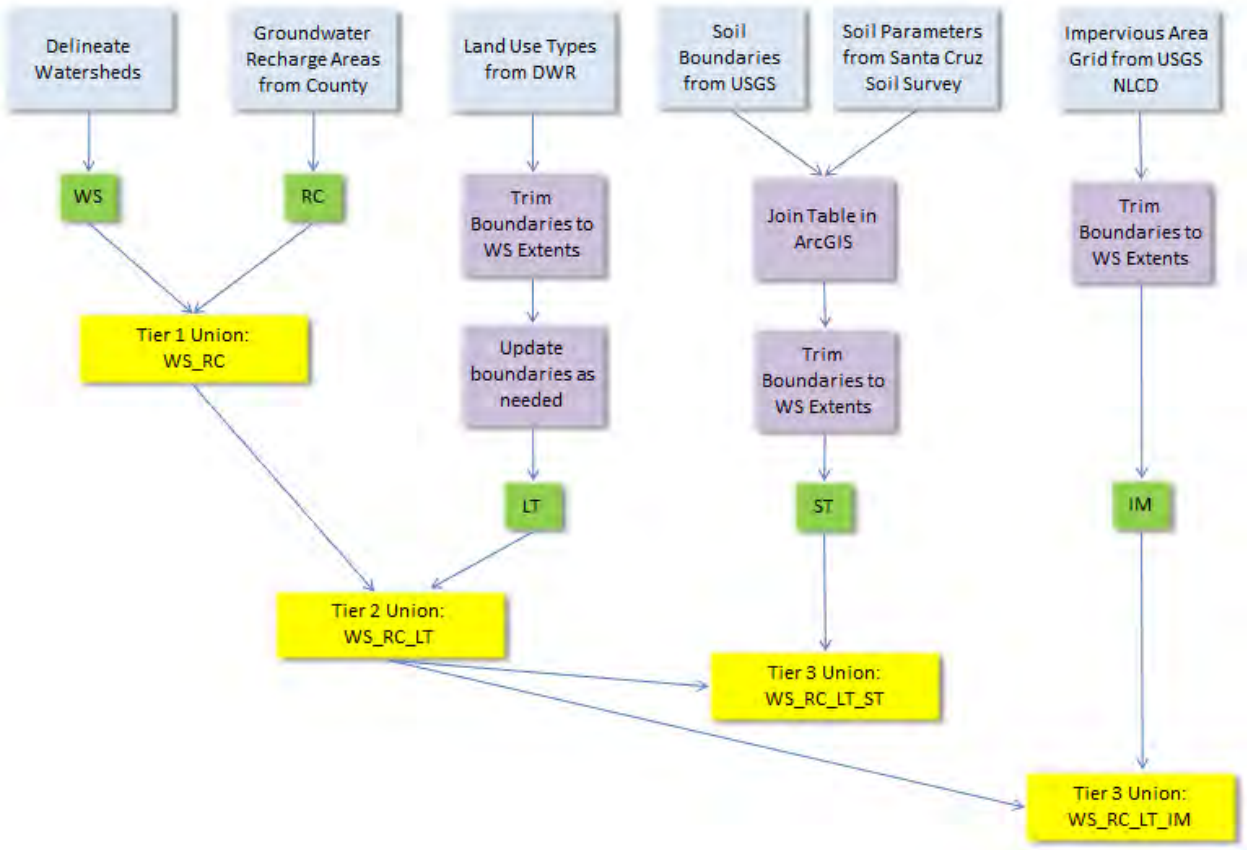
combined soil properties needed to parameterize the SMA component of the HMS model, creating a coverage designated WS_RC_LT_ST. The second Tier 3 coverage was created by the union of the Tier 2 coverage with information on impervious cover to create the WS_RC_LT_IM input.

Figure 5-3. GIS data preparation hierarchy for assembly of input parameters for the HEC-HMS model of the Watsonville Sloughs watershed.



The following sub-sections describe each data source in more detail.

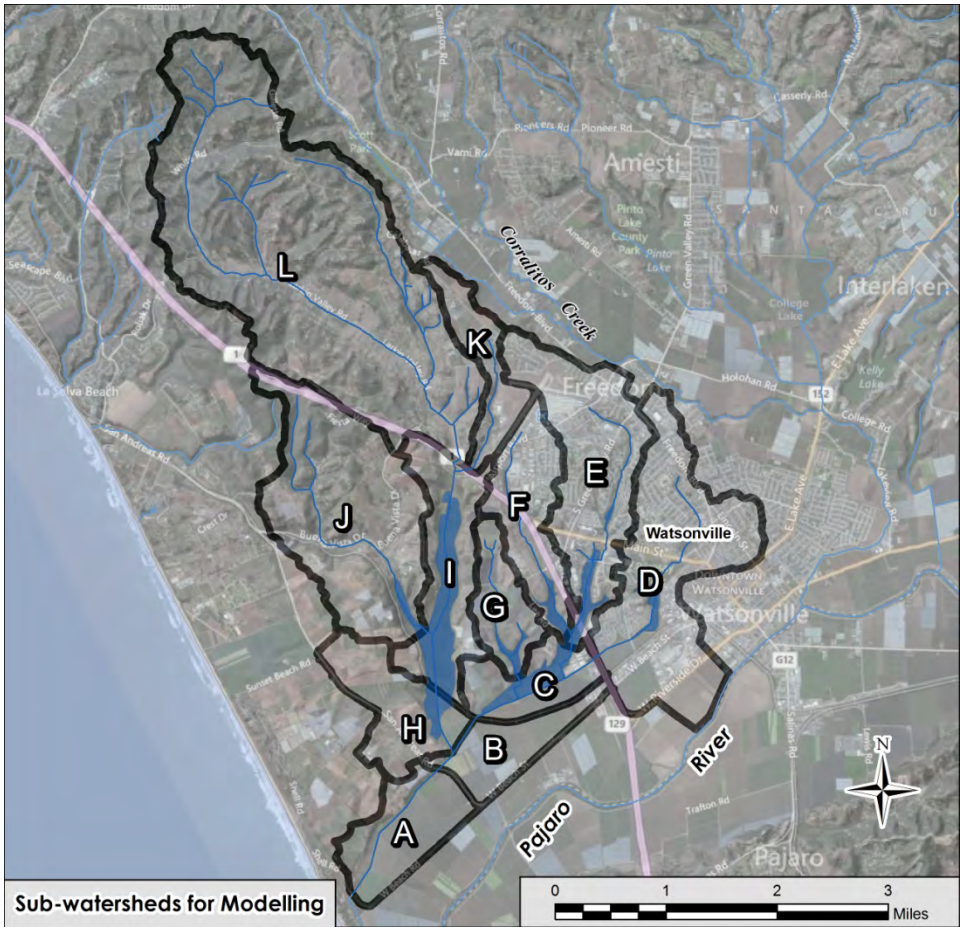
5.2.1 TOPOGRAPHY AND SUB-WATERSHEDS

Topographic information, outside of that collected specifically for the study by EDS (See Chapters 3 and 4), was derived from the county-wide LiDAR database provided by the County of Santa Cruz GIS Department. The information was uploaded into the project GIS database for processing as input data for both the hydrologic and hydraulic modeling work.

The overall watershed was divided into 13 distinct sub-watersheds for modeling and analysis purposes. The sub-watersheds are illustrated in Figure 5-4. The lettered designation for each of the sub-watersheds follows the convention of starting at the

downstream end of the system, then progressing up the Watsonville-Struve branch of the system, followed by the Harkins Slough branch.

Figure 5-4. Sub-watersheds in the Watsonville Sloughs Hydrology Study area.



Area properties by sub-watershed are summarized in Table 5-1. The total watershed area was delineated as 11,867 acres. Sub-watersheds were generally defined based on important points of concentration where main runoff paths combine or flow into the main open water slough areas. The lower watershed, downstream of the flooded bottomland areas, is represented by Sub-watersheds A and B. The Watsonville-Struve branch of the system includes Sub-watersheds C, D, E, F, and G. The remaining Sub-watersheds H, I, J, K, and L constitute the Harkins Slough branch.

The choice of the points of concentration led to a fairly large range in the areas of the sub-watersheds. For example, Sub-watershed L, with an area of 3,742 acres, represents fully 13 percent of the total watershed area. Conversely, the smallest, Sub-watershed G (Hansen Slough), has an area of 352 acres or 3 percent of the total watershed. The extent to which open water has expanded over the last several decades is evident in

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the total ponded area, which was delineated at 548 acres or 5 percent of the total watershed at the time the LiDAR data was collected.

Table 5-1. Sub-watershed designations and overall area parameters.

Sub-Watershed Name	General Location	Total Area	Deep Percolation Area		Shallow Percolation Area		Ponded Area	
		(acres)	*(acres)*	*(%)*	*(acres)*	*(%)*	*(acres)*	*(%)*
A	Watsonville Slough downstream of San Andreas Road	535	147	27	388	73	0	0
B	Watsonville Slough below Railroad	388	16	4	372	96	0	0
C	Watsonville-Struve below Highway 1	409	0	0	319	78	90	22
D	Upper Watsonville Slough	1,469	0	0	1,437	98	31	2
E	East Struve Slough	1,136	0	0	1,070	94	66	6
F	West Struve Slough	607	0	0	575	95	32	5
G	Hansen Slough	356	0	0	330	93	26	7
H	Harkins Slough below Railroad	564	263	47	257	46	44	8
I	Central Harkins Slough	746	38	5	474	64	235	31
J	Gallighan Slough	1,533	653	43	855	56	25	2
K	Harkins Slough - Manfre Tributary	382	0	0	382	100	0	0
L	Upper Harkins Slough	3,742	1,249	33	2,493	67	0	0
Totals		**11,867**	**2,365**	**20**	**8,953**	**75**	**548**	**5**

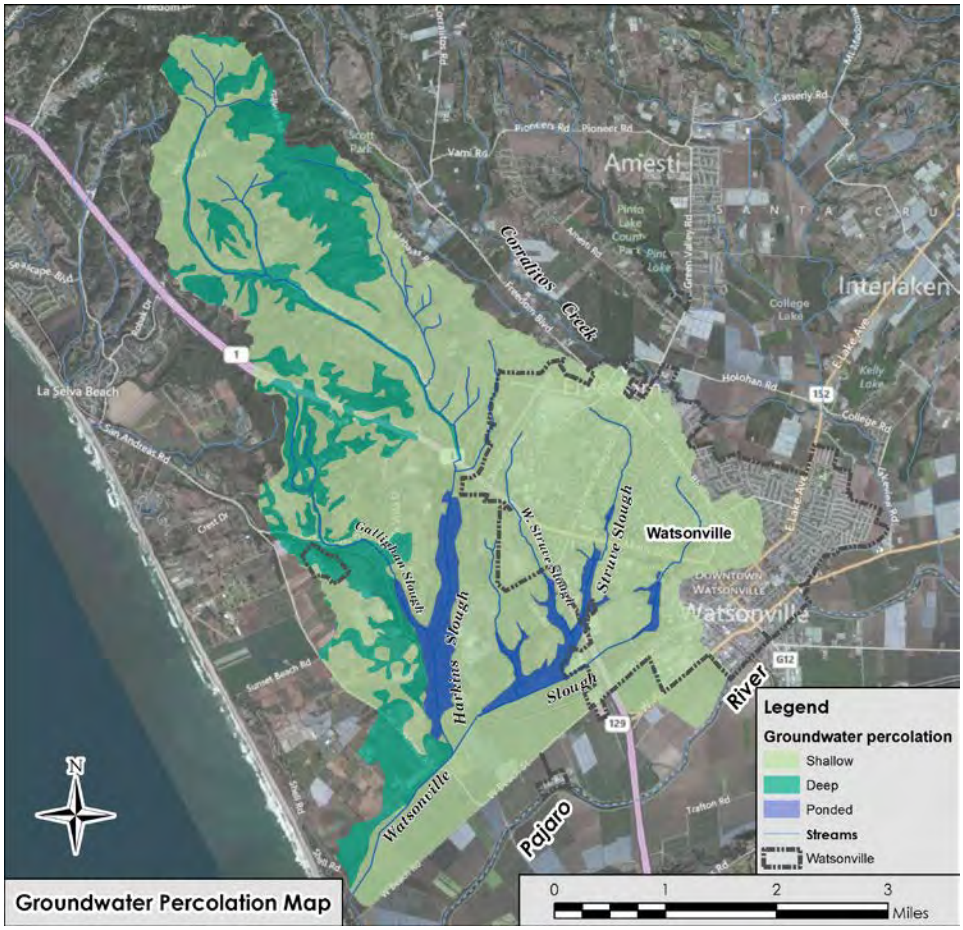
5.2.2 GROUNDWATER RECHARGE AREAS

The potential for recharge to deep groundwater aquifer layers was the primary hydrologic characteristic used to define the basins within each sub-watershed area. The data was compiled from mapping of groundwater recharge areas prepared by the County of Santa Cruz. These areas are shown in Figure 5-5. For GIS data processing, all areas were classified as "D" for deep groundwater recharge potential, "S" for shallow or limited ground water potential, or "P" for areas of ponded water. Those units designated as ponded were not included in subsequent data processing, but were added directly as impervious areas defined as "Ponded" in the final HMS model.

As expected, the areas of deep groundwater recharge potential correspond closely with mapped sandy soil types that are generally classified in Hydrologic Soil Groups A

and B. The mapped recharge areas are asymmetrically distributed, occurring exclusively in the sub-watersheds in the western parts of the watershed. In fact, 6 of the 13 sub-watersheds have no mapped deep recharge areas and only 4 of the 13 have more than 10 percent of their area so mapped.

Figure 5-5. Distribution of deep groundwater recharge areas in the Watsonville Sloughs watershed.



5.2.3 LAND USE TYPES

All sub-watershed basins were further defined on the basis of overall land use type. Source data for these designations came from the California Department of Water Resources and included a fairly large number of land use classifications that were reduced to three overall types as defined in the key provided in the Modeling Tools Users' Guide. The three generalized land use types were: Agricultural, Developed, and Open Space. The distribution of the land use types is illustrated in Figure 5-6.

As seen in Figure 5-5, land use types are more uniformly distributed than recharge areas. Nonetheless, agricultural land uses predominate in the southern portions of the

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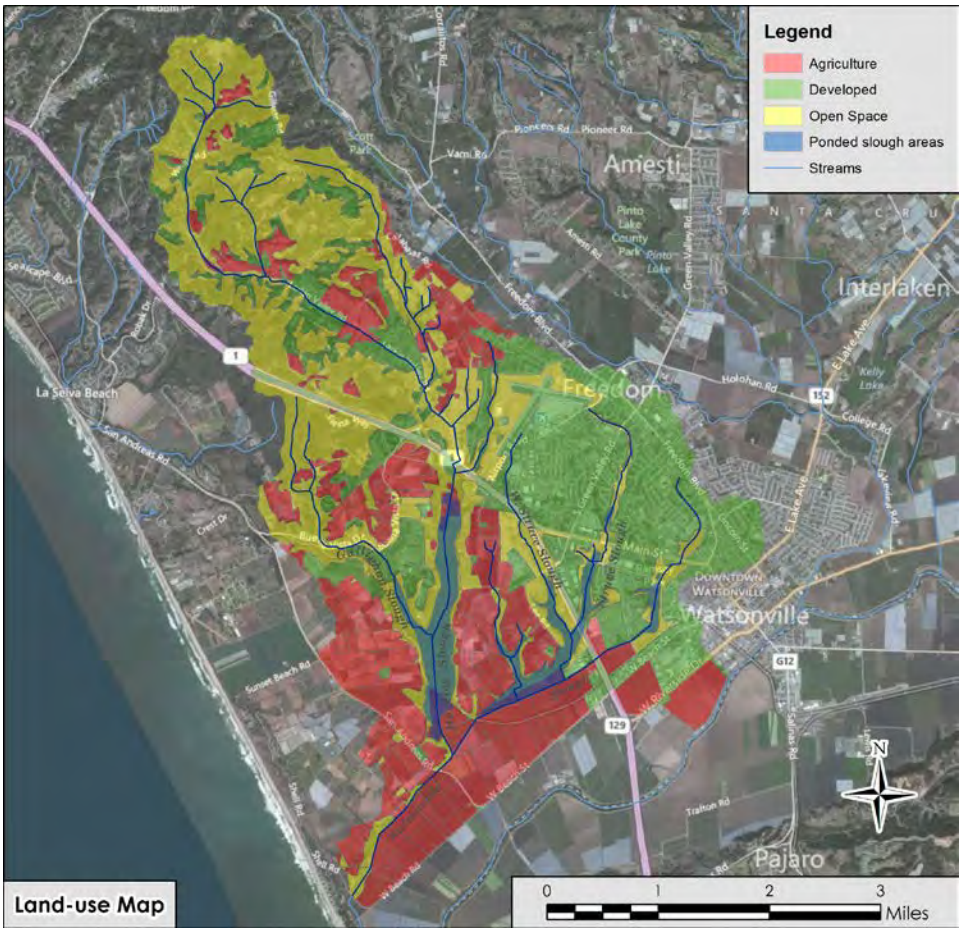
watershed, developed areas in the east (City of Watsonville), and open space areas in the north.

The union of the previously defined sub-watershed and recharge layers in GIS with land use type defined the 54 basins used in the final HMS model.

Figure 5-6. Distribution of the three main land use types within the Watsonville Sloughs watershed per California Department of Water Resources (1997).



5.2.4 SOIL PROPERTIES

Specific hydrologic properties were assigned to the basins in the Tier 3 data processing step. The most complicated of the two Tier 3 data transformations involved the assigning of specific soil properties to each of the 54 basins in the HMS model. Soil property information was compiled from the US General Soil Map (STATSGO2) using NRCS soil classifications. Area weighting was used to average the properties for different soils in the same basin. A roll-up of the data to the full sub-watershed basis is illustrative of the variations in properties across the entire watershed and is provided in Table 5-2.

Table 5-2. Area weighted soil properties summarized by sub-watershed. Note that the areas here are land area only and do not include ponded areas as in Table 5-1.

Sub-Watershed Name	Land Area	Maximum Infiltration	Impervious Cover	Soil Storage	Tension Storage	Soil Percolation
	(acres)	*(acres)*	*(%)*	*(inches)*	*(inches)*	*(in/hr)*
A	535	1.7	3.5	18.5	7.7	1.69
B	388	0.3	8.2	18.6	8.5	0.30
C	319	0.3	12.9	18.5	8.9	0.04
D	1,437	0.4	49.1	18.0	9.5	0.13
E	1,070	0.5	46.9	18.1	8.9	0.04
F	575	0.4	40.3	18.7	9.2	0.04
G	330	0.3	5.7	18.2	9.0	0.04
H	520	3.3	4.2	18.0	6.4	3.27
I	511	1.1	7.3	18.6	7.6	0.50
J	1,508	2.9	7.4	18.4	6.9	2.32
K	382	0.7	13.4	19.4	9.3	0.05
L	3,742	1.9	3.0	18.9	7.2	1.26
Totals or Weighted Average	**11,318**	**1.5**	**16.6**	**18.5**	**7.9**	**1.02**

The summary by sub-watershed shows great variability in maximum infiltration rate (controls water movement into soil profile storage) and soil percolation (used to parameterized movement into and through the groundwater layers). Tension storage (water held in soil profile storage and unavailable for groundwater augment) varied much less. Area weighted soil storage varied hardly at all and could have been treated as a constant with little overall impact on the end results. It is important to note the variation of infiltration and percolation values, with Sub-watersheds H and J standing out as having particularly high values.

5.2.5 IMPERVIOUS COVER

The remaining Tier 3 classification parameter was impervious cover. This information was compiled from the USGS National Land Cover Database 2006 (NLCD 2006) data provided by the County of Santa Cruz GIS Department. The distribution of impervious cover is shown in Figure 5-7 and is included in Table 5-2 as well. Watershed weighted impervious cover is 16.6 percent, but would be much lower except for the high impervious cover values in the urbanized Sub-watersheds D,E, and F.

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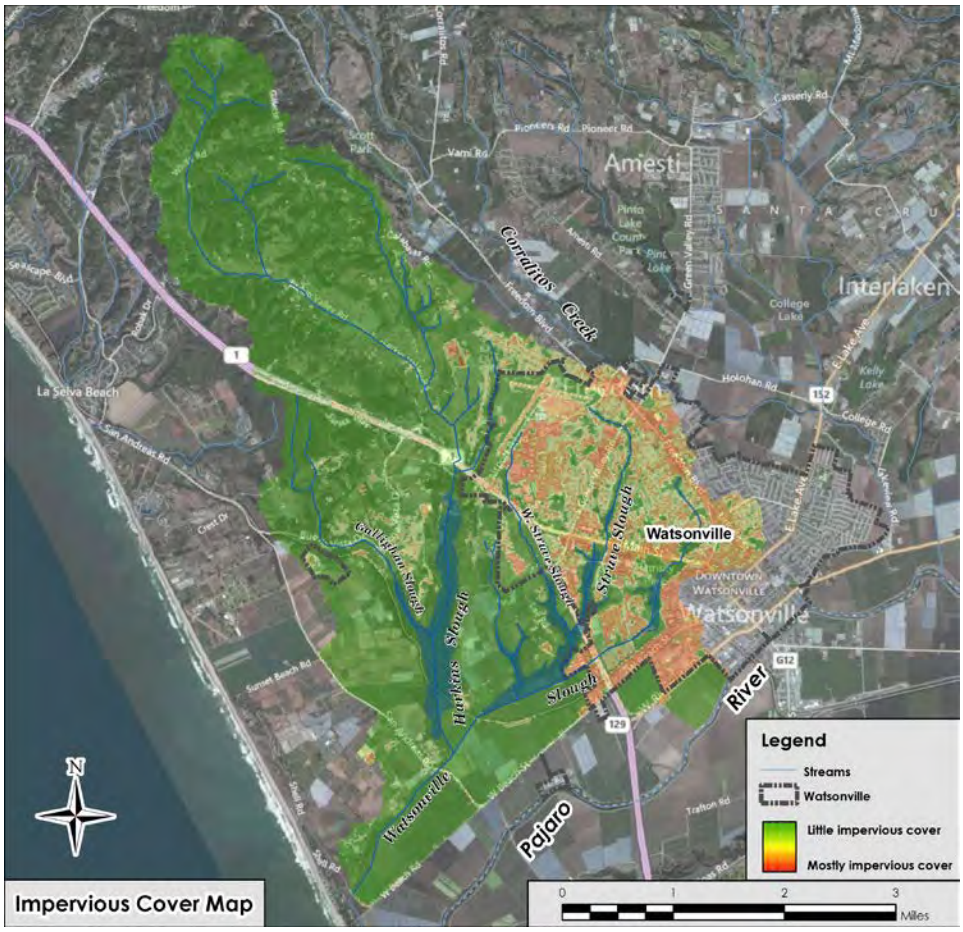
Figure 5-7. Distribution and classification of impervious cover in the Watsonville Sloughs watershed.



5.2.6 METEOROLOGICAL DATA

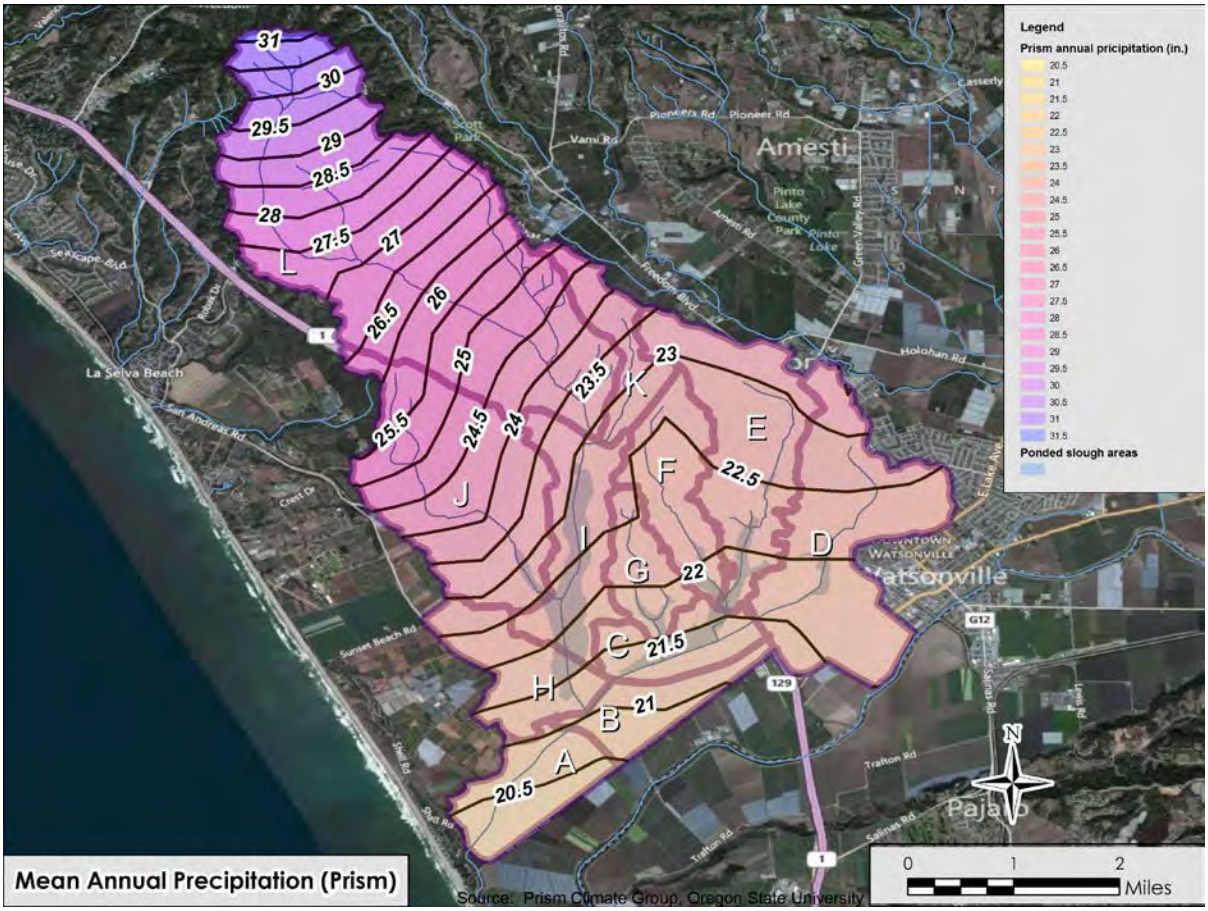
The availability of suitable meteorological data was an important consideration in the selection of the 10-year simulation period used for the HMS modeling. Important considerations included identifying a substantially complete record of hourly data from a location representative of the weather patterns that would be expected over the entire watershed. Additionally, it was felt that more recent data were important such that the model results could be more readily compared to observed conditions.

5.2.6.1 Precipitation

The spatial distribution of rainfall across the study area sub-watersheds was based on mean annual precipitation mapping prepared by the PRISM Climate Group. The data were derived from the 800m raster coverage for the period from WY1981 through WY2010. The associated isohyetal map developed from the data is shown in Figure 5-8. Per this dataset, the mean annual precipitation for the watershed on an area-weighted basis is 23.9 inches, with the lowest local mean annual rainfall on the order of 20 inches

near Shell Road and the highest over 30 inches in the uppermost portions of the Harkins Slough watershed.

Figure 5-8. Mean annual precipitation in the Watsonville Sloughs watershed per data from the PRISM Climate Group, period of analysis WY1981 through WY2010.



Once the basis for spatially distributing rainfall input was identified the remaining steps included selecting the appropriate long-term simulation period and the temporal distribution to be used. With respect to selection of the simulation period, various groupings of 10 consecutive years were reviewed subject to the goal of identifying a relatively recent period for which high quality hourly data were available. The period from WY2003 through WY2012 was eventually selected and has the advantage that the final year in the period overlaps with the first year of field data from the DCP. The period is best compared to the long-term climate conditions by looking at the respective annual totals recorded at the Watsonville Waterworks rainfall gage (WTW) as shown in Figure 5-9.

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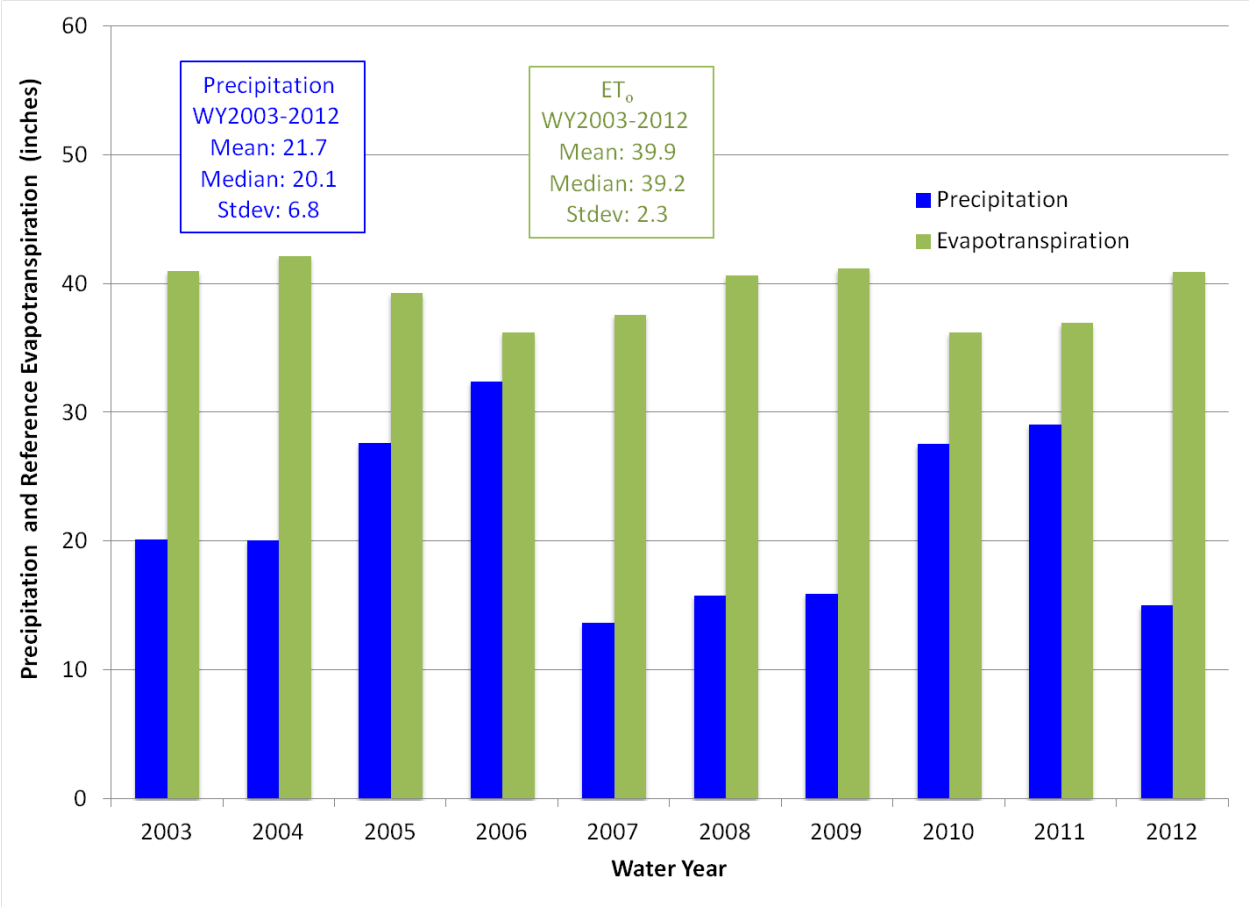
Figure 5-9. Annual rainfall and evapotranspiration values for the 10-year period from WY2003 through WY2012. Precipitation recorded at the Watsonville Waterworks (WTW) and reference evapotranspiration recorded at CIMIS Station 129. Mean precipitation at WTW for the period WY1981-WY2010 is 23.3 inches.



The selected simulation period has somewhat lower mean annual rainfall than the entire 30-year record, the former being 21.7 inches and the latter being 23.3 inches. Overall variability is also less than the 30-year mean as there were no particularly large (e.g. "El Niño") annual totals in the 10-year dataset. Four of ten years in the selected period have annual totals greater than the long-term mean. In particular, it is important to bear in mind that the 10-year hydrologic modeling results developed for the study are based on a dataset that has mean annual precipitation roughly 7 percent below the longer-term average.

The last task in preparing the precipitation input for the model included selecting an appropriate actual gage record for the WY2003-WY2012 period to represent the temporal distribution of storms within that time frame. A number of potential rainfall data sources were evaluated. The hourly rainfall record selected was that recorded at CIMIS Station 129. Although located outside of the Watsonville Sloughs watershed, it had the advantages of close proximity, very little missing data, and a very good correlation with the long-term precipitation dataset collected at the WTW gage. As a

fully-equipped CIMIS weather station, the site had the added advantage of providing a record of other meteorological parameters, especially those used for evapotranspiration calculations within the HMS model.

The hourly precipitation time series from CIMIS 129 was used to create individual time series of precipitation for each sub-watershed rather than applying a single time series across the entire watershed. Therefore, 13 separate precipitation time series were compiled, with each time series prorated by the ratio of the long-term mean for the sub-watersheds derived from the PRISM data and the observed rainfall at CIMIS 129. This is an important data transformation and has the effect of preserving the timing and relative intensity information from the CIMIS gage, but adjusting for the difference in rainfall amount that would be expected based on the pattern of mean annual precipitation from the PRISM analysis (as in Figure 5-8).

It is important to bear in mind that the procedure used to compile the precipitation data input for the model is based on the hourly rainfall record from the single CIMIS 129 station. Therefore, the hydrologic model is not capable (in its present configuration) of accounting for variations in storm patterns that may, and in fact likely do, occur across the watershed. Additional available hourly datasets were carefully examined to assess their potential utility in refining the spatial and temporal distribution of rain events, but none were found with sufficiently complete hourly records and/or acceptable correlation with the long-term WTW precipitation record. Maintenance and calibration of other hourly gaging stations within the watershed has the potential to generate additional robust datasets that can be used to further refine the precipitation inputs to the model in the future.

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5.2.6.2 Evapotranspiration

Evapotranspiration is an important component of the hydrologic cycle, with a very significant impact on the overall water balance. The SMA methodology in HEC-HMS calculates evapotranspiration rates based on the input of solar radiation data and the calculated water in storage that can actually be lost. The latter aspect is of particular importance in coastal California settings such as the Watsonville Sloughs where potential evapotranspiration rates exceed total precipitation in all but the most extremely wet years (see Figure 5-9).[17] Potential evapotranspiration is essentially the loss rate that would occur with continuously available soil moisture and therefore overstates actual evapotranspiration losses in most cases, and particularly were regular irrigation is not applied. The capability of the SMA calculations in this regard allows the model to explicitly account for the cessation of evapotranspiration losses when there is no water stored in the canopy, surface depressions, or the upper soil zone. Conversely, the same calculation framework allows for accounting of the higher losses to be expected where regular irrigation is used (see the next Section).

Solar radiation data is one of the hydrologic parameters collected at CIMIS stations and was therefore directly accessible for CIMIS Station 129 as hourly data.

5.2.7 IRRIGATION DATA

Applied irrigation is another important hydrologic component of the HEC-HMS model and one that can be explicitly tracked within the SMA methodology. Excellent data on overall applied irrigation is available from the PVWMA and was used to parameterize the model.

The primary data source provided by PVWMA covered the period from WY2006 to WY2011, with information for each turnout and well in operation over that period summarized on a quarterly basis. This information was analyzed spatially to aggregate applied water volumes by land use type. The decision was made to only use the data that was clearly associated with areas categorized as agricultural land uses, an important distinction since this means the model does account for irrigation use in open space and developed land use types. Particularly with respect to the latter land use, applied irrigation may be significant, but further work to specifically identify irrigated landscape values was beyond the scope of this study.

The quarterly irrigation data was then fit to an expected monthly distribution based on crop type and evapotranspiration. The resulting monthly mean value was then further

17 Reference evapotranspiration (ETo) is shown in Figure 5-9 and is the potential evapotranspiration for an area of well-watered cool-season grass.

disaggregated to daily totals, which were then appended to the input precipitation record for each basin. Therefore, each basin representing agricultural land uses has a modified HMS precipitation input that represents actual rainfall plus daily irrigation application, with the latter set to occur over a two-hour period in the early morning of each simulation day.[18] Another major simplifying assumption is that the calculated daily irrigation application rates were kept the same for each year in the ten-year simulation period. That is, further refinement to actually correlate applied irrigation in the model to water year type (e.g. "wet", "critically dry") was not pursued and would be a further potential refinement in the model structure in the future. However, review of the five years of data provided by PVWMA indicated that differences between water years are generally on the order ten percent of the longer-term mean.

5.3 Model Calibration

As mentioned previously, the goal of the hydrologic modeling effort was to simulate appropriate patterns and volumes of runoff to the Sloughs. Therefore, model parameterization used total runoff volume (measured in acre-feet) as the primary objective function.

5.3.1 CALIBRATION PARAMETERS

The Modeling Tools Users' Guide details the structure of the input data calculations and transforms from GIS to a spreadsheet environment prior to importing into the HEC-HMS model. The spreadsheet structure was deliberately set up with cells to allow input of adjustment factors to change properties used in the SMA parameterization from those directly extracted from the GIS database. It was expected that these adjustment factors would need to be used to calibrate the model results. However, suitable calibration for overall watershed runoff volume was obtained without the need to significantly alter the values of the GIS-derived parameters.

A number of parameters do not come from the GIS data and were fit by iterative model runs to give the most consistent overall results. These include:

- *Clark unit hydrograph.* These include the time of concentration and storage coefficient, both with units of hours. For the locations not impacted by upstream ponding areas (which are explicitly addressed in the hydraulic model), these parameters were found to be 2 hours and 4 hours respectively. The exception

18 Clearly, actual irrigation applications occur over a wide range of times. However, early morning was selected since the model does not calculate ET during periods of precipitation, and choosing a mid-day irrigation time could therefore significantly impact model generated ET rates.

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was the "ponded" basins, which were run using 1 hour for each parameter in light of the direct rainfall effect of precipitation on these inundated areas.

- *SMA groundwater storage.* The values for GW1 and GW2 storage were set to 12 inches for all basins.
- *SMA groundwater coefficients*. Values for the GW1 and GW2 groundwater coefficients were set to 200 and 600 hours respectively for all basins.
- *SMA simple canopy*. Values for the maximum canopy storage were varied by land use type with 0.12 inches used for open space and developed areas and 0.04 for agriculture areas.
- *SMA simple storage*. Representing the available surface storage in each basin, this parameter was set to 0.20 inches for open space and agriculture areas and 0.10 inches for developed areas.
- *Baseflow*. The model uses the linear reservoir baseflow method, with appropriate fit to observed data found using 6 reservoirs in the GW1 layer and 8 reservoirs in the GW2 layer.

5.3.2 CALIBRATION RESULTS

The results of the calibration efforts were mixed, with very good agreement for the overall Sloughs watershed, and variable results for individual sub-watersheds. Given the limited period of record for the field data on flow rates and the consistently dry conditions over the period of the study, extensive calibration at the individual sub-watershed level was not deemed appropriate until a wider range of data are available.

Calibration efforts focused on runoff from the land areas of the watershed only, since direct rainfall onto ponded areas should reliably all become direct runoff. The calibration target for the overall watershed was calculated by completing a water balance for WY2012 around the ponded areas making use of the measured outflow at San Andreas Road from the data collection efforts.[19] Total runoff from the contributing land areas on this basis was found to be 2,280 acre-feet. This compares to a value of

19 Completing the calculation with San Andreas Road as the downstream limit means that runoff from Sub-watershed A is not included.

2,330 acre-feet from the HMS model. The small difference in total land area runoff (roughly 2 percent) is lower than would be expected for a lumped parameter model, and needs to be verified over a wider range of water year conditions than current data availability allow. Nonetheless, the initial agreement between the observed and modeled values is a good indication that the model can appropriately simulate overall watershed runoff volumes.

As mentioned, calibration at the sub-watershed level was not pursued rigorously due to the short period of available data and the need to better reconcile observed runoff from the urbanized portions of the watershed to expected values. This is particularly true for Sub-watersheds D and E (gaging sites WSHO and SSES respectively) as previously discussed in Chapter 4. Impervious areas in these watersheds are much too high to allow a reasonable calibration to observed data, and calibrating the model to the measured data would result in a much larger difference in the overall watershed calibration. Nonetheless, detailed comparisons were made for Sub-watersheds D, F, J, and K (gaging sites SSWS, GSBL, and HSBV respectively) and are summarized in Table 5-3.

Table 5-3. Gaged and modeled runoff volumes by sub-watershed. Note that the values are only for the period of the gaging records at each site and are not on a full water year basis. Sub-watershed E is not included due to issues discussed in Chapter 4.

Sub-watershed	Gaging Station	Measured Runoff	Prorated to Shed Area	Modeled Runoff	
		(ac-ft)	*(ac-ft)*	*(ac-ft)*	*(% of gaged)*
D	WSHO	490	490	680	139%
F	SSWS	410	620	450	73%
J	GSBL	140	170	180	106%
L	HSVB	270	380	300	79%

The values in Table 5-3 show that there is potential to significantly improve site-specific model fidelity through future calibration efforts once longer-term datasets on sub-watershed runoff are available. In this sense the results for Sub-watershed J are indicative of what a good fit to total runoff volume on a sub-watershed basis would look like, and are illustrated in Figure 5-10.

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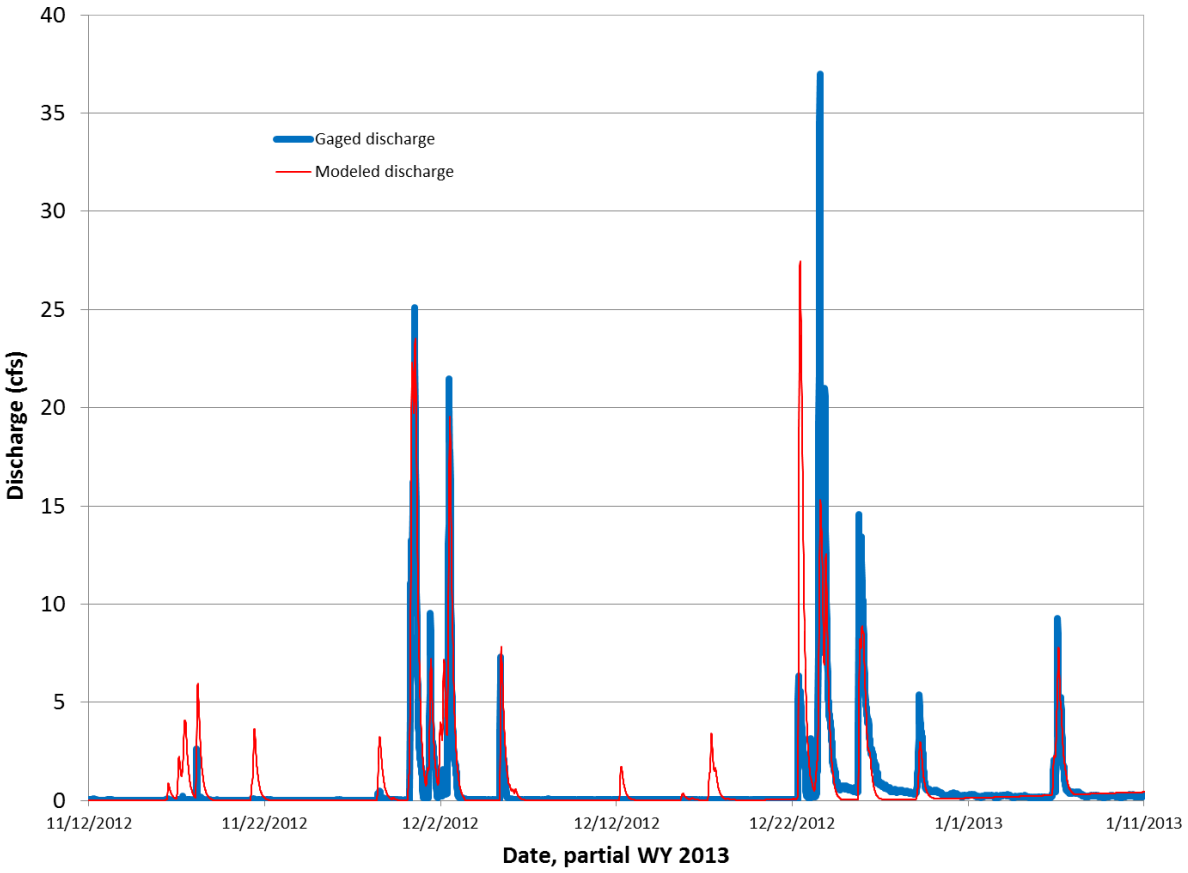
Figure 5-10. Comparison of modeled discharge for Sub-watershed J (Gallighan Slough below Landfill) to the gaged discharge record prorated to the full sub-watershed area. Data presented are for the first portion of WY2013.



The data illustrated in Figure 5-10 show the model generally matches the timing and magnitude of runoff actually observed, with a few notable exceptions. An example of these exceptions is during the storms on and around December 22 to 24, where the peaks are reversed for the two largest runoff pulses. Also, the model predicts more numerous small runoff events than were actually observed at the gage site. A number of factors may be involved and certainly must include potential differences due to the distance between this sub-watershed and the location of the CIMIS 129 gage used as the rainfall input.

5.4 Overview of Hydrologic Model Results

Running the HEC-HMS model in continuous simulation mode for the full 10-year period provides a very large amount of output data that can be analyzed on a spatial, temporal, and parameter-specific basis. A full discussion of the output is beyond the scope of this report and is best undertaken on an issue specific basis in future modeling

applications. However, several generalized aspects of the model output pertinent to the water balance and hydraulic modeling merit discussion.

5.4.1 RESULTS BY SUB-WATERSHED

Model output in terms of total predicted runoff by sub-watershed is summarized in Table 5-4.

Table 5-4. Summary of predicted runoff volumes by sub-watershed from the project hydrologic modeling tool for the 10-year simulation period (WY2003 through WY2012). Note particularly the wide range in unit runoff values which range from a low of 1.5 inches (0.13 ac-ft/ac) up to 9.8 inches (0.82 ac-ft/ac) for highly urbanized Sub-watershed D. These data exclude runoff originating as direct rainfall on ponded areas.

Sub-Watershed	Average WY Runoff	Average Unit Runoff	Maximum WY Runoff	Minimum WY Runoff
	(ac-ft)	*(ac-ft/ac)*	*(ac-ft)*	*(ac-ft)*
A	200	0.38	430	70
B	210	0.55	430	80
C	140	0.44	270	50
D	1,180	0.82	1,840	690
E	860	0.80	1,350	480
F	430	0.74	700	230
G	130	0.39	250	40
H	70	0.13	130	40
I	140	0.26	240	50
J	280	0.19	450	140
K	150	0.39	260	70
L	480	0.13	830	140
Totals or Weighted Average	**4,260**	**0.38**	**7,010**	**2,070**

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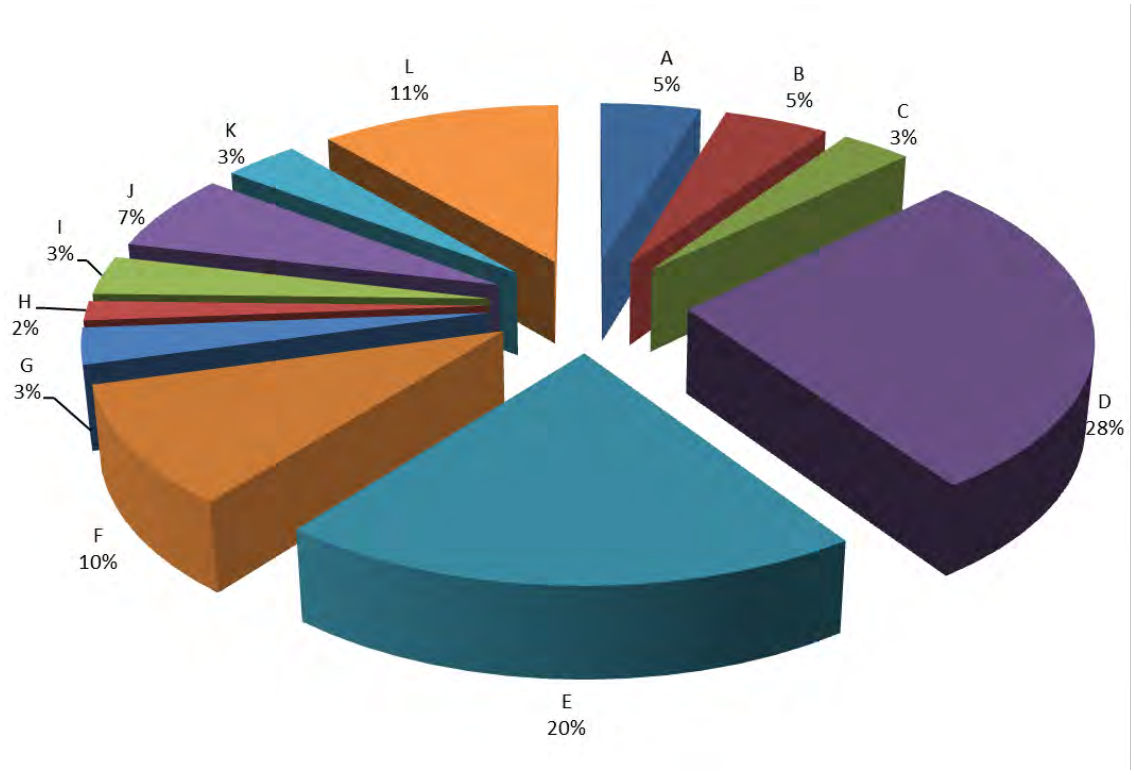
The model output in terms of average water year runoff ranges from a low of 70 acre-feet for Sub-watershed H to a high of 1,180 acre-feet for Sub-watershed D. However, a more appropriate basis for comparison is unit runoff as it accounts for the different physical size of the sub-watersheds. On a unit runoff basis, the values still span a large range from 0.13 acre-feet/acre in the sandier sub-sheds (e.g. H and L) up to 0.82 acre-feet/acre for the most heavily urbanized areas (e.g. Sub-watershed D). The range between the minimum and maximum water year runoff is also important to note, with the developed sub-watersheds generally having lower ratios of maximum to minimum runoff (on the order of 3) indicative of the role of impervious cover in generating runoff.

The values in Table 5-4 underscore the need to verify actual runoff rates from the urbanized sub-watersheds as part of future data collection and model calibration efforts. This can be seen in the relative contributions of each sub-watershed per the model output as illustrated in Figure 5-11. It is particularly evident that predicted runoff rates from Sub-watersheds D and E will be fundamental drivers in the water balance and in the distribution and timing of runoff through the various parts of the system down to the tidally-influenced reaches downstream of Shell Road.

Figure 5-11. Contribution to overall watershed average runoff by sub-watershed for the continuous simulation period from WY2003 through WY2012. The proponderence of developed areas lie in Sub-watershed D, E, and F which combined are predicted to account for nearly 60 percent of the average annual runoff to the Sloughs.



Additional analysis of the output data on the basis of other spatial factors is readily possible using the .dss files generated by the HMS model, but are beyond the scope of this report. Examples include analysis of runoff rates by land use type and/or location with respect to groundwater recharge potential. Manipulation of the underlying variables could then be used to assess how changes in such variables (e.g. expansion of developed areas or agricultural land uses) would impact watershed hydrology and the various components of the overall water balance.

5.4.2 RESULTS BY WATER YEAR

The model output can equally easily be summarized on a water year basis to help understand the range and variability of runoff to the Sloughs. Table 5-5 presents such a summary, along with a disaggregation of the data between the two main branches of the Slough system.

Table 5-5. Summary of predicted runoff volumes by water year from the project hydrologic modeling tool to each branch of the slough system for the 10-year simulation period (WY2003 through WY2012). Runoff from the Watsonville-Struve branch of the system is predicted to account for roughly three-quarters of all the runoff during the period. These data exclude runoff originating as direct rainfall on ponded areas.

Water Year	Entire Watershed	Watsonville Branch		Harkins Branch	
	(ac-ft)	*(ac-ft)*	*(% of total)*	*(ac-ft)*	*(% of total)*
WY2003	3,360	2,570	77%	790	23%
WY2004	3,680	2,690	73%	990	27%
WY2005	6,770	4,890	72%	1,880	28%
WY2006	4,950	3,550	72%	1,400	28%
WY2007	2,070	1,640	79%	430	21%
WY2008	3,150	2,270	72%	880	28%
WY2009	3,240	2,380	73%	860	27%
WY2010	7,010	5,240	75%	1,770	25%
WY2011	5,860	4,220	72%	1,640	28%
WY2012	2,470	2,010	81%	460	19%
Mean	**4,260**	**3,150**	**74%**	**1,110**	**26%**
Median	**3,680**	**2,690**		**990**	

For the entire simulation period, the average modeled annual runoff is 4,260 ac-feet. However, comparison on a water year basis shows a large variability in total runoff from the driest to the wettest years in the simulation period, with the highest predicted runoff (7,010 acre-feet in WY2010) over three times the lowest value (2,070 acre-feet in

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WY2007). The asymmetry is further highlighted by the fact that the median value (3,680 acre-feet) is markedly less than the average, indicative of the impact the several wettest years can have on longer-term average values.

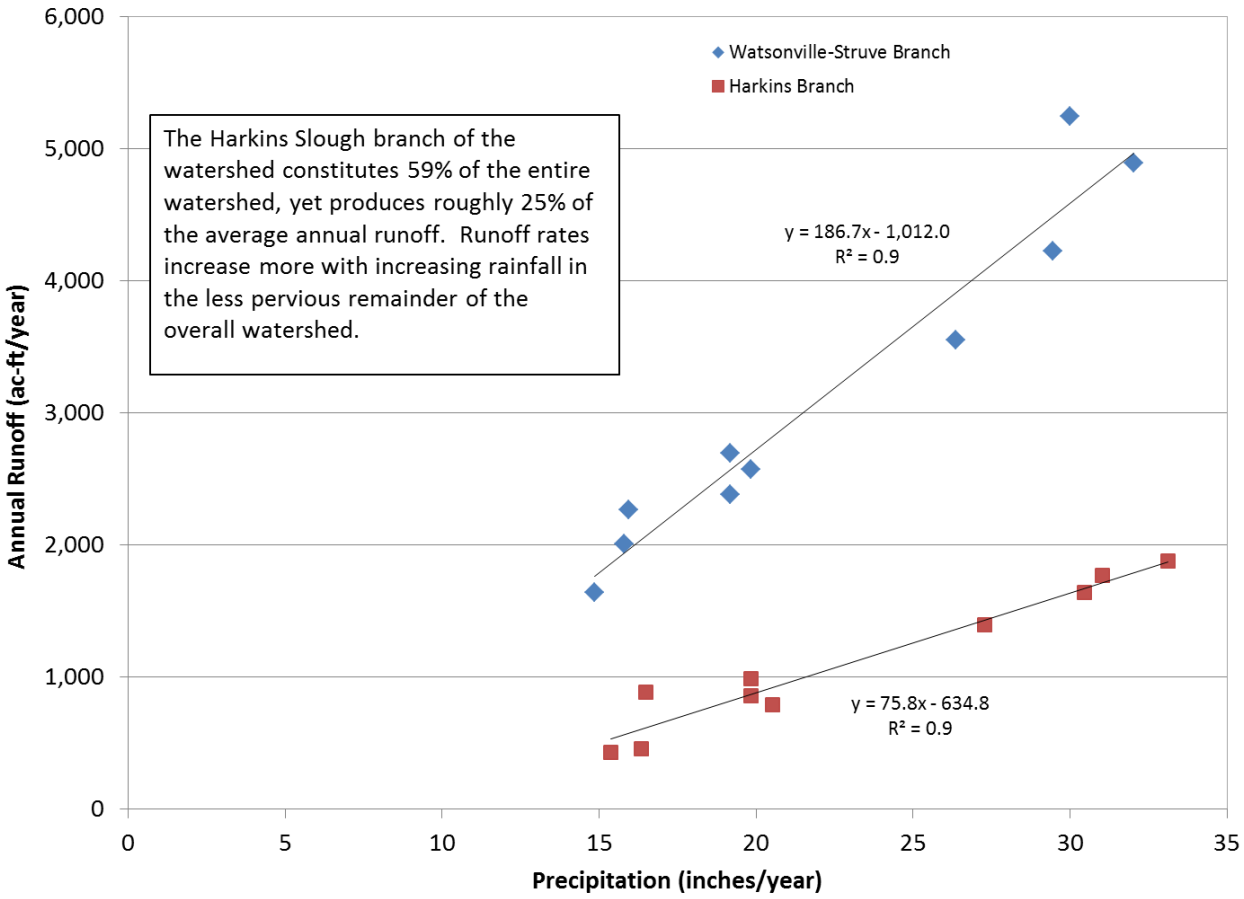
From the standpoint of spatial distribution of runoff, the model output shows 74% of total runoff originates in the Watsonville-Struve Branch of the system, even though the contributing sub-watersheds constitute only 41% of the overall watershed area. Although detailed conclusions should be made only after further model calibration using measured data from a range of water year types, a finding that the vast majority of runoff comes from the Watsonville-Struve Branch of the system is entirely consistent with the field observations made during the study period, which show little total outflow from Harkins Slough.

5.4.3 CORRELATIONS BETWEEN MODEL PARAMETERS

As mentioned previously, the hydrologic model structure (and especially the SMA methodology) provides the opportunity for assessing the sensitivity to, and correlation between, a wide range of parameters. Again, the full sweep of possibilities in this regard cannot be discussed herein and are best left to specific applications of the modeling tool.

That said, a simple example is illustrative of the type of analyses that are possible. Figure 5-12 shows the data from Table 5-4 plotted as a function of annual precipitation for the simulation period.

Figure 5-12. Correlation between predicted runoff volume and annual rainfall per the hydrologic modeling tool for the 10-year simulation period. The low volumes for the Harkins Branch reflect the much higher infiltration capacity and lower level of urbanization in those sub-watersheds.



Although the data show scatter that reflects a number of variables including the spatial distribution of rainfall within any given year, the relationship between runoff and rainfall is fairly well approximated by a linear relationship over the range of annual rainfall values modeled. Of note, is the fact that the model predicts that the combination of recharge type and land use in the Harkins Slough branch of the system leads to a lower sensitivity to total annual precipitation. Results such as this are indicative of how the watershed would respond to longer-term shifts in precipitation patterns, such as those associated with climate change response.

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6 WATSONVILLE SLOUGHS WATER BALANCE

Characterization and quantification of hydrologic fluxes within the Watsonville Sloughs watershed has long been recognized as important for understanding slough dynamics and for informing management decisions. Therefore, an important application of the information generated in the DCP and hydrologic modeling efforts was updating of the water balance calculations completed in previous studies.

6.1 Previous Water Balance Calculations

6.1.1 METHODOLOGY

The most comprehensive previous water balance calculations were presented in the Water Resources Management Plan for Watsonville Slough System, Santa Cruz County (Questa Engineering Corporation, 1995). A detailed discussion of the methodology used and the related findings is included in Section 5 and Appendix C of that report, noting that the terminology used in that study was water "budget" rather than water "balance".

Several aspects of the previous water balance are important in terms of understanding the findings and comparing them to the revised water balance calculations. These include the following:

- *Time step*. The annual water balance for average, typical wet, and typical dry years was developed using a monthly time step, with discrete tabulation of water fluxes during each month based on data available at the time. The monthly values were summed to give overall annual totals.

- *Watersheds*. Calculations were completed for three watersheds Gallighan Slough, Harkins Slough, and Watsonville Slough (the latter including Struve and Hansen Sloughs).

- *Rainfall – runoff conversion*. Rainfall data was converted to runoff using the SCS Curve Number methodology and the TR-55 runoff model. Continuous simulation modeling was not used, but rather multiple discrete storm events were run on a monthly basis to account for differing storm rainfall totals and varying antecedent conditions. The latter were addressed through changes in the curve numbers assigned to each of four distinct land use types (urban commercial, urban residential, open space, and agriculture). The TR-55 model output of total

runoff for each storm event and land use type was summed to give total watershed runoff for each month.

- *Evapotranspiration*. Losses from the system due to evapotranspiration were tabulated independently based on actual (not potential) evapotranspiration data from regional studies. Evaporation rates from the ponded slough areas were tabulated separately.

- *Deep groundwater losses and shallow groundwater return*. The difference between the total losses provided by the TR-55 model and those attributed to evapotranspiration was assumed to have become part of the groundwater system. In watershed areas without a confining soil area (e.g. the deep percolation areas in this study), all groundwater was assumed to be lost to the underlying Aromas aquifer. In other areas, the groundwater flow was assumed to return to the slough system with a lagging of approximately two months from the time of the rainfall.

- *Agriculture return flows and minor fluxes*. Water returned to the slough system from irrigation applications was calculated separately as the difference between use rates and the estimated crop needs based on crop coefficients. Several minor flux terms were also included in the calculations such as groundwater seepage losses and septic system return flows, but these were not significant in the overall accounting.

- *Outflow from the slough system*. All source and loss fluxes were combined and the volume of water needed to close the balance calculations was assumed to be outflow from the respective slough system. Results were presented by watershed and in a combined form for the slough system.

6.1.2 RESULTS OF THE 1995 WATER BALANCE

Results of the previous water balance in terms of an average year for the entire slough system are presented in Table 6-1. Total inflow to the system from precipitation (the sum of runoff and shallow groundwater return flows) was roughly 6,980 acre-feet, equivalent to a volumetric runoff coefficient of 0.34. This is markedly higher than the runoff rates presented in Chapter 5 of the present study, particularly when taking note that the

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runoff rate as defined in the present study includes irrigation applications as an input water source and therefore agriculture return flows are actually part of the runoff coefficient. Calculated agriculture return flows for the 1995 water balance were quite high at 1,145 acre-feet/year and were by definition an inflow not subject to losses. Counting agriculture return flows, total annual inflow to the sloughs in the previous water balance was estimated as approximately 8,120 acre-feet. Ponded slough areas were markedly smaller at the time, so losses due to evaporation were relatively low (e.g. 610 acre-feet/year). Combined, the relatively high inflow rates and low losses led to a predicted outflow from the sloughs on the order of 7,500 acre-feet for an average year.

Table 6-1. Previous water balance results for the overall slough system for an average water year (per Questa Engineering, 1995). Note that the column for pumping from Harkins Slough did not appear in the original calculations, but was added here to facilitate comparisons to the revised water balance work described subsequently.

Month	Precipitation		Runoff	GW Seepage Lost	Loss to Deep GW	Return Shallow GW	Septic Return	AG Return	ET Loss From Wetlands	Pumping Harkin Slough	Outflow Slough System
	inches	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*
Oct	1.10	1,000	101	0	0	0	3	117	22	0	199
Nov	3.25	2,955	429	0	193	0	3	0	0	0	432
Dec	3.25	2,955	768	6	275	140	3	0	0	0	904
Jan	4.50	4,091	1,107	6	469	295	3	0	0	0	1,399
Feb	3.75	3,409	1,075	6	170	547	3	0	0	0	1,619
Mar	4.50	4,091	1,107	6	166	599	3	0	0	0	1,703
Apr	1.50	1,364	343	6	0	339	3	131	0	0	810
May	0.40	364	68	6	0	32	3	155	161	0	90
Jun	0.00	0	0	0	0	0	3	227	157	0	73
Jul	0.00	0	0	0	0	0	3	219	153	0	69
Aug	0.00	0	0	0	0	0	3	107	72	0	38
Sep	0.30	273	28	0	0	0	3	188	44	0	175
Total	**22.55**	**20,502**	**5,026**	**36**	**1,273**	**1,952**	**36**	**1,144**	**609**	**0**	**7,511**

Details of the previous water balance specific to Harkins Slough are presented in Table 6-2 and are important for understanding the difference between the previous work and that derived as part of the present study.

Total inflows from rainfall runoff to the Harkin Slough watershed were calculated as roughly 3,150 acre-feet/year. This is equivalent to a runoff coefficient of 0.24, which is markedly higher than that found per the hydrologic modeling. Precipitation derived inflows to Harkins Slough in the previous water balance would therefore represent approximately 45 percent of all such runoff to the overall slough system. As noted in Chapter 5, the hydrologic modeling runs found that the latter metric is on the order of 25 percent. The previous water balance used relatively low evaporation rate for Harkins Slough. Coupled with the relatively high inflow rates, the water balance predicts outflow from Harkins Slough in every month but August.

Table 6-2. Previous water balance results for Harkins Slough only for an average water year (per Questa Engineering, 1995). As in Table 6-1, the column for pumping from the slough was added for comparison purposes to the revised water balance.

Month	Precipitation		Runoff	GW Seepage Lost	Loss to Deep GW	Return Shallow GW	Septic Return	AG Return	ET Loss From Wetlands	Pumping Harkin Slough	Outflow Harkin Slough
	inches	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*
Oct	1.10	646	10	6	0	-89	3	23	4	0	32
Nov	3.25	1,908	94	6	193	-2	3	0	0	0	97
Dec	3.25	1,908	340	6	259	68	3	0	0	0	404
Jan	4.50	2,642	507	6	442	212	3	0	0	0	716
Feb	3.75	2,202	514	6	162	385	3	0	0	0	896
Mar	4.50	2,642	507	6	158	423	3	0	0	0	927
Apr	1.50	881	155	6	0	262	3	26	0	0	440
May	0.40	235	35	6	0	73	3	31	29	0	107
Jun	0.00	0	0	6	0	-43	3	45	28	0	21
Jul	0.00	0	0	6	0	-50	3	44	27	0	20
Aug	0.00	0	0	6	0	-77	3	6	13	0	-4
Sep	0.30	176	3	6	0	-177	3	38	8	0	36
Total	**22.55**	**13,241**	**2,162**	**72**	**1,215**	**984**	**36**	**213**	**109**	**0**	**3,690**

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6.2 Preliminary Revised Water Balance

6.2.1 METHODOLOGY AND RESULTS

An early work product in the present study was a simple update of the previous water balance using the same methodology as in the original 1995 work (see Table 6-3). Since the methodology was unchanged, this preliminary revised water balance could be calculated as soon as sufficient input data were available. The revisions were explicitly limited to updating watershed area values, parameters for land use, evaporation rates due to increased ponded slough area, and the addition of diversions from Harkins Slough to supply the PVWMA groundwater recharge and recovery project.

Changes in land use were derived from the same GIS datasets used in the hydrologic modeling and were found to be concentrated in the Watsonville Slough watershed, a reasonable finding given that changes since 1995 have been predominately within the City of Watsonville. Overall open space was found to drop by roughly 4 percent, agricultural land decreased by nearly 5 percent (though up slightly in the Harkins Slough watershed), and urban land uses increased by almost 16 percent. Runoff values and associated losses to groundwater (both as deep percolation and shallow groundwater return flows) were prorated to reflect the changes in land use. No updates were made to other fluxes within the water balance such as agriculture return flows

Table 6-3. Preliminary revised water balance for the entire Watsonville Sloughs watershed based on updated input parameters and using the same calculation methodology used in the 1995 work.

Month	Precipitation		Runoff	GW Seepage Lost	Loss to Deep GW	Return Shallow GW	Septic Return	AG Return	ET Loss From Wetlands	Pumping Harkin Slough	Outflow Slough System
	inches	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*
Oct	1.10	983	117	0	0	0	3	117	77	0	160
Nov	3.25	2,906	467	0	193	0	3	0	50	0	421
Dec	3.25	2,906	791	6	270	78	3	0	40	0	833
Jan	4.50	4,023	1,138	6	459	282	3	0	46	116	1,262
Feb	3.75	3,353	1,100	6	168	532	3	0	77	199	1,359
Mar	4.50	4,023	1,138	6	164	586	3	0	128	189	1,410
Apr	1.50	1,341	354	6	0	332	3	131	143	120	557
May	0.40	358	73	6	0	27	3	155	149	53	55
Jun	0.00	0	0	0	0	0	3	227	140	0	90
Jul	0.00	0	0	0	0	0	3	219	145	0	77
Aug	0.00	0	0	0	0	0	3	107	128	0	-18
Sep	0.30	268	32	0	0	0	3	188	107	0	115
Total	**22.55**	**20,160**	**5,212**	**36**	**1,254**	**1,838**	**36**	**1,144**	**1,231**	**678**	**6,320**

Comparison of Tables 6-1 and 6-3 shows a number of changes in water balance fluxes that are expected based solely on changes in land use, extent of ponded water, and changes in pumping for groundwater management. Overall inflow from precipitation increased somewhat to 7,050 acre-feet/year (from 6,980 in the original calculations) due largely to the aforementioned increases in urbanized land uses. Nonetheless, the revised water balance predicts a substantial drop in average annual outflow from the overall slough system, decreasing nearly 16 percent from 7,511 acre-feet to 6,320 acre-feet. This decrease is due almost equally to a doubling of evaporation losses from the ponded slough areas and the inclusion of withdrawals (for groundwater recharge and later recovery for water supply) in the calculations. The latter value was taken as roughly 680 acre-feet/year, the average annual withdrawal rate for the period from WY2003 to WY2011 per records provided by the PVWMA.[21] Another aspect of the preliminary revised water balance is that it shows the overall slough system would have a net loss of water in the month of August (e.g. no outflow with losses due to evaporation exceeding net inflows).

The preliminary revised water balance for Harkins Slough is presented in Table 6-4, and, as expected, shows equally substantial changes in that portion of the overall system.

21 PVWMA is currently permitted to withdraw up to 2,000 acre-feet/year from Harkins Slough, though operational considerations have precluded extraction of the full diversion rate in every year since the withdrawals were authorized.

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Table 6-4. Preliminary revised water balance for the Harkins Slough watershed.

Month	Precipitation		Runoff	GW Seepage Lost	Loss to Deep GW	Return Shallow GW	Septic Return	AG Return	ET Loss From Wetlands	Pumping Harkin Slough	Outflow Harkin Slough
	inches	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*	*ac-ft*
Oct	1.10	629	12	6	0	-89	3	23	42	0	-4
Nov	3.25	1,859	101	6	193	-3	3	0	27	0	77
Dec	3.25	1,859	346	6	259	57	3	0	22	0	384
Jan	4.50	2,574	515	6	442	207	3	0	25	116	584
Feb	3.75	2,145	521	6	162	378	3	0	43	199	661
Mar	4.50	2,574	515	6	158	416	3	0	71	189	674
Apr	1.50	858	158	6	0	254	3	26	79	120	242
May	0.40	229	35	6	0	67	3	31	82	53	0
Jun	0.00	0	0	6	0	-45	3	45	77	0	-29
Jul	0.00	0	0	6	0	-50	3	44	80	0	-33
Aug	0.00	0	0	6	0	-96	3	6	71	0	-62
Sep	0.30	172	3	6	0	-177	3	38	59	0	-15
Total	**22.55**	**12,899**	**2,206**	**72**	**1,215**	**918**	**36**	**213**	**678**	**678**	**2,478**

Inflows to the Harkins Slough portion of the watershed do not change appreciably in the preliminary revised water balance. However, the largest part of the increased evaporation losses and all of the withdrawals for groundwater recharge are in the Harkins Slough watershed. Therefore, the water balance shows an average annual decrease in outflow of 33 percent (from 3,690 acre-feet to 2,480 acre-feet). Equally notable is the monthly pattern in outflows, with the preliminary water balance revision showing outflow from Harkins Slough for only six months of the year (November through April). The calculations indicate that the Harkins Slough watershed should experience a net loss of water in the remaining months, with the deficit peaking in the month of August.

6.3 Final Revised Water Balance

6.3.1 METHODOLOGY

Completion of the hydrologic modeling as described in Chapter 5 provides an entirely different basis for revising the previous water balance work. The soil moisture accounting methodology has many similarities to the water budget approach utilized in the 1995 work, but the various parameters are processed differently and, perhaps most

importantly, on a continuous simulation basis. The latter aspect of the hydrologic modeling provides for a more explicit tracking of antecedent conditions and the variability of the precipitation patterns that are the prime drivers of the watershed-scale water balance. As described previously, the decadal span of the modeling work also provides for an explicit characterization of expected inter-annual variations, allowing "average" conditions to be better placed in context with respect to other statistical measures such as the median.

The final revised water balance was therefore compiled using the output data from the hydrologic modeling runs. Data for key fluxes in the overall balance were extracted and post-processed outside of the HEC-HMS platform. Post-processing was limited to summations by water year, calculation of average values, and, in limited cases, conversion of units from those used in the hydrologic model.

Differences in the parameterization and output between the HEC-HMS hydrologic model and the previous water balance led to a number of changes in the way that fluxes were calculated in the revised balance. These include the following:

- *Hydrologic inputs*. Three hydrologic inputs are tracked in the revised water balance: precipitation, irrigation and direct rainfall. The first two of these account for the inputs to the land areas (e.g. non-ponded) of the overall watershed, bearing in mind that irrigation is accounted for in the hydrologic model as an augment to the base precipitation record. Direct rainfall is tracked separately to provide a separate accounting of the precipitation that falls directly on the ponded slough areas and is not subject to losses before reaching the sloughs. Based on the previous modeling work, septic return flows were assumed to be too small to have an appreciable effect and were not included.

- *Losses*. For the purposes of the water balance, losses are only those from the land areas of the watershed and were simply taken as the sum of precipitation and irrigation less the sum of deep percolation and runoff predicted by the model. This approach is a simplified way of representing the various discrete losses in the soil moisture accounting methodology (e.g. canopy interception, surface storage, evapotranspiration from various soil zones).

- *Deep percolation*. Water modeled as going to the deep groundwater aquifer was treated as a loss from the system, consistent with the approach used in the

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previous water balance calculations. This parameter was readily extracted from the HMS model output files.

- *Runoff*. It is important to note that runoff in the revised water balance is the sum of direct runoff (e.g. immediate surface flow) and shallow groundwater return flows. These two components (three actually, as the HMS model tracks two shallow groundwater layers) can be discretized in the model output, but doing so requires additional effort to appropriately sum the values over the numerous sub-watersheds and respective land areas. For the purposes of this report, these fluxes are treated as one ("runoff") which thus represents all inflow to the ponded areas of the sloughs except for the aforementioned direct rainfall.

6.3.2 RESULTS OF THE MODEL-BASED WATER BALANCE

The revised water balance was compiled from the 10-year hydrologic model output with the average values for each component calculated on a monthly basis (see Table 6-5). The "average" annual result is calculated by summing the monthly average values. Since the revised water balance is based on a specific 10-year dataset, it reflects several idiosyncrasies that are in that period (WY2003-WY2012). For example, no water was diverted to the recharge facility in WY2012, so the average pumping value appears somewhat lower than that in the preliminary revised water balance. Similarly, values for the month of October in WY2010 are impacted by the large atmospheric river storm event that occurred in that month. All entries in the revised balance are rounded to the nearest ten acre-feet to acknowledge limits of precision, even though rounding to the nearest hundred is likely more appropriate.

Table 6-5. Final revised water balance for the overall Watsonville Sloughs watershed based on values generated by the HEC-HMS hydrologic model for the period WY2003 through WY2012.

Month	Precipitation	Irrigation	Losses	Deep Percolation	Runoff	Direct Rainfall	Evaporation	Pumping	Outflow from Sloughs
	(ac-ft)	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*
October	1,590	590	1,750	50	390	70	130	0	330
November	1,680	330	1,770	0	250	80	100	0	230
December	4,550	170	3,640	320	770	210	80	0	900
January	3,580	80	2,280	670	700	160	80	110	670
February	4,030	90	2,830	520	770	190	90	180	690
March	2,980	140	2,000	510	610	140	150	180	420
April	1,870	260	1,550	180	410	90	200	110	190
May	430	440	750	0	120	20	210	50	-120
June	200	590	730	0	60	10	210	0	-140
July	30	860	830	0	60	0	200	0	-140
August	10	860	810	0	60	0	180	0	-120
September	60	780	780	0	60	0	150	0	-90
Total	**21,010**	**5,190**	**19,720**	**2,250**	**4,260**	**970**	**1,780**	**630**	**2,820**

The information summarized in Table 6-5 reveals a number of important differences from the water balances discussed previously. The most significant of these include:

- *Losses to deep percolation*. The annual average loss to deep percolation in the final revised balance is markedly higher at 2,250 acre-feet than the value of 1,250 acre-feet from the preliminary revised balance. This reflects a fundamental difference in how these losses are tabulated between the water budgeting calculations of the earlier calculations and the model output basis of the revised balance. Since irrigation inputs are explicitly accounted for in the hydrologic model, they contribute to soil moisture storage and are therefore available to contribute to groundwater percolation. In this sense, the revised balance represents the net deep recharge from precipitation and applied irrigation water rather than simply that from rainfall in the previous water balance calculations.

- *Inflow to the sloughs*. The revised water balance indicates much less inflow to the sloughs. Combined, runoff and direct rainfall average 5,230 acre-feet/year. This is fully 36 percent less than the 8,230 acre-feet/year (all combined inflows) from the preliminary revision of the 1995 balance. Runoff rates are quite low; calculated on the basis of total precipitation, the runoff coefficient is only 0.30.

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Interestingly, direct rainfall to the ponded areas at 970 acre-feet/year accounts for 19 percent of the total inflow to the sloughs.

- *Evaporation*. Evaporation losses from the ponded slough areas are significantly higher in the final revised water budget (1,780 acre-feet/year vs. 1,230 acre-feet/year in the preliminary revision). This difference reflects the adoption of CIMIS ET as the basis for calculating losses in the hydrologic and hydraulic models as opposed to use of regional relationships as applied in the previous methodology.

- *Pumping from Harkins Slough*. The final model-based water balance has a slightly lower average annual pumped volume extracted from Harkins Slough (630 acre-feet vs. 680 acre-feet in the preliminary revision). This small difference is due to the fact that the model-based value strictly corresponds to the 10-year simulation period, in which there was no pumping in WY2012 due to seawater intrusion.

- *Outflow from the Sloughs*. The model-based water balance yields a value of 2,850 acre-feet as the annual average outflow from the Slough system. This is substantially less than the value of 6,320 acre-feet in the preliminary revision and only 38% of the average 7,510 acre-feet in the 1995 calculations. The low outflow value may seem surprising at first, but it is important to bear in mind that it corresponds very well with findings from the DCP such as the low measured total outflow at San Andreas Road (1,430 acre-feet in the drier than average WY2012) and the lower measured capacity of the Shell Road Pump Station. Figure 6-1 shows a comparison of total Slough outflow values by month for the three versions of the water balance.

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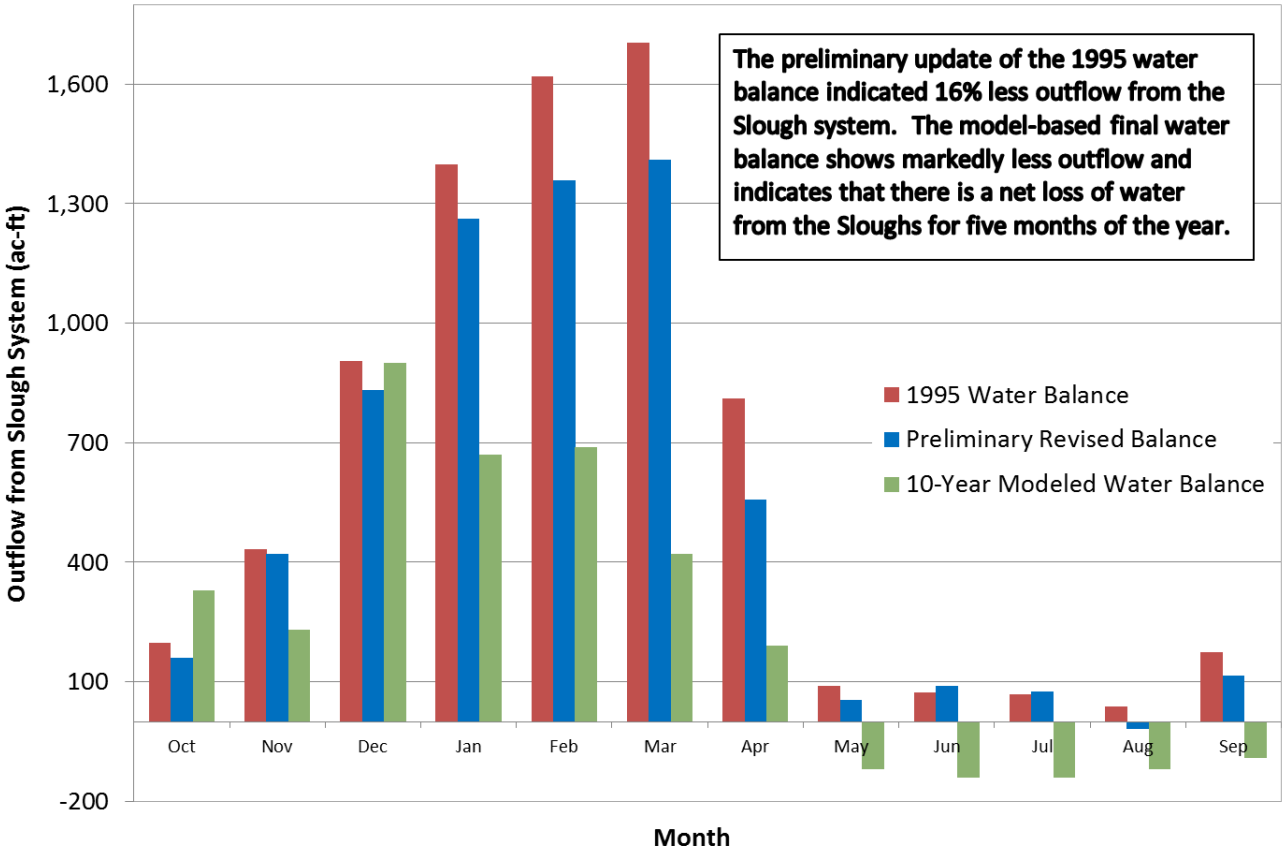
Figure 6-1. Comparison of total predicted outflow from the Watsonville Slough system by month for the three versions of the water balance. Predicted outflow per the model-based water balance is only 38% of that from the 1995 water balance, but corresponds well with conditions measured in the field in WY2012. The negative values in the summer months indicate that the overall system has a net loss of water, due essentially completely to evaporation losses.



Sub-watersheds were selected in preparing the hydrologic model that readily allow for an analogous water balance to be extracted for the Harkins Slough branch of the system. The resulting values are summarized in Table 6-6 and underscore the fact that Harkins Slough is a net sink for water over the course of an average year on the basis of its watershed alone. However, it is important to bear in mind that this form of the water balance does not account for the substantial inflow to Harkins from the Watsonville-Struve branch of the system (e.g. flow at the Harkins weir and across the Knox property).

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Table 6-6. Final revised water balance for the Harkins Slough watershed based on values generated by the HEC-HMS model for the period Water Year 2003 through Water Year 2012. Note that these values do not account for inflow from the Watsonville Slough branch of the system, which were observed to be the "norm" during the field work for this study.

Month	Precipitation	Irrigation	Losses	Deep Percolation	Runoff	Direct Rainfall	Evaporation	Pumping	Outflow from Harkins
	(ac-ft)	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*
October	970	280	1,130	40	80	40	80	0	40
November	1,020	160	1,130	0	50	50	50	0	50
December	2,770	80	2,420	300	130	140	40	0	230
January	2,180	40	1,410	610	190	110	50	110	140
February	2,450	40	1,820	480	190	120	60	180	70
March	1,820	60	1,240	440	200	90	100	180	10
April	1,140	120	980	150	140	60	130	110	-40
May	260	210	430	0	40	10	140	50	-140
June	120	280	380	0	20	10	150	0	-120
July	20	410	410	0	20	0	140	0	-120
August	0	410	390	0	20	0	130	0	-110
September	40	370	390	0	20	0	110	0	-90
Total	**12,790**	**2,460**	**12,130**	**2,020**	**1,100**	**630**	**1,180**	**630**	**-80**

6.3.3 INTER-ANNUAL VARIABILITY FROM THE REVISED WATER BALANCE

As per the summary information provided in Chapter 5, the hydrologic model allows for extraction of output data to create separate water balance summaries for each year in the simulation period. The resulting information gives a detailed picture of the range and nature of the inter-annual variability that characterizes the key fluxes of the hydrologic cycle in the Slough system and is summarized in Table 6-7.

The values in Table 6-7 show that outflow from the Watsonville Sloughs watershed can vary greatly from year to year. In fact, since evaporation losses are large and vary between years, the variability in outflow from the Sloughs is even larger than the variability in runoff from the land areas of the watershed. The highest modeled outflow of 5,620 acre-feet in WY2010 is over seven times the lowest value of 770 acre-feet in WY2007. The results also indicate that substantial flushing of the system likely happens in only the wettest years, as evident by the fact that 54% of the predicted outflow in the 10-year period occurred in the three wettest years.

Table 6-7. Final revised water balance for the overall Watsonville Sloughs watershed summarized by water year for the 10-year simulation period. Differences in nominal values from Table 6-5 result from rounding of values, but are nowhere considered significant. These values do not include inflow from beach overtopping events, such as that observed in WY2012.

Water Year	Precipitation	Irrigation	Losses	Deep Percolation	Runoff	Direct Rainfall	Evaporation	Pumping	Outflow from Sloughs
	(ac-ft)	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*	*(ac-ft)*
WY2003	18,720	5,200	19,190	1,380	3,360	860	1,830	110	2,280
WY2004	18,090	5,210	17,330	2,280	3,680	830	1,770	770	1,970
WY2005	30,200	5,200	24,300	4,340	6,770	1,390	1,880	800	5,480
WY2006	24,870	5,210	21,840	3,300	4,950	1,150	1,890	900	3,310
WY2007	14,020	5,200	17,050	110	2,070	650	1,400	550	770
WY2008	15,050	5,210	14,910	2,190	3,150	690	1,570	790	1,480
WY2009	18,100	5,200	18,400	1,660	3,240	830	1,730	560	1,780
WY2010	28,300	5,210	23,080	3,410	7,010	1,300	1,790	900	5,620
WY2011	27,780	5,210	23,340	3,790	5,860	1,280	1,880	840	4,420
WY2012	14,920	5,210	17,660	0	2,470	690	1,810	0	1,350
Mean	**21,010**	**5,210**	**19,710**	**2,250**	**4,260**	**970**	**1,760**	**620**	**2,850**

The water balance calculated outflow for WY2012 (1,350 acre-feet) is particularly significant as a calibration point with the observed outflow of 1,430 acre-feet at San Andreas Road measured as part of the data collection program. That said, verification of the model-based water balance during a water year of above average precipitation is extremely important.

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7 HYDRAULIC MODELING

As discussed previously, the watershed modeling completed for this study consists of two distinct applications: hydrologic modeling using the HEC-HMS software platform and hydraulic modeling using the HEC-RAS platform. This chapter summarizes the latter work, beginning with a discussion of the model configuration and parameterization, then moving to calibration, and finishing with a review of the model output for existing conditions which constitutes the Base Model for the scenario modeling described in Chapter 8.

7.1 Model Structure

7.1.1 OVERALL MODEL CONFIGURATION

The extents and key features of the hydraulic model are shown in Figure 7-1.

Figure 7-1. Overview of the HEC-RAS hydraulic model. Cross-sections are shown in green and labeled increasing in the upstream direction. The maximum extents of the storage areas are shown in blue.

All modeling was completed using Version 4.1.0 of the HEC-RAS software package. The final model runs were completed in unsteady flow mode for the entire 10-year time simulation period consistent with the hydrologic modeling. Additional configurations were run during the model build process including calibration runs using field data from WY2013 that are not included in the 10-year simulations.

The backbone of the model domain consists of detailed cross-sections extending upchannel from just downstream of Beach Road in the west to the Watsonville Slough channel at Highway 1. Notable features of the model include:

- *Cross-sections*. A total of 56 cross-sections were input into the model geometry. All cross-sections were georeferenced to the project topographic data, allowing for ready modification of individual cross-sections and/or addition of sections as new information is obtained. Georeferencing also provides for easy export of model output to other platforms such as GIS or CAD. As shown in Figure 7-1, the southern limit of cross-sections was Beach Road (which is elevated as it crosses the Pajaro floodplain) and the northern limit was either the toe of the terrace slope along the lower reaches of Watsonville Slough or the open water areas of Harkins, Hansen, and Struve Sloughs.

- *Cross-section detail and density.* Average cross-section spacing is on the order of 300 feet, with a maximum spacing of 900 feet. Resolution of hydraulic behavior, particularly under low flow conditions, predicated much closer section spacing in the vicinity of the previously discussed sediment dams. Cross-sections typically included on the order of 400 elevation points taken primarily from the digital elevation model (DEM) created from the project LiDAR topography base. However, low overall flow rates prevail through much of the channel network modeled, and the information from the detailed site survey work was critical in providing high-definition of elevations in, and immediately adjacent to, the channel. An example of the added detail, especially below the water surface, is illustrated in Figure 7-2. Interpolated cross-sections were used as necessary to achieve model stability and were of particular importance to allow stable run conditions around abrupt changes in channel slope during the driest periods time period (e.g. at sediment dams during the summer months).

- *Channel roughness*. Manning's n values for cross-sections were set based on field reconnaissance and photo archives provided by PVWMA. Essentially all

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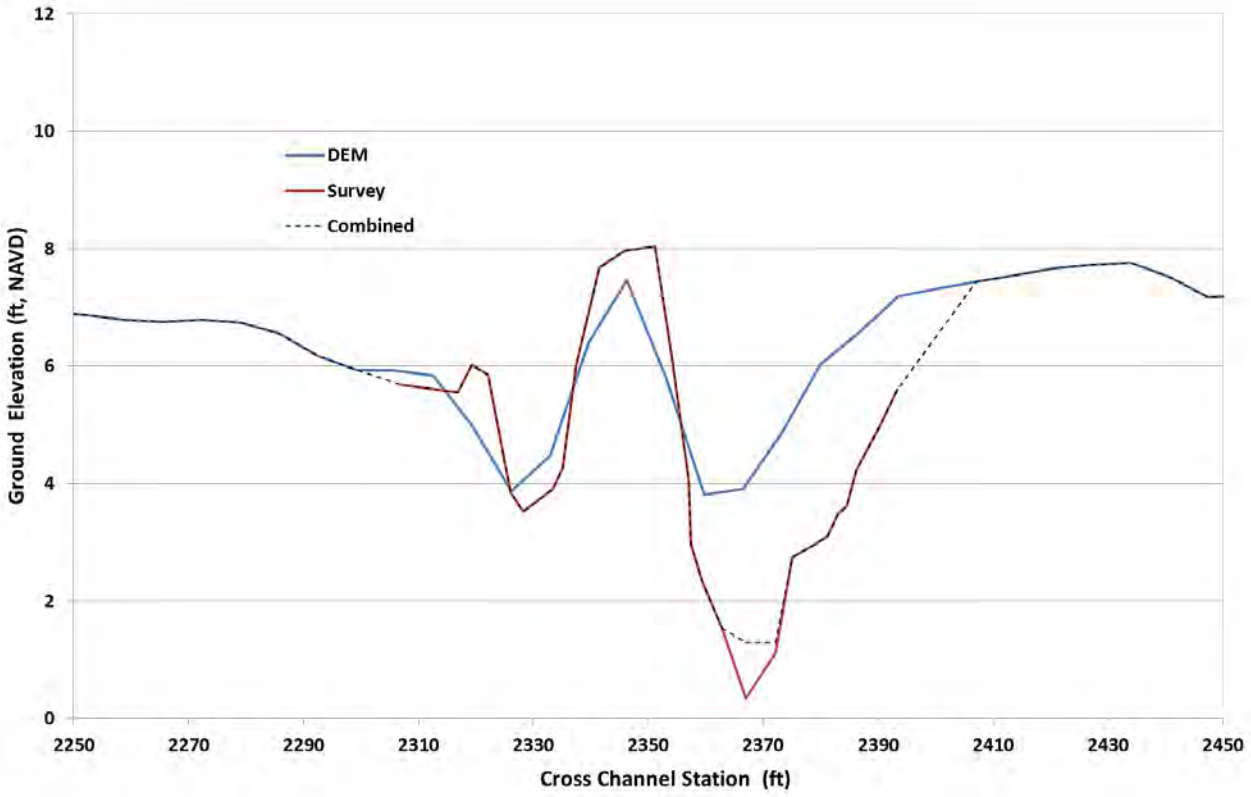
channel reaches were found to be heavily impacted by vegetation of one sort or another, including thick and expansive growth of pennywort and cattails. Field observations showed that peak flow rates were only infrequently high enough to flatten or scour away this vegetation. However, it was noted that the pennywort rode on the channel surface and that the thickest growth of cattails were overtopped at moderate to high flow rates. Therefore, roughness coefficients used in the modeling were set to vary with flow rate in main channel reaches, with higher roughness for low flow conditions trending to lower roughness as flow increased.

Figure 7-2. Portion of HEC-RAS model cross-section 130 comparing DEM to survey ground elevation data. Note that vertical exaggeration is approximately 10X for this view.



7.1.2 STORAGE AREAS

This report has discussed the large extent of open water areas that now characterize the valley bottoms in the Slough system. From a hydraulic perspective, these ponded areas play a key role in attenuating peak flow rates through the system, significantly

impacting the timing and dynamics of the Sloughs' response to rain events and longer-duration factors such as evaporation.

The ponded slough areas were represented in the HEC-RAS model with seven different storage area elements (see Figure 7-1 and Table 7-1), including the following:

- *Lower Watsonville Slough*. Two storage areas, relatively small in size, were used to account for the open water areas on either side of the Shell Road crossing. They are linked by a storage area connection element that actually represents the Shell Road culverts (see Section 7.1.3). The westerly of these two, Shell Road Downstream, is tidal.

- *Harkins Slough.* Harkins Slough was simulated using three storage area elements. By far the largest of these is Harkins Slough Upstream, which represents all of Harkins Slough north of the railroad bridge crossing. The second largest, Harkins Slough Middle, includes the area between the railroad bridge and the farm road bridge crossing to the south. The smallest, Harkins Slough Downstream, encompasses the area between the farm road and the Watsonville Slough channel running along the Knox property. The elevation-area-storage relationship for the three areas combined is shown in Table 7-1, along with the same relationships for Struve and Watsonville Sloughs.

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Table 7-1. Elevation-area-storage relationships as used for the main storage area elements in the HEC-RAS slough system model. Refer to the text for details on the division of Harkins Slough into three distinct storage area elements.

	Harkins		Struve		Watsonville	
Elevation	Area	Storage	Area	Storage	Area	Storage
(feet, NAVD)	*(acres)*	*(ac-ft)*	*(acres)*	*(ac-ft)*	*(acres)*	*(ac-ft)*
0	2	2	3	3	---	---
2	11	14	11	17	---	---
4	71	96	89	117	---	---
6	267	490	178	384	1	2
8	328	1,085	206	768	28	30
10	365	1,778	236	1,211	34	92
12	379	2,522	267	1,714	45	171
14	---	---	292	2,274	60	276
16	---	---	---	---	84	420

The three distinct storage areas for Harkins Slough were used in the model to better simulate any connectivity constraints that might exist at the railroad and farm road bridges. However, model runs showed consistently that there is ample conveyance at each of these crossings, such that the three portions of Harkins Slough function essentially as one for the conditions modeled in this study.

- *Struve Slough*. The largest single storage area element in the model is labeled Struve Slough. Given the perennially flooded conditions prevailing at the time of the study and the location of the ponded area near the upper end of the modeled channel system, the decision was made to use a single element to represent all of the open water area north of the Watsonville Slough channel and upstream of the railroad crossing. Therefore, the Struve Slough element includes West Struve Slough, East Struve Slough, and Hansens Slough combined.

- *Watsonville Slough.* The uppermost storage area element in the model is Watsonville Slough, representing all of the open water areas east of the Highway 1 crossing.

Storage area elements function in a particular manner in HEC-RAS and several aspects are worth noting. They provide a convenient single location for applying boundary conditions or operations where the impact can reasonably be expected to be uniform over a large area (e.g. evaporation losses, see Section 7.2). Nonetheless, the model treats each element as a single point, even though it may represent storage over a very large area. For example, modeled runoff inflow to the uppermost reaches of Struve Slough results in essentially instantaneous and uniform changes in water surface elevation in the Struve Slough storage area, including Hansens Slough and the entire reach to the Watsonville Slough channel. This characteristic of the model is appropriate if high water levels lead to unimpeded connectivity within the areas represented by each element and if changes downstream over short time scales (less than several hours) are not important. Should conditions or modeling resolution demands change, the storage areas can be replaced with additional cross sections, a change that would be particularly appropriate if future management actions lead to lower water levels and the reemergence of significant flow controls at locations such as the Lee Road crossing of Struve Slough or the Harkins Slough Road crossing of Harkins Slough.

7.1.3 PHYSICAL STRUCTURES

A number of physical structures are located within the extents of the slough system covered by the hydraulic model and were explicitly represented within the model geometry. These include:

- *Bridges.* Three bridges are included in the model geometry including the railroad crossing of Harkins Slough, a farm road bridge over Harkins Slough adjacent to the Knox property, and the San Andreas Road crossing of Watsonville Slough. The Highway 1 bridge over Watsonville Slough was not included in the model as it has exceptionally high clearance and would not be expected to impact hydraulic conditions at that crossing point.

- *Culverts.* A total of seven culvert crossings are represented as summarized in Table 7-2. The two farm crossings of Watsonville Slough between Lee Road and the Railroad are in poor condition (partially collapsed). The only culvert array equipped with flapgates and therefore capable of restricting upchannel flow is that at Shell Road. As discussed previously, the Shell Road culverts have vent slots on the downstream side at an approximate elevation of 7.5 feet. Water

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higher than that elevation can leak through the vent slots and bypass the flapgate closures.

Table 7-2. Summary of culvert crossings included in the HEC-RAS hydraulic model geometry. Note that where multiple culvert barrels are listed, the invert elevation is the average of the individual barrel inverts at the upstream end. The listed deck elevation for each crossing is a representative value near the crossing centerline.

Model Station	Phyical Location	Number of Barrels	Diameter	Invert Elevation	Deck Elevation
			(inches)	*(ft, NAVD)*	*(ft, NAVD)*
103	Beach Road	6	48	1.8	8.2
Shell Road	Shell Road (with flapgates)	8	48	1.3	8.8
113	Weir/culvert ~35 feet upstream of Shell Road	7	40	1.7	6.7
163	Railroad crossing of Watsonville Slough	2	60	1.2	14.9
169	Farm crossing of Watsonville Slough ~1,070 feet upstream of RR	1	48	1.6	7.5
179	Farm crossing of Watsonville Slough ~2,780 feet upstream of RR	1	48	1.8	7.1
193	Lee Road	2	60/72	6.2	14.7

- *Levees.* Both the DEM and, particularly, the detailed project survey indicated numerous levees of various sizes. None of these structures appear to be certified levees, with many likely constructed informally as part of adjacent agricultural operations. They were included in the modeling under the assumption that they would be effective in containing and/or directing flow given the low flood depths and velocities found for the modeling period. Other features, such as the elevated bed of the railroad, would also act as levees and were included in the model as well.

- *Lateral weirs.* Given the propensity for overbank flow with the low conveyance of the channels and the numerous low-lying levee structures, eight lateral weirs

were used to represent flow diversions. The lateral weirs allow water to enter and leave the main channel in a manner that permits representation of relatively complex flow patterns even though the model itself is one-dimensional.

- *Pump stations*. Two pump stations have a direct impact on the quantity and timing of flow in the lower slough system. Each of these pump stations was explicitly included in the hydraulic model build.

    - Harkins Slough Pump Station. This pump station is physically located just upstream of the in-line weir that crosses the lower end of the Harkins Slough channel at the confluence with Watsonville Slough. PVWMA operates the pump station and uses it to divert water to groundwater recharge facilities located on the top of the terrace west of Harkins Slough. The permit for the diversion allows up to 2,000 acre-feet/year to be extracted each year, subject to a number of permit restrictions. Those restrictions, and other constraining factors such as turbidity and salinity of the water in Harkins Slough, have limited overall pumping rates, which have never yet reached the total annual volume allowed. The monthly pumping record is illustrated in Figure 7-3 and shows that diversions have typically been limited to four or five months beginning in January, though no pumping occurred in WY2012 due to salinity impacts from the large beach overtopping event that year.

      The pump station is simulated in the model using a pump element that connects to the Harkins Slough Downstream storage area element and discharges out of the model domain. For base scenario conditions the actual pumping record provided by PVWMA was converted into an average monthly pumping rate in cubic feet per second which was then loaded into the model as the pump rule.

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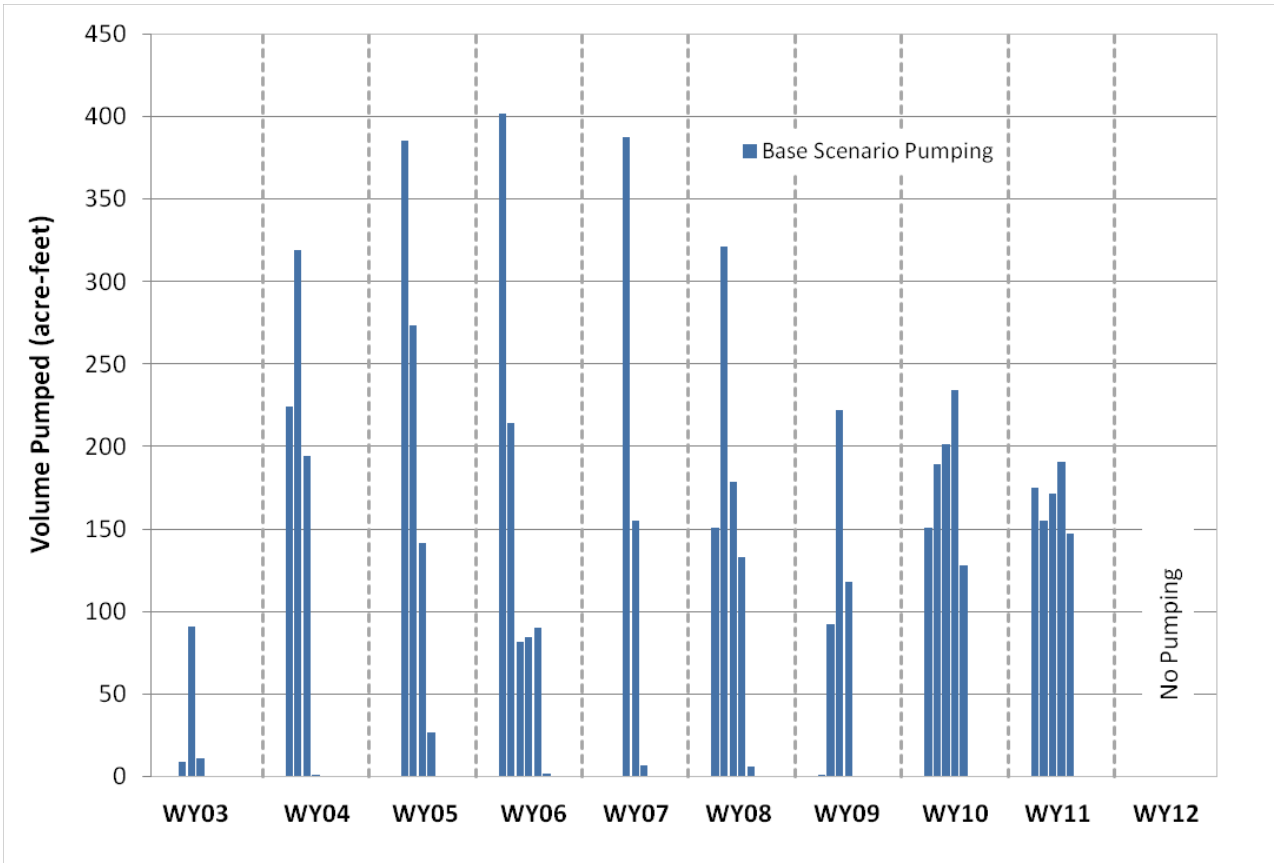
Figure 7-3. Monthly diversion pumping volumes at the Harkins Slough Pump Station per data provided by PVWMA. Note that pumping is confined to mid-winter and spring periods when slough levels are high, with markedly less pumping in dry years such as WY2007. No pumping occurred in WY2012 due to salinity issues.



- Shell Road Pump Station. The Shell Road pump station was described in Chapter 4, and consists of two separate pump units located upstream of Shell Road that are operated on level sensors. One or both pumps operate depending on the water level in the Watsonville Slough channel at the pump station. For modeling purposes, the pump station was represented using a single pump element that withdraws water from the Shell Road Upstream storage area element and discharges to the Shell Road Downstream storage element. The rating curve for the pump element was set based on the calibration measurements completed in May 2013, with trigger elevations of 2.0 feet for the first pump and 2.4 feet for the second pump.

7.2 Hydraulic Model Input and Boundary Conditions

7.2.1 INFLOW HYDROGRAPHS

The output from the HEC-HMS hydrologic model was used directly as the source of the inflow hydrographs for the hydraulic model runs. Specifically, the combined flow path for each sub-watershed in the output .dss file was connected to the HEC-RAS model at specific points as summarized in Table 7-3.

Table 7-3. Connection points in the HEC-RAS hydraulic model for inflow hydrographs from the HEC-HMS hydrologic model. "SA" in this case signifies "Storage Area", the HEC-RAS model elements representing the open water areas of the Slough system.

Inflow Hydrograph	HEC-RAS Connection Location	Physical Location
Sub-watershed A	River Station 148	Watsonville Slough channel just downstream of San Andreas Road
Sub-watershed B	River Station 162	Watsonville Slough channel just downstream of RR crossing
Sub-watershed C	River Station 194	Watsonville Slough channel just upstream of Lee Road
Sub-watershed D	Watsonville SA	Watsonville Slough above Highway 1
Sub-watershed E	Struve SA	Struve Slough
Sub-watershed F	Struve SA	Struve Slough
Sub-watershed G	Struve SA	Struve Slough
Sub-watershed H	Harkin Mid SA	Harkins Slough upstream of farm road
Sub-watershed I	Harkin US SA	Harkins Slough upstream of RR bridge
Sub-watershed J	Harkin US SA	Harkins Slough upstream of RR bridge
Sub-watershed K	Harkin US SA	Harkins Slough upstream of RR bridge
Sub-watershed L	Harkin US SA	Harkins Slough upstream of RR bridge

As noted, the connections for the inflow hydrographs must be made at specific points in the hydraulic model, in this case at storage area elements or at cross-sections. Naturally, this is an approximation, since inflow actually occurs at many points within each sub-watershed. The impact of this approximation is not overly important where the connection point is a large storage area element, but should be borne in mind for those sub-watersheds (such as A and B) that are connected at specific cross-sections

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as total flow is somewhat underestimated immediately upstream of the connection point and, conversely, overestimated immediately downstream of the point.

Since the combined flow output path was used as the inflow hydrograph, it includes direct runoff and outflow from both the Groundwater 1 and Groundwater 2 layers (all per the soil moisture accounting methodology) as well as direct rainfall on the ponded slough areas.

7.2.2 EVAPOTRANSPIRATION LOSSES

Evapotranspiration (ET) losses from the wetlands presented a special challenge as they are an important source of system losses for continuous simulation runs over extended periods of time such as the 10-year simulation period used in this study. As discussed in Chapter 6, ET losses from the ponded slough areas are now equal to roughly 34 percent of the total inflow on a water year basis, and are on the order of 62 percent of total slough outflow. In this case the term evapotranspiration is used for the actual mix of ET from emergent wetland vegetation in the sloughs and direct evaporation from open water areas.

Losses due to ET were simulated in the model through additional pump elements. These pump elements were set to withdraw water from the five upstream storage area elements and discharge outside the model domain. The two Shell Road storage areas were not included: Shell Road Upstream is too small to be significant and Shell Road Downstream is tidal. No specific information on actual evaporation rates from any of the Watsonville Slough water bodies was found. Therefore, reference evapotranspiration information from the CIMIS 129 gage was used as a proxy for open water evaporation rates. This is clearly an approximation, but has support in other studies of wetland environments and resulted in reasonably good drawdown curve fits in the dry season (see calibration discussion below).

The pump elements (labeled ET Pumps in the model) were controlled by a multi-step pump rule written into the HEC-RAS model. Average monthly ET information (in inches) from the CIMIS 129 station was converted into an equivalent flow rate (in cfs) based on the surface area of each storage area at the previous time step. This allows the model to explicitly account for losses that vary both due to monthly changes in ET rates and due to changes in the extent of surface water coverage within each storage area element.

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7.2.3 DOWNSTREAM BOUNDARY CONDITION (TAILWATER)

The hydraulically complicated outlet of the Slough system presented particular challenges with respect to deriving an appropriate downstream boundary condition for the model. As discussed earlier, the Slough system ends at the Pajaro River Lagoon, where actual water surface elevations vary with a wide range of factors including whether or not the barrier bar is open or closed (with implications for tidal and wave energy forcing) and river discharge.

There are no long-term continuous monitoring data of water surface elevations downstream of Beach Road, so a synthetic non-steady state tailwater record had to be developed. The synthetic stage record covers the period from WY2003 through WY2011, with WY2012 using actual water surface elevations from the data collection program. It is important to bear in mind that only one of the ten years in the modeled tailwater relationship represents actual field data. Therefore, though care was taken to create a representative relationship, the model results cannot be expected to precisely recreate hydraulic conditions in the lowermost portions of the Slough system. This is especially true with respect to barrier bar ovetopping events (see discussion below).

The synthetic portion of the downstream boundary condition was compiled using multiple inputs. The first of these inputs was mean daily flow rate for the Pajaro River as measured at the USGS Chittenden gauge, which was used as the measure of whether the barrier bar would be open or closed, with the bar assumed to be open when river flow rates were equal to or greater than a specified threshold ranging from 11 cfs in the summer months to 35 cfs in the period from December through February. This flow threshold was selected based on the pattern of open barrier beach conditions observed in WY2012 and WY2013 and the respective river flow record as measured by the USGS. When open, the tailwater elevations were simply taken to be the same as the concurrent tide elevations in Monterey Bay as measured at Station 9413450 operated by the National Oceanic and Atmospheric Agency (NOAA). Tailwater conditions when river flows were less than the assumed threshold, implying a closed barrier beach, were simply represented by a step function derived from observed data for WY2012 and WY2013 and which incremented water elevations based on the cumulative time since initiation of the previous closure.

The derivation of the synthetic tailwater did not explicitly consider several variables that were evident even in the short duration of actual monitoring data. Foremost among

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these are the effects of higher river discharge rates and various measures of wave energy. High river discharge values (implying open barrier beach conditions), especially those in excess of 100 cfs, lead to a "washing out" of the tidal signal, which largely disappears at river flows in excess of approximately 500 cfs. Therefore, the synthetic relationship likely substantially underestimates tailwater elevations for Pajaro River flow rates in excess of 500 cfs, particularly at periods of low tide. For closed barrier beach conditions, not explicitly accounting for wave energy effects means that the increments in water surface included in the record are not explicitly linked to coastal dynamic effects, and beach overtopping events are therefore likely to both be over- and underestimated in the synthetic record.

Refinement of the downstream boundary condition with respect to peak elevations for both open and closed barrier beach conditions would be desirable as both river flood flow and beach overtopping events can potentially cause tailwater elevations higher than the effective downstream backflow control at Shell Road and lead to significant flow from the lagoon upchannel into the Slough system. Of these potential backflow conditions, those associated with beach overtopping are of most concern as they can lead to salinity intrusions (as already observed in the short monitoring period), while backflow from Pajaro River flood events, though potentially substantial, would be freshwater in nature. Quite accurate models of lagoon water surface have been developed for other coastal California settings (e.g. Rich and Keller, 2013), but would require more extensive monitoring data than are currently available for the Pajaro Lagoon and were beyond the scope of this study.

7.3 Model Calibration

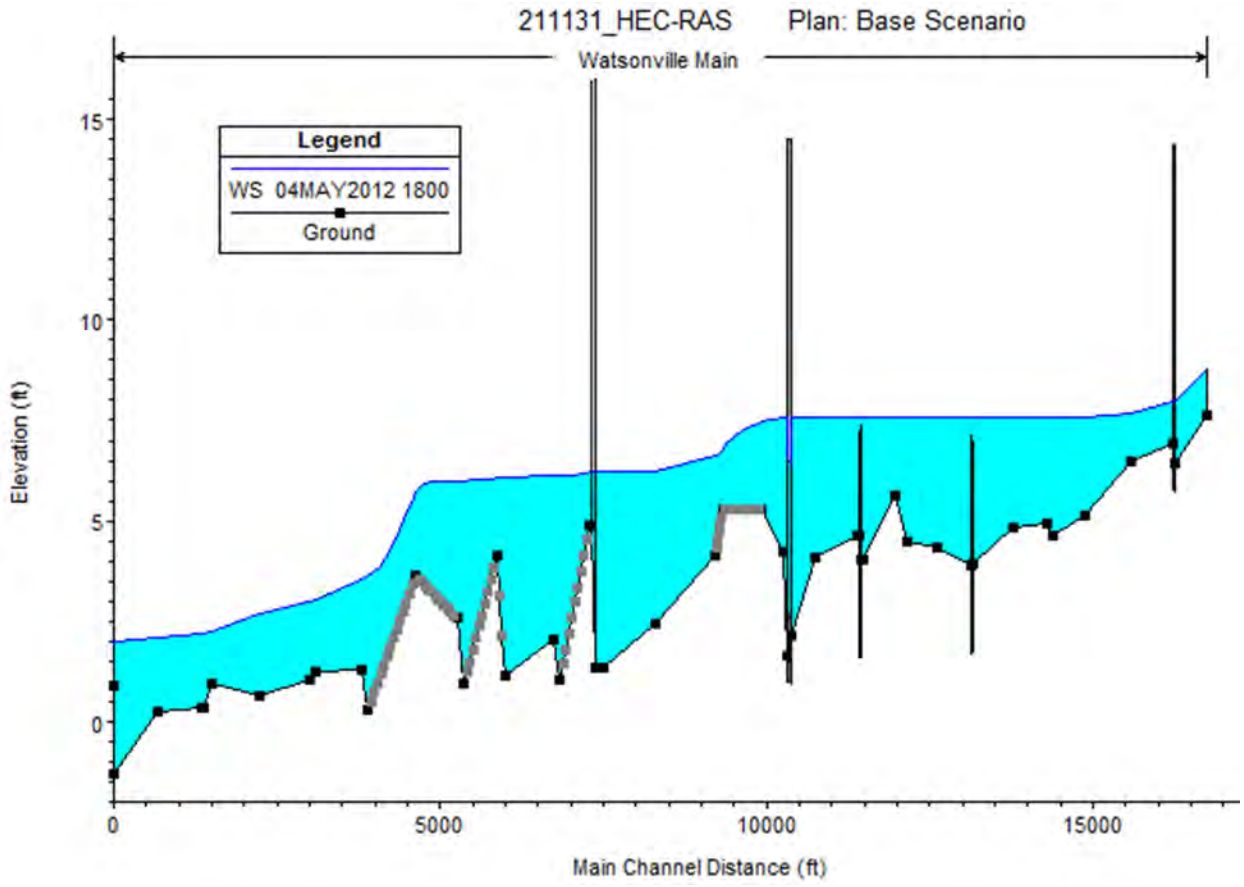
Calibration of the HEC-RAS model was carried out using the long-term continuous simulation dataset augmented by separate model runs that made use of data from WY2013 that was not included in the longer-term model. Calibration metrics focused on water surface elevations at key points in the model that directly corresponded to monitoring locations from the data collection program. However, calibration of the hydraulic model to stage alone was deemed insufficient, and the results were also checked against the compiled flow record from Watsonville Slough at San Andreas Road provided by the data collection program.

Calibration was primarily accomplished through iterative model runs with variations of a few key parameters. The importance of channel conveyance was readily evident in

even the preliminary runs, and the decision was made to control conveyance through adjustments in the channel roughness ("Manning's n") values as the primary variable. Detailed review of the field data showed that channel roughness varied significantly over the range of flow depths at any given location. This result is not surprising given the very thick aquatic vegetation in almost all channels in the Slough system. Calibrated roughness values for low baseflow conditions (e.g. mid-summer until the first rains) were exceptionally high, in some cases as high as 0.25. These values show that vegetation growth is of such an extent that the model is at its limits in terms of simulating what are actually substantially obstructed flow paths.

An excellent, spatially comprehensive, calibration data set is the long water surface profile collected by EDS concurrent with the survey work (see discussion in Chapter 4). An example of the calibration fit for spring baseflow conditions is illustrated in Figure 7-4 based on the data collected on May 4, 2012.

Figure 7-4. Modeled water surface elevations for spring baseflow conditions corresponding to survey data collected by Environmental Data Solutions on May 4, 2012 (see Figure 4-2 for surveyed data).



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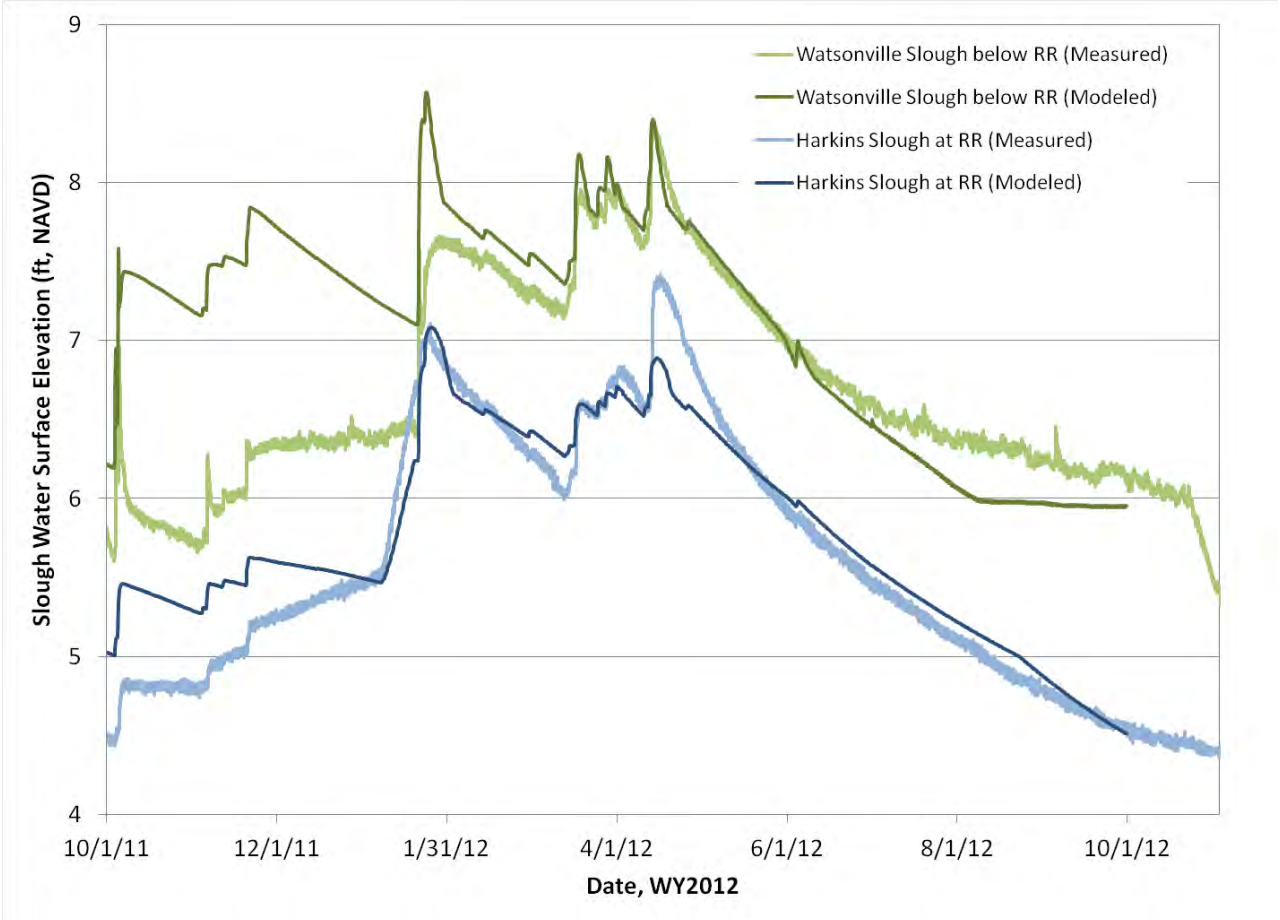
The fit of data for the spring baseflow conditions is quite good at points except those immediately upstream of the Shell Road Pump Station (e.g. approximately from model Station 0+00 to 15+00). The difference at that point was likely due, at least in part, to the relative timing of survey data collection in that reach and triggering of the Shell Road pumps.

The data in Figure 7-4 show calibration over a large area at a specific time. The model fit to observed data was also checked for longer time periods at specific spatial locations. The only significant overlap between the continuous simulation modeling timeframe and that of the data collection program is WY2012. Comparison of the observed water surface elevations in Harkins Slough (at the railroad) and Watsonville Slough channel (just downstream of the railroad) to those from the model are illustrated in Figure 7-5.

Figure 7-5. Comparison of observed water surface elevations to output from the HEC-RAS model for two specific areas of interest for the entirety of WY2012. Observed water surface elevations are as per the data collection program for Watsonville Slough channel upstream of the railroad (WSRR) and for Harkins Slough at the railroad (HSRR).



The fit of the model in the early part of the season (prior to the beach overtopping event and subsequent rain in early January) ranges from fair for Harkins Slough to very poor for the Watsonville Slough channel downstream of the railroad tracks. With respect to Harkins Slough the fit would actually be quite good except for the fact that the long-term modeled stage begins the water year roughly 0.5 feet too high. Increases in stage due to early season rainfall are captured well, however, the modeled stage is sufficiently high to allow runoff from Harkins Slough and out through the channel system leading to drawdown in periods when the observed stage is stable or slowly rising. The wide discrepancy between observed and modeled stage in the Watsonville-Struve branch of the overall system prior to January appears most directly attributable to issues with calibration of overall runoff timing for Sheds D and E as discussed previously in Chapter 5, recalling that the calibration for total system runoff volume as manifest in the field data-informed water balance is quite good (see Chapter 6). Therefore, continued work to refine the hydrologic model results for those particular sub-watersheds can be expected to directly improve the performance of the hydraulic model with respect to replicating observed stage values in the Watsonville-Struve branch in the early portion of each water year.

The fit of the modeled output to observed stage for the remainder of WY2012 is generally very good. This includes the fit for the change in stage associated with the beach overtopping event (early to mid-January) that shows up well in the Harkins Slough stage record. After that event, the fit is quite good (for both branches of the Slough system) with respect to observed stage throughout the series of storms that ran through April and into the summer drawdown period of much diminished to essentially no outflow. The only exception in this time period is a notable underestimate of stage for Harkins Slough in late April 2012.

Validity of the model results was also checked with respect to overall runoff volume as measured in the channel at San Andreas Road, similar to the manner in which the overall water balance from the hydrologic model was compared to the measured values at the same location. The comparison of cumulative runoff volume for WY2012 is illustrated in Figure 7-6 and shows the generally good agreement between the measured total runoff at San Andreas Road and that generated by the model. The inflow of water from the Pajaro Lagoon during the beach overtopping event in January is readily evident as the drop in the cumulative volume (outflow from the Sloughs is positive and inflow from downstream is negative in this analysis). Values for observed

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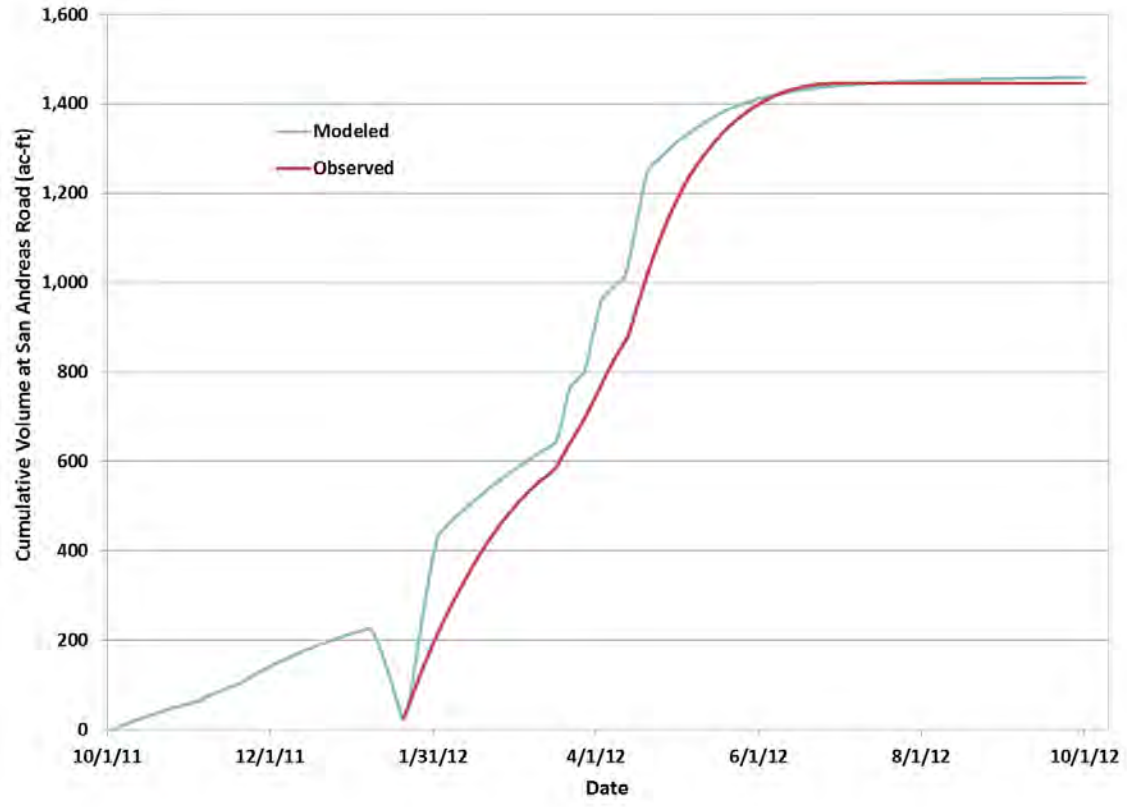
outflow prior to the beach overtopping are not included, but only amounted to a total of approximately 3 to 4 acre-feet. The difference between the modeled and the measured values in early-season period reflects the higher initial storage in the model output, bearing in mind that WY2012 is the final year in the 10-year simulation period.

The data in Figure 7-6 show that the combined hydrologic and hydraulic model can accurately simulate total flow through the Slough system. However, further refinement in the timing of runoff, especially that for early season rains, would markedly enhance the overall fit. Additionally, ongoing data collection to allow for calibration over a wider range of flow conditions is needed. Multi-year data sets would be particularly useful in assuring that the modeling tools represent the correct amount of carryover storage from one water year to the next.

Figure 7-6. Comparison of observed to modeled cumulative discharge at San Andreas Road (HEC-RAS model cross-section 150) for Water Year 2012. Note particularly the good agreement between the measured and modeled results and the distinct drop in cumulative volume due to upchannel flow associated with the beach overtopping event in January. Approximately 42 percent of the total annual runoff occurred in the 5-week period from mid March into April.



7.4 Base Model Output

7.4.1 OVERVIEW AND EXAMPLES

As mentioned previously, the hydraulic model output allows compilation of statistics for a wide-range of hydraulic parameters at a very large number of locations within the model domain. The discussion in Chapter 6 showed that the hydrologic model provides information on runoff rates, volumes, and timing to the Sloughs sufficient to refine water balance calculations for all or part of the overall system. However, the detailed resolution of the hydraulic model leads to a clearer picture of the way water moves within the system. This, coupled with the long-term time period modeled, provides a tool with fine spatial resolution capabilities that can assess responses over timescales ranging from hours to a full decade.

Table 7-4 shows an example of the way in which a specific hydraulic parameter, in this case water surface elevation, can be summarized at various locations over the full 10-year simulation period of the model. The summary statistics are consistent with expectations and show how reduced channel conveyance has led to relatively high water elevations throughout all but the lowermost reaches of the system. The modeled mean water elevation just upstream of Shell Road is 2.4 feet, which is fully 3.4 feet less than the mean of 5.8 feet for the channel at the San Andreas Road crossing. The way in which sediment dams have obstructed channel flow is underscored by the fact that the mean elevation for Harkins Slough is essentially the same as that at San Andreas Road. The greatest variability in water surface elevations (as measured by standard deviation) is found at Shell Road, not an unusual finding given that the location is subject to fluctuations from pumping and outflow through the flap-gated culverts under specific tidal conditions.

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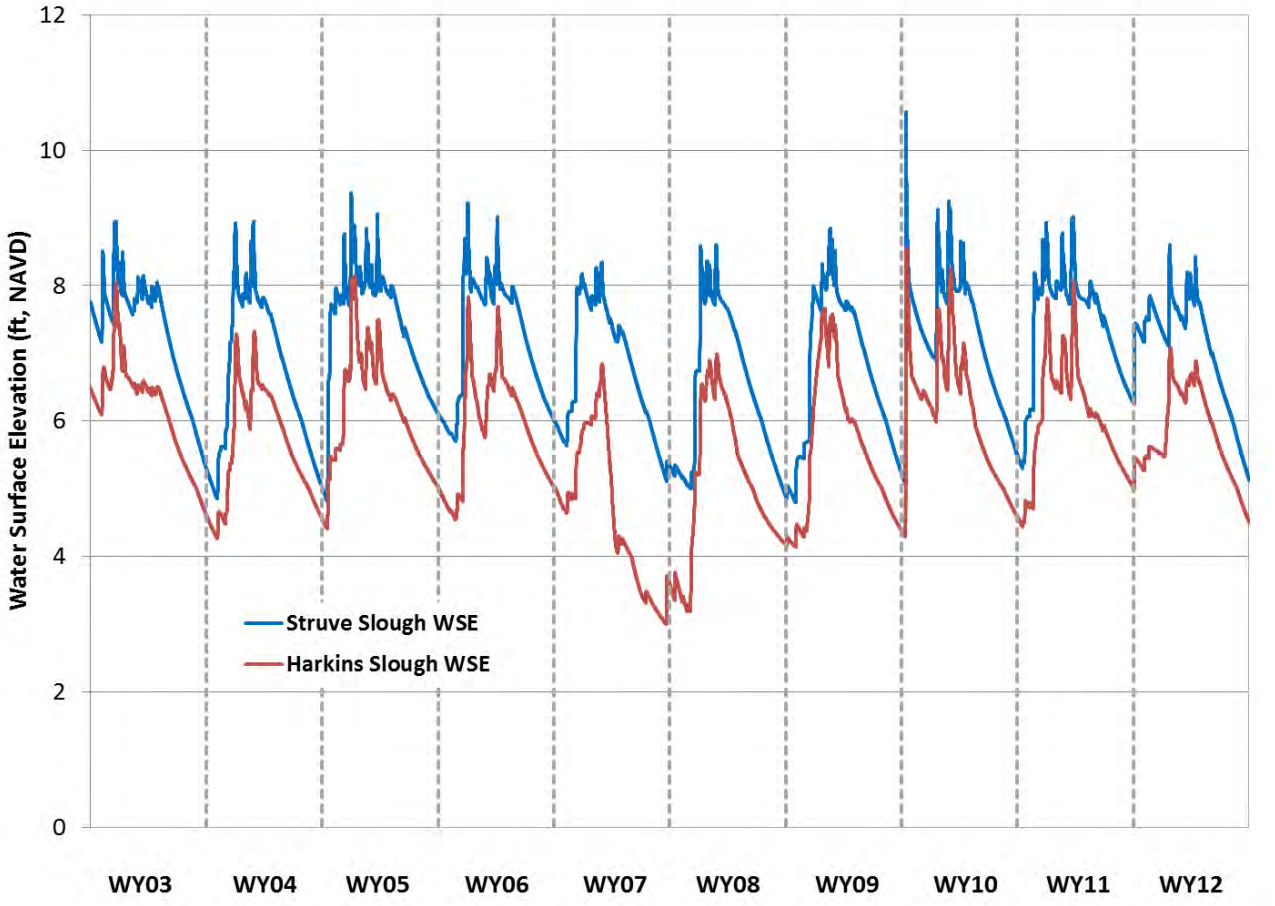
Table 7-4. Summary statistics for modeled water surface elevations per the HEC-RAS hydrualic model at key locations within the Slough system. Values based on modeled output for the entire 10-year continuous simulation period.

	Water Surface Elevation Statistics (*ft, NAVD*)			
Location	Mean	Std Dev	Max	Min
Shell Road	2.2	0.7	8.1	1.7
San Andreas Road	5.7	0.7	8.3	4.9
Harkins Slough	5.6	1.0	8.5	3.0
Watsonville Slough upstream of RR	7.1	0.8	10.6	5.8
Struve Slough	7.0	1.0	10.6	4.8
Watsonville Slough	8.7	0.7	15.0	7.5

In addition to summary statistics such as those in Table 7-4, the model output can be viewed in time series format as per the data previously illustrated in Figure 7-5. Plots of the time series data can provide insight into the intra- and inter-annual variability of selected parameters. Figure 7-7 shows an example of such a plot and illustrates the model output for the water surface elevation in Harkins and Struve Sloughs for the entire simulation period.

Figure 7-7. Simulated water surface elevations for Harkins Slough and Struve Slough per the HEC-RAS hydraulic model for the 10-year period from WY2003 through WY2012. The model output shows Struve Slough with a conistently higher water surface elevation than Harkins Slough. Note the general "capping" of maximum elevations at approximately 9 feet for Struve Slough and 8 feet for Harkins Slough, which shows that the extensive storage capacity in the sloughs compensates for impaired channel conveyance. The spike in elevation in October 2009 (WY2010) is due to a single atmospheric river storm system.



7.4.2 NOTABLE ASPECTS OF THE HYDRAULIC MODEL OUTPUT

As alluded to earlier, it is difficult to summarize the extensive model output without reference to a specific issue or area of concern. That said, a few aspects of the hydraulic model output are worth noting. These include:

- *Flow attenuation.* The limited capacity of the channels draining the Sloughs coupled with the large storage volumes in the inundated areas combine to greatly suppress peak flow rates in the downstream direction. An excellent example from the modeling record is the large storm event that occurred on October 13, 2009. This storm resulted in nearly 6 inches of rain at the CIMIS 129

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gage and produced the largest peak watershed flow rate of roughly 3,000 cfs. However, the peak discharge at Shell Road for the same event was only 138 cfs.

- *Seasonal recovery of water levels*. The limited conveyance capacity (particularly the flow obstructions in the channels) has another noticeable effect: relatively rapid and consistent recovery of water levels in the early winter. This is particularly true in the Watsonville-Struve branch of the system, where more impervious cover associated with developed areas produces enough runoff even in dry years to lead to substantially consistent maximum water elevations by mid- to late-winter. Harkins Slough, with lower contributing area runoff rates and a "dependence" on inflow from the Watsonville Slough channel is more susceptible to dry conditions as evident in the model output for WY2007.

- *Annual low water levels*. The tendency for runoff to be retained in the Sloughs directly impacts annual low water levels. The low stand in the Sloughs is closely correlated with the date of the last several rain events each spring. When rainfall ends earlier, there is more time for evaporation losses, and Slough water surfaces draw down to their lowest levels. Conversely, late season rains can result in enough runoff to offset a significant portion of the evaporation losses and result in higher annual minimum water levels. This effect is readily evident in WY2011 where substantial precipitation was recorded in late May and early June.

- *Annual maximum water levels*. The geometry and topography of the drainage network in the Slough system result in a typical mid-winter base elevation for the ponded areas. This can be seen in Figure 7-7, where Struve Slough generally settles into a winter base level at an elevation of just under 8 feet. Water surface excursions above that level are relatively small (on the order of one foot or less) and recovery to the base level is generally within a week. A similar pattern is seen for Harkins Slough at an elevation of roughly 6.5 feet.

- *Pumped versus gravity outflow at Shell Road*. Despite the relatively low total capacity of the Shell Road Pump Station, the pumps still handle the majority of outflow from the system. This reflects factors such as high levels in the Pajaro Lagoon (which close the flapgates on the Shell Road culverts) and the aforementioned flow attenuation in the Sloughs (which lowers total outflow rates to levels manageable by the pumps). In drier than average years such as WY2012, the pumps discharge approximately 87% of the total annual outflow. In

wetter years, where outflow rates are correspondingly higher, proportionally less of the total outflow is handled by the pumps. For example, in WY2010 the pumps discharged only 53% of the total outflow.

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8 SCENARIO MODELING

Once the base hydraulic model build, calibration, and data runs were completed, work shifted to application of the model through a select number of scenarios selected by the Technical Advisory Committee. This chapter describes the characteristics of each of the selected scenarios followed by a discussion of the results of the scenario model runs.

8.1 Scenario Overview

Table 8.1 gives key characteristics of each of the scenarios. A total of six scenario runs were completed falling into two main groups based on model geometry and management activities. These main groups were given numerical designations, thus Scenario 1 and Scenario 2. Each of these main groups was then further differentiated on the basis of sea level rise as an example of the ability of the modeling platform to simulate future conditions associated with climate change. The sea level rise scenarios are distinguished by a letter designation following the scenario number.

Table 8-1. General characteristics of the base model and scenarios prepared as part of the Watsonville Sloughs Hydrology Study.

Scenario	Characteristics
Base Model	Slough channel conditions as surveyed in spring 2012 Harkins Slough diversion pumping per PVWMA records
Scenario 1	Wetland restoration at Knox Property per 2013 plans Increase Harkins Slough diversion pumping to 2,000 ac-ft/yr maximum
Scenario 1B	As per Scenario 1, but increase tailwater by 14 inches (2050 sea level rise)
Scenario 1C	As per Scenario 1, but increase tailwater by 55 inches (2100 sea level rise)
Scenario 2	Same as Scenario 1, but remove sediment and vegetation from channels downstream of RR crossing to represent regular maintenance
Scenario 2B	As per Scenario 2, but increase tailwater by 16 inches (2050 sea level rise)
Scenario 2C	As per Scenario 2, but increase tailwater by 55 inches (2100 sea level rise)

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8.1.1 SCENARIO 1

Scenario 1 includes both changes in model geometry and in management activities as follows:

- *Model geometry*. Cross-sections and lateral weir elements north and east of the confluence of Harkins Slough with the Watsonville Slough channel were altered per preliminary restoration grading plans for the Knox property. The proposed restoration work includes grading that would increase the connectivity between the Watsonville channel and the Harkins channel downstream of the farm road bridge. As discussed in Chapter 4, overflow and connectivity via this route (though not necessarily intentional) are important characteristics of the lower Slough system at present. The Knox restoration work would essentially formalize and enhance flow routes for this interconnectivity.

- *Management activities*. As noted in Chapter 7, the Base Scenario model explicitly includes the historical diversion pumping from Harkins Slough carried out by PVWMA to supply aquifer recharge operations. Historical pump operations have been based on a historical maximum diversion of roughly 1,000 acre-feet per water year. For Scenario 1, the historical pumping rates are replaced by an updated pump rule for the Harkins Slough Pump Station that allows up to 2,000 acre-feet of diversions per water year as conditions allow.

Scenario 1 has characteristics that can be thought of as representing a short- to mid-term condition for the Slough system since preliminary studies and permitting considerations have already been considered for both the Knox Property and increased diversion aspects.

8.1.2 SCENARIO 2

Scenario 2 includes the changes associated with Scenario 1 and then incorporates a major reconfiguration of the model geometry to simulate how the system would behave if a comprehensive channel maintenance program were pursued to completion. In that sense Scenario 2 can be thought of as a mid- to longer-term scenario in that it would require successful permitting and completion of work

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associated with the Knox property wetland restoration as well as maintenance dredging of the entire lower Watsonville Slough channel from the railroad crossing down to Shell Road, a distance of approximately 10,000 feet (model cross-sections 114 to 162).

The model geometry for Scenario 2 was prepared by modifying the HEC-RAS cross-sections for the reach referred to above. Channel width, depth, and slope were made uniform for the reach, appropriate for representation of channel hydraulics soon after completion of sediment and vegetation removal by dredging. Channel roughness values were reduced as well, consistent with expected conditions for a maintained channel. Combined, these changes in the model result in a very significant increase in channel conveyance capacity. Thus, Scenario 2 represents an upper bound on how the lower Slough system would function given the alterations that would accompany a comprehensive maintenance program. Much, if not most of the impact simulated in this scenario could potentially be achieved with a targeted effort to remove the most constricting of the sediment dams discussed in earlier chapters. However, modeling of varying levels of channel maintenance was beyond the scope of this study.

It is also important to note that, even though Scenario 2 can be considered as a mid- to longer-term condition, it may well come closest of all the scenarios to representing conditions that prevailed in the watershed decades ago when channel maintenance was carried out on a regular basis. It is true that the hydrologic modeling demonstrates that there has been a significant increase in total runoff with increasing urbanization in the watershed over the last 40 years, but in the 10-year time frame used in the continuous simulation hydraulic modeling, pumping for the PVWMA recharge project has been an offsetting factor, at least with respect to Harkins Slough and those portions downstream of the confluence of Harkins Slough and the Watsonville Slough channel.

8.1.3 SEA LEVEL RISE

There are many aspects of climate change that have the potential to significantly impact the hydrologic and hydraulic behavior of the Slough system. These include, but are certainly not limited to, factors such as changes in short- and long-term rainfall amounts, temporal changes in precipitation patterns (both seasonal and inter-annual), alterations in evaporation patterns due to temperature changes or shifts in coastal fog behavior, and changes in vegetation density and type in response to all of the previous. Any or all of these factors could potentially be addressed through

appropriate changes to the base hydrologic and hydraulic models. However, practical considerations led to the selection of sea level rise as the factor of most immediate interest in terms of this study.

Sea level rise is a potential driver of Slough system behavior due to the direct connection and ample interactions that exist between the lower portion of the system and the Pajaro Lagoon as discussed previously in this report. The precise response of the Lagoon to increases in base sea level will naturally be quite complex, involving the interplay of a number of variables that could well be changing concurrently due to climate change (e.g. Pajaro River flow and sediment load, coastal sediment transport, wave dynamics, etc.). Nonetheless, changes in the downstream hydraulic boundary condition represented by the tailwater elevation immediately downstream of the Beach Road crossing (entrance to Pajaro Dunes) can have a significant impact on Slough function and merit consideration.

Alterations in the tailwater condition are readily simulated using the model platform, particularly since they can be restricted to the hydraulic model alone. For the present study, two separate magnitudes of sea level rise were modeled: 14 inches and 55 inches above the observed tidal elevations during the 10-year simulation period. These values correspond to the mean and high average predicted increases in sea level for this portion of the California coast for the years 2050 and 2100 respectively as documented by the California Ocean Protection Council (CaOPC, 2011). In each case, the respective elevation difference was simply added to the tailwater record used in the Base Scenario model build. Model runs for the 14-inch increase scenario were designated with a “B” suffix and those with a 55-inch increase were designated with a “C”.

8.2 Results of the Scenario Model Runs

As with the base model runs, each scenario generates extensive output data files that can be reviewed and summarized in many ways on a temporal and spatial basis. Therefore, this section will present an overview of the scenario model output with an emphasis on the parameters that were discussed in previous chapters.

8.2.1 OVERVIEW OF SCENARIO RESULTS WITHOUT SEA LEVEL RISE

Table 8-2 summarizes the model output with respect to water surface elevations at key points in the Slough system.

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Table 8-2. Summary statistics for modeled water surface elevation at key points in the Watsonville Sloughs system for the base model and Scenarios 1 and 2. All elevations are in feet NAVD.

	Base Scenario			Scenario 1			Scenario 2		
Location	**Mean**	**Max**	**Min**	**Mean**	**Max**	**Min**	**Mean**	**Max**	**Min**
Shell Road	2.2	8.1	1.7	2.1	8.1	1.7	2.2	7.9	1.8
San Andreas Road	5.7	8.3	4.9	5.4	8.3	4.9	2.2	8.1	1.9
Harkins Slough	5.6	8.5	3.0	4.9	8.4	2.7	3.9	7.8	2.5
Watsonville Slough upstream of RR	7.1	10.6	5.8	6.8	10.4	5.8	6.7	10.1	5.9
Struve Slough	7.0	10.6	4.8	6.5	10.4	4.4	6.4	10.1	4.3
Watsonville Slough	8.7	15.0	7.5	8.6	15.0	7.4	8.7	15.0	7.5

The statistical summary of Table 8-2 highlights several important insights from the scenario runs, including:

- *Scenario 1 mean water elevations*. The modeling results indicate that this scenario would have little to no appreciable effect on water surface elevations at the upper end of the system (Watsonville Slough) or at the lower end (Shell Road). However, the enhanced flow path from the Watsonville Slough channel to Harkins Slough provided by the Knox Property restoration and increased diversion pumping from Harkins Slough are predicted to have an impact on water surface elevations in the middle reaches of the system. The most pronounced impact would be in Harkins Slough itself, where the mean modeled water level would drop by nearly a foot. The weir at the Knox Property would reduce the tailwater elevation for outflow from Struve Slough, resulting in a drop in mean water surface of roughly 0.5 feet. Sediment dams would still be in place downstream of San Andreas Road, so mean water surface elevations at that point would be only slightly changed.

- *Scenario 1 extreme water elevations*. With downstream channel obstructions still in place, the basic hydraulics of the overall system would not be

fundamentally changed, with the potential for rapid runoff in large storms and only limited outflow potential. Therefore, maximum water surface elevations would hardly be affected at any point in the system, including the main ponded slough areas. Minimum water surface elevations in Struve and Harkins Sloughs would drop marginally, the former due to increased outflow to Harkins via Knox and the latter due to increased diversion pumping.

- *Scenario 2 mean water elevations*. The impact that restricted channel conveyance has on Slough hydraulics in readily apparent in review of the Scenario 2 results. Regular maintenance of the lower slough channels would have a substantial impact in the middle reaches of the system. Water levels at Shell Road would not change as the combined pump and culvert outflow would keep pace with any changes in flow quantity and timing. Similarly, Watsonville Slough above Highway 1 would be above the area impacted by maintenance. The largest changes would be at San Andreas Road, where removal of downstream sediment dams would lower mean water elevations by more than three feet. Increased capacity in the slough channels would reduce the amount of flow across the Knox property into Harkins Slough, resulting in lower average water levels there as well. Mean levels in Struve Slough would be largely unaffected as channel obstructions would still be in place above the RR crossing.

- *Scenario 2 extreme water elevations*. The changes in extreme water levels would be most pronounced at the same locations where changes in the average values are most evident. Minimum water levels would be particularly low at San Andreas Road and somewhat lower at Harkins Slough (under this scenario diversion pumping would never reach a full 2,000 acre-feet/year due to low water levels). Maximum water levels would be reduced substantially in both Struve and Harkins Sloughs.

Given its central location in the watershed, Harkins Slough would be most impacted by the changes engendered in the scenario runs. Figure 8-1 illustrates the impact of the scenarios on water surface elevations in Harkins Slough on a month-by-month basis.

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Figure 8-1. Modeled monthly mean water surface elevations in Harkins Slough for the base conditions and Scenarios 1 and 2. Note the overall appreciably lower water surface under each scenario, and particularly for Scenario 2.

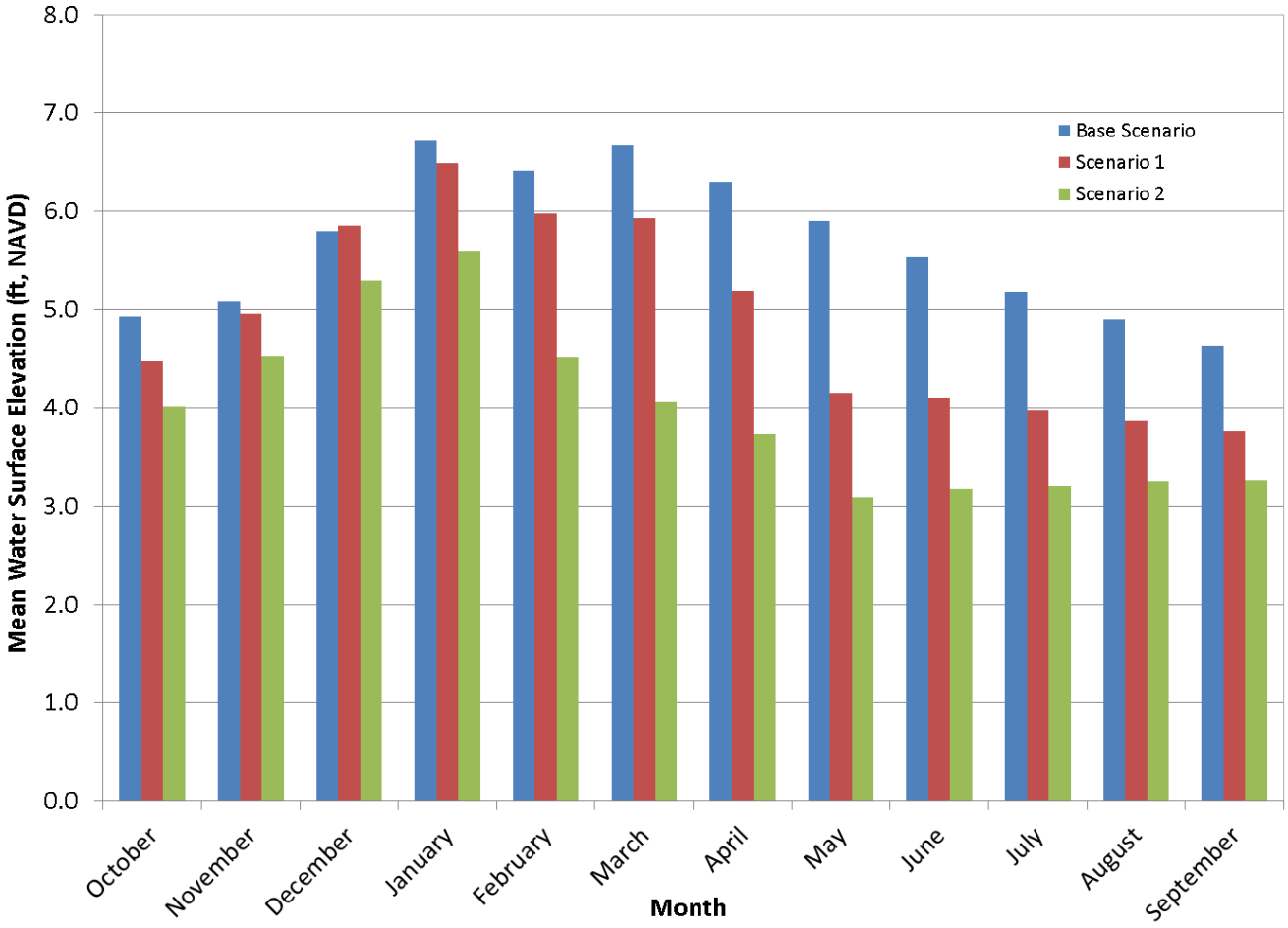
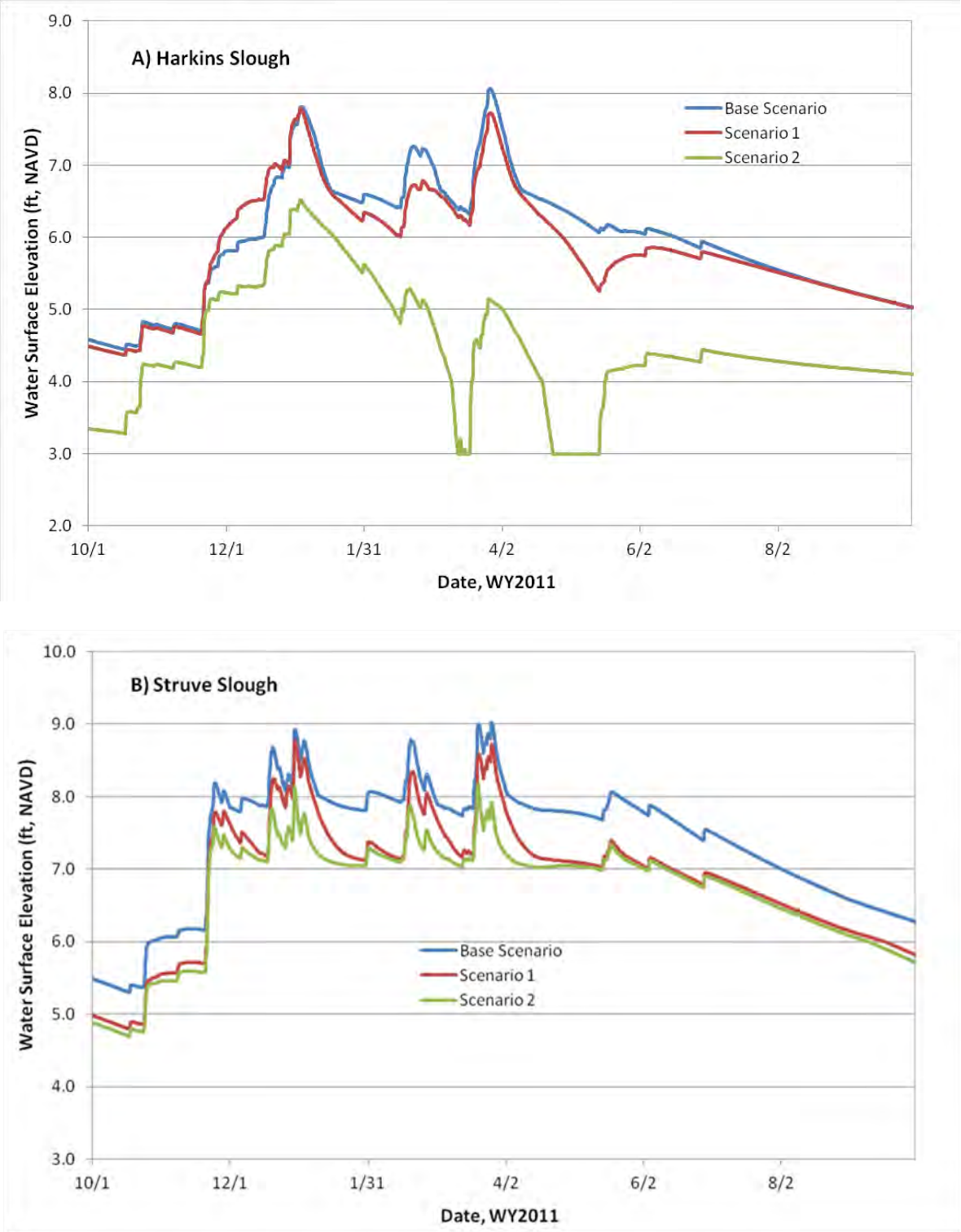


Figure 8-1 shows additional detail on the temporal change in mean water surface that would be expected under the scenarios. The impact under Scenario 1 is somewhat muted, with increased flow across the Knox Property partially offsetting the increase in diversion pumping for the PVWMA recharge system. The impact of the maintained downstream channels in reducing flow across the Knox restoration is especially evident in the late winter and early spring under Scenario 2.

The specific intra-annual variations associated with each scenario may also be of interest from the perspective of managing the Sloughs. Examples with respect to the modeled water surface elevation in Harkins Slough and Struve Slough for WY2011 (wetter than average) are shown in Figure 8-2.

Figure 8-2. Modeled mean water surface elevations in Harkins Slough (A) and Struve Slough (B) for the base conditions and Sceanrios 1 and 2 for WY2011.



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As Figure 8-2 shows in this wetter than average year, the impacts under Scenario 1 would be relatively small and a full 2,000 acre-feet of water could be diverted to recharge. However, the difference under Scenario 2 is pronounced, where the maintained channel system would reduce inflows via the Knox Property and diversion pumping would lead to substantially lower water surface levels. By contrast, water levels in Struve Slough are most impacted under Scenario 1, with reductions in both peak and baseline elevations. Scenario 2 would have little impact on Struve Slough base elevations, but would further reduce peaks associated with winter storm events.

The data can also be analyzed in terms of cumulative discharge at any point within the hydraulic model domain, giving a reference point for changes in the overall water balance for the Slough system. An example is provided in Table 8-3, which summarizes the total annual discharge passing San Andreas Road.

Table 8-3. Summary of modeled total flow at San Andreas Road for base conditions and scenarios without sea level rise. The decrease in mean annual outflow in Scenario 1 results from the increased diverion pumping from Harkins Slough upstream of San Andreas Road. Under Scenario 2, the maintained channels would allow runoff from the Watsonville-Struve branch of the system to bypass Harkins Slough, markedly limiting the potential for increased diversions.

	Base Scenario	Scenario 1	Scenario 2
	Discharge *(ac-ft)*	Discharge *(ac-ft)*	Discharge *(ac-ft)*
10-Year Mean	2,810	1,830	2,530
By Water Year			
WY2003	3,090	2,140	2,710
WY2004	1,900	1,230	2,030
WY2005	5,000	3,920	4,450
WY2006	3,190	2,050	3,040
WY2007	960	240	1,070
WY2008	1,410	840	1,540
WY2009	1,750	670	1,060
WY2010	5,220	4,100	4,660
WY2011	4,050	2,910	3,650
WY2012	1,480	220	1,120

8.2.2 SCENARIO RESULTS WITH SEA LEVEL RISE

The impacts of even modest increases in tailwater elevation (e.g. water surfaces in the Pajaro Lagoon) have the potential to be quite significant. As discussed in Section 8.1, actual increases in lagoon levels due to sea level rise are likely to be complex and a detailed representation of those future conditions is beyond the scope of this study.

Therefore, the simplifying assumption was made that future sea level rise would yield a corresponding increase in tailwater for the system. Since high water stands associated with beach overtopping events already occur (including direct observation of events in both WY2012 and WY2013) and reach or exceed the elevation of the most downstream control (Shell Road), any upward shift in the tailwater boundary condition must result in increased frequency and magnitude of flow reversal in the system. This would be particularly true if mitigation measures such as increased levees and enhancements to the Shell Road Pump Station are not undertaken. No such measures are assumed to apply for the sea level rise scenarios. Thus, the results with respect to sea level rise may well overstate the potential impact, but certainly give a picture of how conditions may change due this single climate change factor.

The potential impact of sea level rise on water surface elevations in the Slough system can be seen in the summary statistics for the Scenario 2 10-year simulation period presented in Table 8-4.

Table 8-4. Summary statistics for modeled water surface elevation at key points in the Watsonville Sloughs system for Scenario 2 with sea level rise. Statistics are for 10-year simulation runs using WY2003 through WY2012 climate data and the Scenario 2 HEC-RAS model structure. All elevations are in feet NAVD.

	Scenario 2			Scenario 2B (*14-inch SLR*)			Scenario 2C (*55-inch SLR*)		
Location	**Mean**	**Max**	**Min**	**Mean**	**Max**	**Min**	**Mean**	**Max**	**Min**
Shell Road	2.2	7.9	1.8	2.5	9.8	1.8	8.5	13.2	1.8
San Andreas Road	2.2	8.1	1.9	2.6	9.7	1.9	8.5	13.1	2.0
Harkins Slough	3.9	7.8	2.5	4.2	9.7	2.5	8.6	13.1	5.4
Watsonville Slough upstream of RR	6.7	10.1	5.9	6.7	10.1	5.9	8.7	13.5	6.8
Struve Slough	6.4	10.1	4.3	6.4	10.1	4.3	8.7	13.5	6.8
Watsonville Slough	8.7	15.0	7.5	8.7	15.0	7.5	9.3	15.5	7.5

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The model output with sea level rise shows how impacts can potentially spread through the system with different increases in downstream elevation. For example, the Scenario 2B results indicate that even moderate increases in sea level can have a significant effect on the system up to, and including, Harkins Slough. In fact, the modeling predicts that sea water incursions would increase to such an extent that the lower Slough system would convert to a mixed hydrologic regime, with beach overtopping events introducing large volumes of saline water during the high wave energy late fall and winter balanced by freshwater runoff from the watershed. Conversion to salt marsh vegetation in the lower Slough channels and southern end of Harkins Slough could be expected and diversion pumping for aquifer recharge would be unlikely in most, if not all, years. By contrast, the limited culvert capacity at the existing RR crossing and modest increase in tailwater elevations would largely protect the Watsonville-Struve branch from major inflows of saline water. Additionally, the greater stormwater runoff rates from the sub-watersheds in this branch would provide a markedly higher rate of freshwater flushing. Therefore, whole-scale conversion of the hydrologic regime in the Watsonville-Struve branch would not be expected.[12]

The data in Table 8-4 for Scenario 2C, however, point to the potential for a fundamentally and completely altered hydrologic regime with large increases in sea level. The results should be viewed as a generalized approximation only, since they imply a level of tidal exchange through the Pajaro Lagoon and up into the Slough system that would have to be accompanied by a wholesale reconfiguration of the barrier beach and its seasonal dynamics. Nonetheless, it is important to bear in mind that mean tide levels in Monterey Bay with 55 inches of sea level rise would be on the order of 7.6 feet NAVD, only fractions of a foot above the present elevation of Shell Road and high enough for inflow through the Shell Road culvert vents. Mean higher high water under such a scenario would be on the order of 10 feet, well above existing downstream control elevations. Under such circumstances, essentially all of the system

12 Again, it is important to point out that these results are without mitigation measures in place. Protection from adverse impacts for modest increases in sea level would likely be straightforward to implement and not particularly challenging from a technical or management perspective. The HEC-RAS model provides an excellent tool to evaluate potential mitigation measures.

up to Highway 1 would likely change to a very brackish to saline character, although mixing effects and the potential for extensive stratification would play critical roles in how freshwater runoff would move through the system.

Table 8-4 presents statistics based on the entire 10-year simulation period and does not identify seasonal differences that would be important factors in determining the hydrologic regime under sea level rise. Figure 8-3 illustrates how mean monthly water surface elevations at one point (Harkins Slough as per Figure 8-1) would be expected to change for the Scenario 2 conditions with and without sea level rise. Comparison of the Scenario 2 and 2B values show that the impacts from modest sea level rise would largely be confined to the winter months when the existing Shell Road pumps and culverts (the latter less efficient due to higher tailwater) would not be able to keep up with the more frequent influxes of sea water. For greater sea level rise, as in Scenario 2C, Figure 8-3 indicates a system that is predominately lagoonal/tidal, with mean water levels peaking during the winter when the highest wave energy and storm events coincide.

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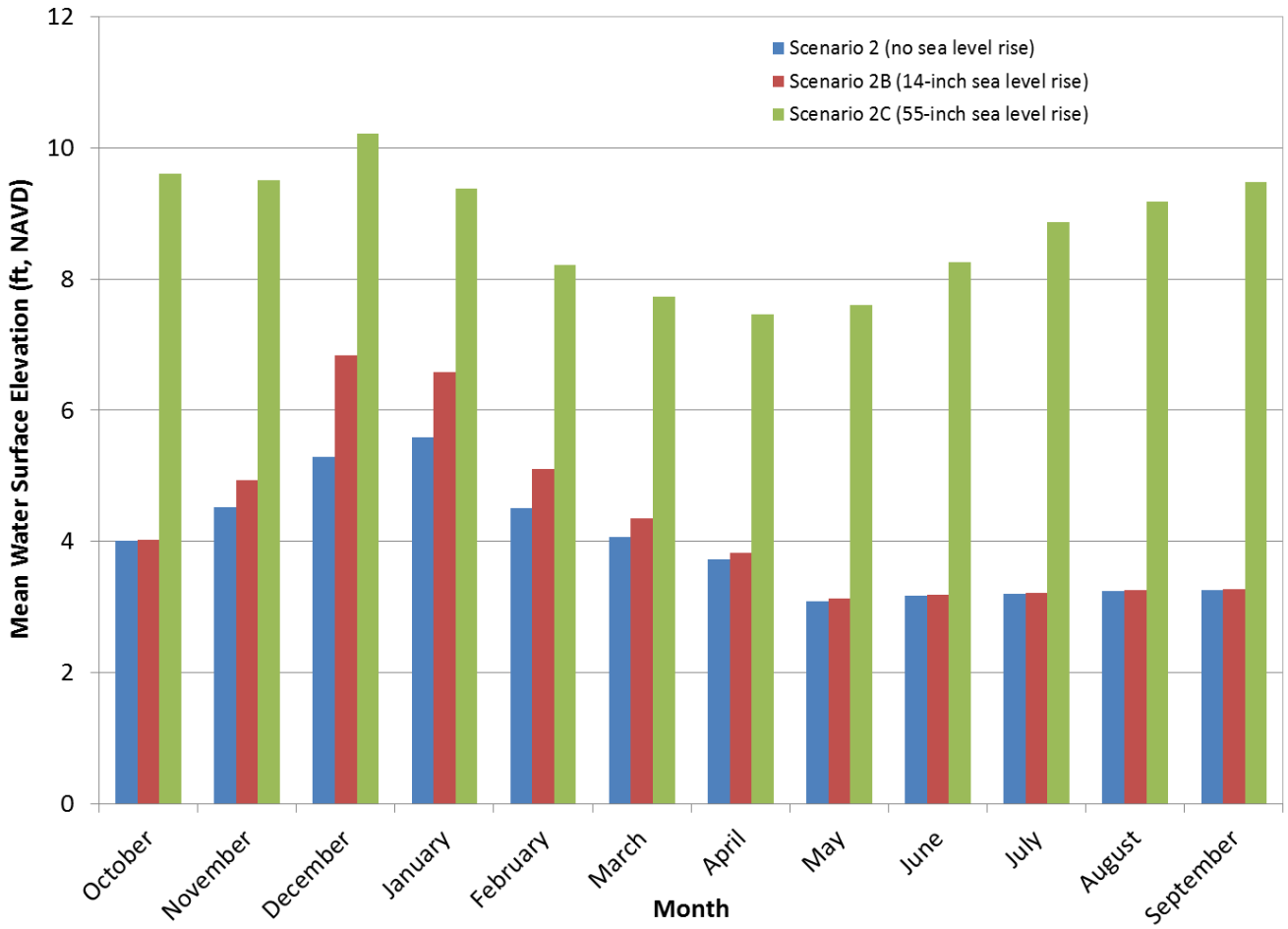
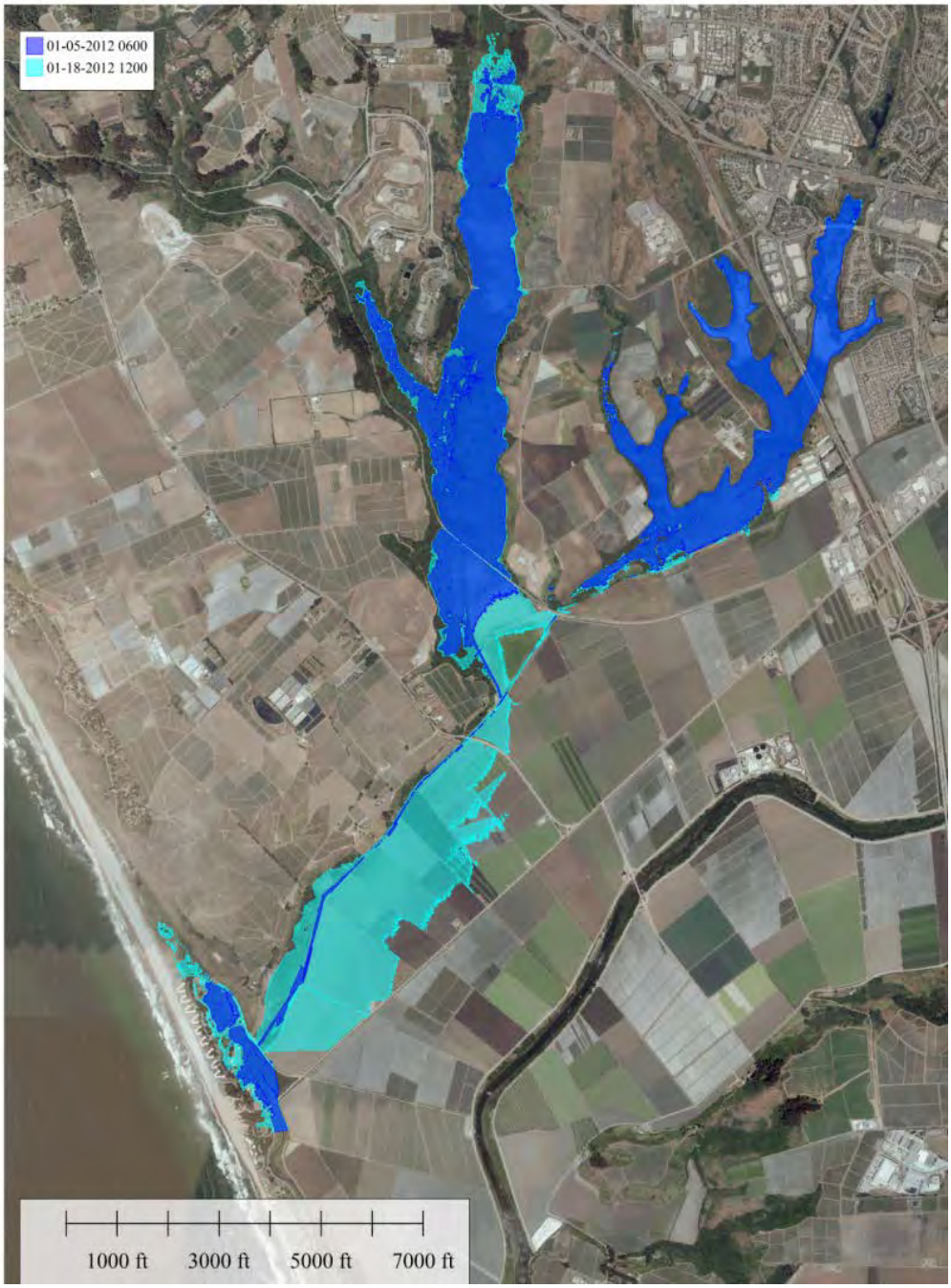
Figure 8-3. Modeled monthly mean water surface elevations in Harkins Slough for Scenario 2 with and without sea level rise. Note the markedly different water levels that would be associated with large increases in sea level and would be associated with a complete change in the hdyrologic regime of the Slough.



The data in Table 8-4 and Figure 8-3 give a statistical sense of the potential impacts of sea level rise, but do not readily present a sense of the impacts in a spatial sense. Fortunately, the output from the hydraulic modeling tool can be viewed in a comprehensive spatial manner for any specific time in the 10-year simulation period. This allows visualization of the model results through export to a GIS environment where parameters such as extent of flooding and water depth can be analyzed.

An example of this functionality of the model is presented in Figure 8-4, which shows the expected extents of inundation from the January 2012 beach overtopping event under Scenario 1B conditions.

Figure 8-4. Modeled extent of inundation for the beach overtopping event of January 2012 under Scenario 1B conditions (14 inches of sea level rise). The dark blue areas show the extent of ponding prior to the event and would roughly correspond to the expected extents under existing conditions. The lighter blue areas are the additional areas that would be flooded by the same wave energy under modest sea level rise conditions. The modeling shows that water levels could rise to the point of flooding much of the area north and west of Beach Road and San Andreas Roads.



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The extent of inundation shown in Figure 8-4 underscores the susceptibility of the low-lying valley bottom locations to impacts from relatively small increases in sea level. The increase in flooding extent at the north end of Harkins Slough results from substantially higher water elevations, which would correspond to significantly larger inflows of seawater. A single such event could lead to prolonged density stratification of Harkins Slough, with a persistent saltwater lens underlying a freshwater cap. Such conditions could be difficult to reverse given the low runoff rates from the Harkins Slough watershed. By contrast, the limited increase in extents in the Watsonville Slough channel and Struve Slough point out how conveyance restrictions (such as an unmaintained channel and the culverts at the RR crossing) would work to limit the influx of saline water further east.

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9 RECOMMENDATIONS

As alluded to at various points in this report, the complexity of the Watsonville Sloughs system cannot be fully understood without long-term data at an appropriate spatial resolution. Similarly, any modeling effort, by its very nature, can be continuously improved as new information and observations become available.

There is a clear need to continue work to characterize and describe key attributes and functional relationship in the Sloughs over the full range of disciplines. Nonetheless, the work in this study points to a number of recommendations that have can have a direct bearing on improving the modeling tools that have been created and will improve their accuracy and utility moving forward.

Specific recommendations include the following, presented in a general order of descending importance:

- *Overall ongoing monitoring*. As discussed previously, hydrologic conditions during the study period were consistently drier than historical longer-term norms. Therefore, calibration and validation of the modeling tools was not possible for conditions more representative of average or above-average annual rainfall conditions. Therefore, it would be highly desirable to continue data collection efforts to provide a minimum core of field data (see below) for a water year with above normal precipitation, and preferably one where the watershed area-weighted precipitation exceeds 25 inches.

- *Critical gage locations.* Continued operation of the gaging locations listed in Table 9-1 is highly recommended to provide data for future model calibration and verification and informed management of the Slough system. Several of the sites merit additional comment including:

    - Watsonville Slough downstream of Shell Road (WSDS). This station is critical for providing ongoing information related to the dynamic behavior of the Pajaro Lagoon, including frequency and timing of breaches and overtopping events, frequency and magnitude of backwatering from Pajaro River flows, and tidal benchmarking for open barrier bar conditions.

    - Harkins Slough at Buena Vista Road (HSBV) and Gallighan Sough below the Landfill (GSBL). These stream gages are needed to provide information on runoff from primarily open space/agricultural sub-watersheds and characterize runoff from the majority of lands draining to Harkins Slough.

    - West Struve Slough at High School (SSWS) and Watsonville Slough at Highway One (WSHO). These stream gages would provide important data on runoff from the highly urbanized areas draining to the Watsonville-Struve branch of the system.

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- Struve Slough. Though not listed in Table 9-1, we recommend water level monitoring for Struve Slough itself and in fact project wrap up work has included installation of a new logger at Lee Road. We suggest this station be coded as SSLR and monitored to provide direct measurement of levels in Struve Slough.

Table 9-1. Recommended minimum monitoring locations list for ongoing data collection.

Project Stations[1]	Code	Latitude	Longitude	Objective	Equipment
West Struve Slough at High School	SSWS	36.918	-121.791	Flow	PT
Watsonville Slough at Highway One	WSHO	36.903	-121.779	Flow/Salinity	PT/SCT (Telemetered)
Watsonville Slough upstream of Shell Road	WSUS	36.871	-121.818	E/Salinity/Rain	PT/SCT,TB (Telemetered)
Watsonville Slough downstream of Shell Road	WSDS	36.871	-121.819	WSE/Salinity	PT/SCT
Harkins Slough at Buena Vista Road	HSBV	36.938	-121.808	Flow/Salinity	PT/SCT
Gallighan Slough below Landfill	GSBL	36.911	-121.813	Flow/Salinity	PT/SCT
PVWMA Stations[2]					
Watsonville Slough at Railroad	WS@RR	36.894	-121.799	WSE/Salinity	PT/SCT
Watsonville Slough at San Andreas Road	WS@SA	36.888	-121.805	Flow/Salinity	PT/SCT
Watsonville Slough at Shell Road	WS@SR	36.871	-121.818	WSE/Salinity	PT/SCT
Harkins Slough at Pump Station	HS@HSPUMP	36.890	-121.803	WSE/Salinity	PT/SCT
Watsonville Slough at Railroad	WS@RR	36.894	-121.799	WSE/Salinity	PT/SCT
Harkins Slough at Railroad	HS@RR	36.898	-121.805	WSE/Salinity	PT/SCT

Notes:
1. Installed stations were those set up by Balance Hydrologics in support of project objectives.
2. Existing stations are part of the previously established PVWMA monitoring network.

- *Urban runoff measurements.* This report has noted repeatedly the importance of appropriately characterizing runoff response from the urbanized portions of the watershed. This is particularly important given that measured runoff at two of the gaging locations (WSHO and SSES) were well below anticipated levels and inconsistent with the downstream response in water surface elevations. Table 9-1 includes WSHO as an ongoing station. However, it will be very important to ascertain the runoff behavior from Sub-

watershed E, which should be the primary source of runoff to the Struve Slough portion of the system. Data collected at the SSES station confirms that it is not fully characterizing conditions in that sub-watershed and that supplemental work should be completed to clarify the timing and quantity of runoff from all the urbanized sub-watersheds.

- *Rain gages.* Refinement and improved calibration of the modeling tools would be greatly facilitated by more comprehensive availability of hourly rainfall data, particularly from locations within the watershed. At a minimum, we would recommend that stakeholders coordinate and/or support calibration and maintenance of CIMIS Station 209 and of hourly rainfall recording at either the Watsonville Treatment Works (WTW) site or the Watsonville Municipal Airport, noting that multiple and even redundant gages are valuable.

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10 REFERENCES

Adelman, K., and Adelman, G., 2012, California Coastal Records Project, Image 8712076 acquired June, 1987, http:www.californiacoastline.org (accessed August 16, 2013).

Clark, R., and O'Connor, K., California bar-built estuaries, Utility of a newly developed rapid assessment tool for describing the condition and ecological services of California lagoons, Moss Landing Marine Labs presentation prepared for the Central Coast Wetlands Group, 32 slides.

Klingaman, N.P., Kevia, D.F., and Frost, E.E., 2007, A comparison of three canopy interception models for a leafless mixed deciduous forest stand in the Eastern United States, Journal of Hydrometeorology, vol. 8, pp. 825-836, doi:10.1175/JHM564.1.

PRISM Climate Group, Oregon State University, http://prism.oregonstate.edu, created 31 January 2012.

Questa Engineering Corporation, 1995, Water resources management plan for Watsonville Slough System, Santa Cruz County, prepared for Association of Monterey Bay Area Governments, project #94028, 184 p.

Smith, J., 1993, Pajaro River lagoon management plan, Technical Appendix A, Aquatic habitat and fisheries, 17 p.

Straub, T.D., Melching, C.S., and Kocher, K.E., 2000, Equations of estimating Clark unit hydrograph parameters for small rural watersheds in Illinios: U. S. Geological Survey Water-Resource Investigations Report 00-4184.

Swanson Hydrology and Geomorphology and others, 2003, Watsonville Sloughs watershed resource conservations and enhancement plan, prepared for County of Santa Cruz planning department, 141 p.

U.S. Army Corps of Engineers (USACE), Hydrologic Engineering Center, 2008, HEC-RAS river analysis system hydraulic reference manual, v 4.0, 411p.

USACE, Hydrologic Engineering Center, 2010, HEC-RAS river analysis system user's manual, v 4.1, 790 p.

Vrugt, J.A., Dekker, S.C., and W. Bouten, 2003, Identification of rainfall interception model parameters from measurement of throughfall and forest canopy storage, Water Resources Research, vol. 39, no. 9, 1251, doi:10.1029/2003WR002013.

Modeling Conference, April 2-6, Reno, Nevada, Advisory Committee on Water Information, Subcommittee on Hydrology, Washington, DC, CD-ROM.

Xiao, Q., McPherson, E.G., Simpson, J.R., and Ustin, S.L., 1998, Rainfall interception by Sacramento's urban forest, Journal of Arboriculture, vol. 24, no. 4, 10 p.

Xiao, Q., McPherson, E.G., Ustin, S.L., Grismer, M.E., Simpson, J.R., 2000, Winter rainfall interception by two mature open-grown trees in Davis, California, Hydrological Processes, vol. 14, pp. 763-784.

Xu, C., and Steinman, A., 2009, Event and continuous hydrologic modeling with HEC-HMS, Journal of Irrigation and Drainage Engineering, vol. 135, issue 1, pp. 119-124.

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