Hydrodynamic Modeling and GIS Analysis of the Habitat Potential and Flood Control Benefits of the Restoration of a Leveed Delta Island

by

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TABLE OF CONTENTS

Abstract **1**

Introduction **4**

Study Area **6**

Model Description **15**

Modeling Approach 15

MIKE 11 Description 17

MIKE 11 GIS Description 19

Data Requirements 20

Model Limitations 20

Model Construction **22**

Introduction to the Cosumnes-Mokelumne-North Delta Model 22

Model Network Alignment 22

Boundary Conditions 24

Estimated Boundary Conditions 25

Geometry 29

Datums 32

Manning Coefficient 33

Data Uncertainties 35

Flood Frequency of Time Periods of Simulation 35

Model Calibration and Validation **38**

Calibration and Validation Methodology 38

Sensitivity Analysis 41

Comparison to Observed Data 47

Scenario Evaluation 60

Scenario Descriptions 60

Habitat Methodology 67

Habitat Zone Quantification and Spatial Flood Depth Values 67

Tidal Characteristics 68

Habitat Zone Delineation 70

Effect of Tidal Conditions on Areal Habitat Extent 70

Scenario Simulations 73

Discussion and Conclusions 92

References 95

Appendix A – Model Input Details 100

Appendix B - Sensitivity Analysis Results 108

Appendix C – Scenario Simulation Results 114

ABSTRACT

Over 50% of the wetland ecosystems throughout the conterminous United States have been severely degraded or destroyed for the purpose of agricultural or urban land uses (Dahl and Allord 1996). A realization of their irreplaceable ecosystem functions and value has lead to nation wide efforts to rejuvenate, enhance and restore many of these damaged ecosystems. One of these damaged ecosystems, the Sacramento-San Joaquin Delta of California, previously one of the richest ecosystems in the Americas, currently exists in a highly altered state due to the reclamation of tidal wetland areas for agricultural purposes (Atwater 1980). It has been estimated that over 90% of the tidal freshwater wetlands of the Delta region have been leveed, removing them from tidal and floodwater inundation (Simenstad et al. 2000). In an effort to restore ecosystem health, a program comprised of over 20 state and federal agencies, the California-Federal Bay/Delta Program (CALFED) has proposed the restoration of tidal freshwater marsh ecosystems by reconnecting regions currently managed for agricultural purposes to their adjacent rivers and sloughs (CALFED 2000). One element of such restoration efforts that has not been adequately addressed is the impact that restoration efforts are likely to impose on both regional and local flood stages.

This study tests the hypothesis that habitat restoration and flood mitigation can be compatible. A one-dimensional unsteady hydraulic model is used to evaluate the flood stage impacts of seven management scenarios for the McCormack-Williamson Tract, located in the northern Sacramento-San Joaquin Delta. The seven management scenarios

studied are based upon conceptual input from members of The Nature Conservancy (TNC), the California Department of Water Resources (CA-DWR), CALFED and the Cosumnes Research Group (CRG), which bracket the range of potential management possibilities ranging from solely flood control to the restoration of tidal marsh habitat. Scenario features include weirs, levee breaches, levee removal, and internal levee construction in a variety of configurations. In addition to quantifying flood impacts, the model results are used to quantify the potential areal extent of subtidal, intertidal, and supratidal habitat zones within the project area and volume of tidal exchange for each of the scenarios.

The results of the modeling effort indicate that the restoration of tidal marsh habitat within the McCormack-Williamson Tract would have a minimal impact upon flood stage during a range of flooding conditions, including rare, large flooding events. In addition, the results suggest that the configuration of levee breaches can be optimized for the creation of intertidal habitat within the tract.

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The opportunity to apply the MIKE 11 hydrodynamic modeling package in this research was made possible by the Danish Hydraulic Institute and their provisions for the application of their modeling tools in academic research. In addition their international staff provided invaluable support and assistance throughout the course of this project.

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INTRODUCTION

Over 50% of the wetland ecosystems throughout the conterminous United States have been severely degraded or destroyed for the purpose of agricultural or urban land uses (Dahl and Allord 1996). A realization of their irreplaceable ecosystem functions and value has lead to nation wide efforts to rejuvenate, enhance and restore many of these damaged ecosystems. One of these damaged ecosystems, the Sacramento-San Joaquin Delta of California, previously one of the richest ecosystems in the Americas, currently exists in a highly altered state due to the reclamation of tidal wetland areas for agricultural purposes (Atwater 1980). It has been estimated that over 90% of the tidal freshwater wetlands of the Delta region have been leveed, removing them from tidal and floodwater inundation (Simenstad et al. 2000). In an effort to restore ecosystem health, a program comprised of over 20 state and federal agencies, the California-Federal Bay/Delta Program (CALFED) has proposed the restoration of tidal freshwater marsh ecosystems by reconnecting regions currently managed for agricultural purposes to their adjacent rivers and sloughs (CALFED 2000). One element of such restoration efforts that has not been adequately addressed is the impact that restoration efforts are likely to impose on both regional and local flood stages.

This study tests the hypothesis that habitat restoration and flood mitigation are not mutually exclusive. A one-dimensional unsteady hydraulic model is used to evaluate the flood stage impacts of seven management scenarios for the McCormack-Williamson Tract, located in the northern Sacramento-San Joaquin Delta. The seven management scenarios studied are based upon conceptual input from members of The Nature Conservancy (TNC), the California Department of Water Resources (CA-DWR), CALFED and the Cosumnes Research Group (CRG), which bracket the range of potential management possibilities ranging from solely flood control to the restoration of tidal marsh habitat. Scenario features include weirs, levee breaches, levee removal, and internal levee construction in a variety of configurations. In addition to quantifying flood impacts, the model results are used to quantify the potential areal extent of subtidal, intertidal, and supratidal habitat zones within the project area and the volume of tidal exchange for each of the scenarios.

STUDY AREA

The McCormack-Williamson Tract is a 652-ha (1,612-a) parcel located in the northern portion of the Sacramento-San Joaquin Delta of California (Figure 1), which historically supported tidal freshwater marsh and riverine floodplain habitats (USGS 1911; Brown and Pasternack In Prep.) Sediment deposited from the overbank flow of floodwaters in this region created natural levees at least one meter high (Atwater 1980). In 1919, the natural levees around the area currently known as McCormack-Williamson Tract were raised in an effort to reclaim the land for agricultural uses removing it from frequent tidal and floodwater inundation (State of California Reclamation Board 1941). In the 80 years which follow, these levees were raised, improved, accidentally breached and repaired a number of times. Around its perimeter, the McCormack-Williamson Tract is bordered by the Mokelumne River to the southeast, Snodgrass Slough to the west and an artificial dredging canal named Lost Slough to the north (Figure 2). The McCormack-Williamson Tract is located roughly 2.4 km downstream of the confluence of the Cosumnes and Mokelumne Rivers, and 1.3 km east of the Sacramento River, which is at times hydraulically connected to Snodgrass Slough via the Delta Cross Channel.

Situated in the northern region of the Sacramento-San Joaquin Delta, the McCormack-Williamson Tract is located in the midst of a very complex system. Tidal and fluvial forcings drive water through a heavily manipulated system of channels confined by levees, and subject to backwater conditions caused by road crossings and railroad embankments. The McCormack-Williamson Tract lies near the upstream extent of tidal fluctuation, experiencing a semi-diurnal tidal pattern with an average tidal range during

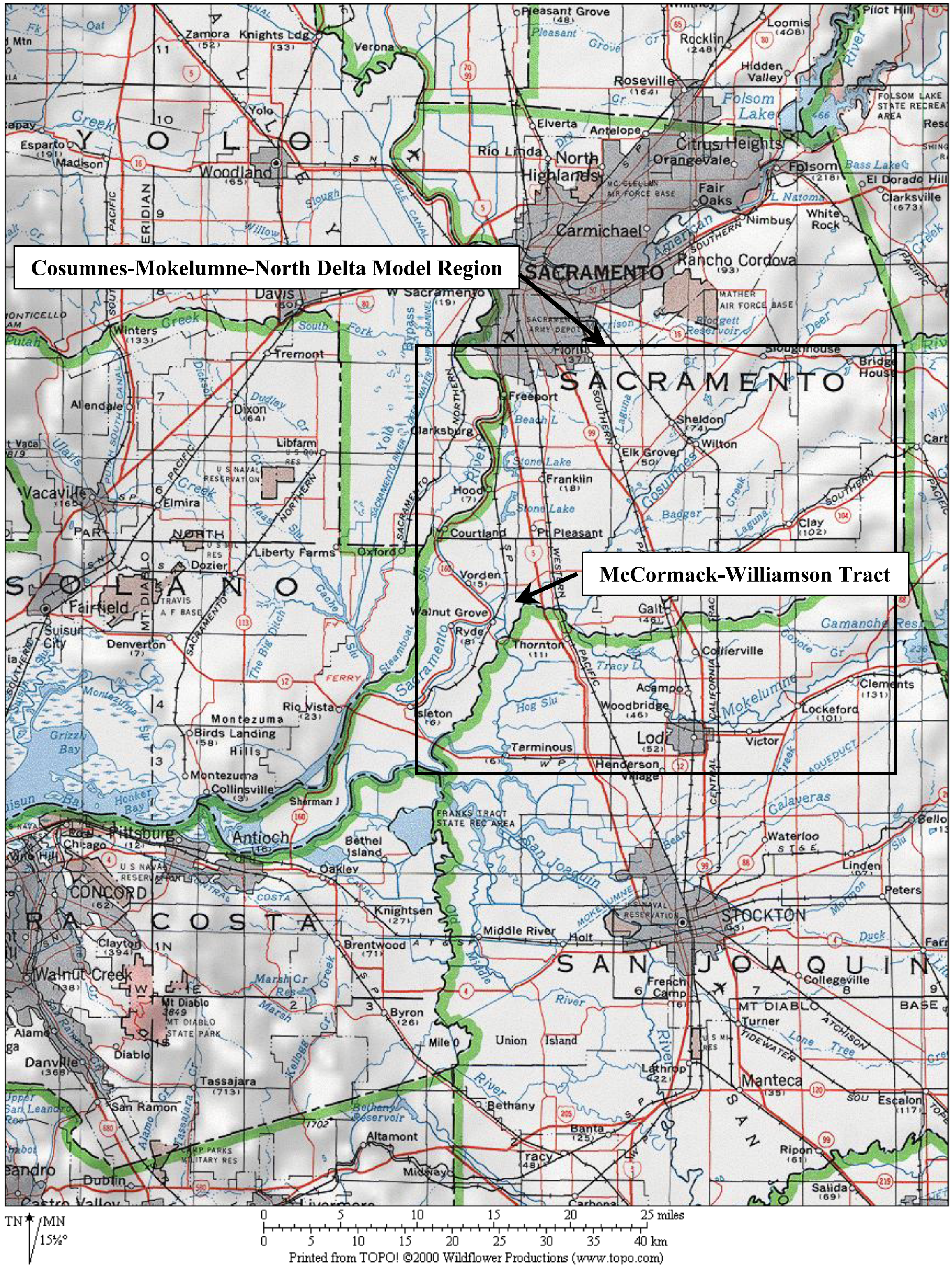


Figure 1. Study area of Cosumnes-Mokelumne-North Delta modeling effort.

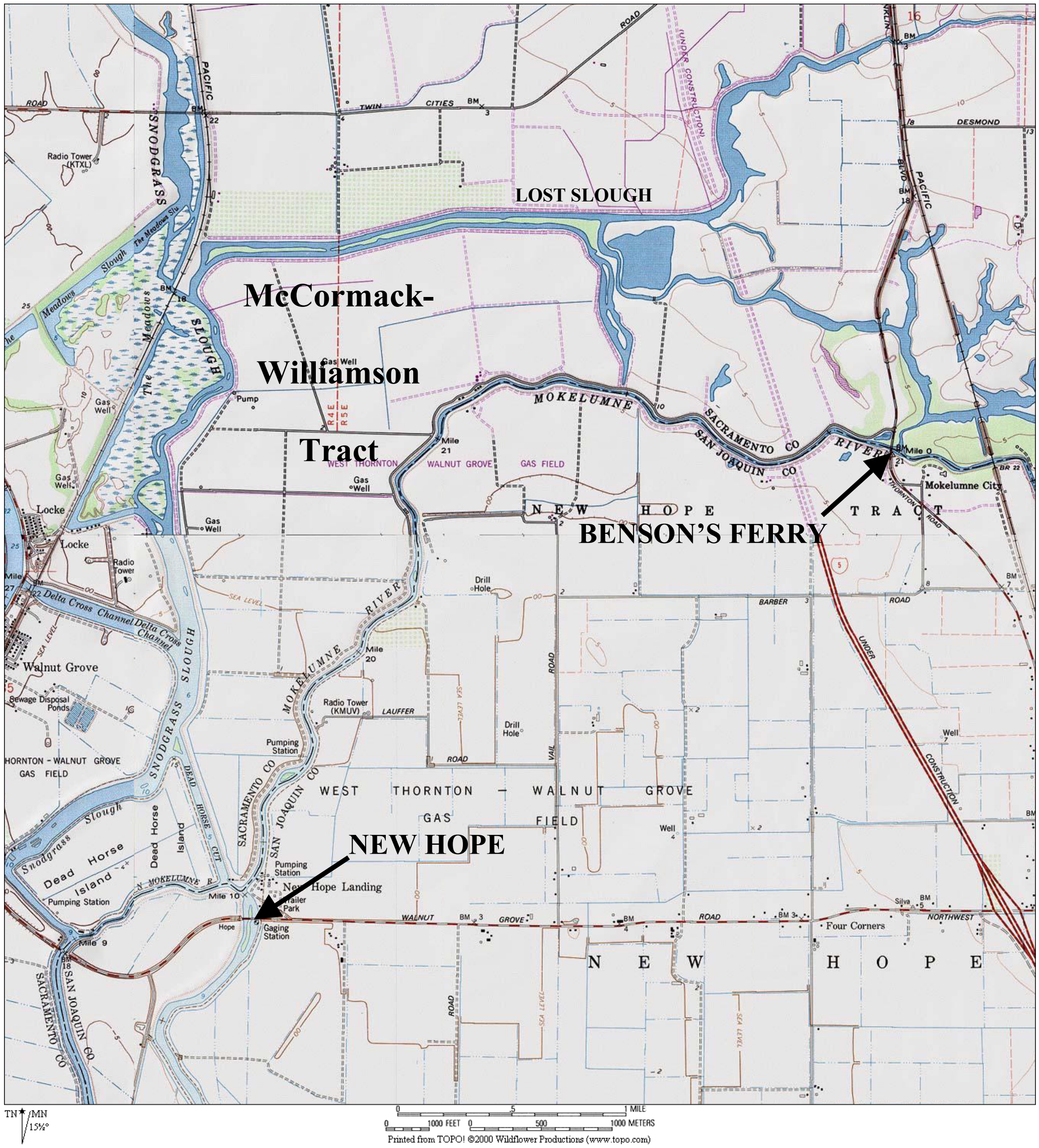


Figure 2. McCormack-Williamson Tract location map, showing the locations of New Hope and Benson's Ferry. The McCormack-Williamson Tract is bordered by Lost Slough to the north, the Mokelumne River to the east, and Snodgrass Slough to the west. The Delta Cross Channel (DCC) hydraulically connects the Sacramento River to Snodgrasss Slough when DCC gates are open.

low river flow conditions of ~1 m (NOAA 2002). Tidal oscillation dominates the hydraulics of the study area at the semi-diurnal to monthly time scales. During the winter and spring, storm and snowmelt events influence the hydraulics of the regional system. The operations of water resource facilities (reservoir releases and Delta Cross Channel gates) also influence regional water levels and system hydraulics.

The McCormack-Williamson Tract lies approximately 2.4 km downstream of the confluence of the Mokelumne and Cosumnes Rivers at Benson's Ferry (Figure 2). The Cosumnes River is one of the last unregulated rivers in California, maintaining its natural flood regime, sending flood pulses downstream in response to major precipitation events. The Mokelumne River is regulated by several dams managed by East Bay Municipal Utility District (EBMUD) and Pacific Gas and Electric Company, which impound flood flows for storage, power generation and use as municipal water supply. In addition, the Morrison Creek group a tributary to Snodgrass Slough, and Dry Creek contribute flow to the North Delta region.

Due to the unregulated nature of the Cosumnes River and its tributaries, and extensive levee construction, the North Delta region has experienced significant flooding on several occasions. During two recent instances, the large flood events of 1986 and 1997, the eastern levee of the McCormack-Williamson Tract was overtopped resulting in uncontrolled levee breaches. On both occasions, floodwaters inundated the McCormack-Williamson Tract and flowed to the southern portion of the tract. This pulse of floodwaters caused an inside out failure of the levee, returning water into the already swollen North and South Forks of the Mokelumne River, further compromising downstream levees (U. S. Army Corps of Engineers 1988). Due to its geographic location, and flooding history, any manipulation to the manner in which water moves around and through the McCormack-Williamson Tract will likely impact flood flows.

A majority of the Delta region currently lies below mean sea level, due to subsidence associated with oxidation of peat soils (Rojstaczer et al. 1991). Located at the upslope fringe of the Delta, the current topography of the interior of the McCormack-Williamson Tract ranges from –0.9 m to 1.5 m (-3 ft to +5 ft) in elevation NGVD as shown in Figure 3 (California Department of Water Resources 1992, California Department of Water Resources 2002). The elevation range of the McCormack-Williamson Tract presents the opportunity for the creation of a habitat mosaic consisting of tidal freshwater marsh, seasonally inundated floodplain, and shallow open water habitat types without the need for material import and land surface grading. The areal extent of each of these habitat types will initially be determined from the existing topography, and the degree of connectivity of the McCormack-Williamson Tract to the adjacent river channels. The size, shape, elevation, and location of the levee breaches will determine the degree of connectivity to the surrounding network. The areal extent of each of these habitat types will undoubtedly change as the tract evolves biologically and geomorphically to reconnection to the adjacent tidal and storm influenced fluvial system.

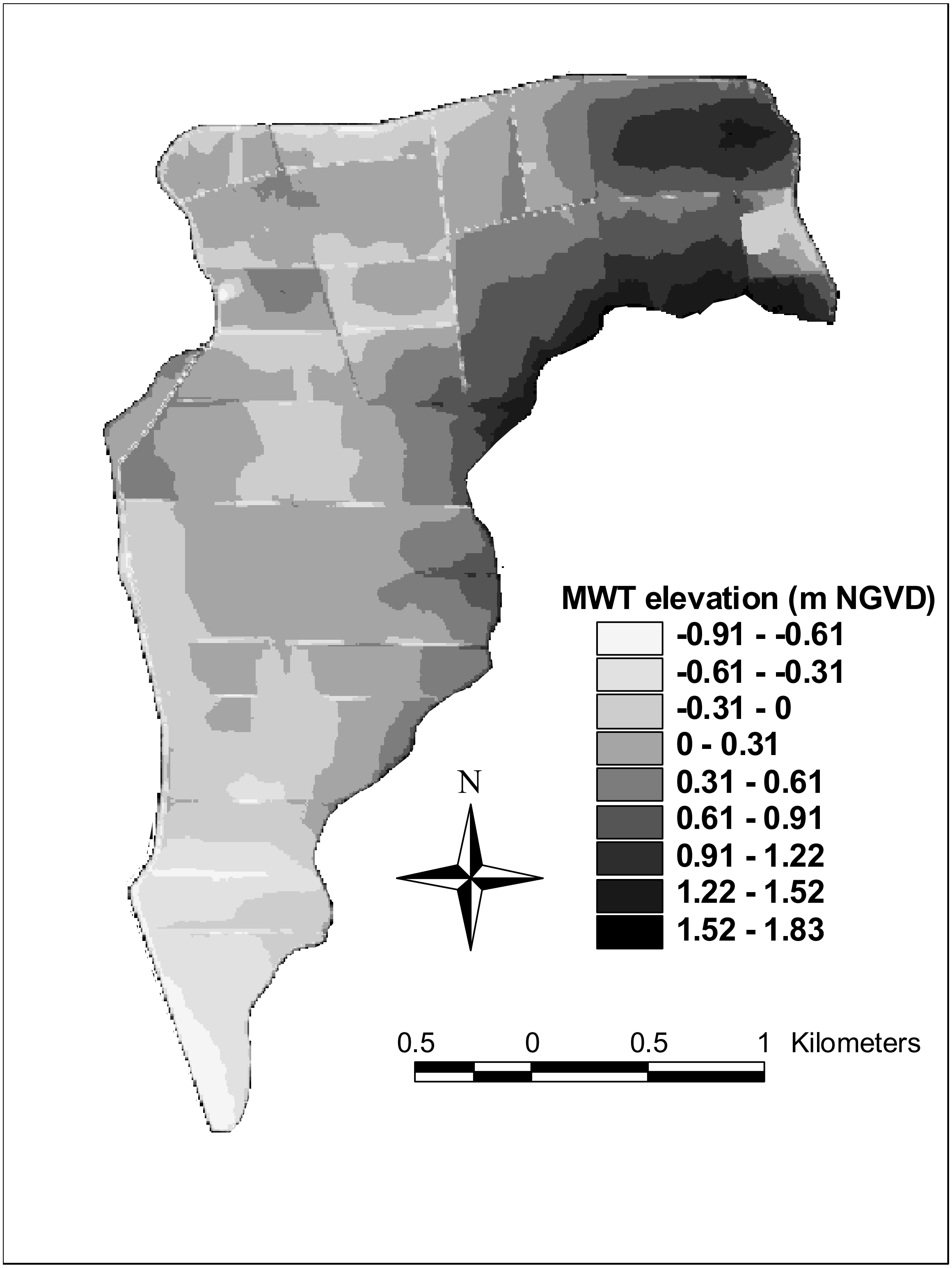


Figure 3. Current topography of the McCormack-Williamson Tract. Non-levee regions range from –0.91 m (light color) in the southern end to 1.5 m (dark color) in the northeastern area. Topography data based upon CA-DWR North Delta Study (1992) and MWT survey (2002).

BACKGROUND

Hydrology is the primary forcing function in wetland ecosystems (Mitsch and Gosselink 2000). In wetlands, the dynamics of inundation have been shown to dictate the interdependence between hydrological and biological processes (Junk et al. 1989), and play a vital role in composition and distribution of plants, aquatic animals and invertebrates (Franz and Bazzaz 1977, Benke et al. 2000, Pasternack et al. 2000). Other processes related to inundation include sediment transport, methane emission, soil nutrient dynamics, and water quality (Gee et al. 1990, Pasternack and Brush 1998, Benke et al. 2000, Knight and Pasternack 2000). One example of the affect of inundation pattern on ecology is the development of a toposequence of wetland vegetation communities. In a toposequence, different plants are located in different zones depending partly on abiotic factors including the frequency and duration of flooding, manifested by topography, and the relative elevation of different areas to the local water level fluctuations. This has been demonstrated in riparian forests of the Sacramento Valley (Conard et al. 1977), as well as in tidal freshwater marshes (Atwater 1980, Pasternack et al. 2000). Atwater (1980) documented this pattern for Delta Meadows, a remnant intertidal wetland directly adjacent to the McCormack-Williamson Tract. A detailed understanding of the dynamics of inundation is vital to the analysis of these ecosystem processes and functions, especially in the context of the restoration of a tidal marsh.

The environmental, ecological, and water resource aspects of the lower Cosumnes River Basin, North Sacramento-San Joaquin Delta, and the surrounding regions of the Morrison Creek and Mokelumne River watersheds have been the focus of previous study. Such studies have detailed the hydrology (U.S. Army Corps of Engineers 1936, U.S. Army Corps of Engineers 1965, U.S. Department of the Interior 1979, Bertoldi et al. 1991; Environmental Science Associates Inc. 1991, U.S. Army Corps of Engineers 1991, U.S. Army Corps of Engineers 1996, U.S. Army Corps of Engineers 1998, U.S. Army Corps of Engineers 1999) and hydraulics (Guay et al. 1998; Simspon 1972, U.S. Army Corps of Engineers 1988, Wang et al. 2000) of various areas of the region. The frequency and magnitude of flooding within the study area is of considerable interest to many involved parties and, as a result, several studies have focused on the hydraulic modeling of floods in the Cosumnes-Mokelumne-North Delta region.

Many studies have been conducted within the hydraulic domain of the study area and present a foundation for the modeling work conducted in this study. The USGS investigated the channel capacity of the Mokelumne River between Camanche Dam and the confluence with the Cosumnes River (Simpson 1972) and modeled the inundation of storms of various magnitudes on the upper main stem of the Cosumnes River (Guay et al. 1998). The California Department of Water Resources (CA-DWR) developed a DWOPER model in its North Delta Program study, in addition to modeling part of the region with its DSIM2 program. The U. S. Army Corp of Engineers has studied the region extensively in the context of flood control, modifying the DWOPER hydraulic model used by CA-DWR to evaluate the impact of modifications to the hydraulic system (U.S. Army Corps of Engineers 1988) in addition to the South Sacramento Streams Investigation Study. This same model was again modified and utilized by a consultant in the Sacramento County Beach Stone Lakes Flood Control Study, and later in a report titled "North Delta Flood Control Scenarios" to assess the hydraulic effect of using the McCormack-Williamson Tract and other local tracts as flood storage areas with a synthetic hydrograph modeled after the February 1986 event (Ensign and Buckley 1998).

Ecologically focused studies have been conducted on the lower Cosumnes floodplain (Swanson and Hart 1994), the river reaches between Michigan Bar and the Delta (Hart and Engilis 1995), and along the entire mainstem of the Cosumnes (Vick et al. 1997). Blake (2001) drew data from many of the above-mentioned studies and compiled a one-dimensional unsteady hydraulic model based in MIKE 11, for the purpose of investigating floodplain dynamics on the Cosumnes River Preserve. This model provides the foundation for the work conducted in this study.

MODEL DESCRIPTION

Modeling Approach

Hydraulic engineers and scientists have used various approaches to analyze riverine environments. With advances in computer technology, a majority of the analysis of rivers and their floodplains has been conducted with numerical models. These efforts have utilized a wide variety of techniques and methods, including the use of one-dimensional finite difference hydraulic models (Shumuk et al. 2000, Snead 2000, Blake 2001, Mishra et al. 2001), two-dimensional finite difference and finite element hydraulic models (Gee et al. 1990, Bates et al. 1992), and two-dimensional finite element hydraulic models coupled with hydrologic models (Bates et al. 1996, Stewart et al. 1999). Hydraulic models in their variety of forms have been employed to assess the performance of canal systems (Mishra et al. 2001), to quantify flood magnitude and floodplain inundation (Gee et al. 1990, Bates et al. 1992, Bates et al. 1996, Stewart et al. 1999, Shumuk et al. 2000), as well as to quantify the effect of levee breaches on flood mitigation (Kozak 1975, Sanders and Katopodes 1999a, Sanders and Katopodes 1999b, Jaffe and Sanders 2001). Hydraulic models have also been utilized to assess possible changes to riparian vegetation based upon simulated changes in reservoir release patterns (Auble et al. 1994). While many methods have been utilized for a variety of purposes, Bates et al. (2000) suggest that at present we do not know what processes must be included to facilitate the accurate prediction of inundation, the appropriate tool to use is generally determined by 1) the purpose of the effort, 2) the amount of available data and

3) the degree of accuracy required. Of these factors, the amount of available data will dictate the approach taken in this study as discussed below.

Based upon a review of the available literature with regard to methods utilized in modeling the hydraulics of riverine systems, and previous modeling efforts within the study area, a one-dimensional unsteady hydraulic model based in MIKE 11 was chosen for this investigation. The availability of hydraulic gage data dictates the domain of the hydraulic model. Given the limited amount of available topographic and hydraulic gage data, combined with the complexity of the study area, a dynamic one-dimensional hydraulic model is best suited for this study. While a two or three-dimensional hydraulic model can provide more information regarding velocities, velocity spatial gradients, and inundation gradients on the McCormack-Williamson Tract, such models would need boundary condition input from a one-dimensional hydraulic model as such information is not available for the local region. Two-dimensional hydraulic models have been proven to more realistically model the dynamics of inundation, however they require more topographic, boundary condition and internal observation data. All three of these elements are lacking in this particular study region. Therefore, with the data that are currently available, and the scope of work proposed, a one-dimensional hydraulic model is appropriate. The integration of the hydraulic model results with Geographic Information Systems (GIS) with MIKE 11 GIS facilitates the analysis of inundation statistics providing for the evaluation of habitat potential.

MIKE 11 Description

To investigate the local and regional impact of various management scenarios on the McCormack-Williamson Tract a hydraulic model, MIKE 11, was utilized. The MIKE 11 hydraulic model, developed in 1987 by the Danish Hydraulic Institute, is a dynamic, one-dimensional modeling package, which simulates the water level and flow throughout a river system (DHI 2000). In addition to simulating hydrodynamics, the commercially available MIKE 11 modeling package also includes modules for advection-dispersion, sediment transport, water quality, rainfall-runoff, flood forecasting and GIS floodplain mapping and analysis. The GIS floodplain mapping and analysis module, MIKE 11 GIS, is used in this study to generate and analyze inundation statistics, and is described in more detail below.

When applied with the fully dynamic wave approximation, as in this study, MIKE 11 solves the vertically integrated equations of conservation of volume and momentum, known as the St. Venant equations. The St. Venant equations are derived from the standard forms of the equations of conservation of mass and conservation of momentum based upon the following four assumptions:

1) The water is incompressible and homogeneous; therefore there is negligible variation in density.
2) The bottom (bed) slope is small, therefore the cosine of the slope angle can be assumed to equal 1.

3) The water surface elevation wavelengths are large compared to the water depth, which ensures that the flow everywhere can be assumed to move in a direction parallel to the bottom.

4) The flow is subcritical. Supercritical flow conditions are solved with a reduced momentum equation, which neglects the nonlinear terms.

With these assumptions applied, the standard forms of the equations of conservation of mass and momentum can be transformed into equations 1.1 and 1.2 (below). These transformations are made with Manning's formulation of hydraulic resistance in SI units, and the incorporation of lateral inflows in the continuity equation.

Continuity Equation:

$$\frac{\partial Q}{\partial x}+\frac{\partial A}{\partial t}=q \qquad (1.1)$$

Momentum Equation:

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(\alpha\frac{Q^2}{A}\right)}{\partial x}+gA\frac{\partial h}{\partial x}+gA\frac{\partial h}{\partial x}+\frac{n^2 gQ|Q|}{AR^{4/3}}=0 \qquad (1.2)$$

where:

Q : discharge [m^3/s]
A : cross section area [m^2]
x : downstream direction [m]
t : time [s]
q : lateral inflow [m^2/s]
α : vertical velocity distribution coefficient
g : gravitational acceleration [m/s^2]
h : stage above datum [m]
n : Manning coefficient [m/s$^{1/3}$]
R : hydraulic radius [m]

Within the MIKE 11 program code the above equations are transformed into a set of implicit finite difference equations, which are solved for each point in the grid (at each node). The above formulations of the St. Venant equations are simplified for application in a rectangular channel. Natural river cross sections are rarely rectangular, so the MIKE 11 model integrates the equations piecewise in the lateral direction (DHI 2000).

MIKE 11 GIS Description

The MIKE 11 GIS software package integrates MIKE 11 hydrodynamic model output with the spatial analysis capabilities of the ArcView GIS software developed by Environmental Science Resource Institute. MIKE 11 GIS is a fully integrated extension of ArcView GIS, which among other things, projects the water levels calculated within MIKE 11 as an interpolated water surface over a digital elevation model (DEM). The difference between the water level and the ground elevation is determined throughout the domain and visually presented based upon user defined flood depth increments. Several products are available from the MIKE 11 GIS software package, but the main flood inundation outputs include depth, duration, and comparison maps. This software is designed to assess flood extent as a water resource and flood management tool, however it is also able to provide insight with regards to the regional ecology driven by the disturbance of flooding. In this study, depth inundation maps, and associated inundation statistics generated by MIKE 11 GIS are employed to evaluate the habitat restoration potential of each of the scenarios. This provides a powerful tool when evaluating each scenario based upon defined management objectives.

Data Requirements

In order to operate the MIKE 11 model, several data inputs are required, including the river network alignment, channel and floodplain cross sections, boundary data, and roughness coefficients. The primary inputs to the MIKE GIS program include the results from a MIKE 11 hydrodynamic simulation, the user defined connectivity of river channels to adjacent floodplains, and a digital elevation model (DEM) of the area of interest. The acquisition and development of this data for use in this study is discussed in more detail below.

Model Limitations

It is important to understand the simplifications and assumptions which are made when applying a model and evaluating the model's results. First, the MIKE 11 hydrodynamic model is hydraulic not hydrologic. Important hydrologic elements of river and floodplain systems, which are ignored, include the surface water's interaction with groundwater (infiltration, upwelling, bank storage), with the atmosphere (evaporation, and direct precipitation input), and with vegetation (evapotranspiration). Water movement is simulated purely based upon water forces, and assumed to only act in the longitudinal direction. Thus an eddy or a rapid formed by a constriction in the river channel or at a levee breach is not recognized and therefore the effects of which are not simulated.

The distributed floodplain mapping results obtained through MIKE 11 GIS are directly dependent upon the accuracy of many elements, including the results from a MIKE 11 hydrodynamic simulation, the user defined connectivity of river channels to adjacent floodplains, and a digital elevation model (DEM) of the area of interest. The accuracy of the hydraulic model is discussed later, and while gage data provide point comparisons of the model's output at Benson's Ferry and New Hope Landing, few other locations exist for comparison of the model simulation results with observed data. In addition, it is important to acknowledge that the coupling of a one-dimensional hydraulic model with GIS merely projects the one-dimensional model results in two dimensions; it does not increase the complexity or dimensionality of the results.

MODEL CONSTRUCTION

Introduction to the Cosumnes-Mokelumne-North Delta Model

This effort has utilized the existing MIKE 11 hydrodynamic model created by Steven Blake in his graduate work under Dr. S. Geoffrey Schladow (Blake 2001), with spatial and temporal modifications. The river alignment, cross section geometry and boundary conditions compiled by Blake were verified and modified as needed for this study. The previous effort focused on modeling the flood periods of 1996, 1998, 1999, and 2000. These flood years represent a range of flows of varying magnitude, including flood pulses with ~2.5 year (1996 & 2000), ~5 year (1999), and ~10 year (1998) return period frequencies based upon Cosumnes River discharge measured at Michigan Bar (Guay et al. 1998). In addition to these years, the flood period of 1986 (~25 year return period) (Guay et al. 1998), which caused considerable flooding in the North Delta region has been simulated as part of the present work. This required the acquisition of the available gage data for 1986, and an expansion of the model network to encompass the regions inundated by floodwaters during the 1986 flood.

Model Network Alignment

The alignment of each river channel, floodplain area and slough in the model region provides the skeleton of the hydraulic system. In MIKE 11 this is referred to as the model network, and provides a digital representation of the planform alignment of the system. Each river reach or branch is assigned a name and length in addition to its connectivity with the other branches in the model domain. In addition to the planform alignment of the hydraulic system and the systems connectivity, hydraulic structures are also defined in the model network file. Examples of such structures include weirs, culverts, bridges and dam breaks or levee failures. A graphical and tabular description of the Cosumnes-Mokelumne-North Delta Model network is provided in Appendix A.

In constructing the Cosumnes-Mokelumne-North Delta Model used in this study, the EPA river reach file (1:100,000 scale) was imported into MIKE 11 as a geo-referenced background graphic. Nodes (points) were then digitally placed along each branch (river, creek, or slough) at an adequate spacing to capture the sinuosity of each branch. This base river alignment was modified as necessary to reflect the current status of the hydraulic system, for example the connectivity of Dry Creek. Most maps show Dry Creek as a tributary to the Mokelumne River, however in the current condition, Dry Creek flows (except in extreme floods) are conveyed to the Cosumnes River via Grizzly and Bear Sloughs. When observed from the Mokelumne River, the historic Dry Creek confluence is barely discernable (Jim Smith personal communication).

In one-dimensional hydraulic modeling, various methods are available for incorporating floodplain areas in the model domain. In this study, floodplains are identified as separate reaches in the model network, placed adjacent to the channel. The floodplain is then connected to the river reach with “link channels,” which are simplified branches in which flow through the branch is calculated as flow over a broad crested weir, with user defined weir geometry. All levee breaches in this study, in addition to floodplain connections have been simulated with this approach, providing a pseudo two-dimensional description of floodplain flow.

While the EPA river reach file is sufficient in describing the alignment and connectivity of major river channels and sloughs it provides little information about floodplain regions and their connectivity to main channels. To gain a better understanding of the off channel flow mechanisms local individuals were consulted. Keith Whitener of TNC provided insight into the manner in which floodwaters proceed through the Cosumnes River Preserve area. Walt Hoppe, local resident of Point Pleasant, provided invaluable historical data of the 1986 event, including levee breach locations, flood distribution, and flood flow paths.

Boundary Conditions

In hydraulic modeling, boundary conditions are required to provide the model input at the edges of the domain. Boundary conditions are typically hydraulic monitoring gages where river stage data are recorded at some time interval. In some locations rating curves have been developed based upon field measurements of velocity and channel geometry, and allow for the conversion of stage data into flow data. In other locations ultrasonic velocimeters have been utilized to monitor flow without the development of rating curve.

Data exist from a number of gages in the study area, and have been provided by a number of agencies including United States Geological Survey (USGS), California Department of Water Resources (CA-DWR), East Bay Municipal Utilities District (EBMUD), and

Sacramento County Flood Control Agency (SAFCA). The availability of hydraulic gage data dictate the domain of the hydraulic model, as the model extends upstream to hydraulic gages located at Michigan Bar on the Cosumnes River, Wilton Road on Deer Creek, above Galt on Dry Creek, Woodbridge on the Mokelumne River, and to Lambert Road at the Stone Lakes Outfall (Figure 4). To the west, the model domain includes a short portion (~8 km) of the Sacramento River extending from above the Delta Cross Channel to below the divergence of Georgiana Slough. The inclusion of the Sacramento River was necessary to act as an upstream boundary for flows through Georgiana Slough, as well as to allow for incorporation of the Delta Cross Channel operations. A gage located below the confluence of Georgiana Slough on the Mokelumne River delineates the downstream end of the model domain. A table detailing each gage type, location and operating agency is provided in Appendix A.

In addition to utilizing gage data as boundary conditions to drive the simulated hydraulic system, gage data from locations within the model domain are used to calibrate and validate the model results. Model output is compared to the observed data to evaluate the quality of the model. Two locations, Benson's Ferry and New Hope Landing have been used primarily in this study, due to their close proximity to the McCormack-Williamson Tract (Figures 2 and 4).

Estimated Boundary Conditions

The data record from each of the hydraulic gages utilized is often not continuous, or of sufficient length. To allow modeling to proceed, estimation of the absent boundary

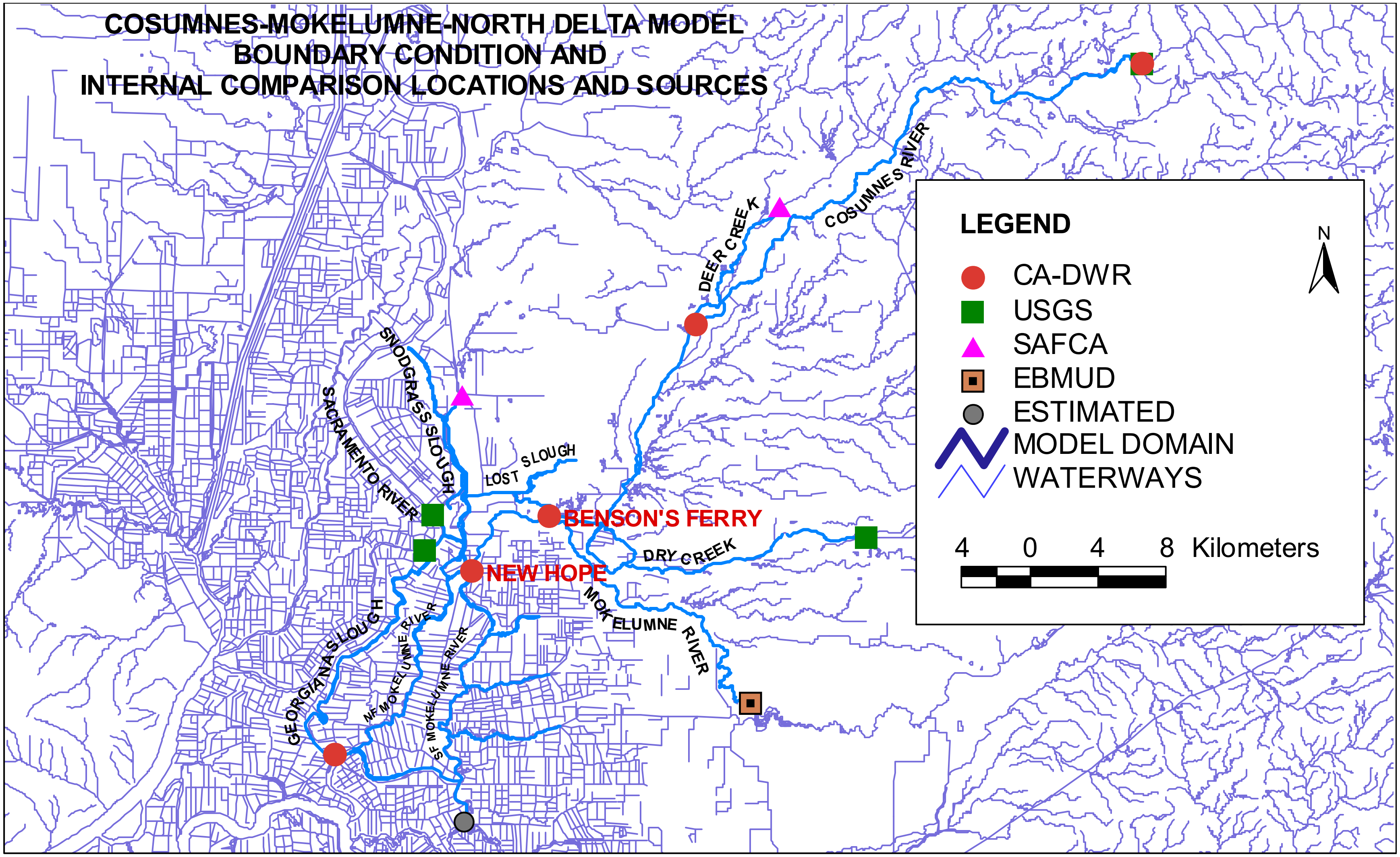


Figure 4. Boundary condition and internal comparison point locations and data sources used in the Cosumnes-Mokelumne-North Delta Model.

condition data was necessary. Boundary condition estimation was required for Deer Creek at Wilton Road, Dry Creek above Galt, Stone Lakes Outfall at Lambert Road, and Little Potato Slough below Terminous, for various time periods of the study as shown in Table 1.

The Dry Creek watershed is 917 km^2 and is known to contribute significant flows to the Cosumnes-Mokelumne-North Delta region during storm periods. The present study includes the lower portion of Dry Creek from above the town of Galt downstream to Bear and Grizzly Sloughs. Gage data at the Dry Creek Galt gage is available for limited periods, however, not for the flood periods of recent years. In order to simulate the years of 1998, 1999, and 2000 an estimation of the Dry Creek flow contribution was required. A comparison of daily average discharge values in 1986 suggests that during storm events Dry Creek Galt discharge is roughly 40% of the Cosumnes River discharge at Michigan Bar. Based upon this simple comparison of historic discharge data the Dry Creek at Galt boundary condition has been estimated for all model runs except 1986 to be 40% of the discharge of the Cosumnes River at Michigan Bar. For the 1986 runs, data from Dry Creek were available and were utilized. A limitation to this approach is that it overestimates Dry Creek discharge during low flow conditions, and may underestimate Dry Creek discharge during flood pulses.

Data from the stage gages located at Wilton Road on Deer Creek and Lambert Road at the Stone Lakes Outfall, both operated by SAFCA does not exist for 1986. For the Wilton Road gage, a correlation to an adjacent gaging station for which data were

Table 1. Hydraulic gages used as boundary conditions and internal comparison points in the Cosumnes-Mokelumne-North Delta Model.

Hydraulic Gage Location	Sensor ID	Operating Agency	Data Type/Simulation Year			
			1986	1998	1999	2000
Upstream Boundary						
Cosumnes River at Michigan Bar	RCSM075	USGS	Q & h	Q & h	Q & h	Q & h
Sacramento River upstream of Delta Cross Channel	RSAC128	USGS	NA[2]	Q & h	Q & h	Q & h
Dry Creek upstream of Galt	DRY1	USGS	Q	e	e	e
Mokelumne River at Woodbridge	RMKL070	EBMUD	Q & h	Q & h	Q & h	Q & h
Deer Creek at Wilton Road	DEER2	SAFCA	e	Q & h	Q & h	Q & h
Stone Lakes Outlet at Lambert Road	SGS1	SAFCA	e	h	h	h
Downstream Boundary						
Sacramento River downstream of Georgiana S.	RSAC121	USGS	h[2]	Q & h	Q & h	Q & h
Mokelumne River at Georgiana Slough	RMKL005	CA-DWR	h	h	h	h
Little Potato Slough downstream of Terminous	-	-	e	e	e	e
Internal						
Cosumnes River at McConnell	RCSM025	CA-DWR	h	h	h	h
Mokelumne River at Benson's Ferry	RMKL027	CA-DWR	h	h	h	h
South Fork Mokelumne River at New Hope	RSMKL024	CA-DWR	h	h	h	h

Notes:

1) Q = discharge, h = stage, e = estimated as explained in text.
2) For the 1986 simulation, RSAC121 stage data was used at the upstream end of Georgiana Slough and the Sacramento River reach removed from the model network.

available was not attempted. Instead an average low flow water level elevation of 16.4 m was assumed. This value was chosen by inspection of available data for the period of 1998-2000. No attempt was made to synthesize flood pulse water levels. At the Stone Lakes Outfall at Lambert Road, a control structure prevents water from flowing south to north at this location. For a brief period during the large flood of 1986, flow traveled over Lambert Road north into the Stone Lakes Region (U. S. Army Corps of Engineers 1988). For 1986 model simulations a weir was inserted at Lambert Road, which prevented flow during non-flood conditions, but allowed some water to travel north over Lambert Road during the peak of the flood pulse.

At the lower boundary of the study domain, two channels, the Mokelumne River and Little Potato Slough, convey flow south to the San Joaquin River. River stage gage data are available for the Mokelumne River at the confluence of Georgiana Slough, but not for Little Potato Slough. Available data have been analyzed and show that magnitude differences in river stage are negligible, therefore Little Potato Slough water levels were estimated as the adjacent Mokelumne River stage.

Geometry

Geometric data in the form of cross sections and digital elevation models, from a variety of sources including USGS, CA-DWR, University of California at Davis (UCD), EBMUD, SAFCA, Phillip Williams and Associates (PWA), California Department of Transportation BIRIS system (BIRIS), Sacramento County Public Works Department, San Joaquin County Public Works Department, and the National Oceanic and

Atmospheric Administration (NOAA) are utilized in this effort. These data have been collected in a variety of forms, including DEMs, AutoCAD drawings, binary data sets used in other modeling platforms, field surveys, as-built drawings of bridge plans, and output from an NOAA NOS lidar mission. All data have been location and datum verified, processed and compiled into a cross sectional database. Figure 5 presents the location, source and time collected (where available) of each cross section used in this effort.

Topographic data for large floodplain areas where no formal survey data exists were extracted from the USGS 30-meter DEM. These areas include Glanville Tract, Dead Horse Island, Erhardt Club, New Hope Tract and Tyler Island. In addition, topography data for several other smaller floodplain areas were also extracted from the DEM, including the region bounded by McCormack-Williamson Tract, Lost Slough, Interstate 5, the Mokelumne River, and some floodplain regions of the Cosumnes River. Cross sections were extracted from the 30-meter DEM in the form of a binary data set. This method provides data for regions where little topographic data exists, however, for in channel, near channel, and leveed areas the elevation coordinates are suspect due to the averaging of elevations over a 900 m^2 area. This averaging obscures the true crown elevation of levees, and true depth of channels, so for this reason, data from this source was only used for large reasonably flat areas where large variations in elevation did not exist. It was not trusted in channel, near-channel, and leveed areas.

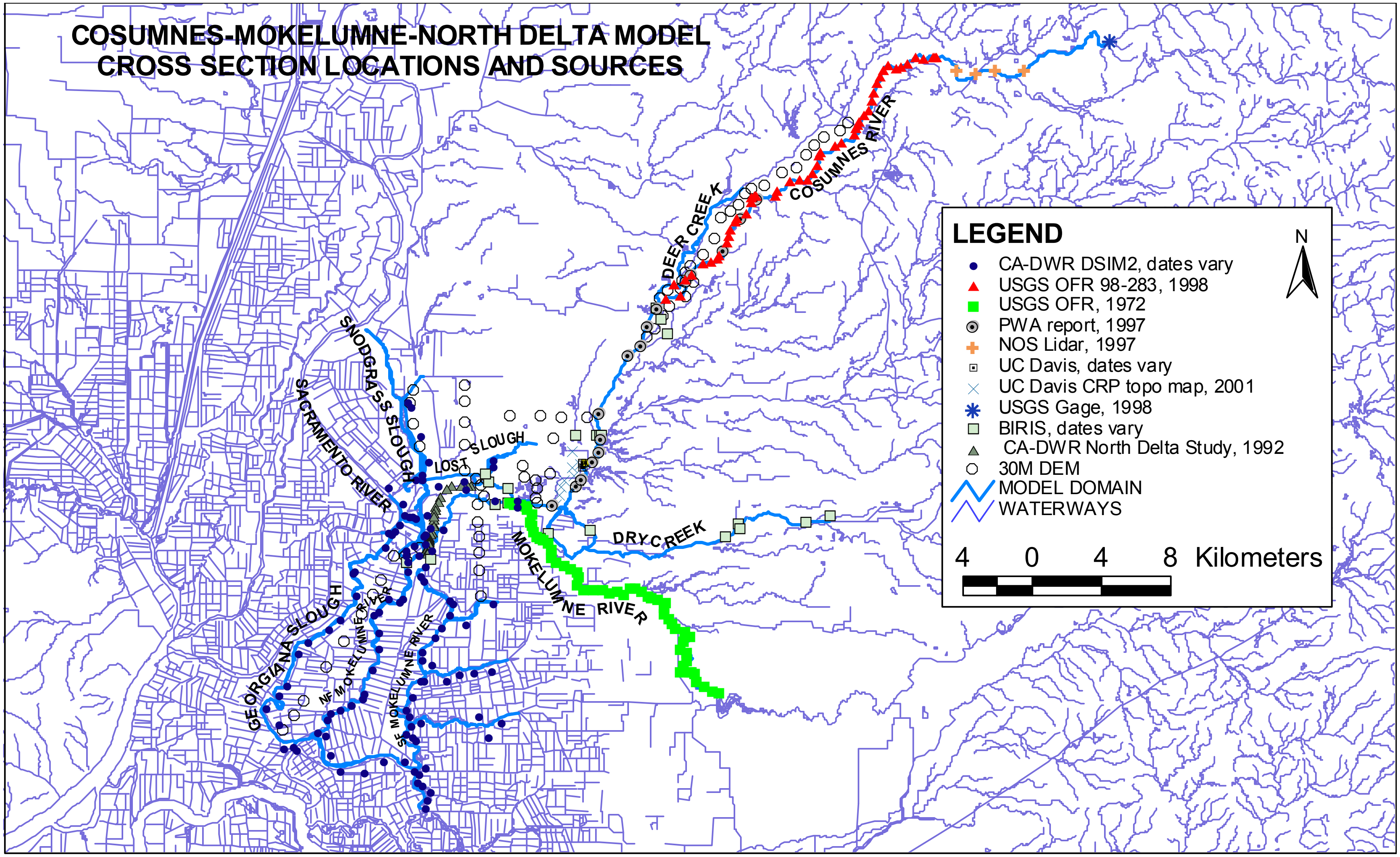


Figure 5. Cross section locations and data sources used in the Cosumnes-Mokelumne-North Delta Model.

Topography data for the McCormack-Williamson Tract were obtained from the North Delta Study (NDS) conducted in 1992 by the CA-DWR. Cross sections were extracted from topographic maps of the area available in an AutoCAD drawing format. The datum for this study was NGVD 29 (Paul Ladyman personal communication). This was verified by comparing the elevations of the levee crown on the NDS drawing with a levee centerline survey performed by MBK in August of 1989 (MBK 1989). To ensure the topography of the McCormack-Williamson Tract had not been significantly altered since the NDS, a survey crew from CA-DWR conducted a partial resurvey of the tract (California Department of Water Resources 2002). This survey focused on the centerline of roads, the perimeters of each agri-cell/field, the location of the television tower and its guy wire foundations. Perimeter values of each cell were compared to contour lines on the NDS topography drawing, and found to be in good agreement. Elevations with in the McCormack-Williamson Tract were found to not have changed significantly in the last ten years. In addition, water levels at Benson's Ferry and New Hope Landing were surveyed (and times noted) to allow for comparison to the reported gage water level elevations. This was conducted to verify the datum of each of the reported gage values.

Datums

Data collected at different times, and by different agencies do not always utilize the same reference datum, and in some cases do not document the reference datum that is used. Such issues can cause considerable confusion, and lead to errors in simulation results. To ensure uniformity, and confidence in the modeling results, data from each source have been datum checked and converted as needed to the National Geodetic Vertical Datum of 1929 (NGVD 29).

Time series water level data from a number of gages are used extensively in this study, in several locations as external boundary conditions, and for two locations as internal comparison points. The datum for many of these gages is the United States Engineering Datum (USED), which in the absence of other information is assumed to be 3 feet below NGVD 29. Considerable effort has been undertaken to ensure the accuracy and consistency of all elevation measurements (geometric and gage) in this study.

Manning Coefficient

Hydraulic models like MIKE 11 require an input of channel roughness in each reach. Typically this parameter is input as the Chezy coefficient or, as in this study, Manning coefficient (n). The value of the Manning coefficient depends upon many things, but primarily upon surface roughness (size, shape and distribution of material that lines the bed), the amount of vegetation, and channel irregularity. Other factors, which influence the Manning coefficient to a lesser degree, include stage, scour and deposition, and channel alignment (Chaudhry 1993). Therefore the roughness of a straight lined trapezoidal canal is very different than that of a meandering vegetated cobble bottomed river. Several methods have been developed to aid in the estimation of the Manning coefficient, including n-value tables, equations, and photographs for comparison.

In this study, a combination of n-value tables and photographs were used to estimate n values for various regions of the model domain. Barnes (1967) compiled a collection of stream cross sections, photographs, and calculated n values (from known flow and stage data) for 50 stream channels. A more recent attempt by Coon (1998) reviewed the available data and methodologies and estimated the roughness coefficients for natural stream channels with vegetated banks. These two references were reviewed along with several n-value tables in order to estimate the roughness coefficients. While these sources provided initial values, various values were adjusted as part of the calibration effort discussed below. The final Manning coefficient values used are provided in Table 2.

Table 2. Manning coefficient (n) values used in the Cosumnes-Mokelumne-North Delta Model.

Location	**Manning Coefficient - n**
Global Value[1]	0.036
Cosumnes River[2]	0.040
Deer Creek	0.050
Dry Creek	0.050
Delta Islands and Tracts	0.050
Floodplain Regions	0.100

Notes:

1) The global value is applied to all model regions unless otherwise specified.
2) For the 1986 runs, Cosumnes River n value was increased to 0.045 to account for the increased effect of vegetation at high water levels.

Data Uncertainties

A great deal of real data have been utilized in compiling, calibrating and validating the model, however many crucial data elements including cross sectional geometry, boundary conditions and system connectivity are not available and have been estimated. As previously mentioned, boundary condition data are not available for several hydraulic gages for various periods. In these situations simple estimates were used and provide one element of uncertainty. Other uncertainties arise when using cross sectional data, which were measured at different times with different methods. For example, data from as early as 1934 are used in the model. Yet another element of uncertainty is the lack of channel cross sectional data in some reaches, with 3.5 km between cross sections in some cases. The bathymetry of Dry Creek is very poorly represented with only a few cross sections. In regions with insufficient cross section geometry, the 30 m DEM was used. While the vertical accuracy of this DEM meets USGS mapping standards and is hoped to be ± 15 cm, it is rarely better than ± 7 m. In addition, the connectivity of the hydraulic system, in particular the manner in which floodwaters access floodplain environments, is an area of uncertainty. In these situations assumptions and estimations have been made. When compounded these elements create high degree of uncertainty, and influence the accuracy of the model results.

Flood Frequency of Time Periods of Simulation

To properly evaluate the impacts of altering the current hydraulic system of the North Delta, a wide range of flows must be considered, because the tract's influence upon regional hydraulics may be very different in different floods. The Cosumnes River is the dominant source of floodwaters to the North Delta region, so Cosumnes River discharge (at Michigan Bar) for various flood pulses has been used as the primary distinguishing variable. In addition to the magnitude of each storm, the recurrence interval of the flood pulse is of interest. Flood recurrence interval is defined as the expected period of time within which a flood of a given magnitude will be equaled or exceeded. For example, the chance that a 50-year recurrence interval flood will occur in a given year is 1 in 50. Flood frequency analyses were performed by the USGS, for the Cosumnes River based upon 91 years of data (1907-1997) recorded at the Michigan Bar gaging station (Guay et al. 1998). PWA performed another flood frequency analysis, for the Cosumnes River based upon 89 years of data (1907-1995) recorded at the Michigan Bar gaging station (Vick et al. 1997). These flow frequency analyses have been used to describe the recurrence intervals of flood pulses in this study.

To evaluate the hydraulic impact of the various management scenarios, flood pulses of various recurrence intervals were simulated (Figure 6). The largest flood observed on the Cosumnes River in 2000 had a maximum hourly averaged discharge of 334 cms (11,790 cfs), which corresponds to a recurrence interval of ~2.5+ years. The largest flood observed in 1999 had a maximum discharge of 625 cms (22,060 cfs), corresponding to a recurrence interval of approximately 5 years. The largest flood observed in 1998 had a maximum discharge of 928 cms (32,780 cfs), which is close to the 10-year recurrence interval (Q=968 cms or 34,200 cfs) for this system. The largest flood modeled in this study occurred in 1986 and had a maximum discharge of 1,169 cms (41,290 cfs), which corresponds to roughly a 25-year storm. In addition to large flood pulses, low river flow (tidally dominated) conditions are simulated in each of the four years studied.

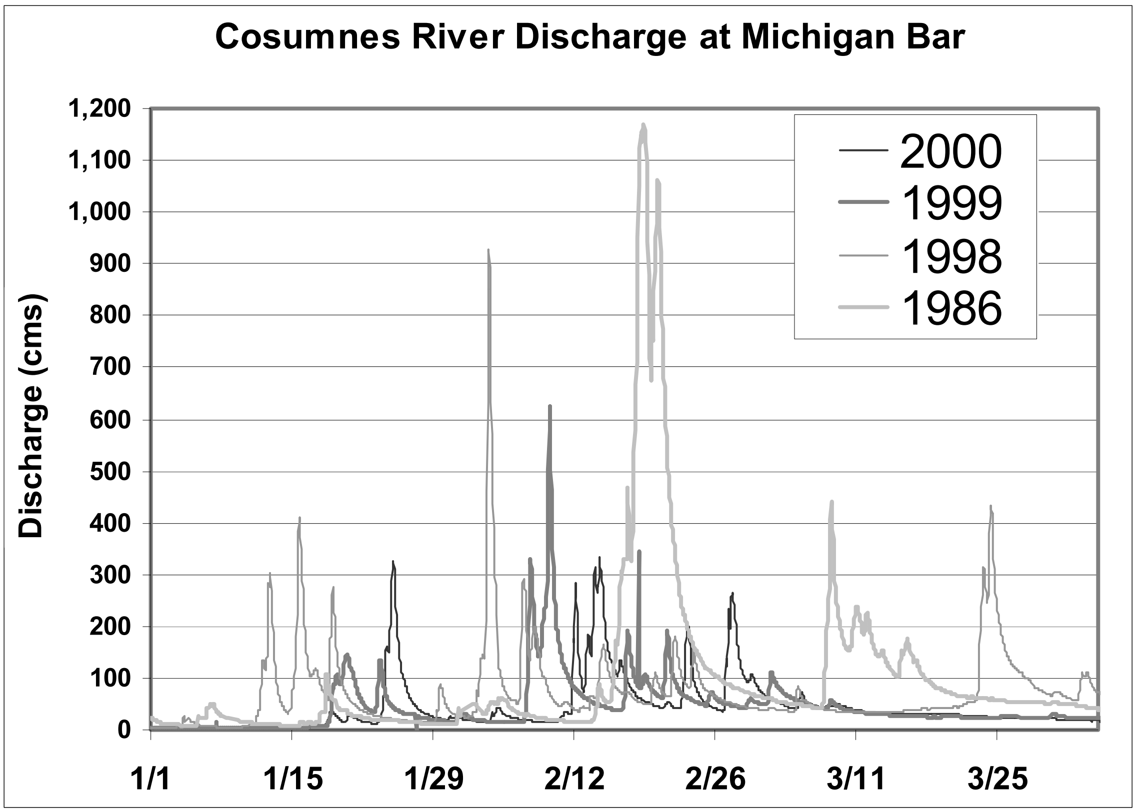


Figure 6. Superimposed Cosumnes River discharge at Michigan Bar for time periods simulated with the hydraulic model. The maximum annual discharges are 334 cms (11,790 cfs), 625 cms (22,060 cfs), 928 cms (32,780 cfs) and 1,169 cms (41,290 cfs) corresponding to the years of 2000, 1999, 1998, and 1986 respectively.

MODEL CALIBRATION AND VALIDATION

Calibration and Validation Methodology

Once the required data were collected and processed for use in the MIKE 11 modeling platform, simulations were undertaken. Initial run results were compared to observed data at a number of locations within the model domain to determine the accuracy of the results. While the model appeared to represent the major elements of the observed stage hydrograph including reasonable tidal oscillation and flood pulses, the magnitude and timing required improvement. In many modeling projects, the required data set is complete and accurate allowing the investigator to trust the data, and calibrate the model through the manipulation of the Manning coefficient. As discussed previously, a high degree of uncertainty exists for many model input parameters including cross sectional and boundary condition data as well as system connectivity. The large number of uncertainties associated with this project made the calibration and validation of the model a time intensive undertaking as the model's sensitivity to various items was investigated individually. Adjustments to the channel geometry, assumed boundary conditions, and system connectivity were necessary to achieve the quality of the final model simulation results.

The model improvement and calibration proceeded in two phases, focusing on different flow conditions. Initially, the low flow, tidally dominated portion of the hydrograph was improved. In initial model simulations the tidal amplitude was muted. Inaccurate geometry in the Cosumnes River Preserve (CRP) region was hypothesized as the reason.

The initial geometric configuration of the CRP region was based upon data from the 30m DEM of the area, in addition to estimates of channel size, and the area contained too much subaerial volume. The cross sections used to define reaches in this area were refined through the incorporation of new field data and better estimates in regions without data. In addition, several reaches in the CRP region: Bear Slough, Grizzly Slough, Dry Creek, Middle Slough and Lost Slough are described poorly in the model. This poor description is a result of little actual topographic data. In these regions old, and in some cases estimated cross sections are used. An inspection of the cross sectional information in this region was conducted and values refined, yielding improved model results. During this phase errors in some Mokelumne River cross sections (upstream of Benson's Ferry) were found. Original source data was referenced and cross sections corrected as necessary. By improving the regional low flow geometry, subsequently reducing the volume below mean sea level, the amplitude of the tidal signal increased and timing improved, resulting in a better agreement with the observed results.

The second phase of model calibration focused on improving the timing, magnitude and hydrograph shape of various flood pulses. In conjunction with this, many parameters were adjusted. While the influence of many estimated cross sections were investigated in this aspect of the model calibration, ultimately refinement to the connectivity of the simulated hydraulic system resulted in the best agreement. In particular this refers to the manner in which Cosumnes River channel flow accesses (through overtopping, breaching, etc.) floodplain regions, and the effect of such regions on attenuating flood pulses. When available, data from local residents were utilized (Walt Hoppe personal communication). At other times, educated trial and error was used to investigate the influence of various floodplain storage regions, and access configurations. This approach was specifically applied to the shared Cosumnes River – Deer Creek floodplain, as the location, size and elevation of access breaches was manipulated. The connectivity of the Cosumnes River to the region north of Twin Cities Road was manipulated for the 1986 event as advised by Walt Hoppe. Further downstream, the connectivity of the Mokelumne River (west of Interstate 5 and east of the McCormack Williamson Tract) to its adjacent floodplain and subsequently Middle and Lost Sloughs was manipulated. These manipulations to the channel-floodplain connectivity resulted in a simulated flood hydrograph, which more accurately mimicked the observed gage data.

Beyond improvements to cross sectional geometry, and system connectivity, the magnitude of Dry Creek flow contributions was considered. While many methods for estimating the discharge from this flashy tributary were attempted, time constraints lead the investigator to use the simple method discussed above. Final calibration involved the manipulation of Manning n values throughout the network. Adjustments to both the global value, and to values applied to individual reaches were made, resulting in the values used throughout the scenario simulations presented in Table2.

The final product of this extensive effort is a model, which simulates water movement in this system across a range of tidally and fluvially dominated conditions, as validated by the simulation of various flood periods with varying flood pulse magnitudes. Simulation results are provided in Figures 7 through 11 and discussed following the sensitivity analysis below.

Sensitivity Analysis

To determine the sensitivity of the model's results to various input parameters, a sensitivity analysis was performed. In conducting a sensitivity analysis, one input parameter is adjusted while all other parameters are left untouched. The model sensitivity to three types of input parameters was investigated: the timing and magnitude of upstream discharge (Cosumnes River at Michigan Bar, Dry Creek above Galt, Mokelumne River at Woodbridge and the Sacramento River at Georgiana Slough), downstream water level (Mokelumne River at Georgiana Slough and Little Potato Slough near Terminous), and channel roughness.

The first four months of 1998 (1/3/98 to 4/30/98) were chosen for the sensitivity analysis, to allow for the analysis of tidally dominated/low river flow conditions in addition to flood events of varying magnitude (up to ~10 year return). The period of 1/25/98 to 2/25/98 is shown in Appendix B for each sensitivity simulation, as it displays the tidally dominated, flood peak, and recession portions of the stage hydrograph. Times series of water surface elevations are presented for Benson's Ferry and New Hope and compared to the base simulation results to determine the model's sensitivity to each parameter, at each location. Changes to the maximum water surface elevation for each sensitivity simulation are provided in Table 3 for both locations.

Table 3. Simulation results from the sensitivity analysis showing the change in maximum water surface elevation and percent change in maximum water surface elevation at Benson's Ferry and New Hope for various model sensitivity runs. Refer to Appendix B for more sensitivity analysis results.

Location	**Benson's**	**Ferry**	**New**	**Hope**
	Δ h[2] m	**%Δ[3]**	**Δ h[2] m**	**%Δ[3]**
Upstream Discharge Magnitude				
Cosumnes River (at Michigan Bar) Q +10%	0.11	2.32	0.06	2.30
Cosumnes River (at Michigan Bar) Q - 10%	-0.13	-2.83	-0.04	-1.60
Mokelumne River (at Woodbridge) Q + 10%	0.03	0.71	0.02	0.66
Mokelumne River (at Woodbridge) Q - 10%	0.00	0.06	0.00	-0.16
Dry Creek (above Galt) Q=0	-0.57	-12.19	-0.21	-7.98
Sacramento River (at Georgiana Slough) Q=0	0.00	0.00	0.00	0.00
Upstream Discharge Timing				
Cosumnes River (at Michigan Bar) Q + 6hrs	-0.05	-1.16	-0.06	-2.18
Cosumnes River (at Michigan Bar) Q - 6hrs	0.04	0.82	0.07	2.72
Mokelumne River (at Woodbridge) Q + 6hrs	0.01	0.13	0.01	0.23
Mokelumne River (at Woodbridge) Q - 6hrs	0.03	0.64	0.02	0.62
Dry Creek (above Galt) Q + 6hrs	0.09	1.82	0.01	0.51
Dry Creek (above Galt) Q - 6hrs	-0.07	-1.59	-0.02	-0.62
Downstream Water Surface Elevation[1]				
Mokelumne River (at GS) h + 0.25 m	0.02	0.43	0.13	5.06
Mokelumne River (at GS) h - 0.25 m	-0.01	-0.17	-0.12	-4.67
Channel Roughness				
Manning's roughness coefficient n + 10%	0.06	1.31	0.01	0.35
Manning's roughness coefficient n - 10%	-0.07	-1.48	-0.01	-0.19

Notes:

1) On the downstream water surface elevation sensitivity run, the estimated Little Potato Slough (below Terminous) time series was also raised and lowered by 0.25m.
2) Calculated by $h_{\max-condition} - h_{\max-base}$
3) Calculated by $\dfrac{h_{\max-condition} - h_{\max-base}}{h_{\max-base}} * 100$
4) +6hrs = delay of 6 hours, -6hrs = advance or 6 hours

Upstream boundary condition data were manipulated in a number of ways. For the Cosumnes and Mokelumne Rivers, gage flow data were increased and decreased by ten percent of the discharge values at Michigan Bar and Woodbridge respectively. This is judged to be an appropriate amount of variance, because the USGS states that their posted flow values are within ten percent of the actual amount. In addition the flow hydrograph at each of these locations was advanced and delayed by six hours. The Dry Creek boundary in the 1998 simulations is artificial, estimated as 40 percent Michigan Bar flow, so rather than increasing/decreasing the estimated values by ten percent, the flow value was set to zero to demonstrate the influence of Dry Creek on the region surrounding the McCormack-Williamson Tract. The hydrograph timing was advanced and delayed by six hours, as with the Cosumnes and Mokelumne runs.

The model's sensitivity to Sacramento River flows on the McCormack-Williamson Tract area was examined by substituting a no flow boundary condition at the upstream end of Georgiana Slough. No adjustments to the timing of Sacramento River discharge were studied. For the downstream water level sensitivity run, the water level at the Mokelumne River (at the confluence with Georgiana Slough) and at Little Potato Slough was increased and decreased by 0.25 m. This was judged to be an appropriate perturbation as this is the average difference between spring and neap high tides in this area.

Hydraulic models rely heavily upon the user defined channel roughness in calculating water levels and flow. Due to the dependence upon this value, it is often used in calibrating model results to better fit observed data. In recognition of this, the Manning's n values throughout the model domain were increased and reduced by ten percent to determine the sensitivity of the model results. This magnitude of change is appropriate, as the altered values still fall within reasonable values of roughness for this system.

Cosumnes River simulations show that water levels in the McCormack-Williamson Tract region are sensitive to both the magnitude and timing of Michigan Bar discharge values during flood periods, with a higher level of sensitivity seen at Benson's Ferry than at New Hope. This displays the role of the Cosumnes River as the dominant producer of floodwaters in this system. A ten percent increase of flow from Michigan Bar resulted in stage increases 0.11 m (2.3% change of maximum water level) and 0.06 m (2.3%), while a ten percent decrease resulted in 0.13 m (-2.8%) and 0.04 m (-1.6%) stage reductions observed at Benson's Ferry and New Hope respectively. A high degree of sensitivity to the timing of Michigan Bar flow is also seen, as a six hour delay (+ 6 hrs) resulted in stage reductions of 0.05 m (-1.2%) and 0.06 m (-2.2%), while a six hour advance (-6 hrs) resulted in 0.04 m (0.8%) and 0.07 m (2.7%) stage increases observed at Benson's Ferry and New Hope respectively.

Mokelumne River simulations show that peak water levels in the McCormack-Williamson Tract are less sensitive to the magnitude and timing of flows from Woodbridge, however Mokelumne River discharge becomes more important later in the season. At this point, Mokelumne River discharge dominates flow to the McCormack-

Williamson Tract region, after flood peaks from the Cosumnes River have already occurred (Appendix B). A ten percent increase of flow from Woodbridge resulted in maximum stage increases 0.03 m (0.7%) and 0.02 m (0.7%), while a ten percent decrease resulted in negligible affects observed at Benson's Ferry and New Hope respectively. Manipulations to the timing of the Mokelumne River discharge also showed small deviations in water levels at both locations.

Dry Creek simulations illuminate the importance of discharge from this tributary to the study area. The no flow simulation demonstrates the role of Dry Creek, as peak water levels are reduced by 0.57 m (-12.2%) and 0.21 m (-8.0%) at Benson's Ferry and New Hope respectively. The timing of Dry Creek discharge is also important, as a 6-hour advance reduces maximum water levels by 0.07 m (-1.6%) and 0.02 m (-0.6%) at Benson's Ferry and New Hope respectively. A 6-hour delay of the assumed boundary condition produces a 0.09 m (1.8%) and 0.01 m (0.5%) increase in maximum water levels at Benson's Ferry and New Hope respectively.

The sensitivity simulation for Sacramento River discharge suggests that for the period modeled, maximum water levels are not influenced by conditions in the Sacramento River. A total removal of the Sacramento River forcing (Q=0) did not alter maximum water levels at either location. This may be different under different flow conditions on the Sacramento system, and most certainly would be different during periods when the Delta Cross Channel is open.

Sensitivity simulations for alterations to downstream stage suggest a high level of sensitivity of water levels below the McCormack-Williamson Tract. Raising the downstream boundary condition by 0.25 m resulted in a maximum stage increase of 0.02 m (0.4%) and 0.13 m (5.1%) at Benson's Ferry and New Hope respectively. A decrease of downstream water levels by 0.25 m resulted in reductions of maximum stage of 0.01 m (-0.2%) and 0.12 m (4.7%) at Benson's Ferry and New Hope respectively. An inspection of the simulation results indicates that downstream stage has a large influence upon simulated water levels during low flow tidally dominated conditions at both locations, however significant effects to maximum stage are only observed at New Hope.

Model simulation results show a high degree of sensitivity to alterations of channel roughness (n) at Benson's Ferry, however impacts at New Hope are not as significant. A ten percent increase in channel roughness throughout the model region resulted in an increase of 0.06 m (1.3%) at Benson's Ferry, but an increase of only 0.01 m (0.4%) at New Hope. A ten percent reduction of channel roughness values resulted in a decrease of maximum stage of 0.07 m (-1.5%) at Benson's Ferry, but only 0.01 m (-0.2%) reduction at New Hope. The most significant differences are apparent upon inspection of the recessional limb, or the period after the major flood peak (Figure B.9, Appendix B). A reduction of the n value results in the earlier arrival of flood pulses, while an increase results in a delay of flood pulse arrival.

In summary, sensitivity analysis indicates that the model is sensitive to alterations of most input parameters, with varying degrees of sensitivity observed at New Hope and

Bensons Ferry. Maximum water levels are sensitive to both the timing and magnitude of discharge from both the Cosumnes River and Dry Creek. Mokelumne River discharge magnitude plays a larger role in water levels later in the spring when reservoir releases dominate flow to the North Delta region. The model shows little sensitivity to Sacramento River discharge during the period evaluated in the sensitivity analysis, although it is noted that the sensitivity to Sacramento River conditions would likely increase during periods when the Delta Cross Channel is open. Adjustments to the water level at the downstream boundaries (Mokelumne River below Georgiana Slough and Little Potato Slough) resulted in significant effects when observed at New Hope, yet negligible effects when observed at Benson's Ferry. Adjustments to channel roughness resulted in significant effects when observed at Benson's Ferry, but lesser effects when compared at New Hope.

Comparison to Observed Data

In the current study, the hydraulic model has been applied to simulate the flooding period of four years: 1986, 1998, 1999, and 2000. Simulations begin in early January while river discharges are low and extend beyond the significant flood pulses of each year. Benson's Ferry and New Hope are used for comparison, based on their proximity to the McCormack-Williamson Tract. Plots of water surface elevation vs. time are provided in Figures 7 through 11 for each location and study year comparing the model simulation results to the observed gage data. Figure 7 focuses on one week in January of 2000, in which low river flow tidally dominated conditions existed. These plots are provided to display the ability of the hydraulic model to simulate flow conditions ranging from purely

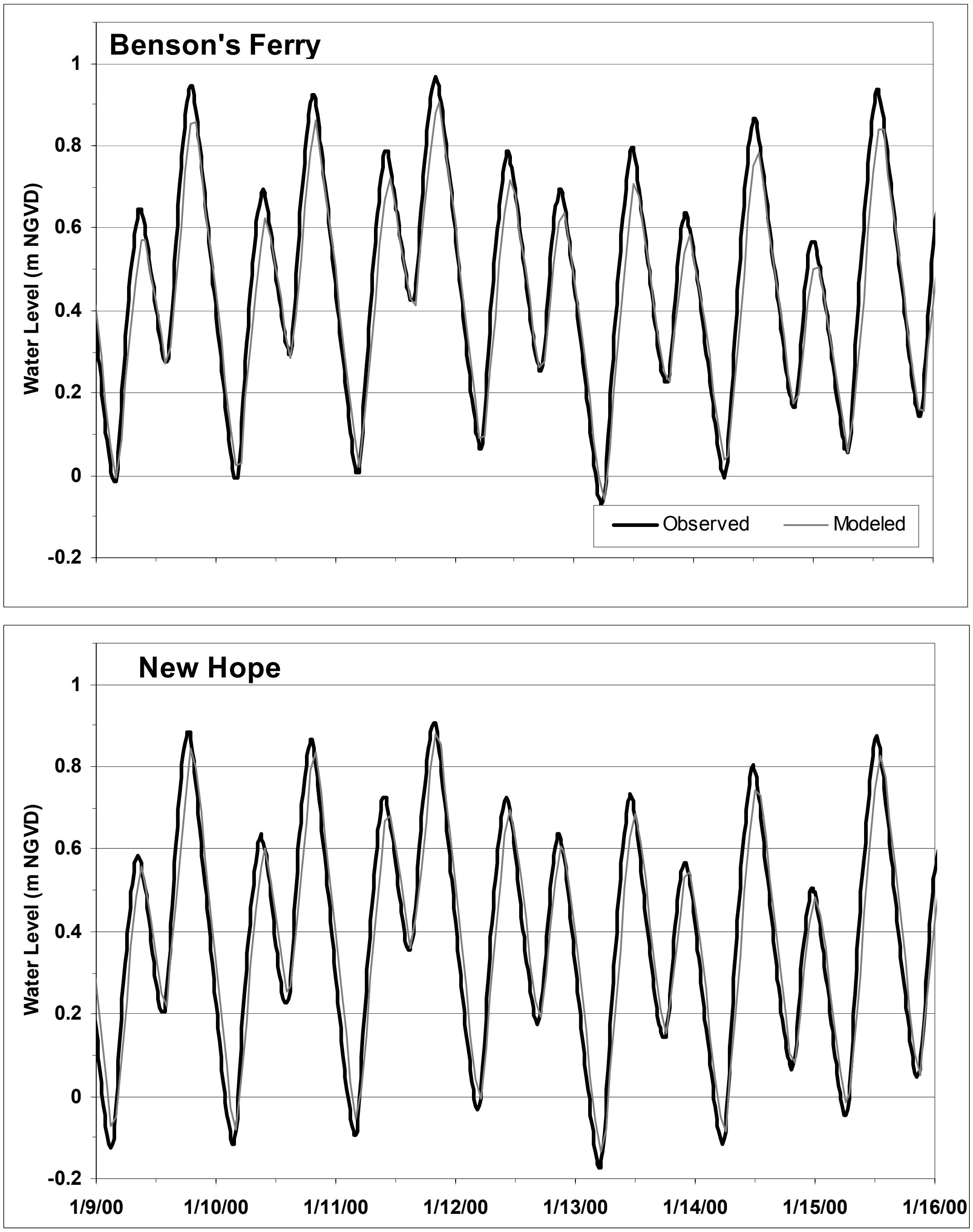


Figure 7. Comparison of model simulation results to observed gage data at Benson's Ferry and New Hope for a one week low flow period in 2000. For the locations of Benson's Ferry and New Hope relative to the McCormack-Williamson Tract refer to Figure 2.

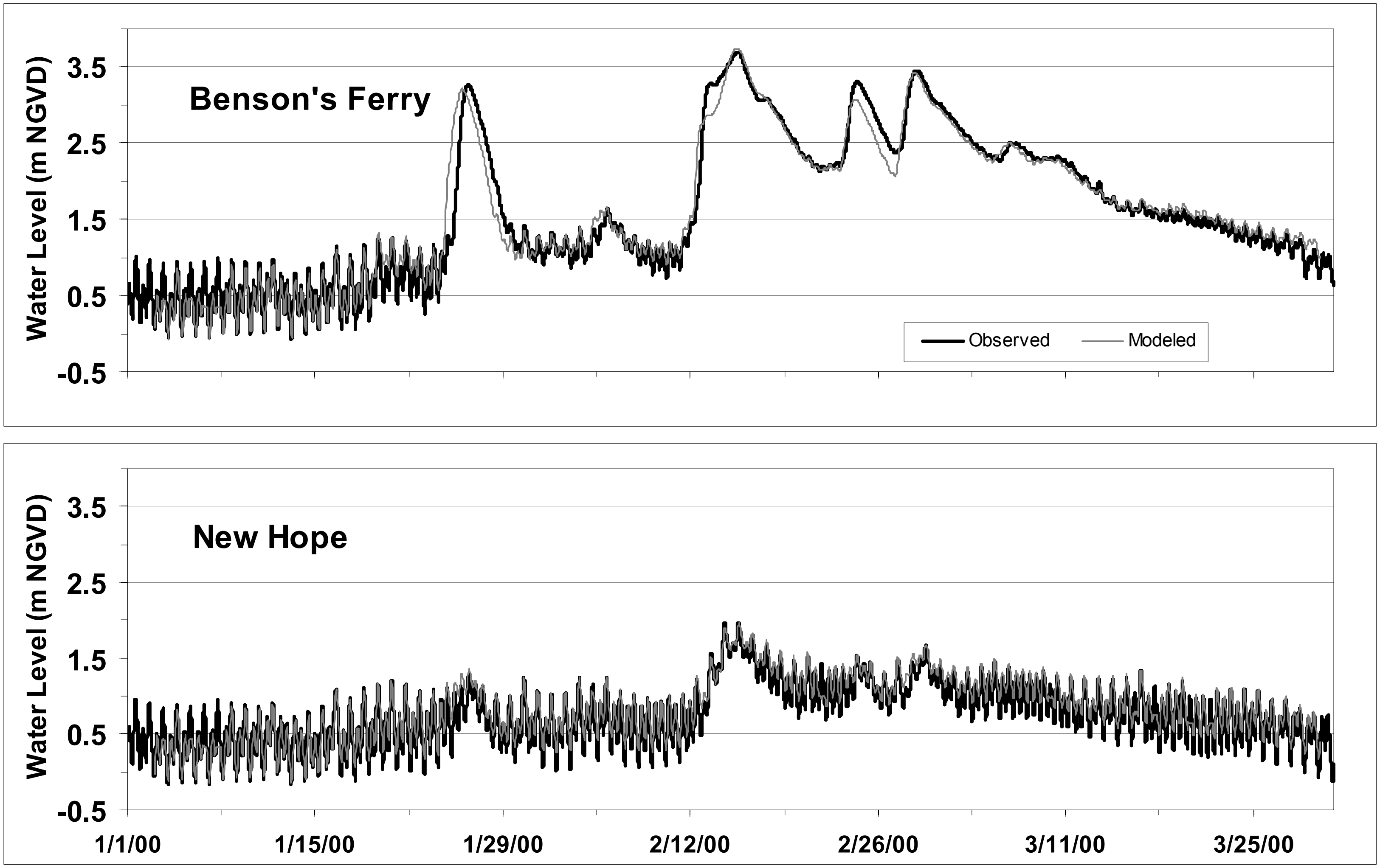


Figure 8. Comparison of model simulation results to observed gage data at Benson's Ferry and New Hope for the 2000 flood period. For the locations of Benson's Ferry and New Hope relative to the McCormack-Williamson Tract refer to Figure 2.

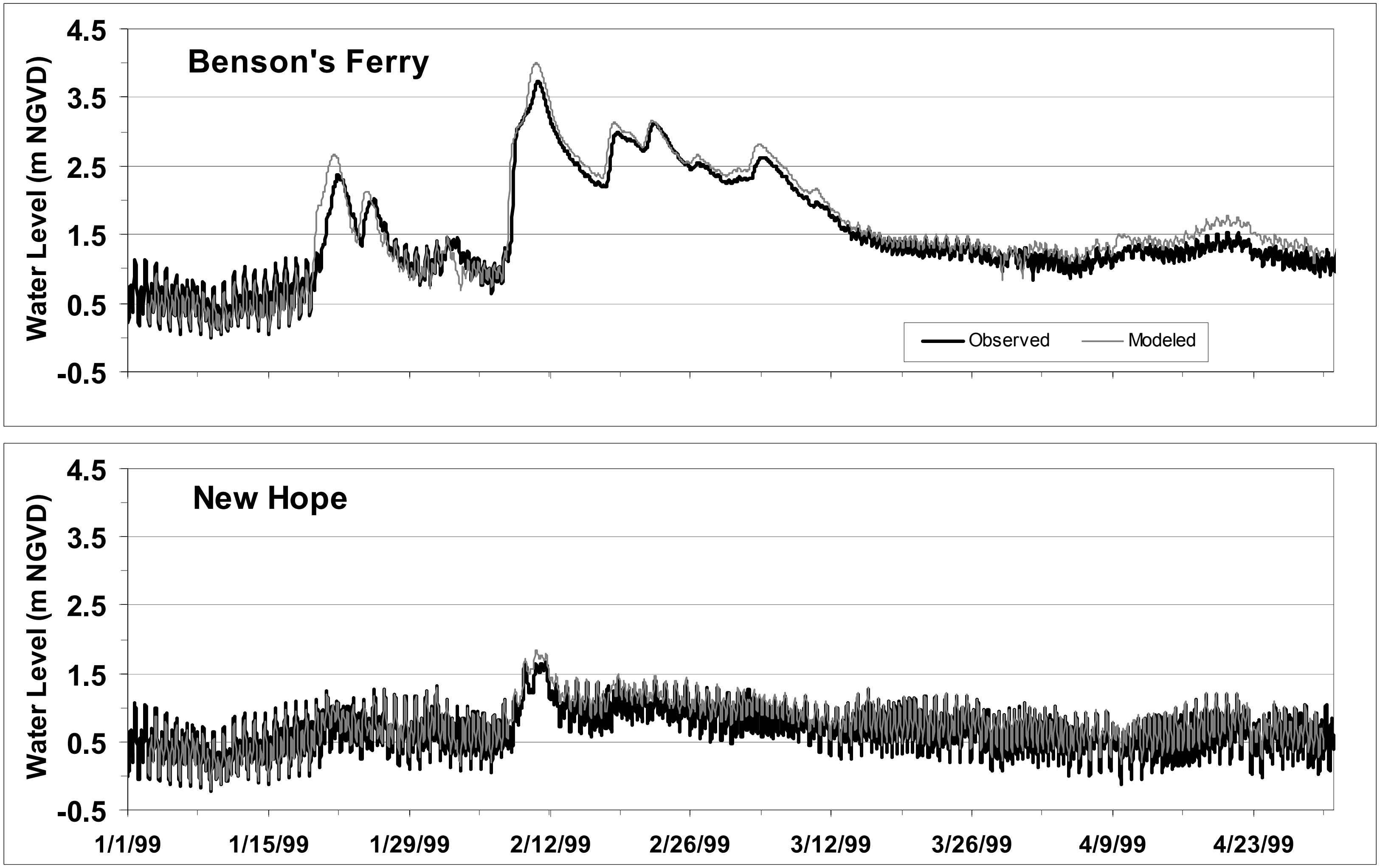


Figure 9. Comparison of model simulation results to observed gage data at Benson's Ferry and New Hope for the 1999 flood period. For the locations of Benson's Ferry and New Hope relative to the McCormack-Williamson Tract refer to Figure 2.

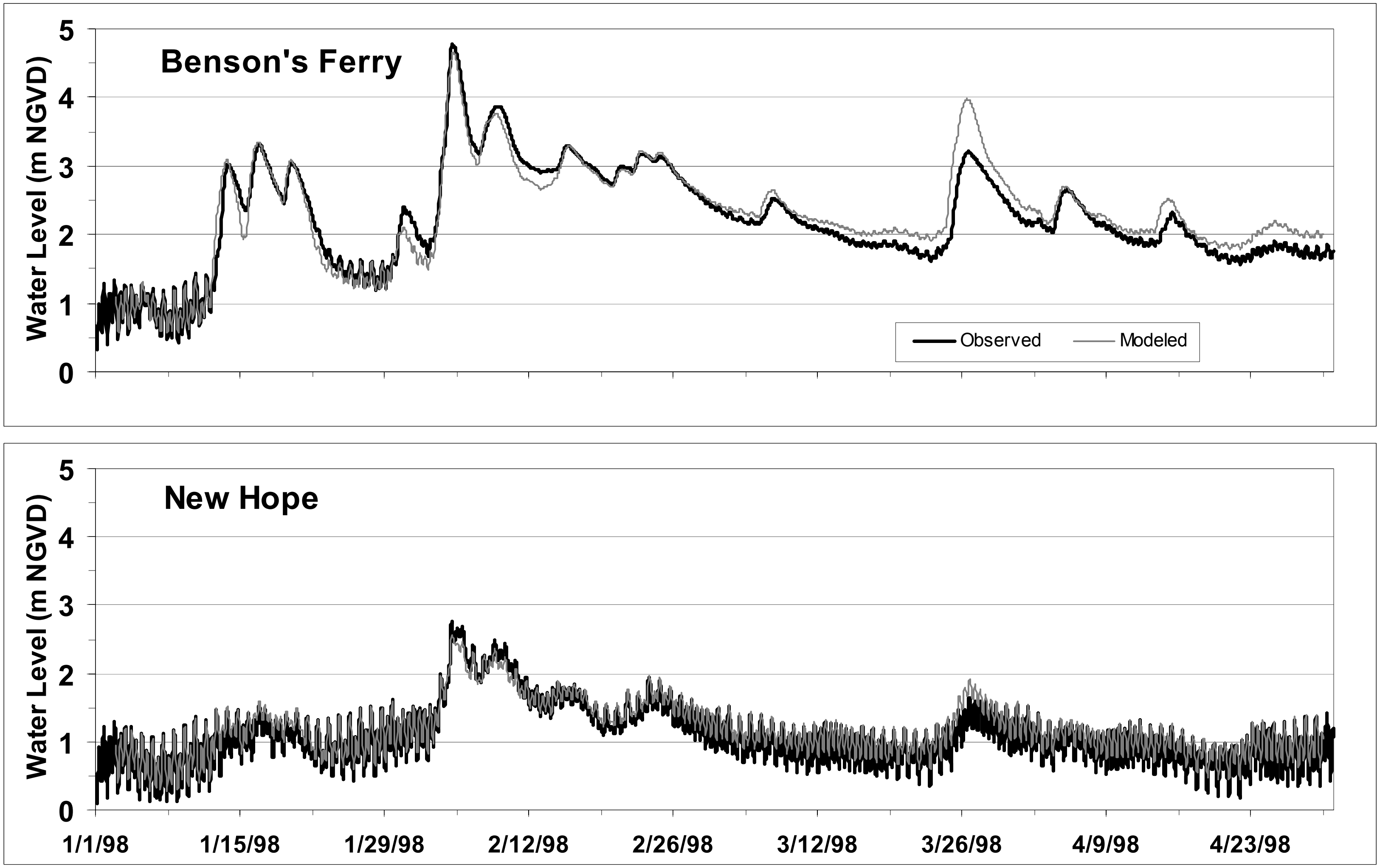


Figure 10. Comparison of model simulation results to observed gage data at Benson's Ferry and New Hope for the 1998 flood period. For the locations of Benson's Ferry and New Hope relative to the McCormack-Williamson Tract refer to Figure 2.

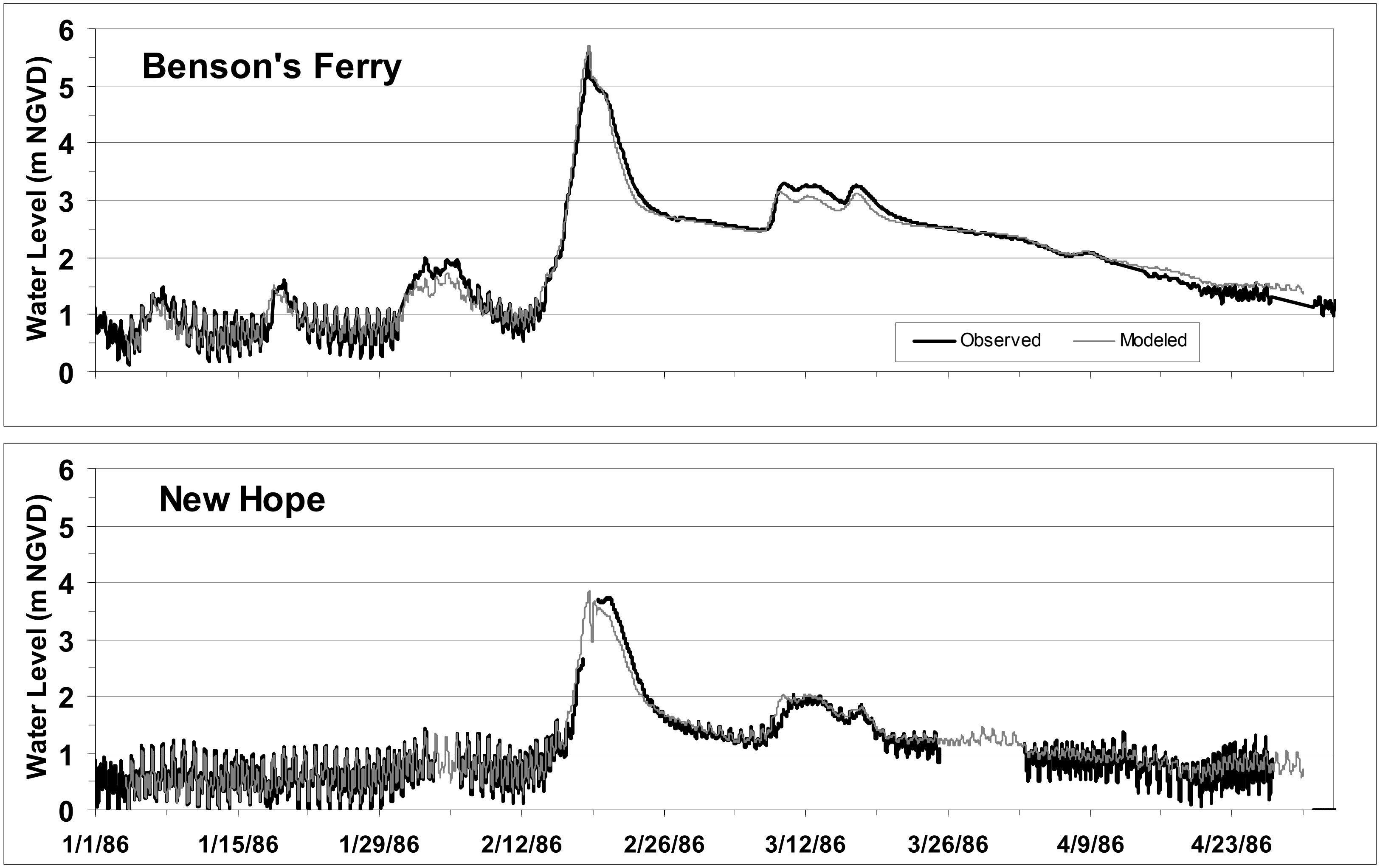


Figure 11. Comparison of model simulation results to observed gage data at Benson's Ferry and New Hope for the 1986 flood period. For the locations of Benson's Ferry and New Hope relative to the McCormack-Williamson Tract refer to Figure 2.

tidal to floods of varying magnitude. A comparison of simulated maximum water levels to observed data for each period modeled is provided in Table 4, as is the correlation of simulated results for the entire period.

Table 4. Error estimation for model run simulation results for each time period simulated at Benson's Ferry and New Hope.

Simulation Year	**2000**	**1999**	**1998**	**1986**
Benson's Ferry Results				
Correlation Coeficient[1]	0.978	0.978	0.962	0.989
Peak Error[2]	-0.009	-0.070	0.024	-0.022
Observed MWL Peak (m)	3.696	3.726	4.770	5.580
Simulated MWL Peak (m)	3.735	4.006	4.657	5.706
MWL Difference[3] (m)	-0.039	-0.280	0.113	-0.126
New Hope Results				
Correlation Coeficient[1]	0.961	0.953	0.978	0.979
Peak Error[2]	0.007	-0.092	0.074	0.044
Observed MWL Peak (m)	1.966	1.676	2.760	3.724[4]
Simulated MWL Peak (m)	1.952	1.850	2.571	3.844
MWL Difference[3] (m)	0.014	-0.174	0.189	-0.120

Notes:

1) Calculated by $\dfrac{Cov(h_{obs}, h_{sim})}{\sigma_{h_{obs}} \bullet \sigma_{h_{obs}}}$
2) Calculated by $\dfrac{h_{obs-\max peak} - h_{sim-\max peak}}{h_{sim-\max peak}}$
3) Calculated by $h_{obs-\max peak} - h_{sim-\max peak}$
4) Highest water level recorded in the incomplete gage record.

2000

The model simulation results for this time period show good agreement with the gage data at both comparison locations. During the initial low river flow, tidally dominated condition, the model accurately captures the magnitude and timing of the tidal oscillation at both locations (Figure 7), although slight muting is observed as the model under estimates high tide values by ~0.07 m, and lags the observed tidal peaks by ~45 minutes at Benson's Ferry, and underestimates high tide values by ~0.05 m, and lags the observed tidal peaks by ~30 minutes at New Hope.

Comparison of flood peak magnitudes shows good agreement with the gage data, although simulated flood peaks arrive early (Figure 8). The largest flood peak in this period occurred on 2/15/00 and corresponds to roughly a 2.5+ year return interval, based upon maximum Michigan Bar discharge (Q_{MB}=334 cms, 11,790 cfs). Simulation maximum water levels for this peak are 3.74 m and 1.95 m, which are 0.04 m above and 0.01 m below observed data at Benson's Ferry and New Hope respectively. In general the model does well at simulating this flood period, although water surface elevations are observed to be slightly elevated during the end of March, possibly due to the overestimation of lower flows from the assumed Dry Creek boundary.

1999

The model results show reasonable agreement with gage data for this flow period (Figure 9). In general the model results are above the gage data at both comparison locations. The largest flood peak in this period occurred on 2/11/99, and corresponds to roughly a

5+ year return interval, based upon maximum Michigan Bar discharge (Q_{MB}=625 cms, 22,060 cfs). Simulation maximum water levels for this peak are 4.01 m and 1.85 m, which are 0.28 m and 0.17 m above observed data at Benson's Ferry and New Hope respectively. A possible explanation of the discrepancy between model and actual water levels again is hypothesized as the estimated Dry Creek boundary. This is easily displayed during the snowmelt pulse, which can be seen starting on 4/15/99. The Dry Creek watershed is much lower in elevation than the upper Cosumnes River watershed and receives very little snow, thus the Dry Creek flow contribution, estimated as 40% of Michigan Bar discharge, is clearly an overestimate in these conditions. In addition to the estimated Dry Creek boundary the storage and peak attenuation which results from overbank flooding into floodplains in the Cosumnes River, is likely more significant than is represented in the model, as the connectivity of the river channel to the adjacent floodplain regions is one element of uncertainty in this study.

1998

1998 simulation results show good agreement with gage data (Figure 10). The largest flood peak in this period occurred on 2/4/98 and corresponds to roughly a 10 year return interval, based upon maximum Michigan Bar discharge (Q_{MB}=928 cms, 32,780 cfs). Simulation water levels for this peak are 4.66 m and 2.57 m, which are 0.11 m and 0.19 m below the observed data at Benson's Ferry and New Hope respectively. Modeled water levels for other flood peaks in this period are generally within 0.1 m of observed values. Water surface elevations during the later portion of the simulation period are again higher than the actual levels, presumably a result of the simple Dry Creek boundary estimate.

1986

Agreement between the simulation results and the actual data is strong at both locations throughout the simulation (Figure 11). The largest flood peak in this period occurred on 2/18/86 and corresponds to roughly a 25 year return interval, based upon maximum Michigan Bar discharge (Q_{MB}=1,169 cms, 41,285 cfs). Peak stage at Benson's Ferry is overestimated by 0.13 m; with simulated values at 5.71 m while the peak stage recorded at the gage was 5.58 m. A comparison of peak water levels at New Hope is not possible as the gage record is incomplete during this period. Pre and post flood peak water levels agree well with actual data, however as seen in other model runs, flood peaks reach the McCormack-Williamson Tract region early.

A partial explanation of the over estimation of the water elevation peak at Benson's Ferry, lies in the early arrival of flood pulses, combined with the timing of the major levee failures. The flood pulse arrives and water levels rise to a peak until levee failures begin, which drastically reduce water levels. Since the peak arrives early, water levels rise to an elevated level prior to breach initiation and subsequent stage reduction. If breaches were simulated earlier, or if the flood pulse was delayed further, a better agreement with observed maximum stage would likely result.

The major levee failures on McCormack-Williamson Tract, Glanville Tract, Tyler Island, and New Hope Tract all had significant influence upon water levels in the study area, particularly as observed at Benson's Ferry and New Hope. The effect of each breach is evident as a distinct change in slope of the stage hydrograph is observed corresponding to each failure. While estimates of the timing of breach initiation are available (U. S. Army Corps of Engineers 1988) and provided in Appendix A, few details are available regarding the size, elevation, and rate of opening of each breach. Water levels in the region are sensitive to each of these breach parameters, and the model was calibrated to optimize agreement with observed water levels.

Other time periods of the 1986 simulation results agree well with observed data at both locations except for the magnitude of a series of small pulses which arrive starting 3/11/86, and the magnitude of tidal oscillation later in the simulation time period. While tidal oscillations (peaks, troughs, and timing) are well represented in the pre-peak portion of the 1986 simulation, modeled tidal fluctuation is dampened in the late season. The reason for this tidal muting lies with the levee breaches and large volumes of stored water within each flooded region. During the model simulations, levee breaches were never closed or repaired, removing the effect of storage of each tract or island. The most notable in dampening the tidal oscillation is Tyler Island due to its size, location, and elevation range.

One additional method of evaluating the model results for the 1986 flooding event is a comparison of maximum floodwater volume stored in the various areas flooded as levees failed. Maximum floodwater storage in McCormack-Williamson Tract, Glanville Tract, Dead Horse Island, Tyler Island, and New Hope Tract was estimated by the Sacramento District of the U. S. Army Corp of Engineers (1988) and is provided in Table 5, as are the results of the 1986 model simulation. Reasonable agreements between the estimated and simulated values are seen given the degree of uncertainty of various model inputs (breach and tract geometry). Most notable is the geometric representation of the various regions, which are described in the model by a series of cross sections (Figure 5). All regions except the McCormack-Williamson Tract are described by cross sections extracted from the 30m DEM, while the McCormack-Williamson Tract is described by higher resolution geometry from the North Delta Study (California Department of Water Resources 1992). Maximum storage volume is most similar to the estimated values for the McCormack-Williamson Tract, presumably due to the quality and resolution of the geometric data.

Table 5. Comparison of model simulation results to estimated values of maximum floodwater storage for each flooded island or tract during the 1986 flood event.

Flooded Region	**Maximum Floodwater Storage (ac-ft)**	
	Simulation	**Estimated**[1]
Glanville Tract	48,900	45,000
McCormack-Williamson Tract	18,900	17,000-20,000
Dead Horse Island	2,700	2,000-3,000
Tyler Island	108,000	130,000-150,000
New Hope Tract	49,300	60,000

Note:

1) Estimated maximum floodwater storage values obtained from U. S. Army Corps of Engineers 1988.

Comparison Summary

The model has been demonstrated to accurately simulate water levels in the region around the McCormack-Williamson Tract for flow conditions ranging from purely tidal to floods of various magnitudes, including one large event (1986), which caused several levee failures and resulted in widespread flooding. With a validated model, investigation of various management scenarios for the McCormack-Williamson Tract could be undertaken.

SCENARIO EVALUATION

Scenario Descriptions

A wide range of possibilities exist for the future of the McCormack-Williamson Tract ranging from a completely hydraulically connected tract supporting a mosaic of habitat types, to a design which maximizes farming and flood control. Options include the removal or breaching of levees, in addition to the construction of new levees to partition the interior of the tract. In an attempt to bracket the range of possibilities several scenarios were developed with different ecological and flood control objectives in mind. These scenarios were developed based upon input from members of The Nature Conservancy, CALFED, California Department of Water Resources, and the Cosumnes Research Group in an attempt to represent some potential restoration designs in addition to purely flood control options. An objective of each scenario is the reduction of the flood pulse caused by the unplanned rapid levee failures as observed in 1986 and 1997. Each scenario is described below, detailing the hydraulic features of any breach, weir, and levee the scenario contains, as well as the intended management objectives of the scenario. In addition, graphical (Figure 12) and tabular (Table 6) explanations of each scenario are provided. For all breaches the side slopes were set at 1:1 (45 degrees), and base elevations were chosen to correspond to the local land surface behind the levee at each location.

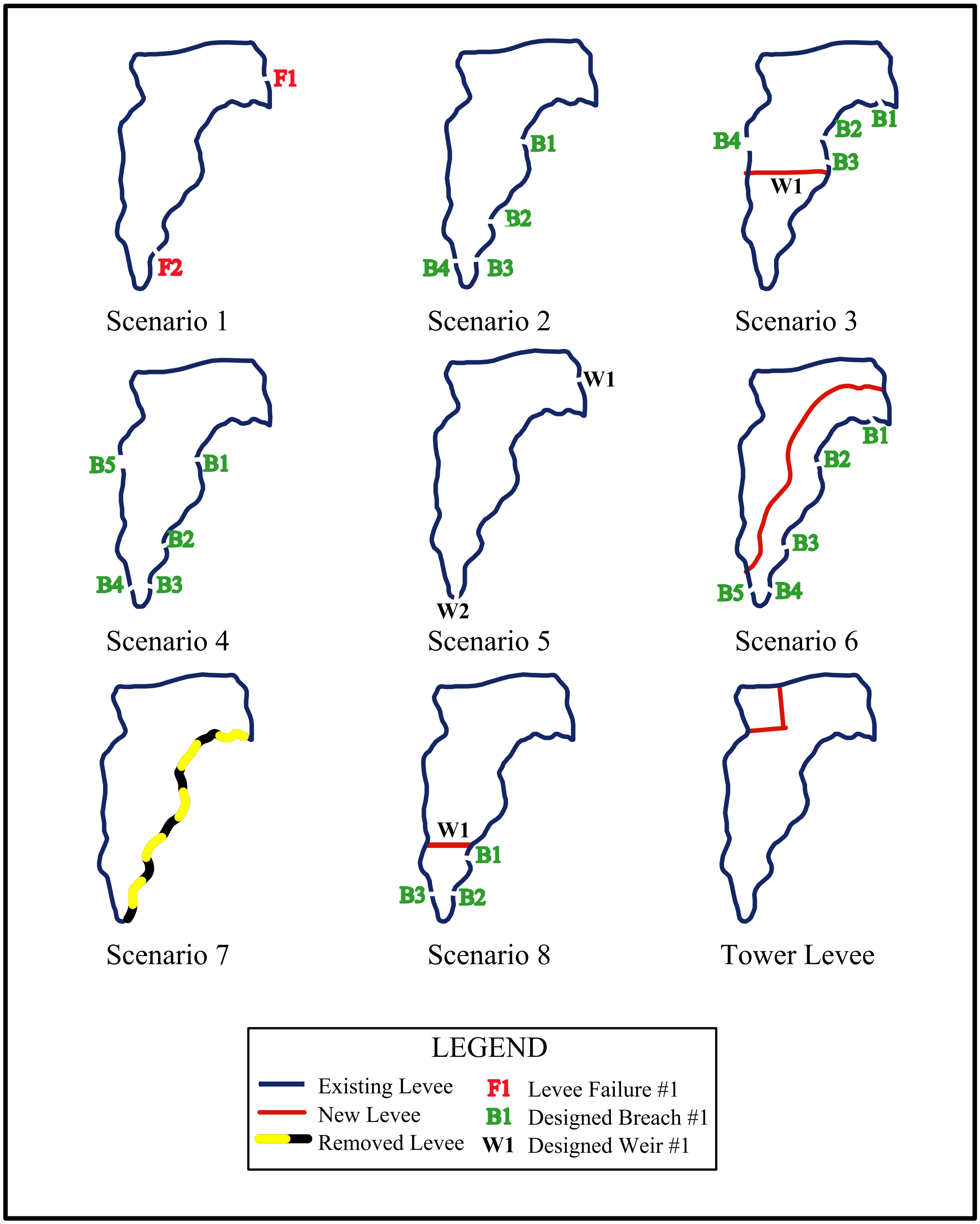


Figure 12. Schematics of various McCormack-Williamson Tract breaching scenarios evaluated with the hydraulic model. Refer to Table 6 for breach weir, and levee dimensions. The tower levee case was analyzed for all scenarios except 1 and 6.

Table 6. Breach width and elevation values utilized when simulating the various scenarios with the hydraulic model. A graphical explanation of each of the various scenarios in addition to the locations of levees, breaches and weirs is found in Figure 12.

Scenario No.	Breach 1		Breach 2		Breach 3		Breach 4		Breach 5		Weir 1		Weir 2	
	w	z	w	z	w	z	w	z	w	z	w	z	w	z
2	50	0.91	50	0.00	50	-0.31	50	-0.61	-	-	-	-	-	-
3	50	1.52	50	1.52	50	0.91	100	0.00	-	-	1610	2.70	-	-
4	50	0.91	50	0.00	50	-0.31	50	-0.61	300	0.00	-	-	-	-
5	-	-	-	-	-	-	-	-	-	-	100	3.50	100	2.50
6	50	1.52	50	0.91	50	0.00	50	-0.31	50	-0.61	-	-	-	-
7	1000	1.52	1000	1.52	1000	0.91	1000	0.00	1000	-0.31	-	-	-	-
8	50	0.00	50	-0.31	50	-0.61	-	-	-	-	800	2.40	-	-

Notes:

1) W = breach width at base.
2) Z = breach elevation at base, referenced to NGVD 29.
3) All breaches and weirs are modeled as broad crested weirs with 1:1 side slopes.
4) Internal levees are modeled as broad crested weirs.

Scenario 1-Current Condition

This is the no-project scenario, configured as the tract in its current condition. During the flood pulses of 1998, 1999 and 2000, the McCormack-Williamson Tract levees remained intact and prevented inundation. In 1986, two large back to back storms caused high river stages, resulting in flooding for several islands and tracts in the North Delta region (McCormack-Williamson Tract, Glanville Tract, Dead Horse Island, New Hope Tract, and Tyler Island). To simulate these levee failures the Dam Break module of MIKE 11 was used. On the McCormack-Williamson Tract two levee failures were simulated, one at the top of the tract along the eastern levee, and one at the bottom of the tract along the southern end of the Mokelumne River. Additional levee failures were simulated on Glanville Tract, Dead Horse Island, Tyler Island, and New Hope Tract. While the approximate timing of most of the failures is known (U.S. Army Corps of Engineers 1988), the breach dimensions are not well documented, and were determined from available data (aerial photos, residual scour hole, local individual knowledge), and otherwise assumed in the modeling effort. Refer to Table A.3 in Appendix A for details regarding the dimensions and timing of these breaches.

Scenario 2

This scenario is comprised of four breaches, each 50 m wide. Three of the breaches are located at meander bends on the Mokelumne River, and the fourth is located along Dead Horse Cut at the southern end of the tract. This scenario attempts to reconnect the tract with the adjacent river channels allowing tidal and flood pulse inundation, so the weir elevations are set to the ground elevation behind the existing levee. The sill elevations are 0.91 m, 0.0 m, -0.31 m, and -0.61 m for breaches B1, B2, B3 and B4, respectively (Figure 12). Mokelumne River breach locations were chosen to encourage the recruitment of sediment from the Mokelumne River onto the McCormack-Williamson Tract; based upon the results of Florsheim and Mount (2002), which demonstrated the importance of levee breaches on sediment deposition on the lower Cosumnes River floodplain. The objectives of this scenario include the creation of seasonal floodplain, tidal freshwater marsh, and shallow water habitats, in addition to sediment recruitment onto the tract.

Scenario 3

This scenario divides the tract into two regions with the construction of a low levee across the tract. The east-west trending levee isolates an area of approximately 200-ha (500-a) at the southern end of the tract separating the lower subtidal zone from frequent inundation. The levee is modeled as a broad crested weir, at an elevation of 2.7 m and a length of 1610 m. This elevation was chosen to prevent flooding in the lower region for up to a 10 year event, but to allow floodwaters to access the lower region by overtopping in a larger event. Three 50 m wide breaches are located along the Mokelumne River, with bottom elevations at 1.52 m, 1.52 m, and 0.91 m (B1, B2 and B3), and an additional 100 m wide breach set at 0.0 m along Snodgrass Slough (B4). The objectives of this scenario are the creation of tidal freshwater marsh and seasonal floodplain habitat. In addition, the objectives of the cross levee are 1) the reduction of permanent shallow water habitat, 2) flood storage for large events and 3) continued farming in the isolated southern region.

Scenario 4

This scenario is similar to Scenario 2, with the addition of one large 300 m wide breach along the west side of the McCormack-Williamson Tract opposite the Delta Cross Channel, opening the region to flow from Snodgrass Slough (B5). The elevation of breach B5 is 0.0 m. Similar to scenario 2, the objectives of this scenario include the creation of seasonal floodplain, tidal freshwater marsh, and shallow water habitats, in addition to sediment recruitment. The objective of the additional large breach is to encourage the exchange of a large volume of water, without creating significant scour in the region adjacent to the breach, as well as to enhance connectivity to the existing tidal marsh habitat found in nearby Delta Meadows.

Scenario 5

This scenario investigates the flood control benefit of using the McCormack-Williamson Tract as an off stream reservoir to store and detain floodwaters. It is composed of two 100 m wide weirs, one at the top of the McCormack-Williamson Tract, and one at the bottom of the tract. The upper weir (W1) is placed along the eastern levee and set at an elevation of 3.5 m. The lower weir (W2) is located at the southern tip of the island and is set at an elevation of 2.5 m. The objective of this scenario is continued farming of the tract and utilization for flood control in large (~10 year return interval) events.

Scenario 6

Scenario 6 includes a setback levee along the Mokelumne River. The levee is placed 500 m back from the current levee alignment and runs the length of the tract. Five 50 m wide breaches are placed in the existing levee as shown in Figure 6, with base elevations at 1.52 m, 0.91 m, 0.0 m, -0.31 m, and -0.61 m, from north to south. In this scenario, overtopping of the setback levee was not considered. The objectives of this scenario are to expand the flood corridor of the Mokelumne River, and the creation of seasonal floodplain, and tidal marsh, while retaining some land for agriculture.

Scenario 7

This scenario involves the removal of the entire southeastern levee, which parallels the Mokelumne River. In the model this was simulated through five large (1000 m) broad crested weirs set at the average land surface elevation behind the existing levee. The objectives of this scenario include creation of seasonal floodplain and tidal marsh habitat, in addition to initiating meander migration of the Mokelumne River.

Scenario 8

This scenario includes a low east-west trending levee, which separates the southern portion of the tract, which is below sea level. The area of the lower leveed region is roughly 70-ha (200-a), and is accessed by three breaches, two located at the outside of meander bends along the Mokelumne River, and one located along Dead Horse Cut. The breach sill elevations are 0.0 m, -0.31 m, and –0.61 m (B1, B2 and B3). The cross levee is modeled as a broad crested weir with a width of 100 m at an elevation of 2.3 m, widening to 800 m by the elevation of 2.4 m.

Tower Levee Case

A large television broadcast tower is currently located in the northwestern corner of the McCormack-Williamson Tract. With the long-term future of the tower uncertain, it has been considered in the restoration planning. For all scenarios except for 1 and 6, model simulations were run to evaluate the hydraulic impact of constructing a levee to protect the tower and guy wire foundations from inundation. The most conservative case has been considered, which does not allow any inundation to occur within the tower levee region. The area enclosed by the tower levee was modeled as approximately 70 ha, and isolates the area indicated in Figure 12.

Habitat Methodology

Habitat Zone Quantification and Spatial Flood Depth Values

The incorporation of results from the hydraulic model with GIS software provides the ability to quantify the spatial extent of flooding in a region. Using the MIKE 11 GIS package, water elevations from a hydrodynamic model simulation are projected upon a DEM of the McCormack-Williamson Tract for the calculation of areal extent and distribution of water depths for a particular moment. The DEM used, is composed of a 9 m grid of the tract based upon topography data collected in the North Delta Study (CA-DWR 1992) as shown in Figure 3. The surface was modified to include the levee borrow pit in the northeast region, however further refinement of the surface was not done to include the many irrigation channels which dissect the tract as this study aims at quantifying large areal differences between scenarios. The vertical difference between the water surface and the ground surface (DEM) is then calculated over the entire tract. Flooded regions are separated into depth classes, and the areal extent of each class quantified. Due to software limitations, flood depths of less 0.1 m cannot be plotted and are considered dry. Based upon the resolution of the data used in creating the DEM and the precision of the hydrodynamic model results this is not considered significant.

Tidal Characteristics

Water levels in the northern Sacramento-San Joaquin Delta depend upon tide conditions, river discharge, and water resource facility operations (DCC gates and reservoir releases). Unregulated discharge from the Cosumnes River watershed and to a lesser degree the Dry Creek watershed dominate inflow to the study area in the winter and early spring, while Mokelumne River and Morrison Creek discharge play a larger role in the later spring, and summer months. At New Hope, a tidal signal is present in most flow conditions, however in large flood events, as in 1997 and 1986 the tidal signal is overwhelmed by river discharge. Tidal characteristic indexes (mean higher high water-MHHW, mean high water-MHW, mean tidal level-MTL, mean low water-MLW, and mean lower low water-MLLW) reflect the range of expected tidal conditions at a location based upon the period of data the statistics are derived from. These values are calculated from a time series of gage data, and reflect the effect of hydrologic conditions and facility operations.

Published tidal characteristic values (MHHW, MLLW, etc.) calculated by the National Oceanic Service (NOS) of the National Oceanic and Atmospheric Administration (NOAA) from New Hope gage data for the period of November 1978 to October 1979, are presented in Table 7. These values are used as the reference tidal values in the habitat evaluation discussed below.

Table 7. Tidal characteristic index values for the Mokelumne River at New Hope used as representative tidal conditions in the habitat analysis.

Tide Level[1] (m)	Tidal Datum (MLLW=0)[2]	NGVD 29 (MLLW=0.07m)[4]
MHHW[3]	0.94	1.01
MHW	0.82	0.89
MTL	0.47	0.54
MLW	0.11	0.18
MLLW	0.00	0.07

Notes:

1) MHHW = mean higher high water, MHW = mean high water, MTL = mean tidal level, MLW = mean low water, MLLW = mean lower low water
2) Values calculated from 1979 water year data, and obtained from NOAA 1982.
3) Not specified in Bench Mark sheet (NOAA 1982). Calculated by adding 0.12 m, the difference between MHW and MHHW from other tidal summary values (NOAA 2002) to MHW.
4) Vertically translated based upon elevation data, MLLW = 0.07 m NGVD, from the Primary Bench Mark Stamping: Hope 1931 (PID: JS1243).

Flood inundation maps are created based upon model water level results for a specific moment in time. To convey the inundation statistics for a given tidal index, MLLW for example, a single moment (lower low tide on January 14, 1986) from the scenario 1 (as-is or no-project condition) results is selected as a representative tide. This representative tide is chosen because the New Hope low tide value from the scenario 1 model result is equal to the index value stated in Table 7. This moment or representative tide is then used to evaluate each scenario. Each scenario has different water levels within the tract and at New Hope at the representative tide due to the varying magnitudes of tidal dampening caused by each scenarios configuration. Therefore, while each inundation map reflects the representative tide, each may have a different minimum water level value at New Hope, and associated water levels throughout the tract.

Habitat Zone Delineation

Several types of habitat (subtidal, intertidal, supratidal/floodplain, and ecologically sensitive farming) are potentially obtainable on the McCormack-Williamson Tract. For the purpose of this study, subtidal habitat is defined as the region that remains inundated at MLLW. Intertidal habitat is the region inundated at MHHW but not at MLLW. The region inundated by above average tidal levels and flood pulse flows, but not at MHHW is defined as supratidal habitat. Other regions separated entirely (as in scenario 6, or the tower levee case) are considered excluded. Regions isolated by low levees, which are subject to flooding by larger less frequent events (as in scenario 3 and 8) are considered flood storage, and considered to offer little habitat value beyond that which may be obtained through wildlife sensitive farming practices.

Effect of Tidal Conditions on Areal Habitat Extent

The areal extent of each habitat type is directly dependent upon the MHHW and MLLW values used. The index values used to delineate habitat zones are mean values that reflect the time period they summarize, which may be significantly different than the average values for any given year. It is important to understand that the area inundated at any given lower low tide may be significantly greater or less than the value calculated based upon variation of tidal levels from the mean, due to spring and neap tidal cycles, or differences in facility operations (reservoir releases and Delta Cross Channel), in addition to variations in inter-seasonal and inter-annual hydrologic conditions. Based upon the tidal muting observed in this study, modifications to Staten Island (setback levees and levee breaches) may alter tidal propagation to the McCormack-Williamson Tract, and subsequently alter the extent of each habitat zone.

To demonstrate the variation of tidal levels from year to year, MHHW, MHW, MTL, MLW and MLLW values for a four month period (May through August) were calculated for 1998 and 2001 and are provided in Table 8. These years were chosen because they represent different hydrologic conditions, and reservoir releases. 1998 was a wet year with discharge at Woodbridge ranging from 42 to 90 cms (1,475 – 3,185 cfs) during the four month period. In contrast, 2001 was a dry year with discharge at Woodbridge ranging from 1.4 to 5.7 cms (51 – 200 cfs). This difference in Mokelumne River inflow to the study area alters the MHHW value by 0.22 m and the MLLW by 0.33 m. Subsequently the tidal range is reduced during the period of higher discharge (1998).

Table 8. Tidal characteristic values for the Mokelumne River at New Hope for a four month period (May through August) of 1998 and 2001 used to demonstrate the sensitivity of habitat extent to tidal values used. Note how these values differ from the published tidal benchmark values provided in Table 6.

Tide Level (m NGVD)	**1998**	**2001**
MHHW	1.26	1.04
MHW	1.14	0.90
MTL	0.83	0.56
MLW	0.49	0.20
MLLW	0.39	0.06

Note:

1) MHHW = mean higher high water, MHW = mean high water, MTL = mean tidal level, MLW = mean low water, MLLW = mean lower low water

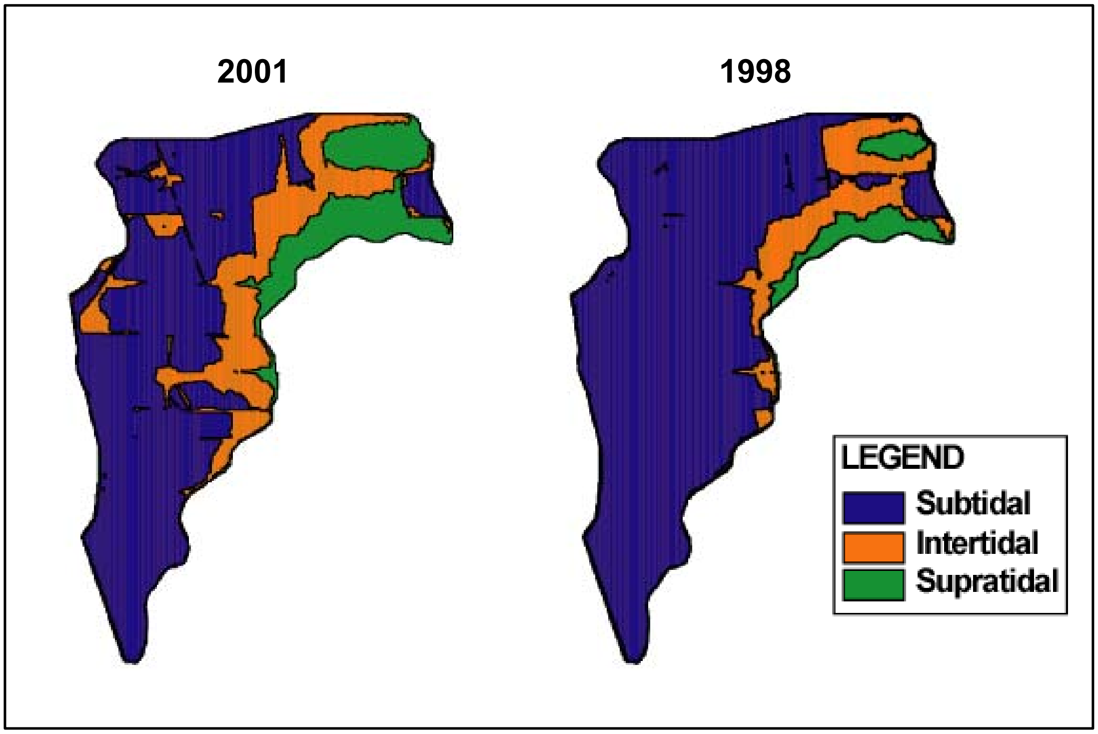


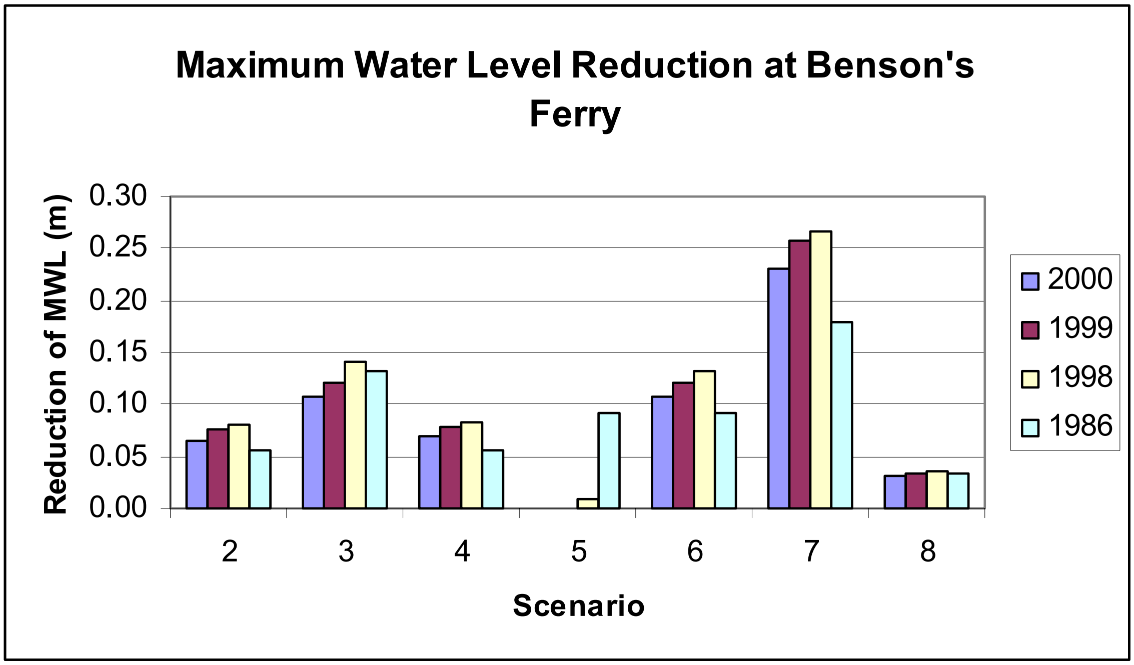
Figure 13. Areal extent of subtidal, interitdal, and supratidal habitat zones calculated with MLLW and MHHW values calculated from a four month (May through August) period of 1998 and 2001. Note the difference in areal extent and location of each of the habitat zones between the two years.

Scenario 2 was chosen to demonstrate the impact of the use of these different values with respect to areal habitat extent. Figure 13 shows the subtidal, intertidal and supratidal extent for each of these tidal conditions. Areal extent of each of the habitat zones changes dramatically from the wet condition to the dry condition. In the wet condition, the summer of 1998, scenario 2 provides 508-ha of subtidal, 88-ha of intertidal, and 44-ha of supratidal habitat. In contrast, the dry condition, 2001, provides 399-ha of subtidal, 146-ha of intertidal, and 95-ha of supratidal habitat. Dramatically different results are obtained depending on the period used when calculating the tidal characteristic values.

Therefore it is important to realize that the habitat extent values discussed below are for comparison, and are unlikely to represent the actual extent or range of various habitat types.

Scenario Simulations

Seven alternative scenarios (scenarios 2 through 8) are evaluated from two perspectives, the impact upon the hydraulics of the local region, and the potential for the restoration/creation of habitat. In addition several scenarios (all but 1 and 6) were also analyzed with the inclusion of a tower levee as discussed previously and shown in Figure 12. The hydraulic evaluation investigates the impact of each scenario upon water levels during flood pulses, with maximum water levels compared to the results from scenario 1, the as-is or no-project condition. The results of this aspect of the study are presented in Figure 14 and Tables 9 through 12 for each of the four periods modeled at both Benson's Ferry and New Hope. In addition to peak water levels, each table includes the difference between each scenario and scenario 1, the percent reduction of each peak at each location, and a ranking of each scenario based on the normalized reduction. For all scenarios the maximum water level at Benson's Ferry, is unaltered or reduced in every period studied. Scenarios 6 and 7 in the 1986 flood simulation yielded slightly higher water levels (6 cm and 4 cm respectively) at New Hope than the as-is or no-project condition (scenario 1).



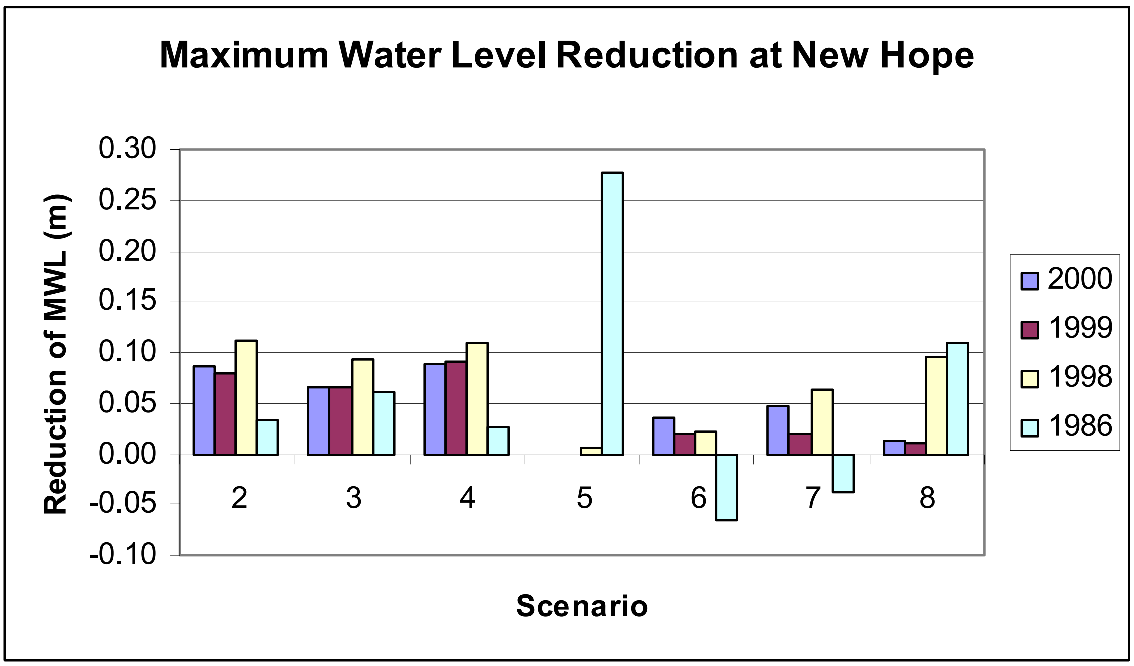


Figure 14. Maximum water level (MWL) reduction values for each of the scenarios simulated. Scenario 5 only reduces peak values during the 1998 and 1986 events; scenarios 6 and 7 increase the maximum water level at New Hope during the 1986 model simulations.

Table 9. Simulation results showing maximum water level (MWL), change of maximum water level from base case, percent change and normalized change values for each scenario and period studied at Benson's Ferry.

Benson' Ferry

Scenario	**1-BASE**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
2000								
MWL (m)	3.74	3.67	3.63	3.67	3.74	3.63	3.51	3.70
Change from Base[1]	0.00	-0.07	-0.11	-0.07	0.00	-0.11	-0.23	-0.03
Percent Change[2]	0.00	-1.80	-2.95	-1.88	0.00	-2.98	-6.56	-0.84
Normalized Change[3]	0.00	0.29	0.47	0.30	0.00	0.47	1.00	0.13
1999								
MWL (m)	4.01	3.93	3.89	3.93	4.01	3.89	3.75	3.97
Change from Base[1]	0.00	-0.08	-0.12	-0.08	0.00	-0.12	-0.26	-0.03
Percent Change[2]	0.00	-1.91	-3.09	-2.01	0.00	-3.11	-6.86	-0.83
Normalized Change[3]	0.00	0.29	0.47	0.31	0.00	0.47	1.00	0.13
1998								
MWL (m)	4.66	4.58	4.52	4.57	4.65	4.52	4.39	4.62
Change from Base[1]	0.00	-0.08	-0.14	-0.08	-0.01	-0.13	-0.27	-0.04
Percent Change[2]	0.00	-1.77	-3.15	-1.81	-0.17	-2.94	-6.06	-0.76
Normalized Change[3]	0.00	0.30	0.53	0.31	0.03	0.50	1.00	0.13
1986								
MWL (m)	5.71	5.65	5.58	5.65	5.62	5.61	5.53	5.67
Change from Base[1]	0.00	-0.06	-0.13	-0.06	-0.09	-0.09	-0.18	-0.03
Percent Change[2]	0.00	-0.97	-2.35	-0.97	-1.62	-1.64	-3.22	-0.60
Normalized Change[3]	0.00	0.31	0.74	0.31	0.51	0.52	1.00	0.19

Notes:

1) Calculated by $d = WL_{S1_{\max}} - WL_{Si_{\max}}$
2) Calculated by $\dfrac{WL_{S1_{\max}} - WL_{Si_{\max}}}{WL_{S1_{\max}}} * 100$
3) Calculated by $\dfrac{WL_{S1_{\max}} - WL_{Si_{\max}}}{d_{\min}}$

Table 10. Simulation results showing maximum water level (MWL), change of maximum water level from base case, percent change and normalized change values for each scenario and period studied at New Hope.

New Hope

Scenario:	**1-BASE**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
2000								
MWL (m)	1.96	1.87	1.89	1.87	1.96	1.92	1.91	1.94
Change from Base[1]	0.00	-0.09	-0.07	-0.09	0.00	-0.04	-0.05	-0.01
Percent Change[2]	0.00	-4.60	-3.44	-4.71	0.00	-1.88	-2.41	-0.62
Normalized Change[3]	0.00	0.98	0.74	1.00	0.00	0.41	0.52	0.14
1999								
MWL (m)	1.85	1.77	1.78	1.76	1.85	1.83	1.83	1.84
Change from Base[1]	0.00	-0.08	-0.07	-0.09	0.00	-0.02	-0.02	-0.01
Percent Change[2]	0.00	-4.52	-3.70	-5.11	0.00	-1.04	-1.09	-0.54
Normalized Change[3]	0.00	0.89	0.73	1.00	0.00	0.21	0.22	0.11
1998								
MWL (m)	2.57	2.46	2.48	2.46	2.57	2.55	2.51	2.48
Change from Base[1]	0.00	-0.11	-0.09	-0.11	-0.01	-0.02	-0.06	-0.10
Percent Change[2]	0.00	-4.55	-3.75	-4.43	-0.23	-0.86	-2.51	-3.84
Normalized Change[3]	0.00	1.00	0.83	0.97	0.05	0.20	0.56	0.85
1986								
MWL (m)	3.84	3.81	3.78	3.82	3.57	3.91	3.88	3.73
Change from Base[1]	0.00	-0.03	-0.06	-0.03	-0.28	0.06	0.04	-0.11
Percent Change[2]	0.00	-0.89	-1.64	-0.71	-7.74	1.66	1.00	-2.95
Normalized Change[3]	0.00	0.12	0.22	0.10	1.00	-0.24	-0.14	0.40

Notes:

1) Calculated by $d = WL_{S1_{\max}} - WL_{Si_{\max}}$

2) Calculated by $\dfrac{WL_{S1_{\max}} - WL_{Si_{\max}}}{WL_{S1_{\max}}} * 100$

3) Calculated by $\dfrac{WL_{S1_{\max}} - WL_{Si_{\max}}}{d_{\min}}$

Table 11. Tower levee case simulation results showing maximum water level (MWL), change of maximum water level from base case, percent change and normalized change values for each scenario and period studied at Benson's Ferry. Scenario 6 was not evaluated with a tower levee.

Benson' Ferry

Scenario	**1-BASE**	**2T**	**3T**	**4T**	**5T**	**6T**	**7T**	**8T**
2000								
MWL (m)	3.74	3.67	3.63	3.67	3.74	-	3.51	3.70
Change from Base[1]	0.00	-0.07	-0.11	-0.07	0.00	-	-0.23	-0.03
Percent Change[2]	0.00	-1.80	-2.95	-1.88	0.00	-	-6.53	-0.84
Normalized Change[3]	0.00	0.29	0.47	0.30	0.00	-	1.00	0.14
1999								
MWL (m)	4.01	3.93	3.89	3.93	4.01	-	3.75	3.97
Change from Base[1]	0.00	-0.08	-0.12	-0.08	0.00	-	-0.26	-0.03
Percent Change[2]	0.00	-1.91	-3.09	-1.99	0.00	-	-6.83	-0.83
Normalized Change[3]	0.00	0.29	0.47	0.30	0.00	-	1.00	0.13
1998								
MWL (m)	4.66	4.58	4.52	4.58	4.65	-	4.40	4.62
Change from Base[1]	0.00	-0.08	-0.14	-0.08	-0.01	-	-0.26	-0.04
Percent Change[2]	0.00	-1.75	-3.12	-1.79	-0.17	-	-5.96	-0.76
Normalized Change[3]	0.00	0.31	0.54	0.31	0.03	-	1.00	0.13
1986								
MWL (m)	5.71	5.65	5.58	5.65	5.62	-	5.53	5.68
Change from Base[1]	0.00	-0.05	-0.13	-0.05	-0.09	-	-0.18	-0.03
Percent Change[2]	0.00	-0.94	-2.29	-0.94	-1.62	-	-3.16	-0.49
Normalized Change[3]	0.00	0.30	0.73	0.30	0.52	-	1.00	0.16

Notes:

1) Calculated by $d = WL_{S1_{\max}} - WL_{Si_{\max}}$

2) Calculated by $\dfrac{WL_{S1_{\max}} - WL_{Si_{\max}}}{WL_{S1_{\max}}} * 100$

3) Calculated by $\dfrac{WL_{S1_{\max}} - WL_{Si_{\max}}}{d_{\min}}$

Table 12. Tower levee case simulation results showing maximum water level (MWL), change of maximum water level from base case, percent change and normalized change values for each scenario and period studied at New Hope. Scenario 6 was not evaluated with a tower levee.

New Hope

Scenario:	**1-BASE**	**2T**	**3T**	**4T**	**5T**	**6T**	**7T**	**8T**
2000								
MWL (m)	1.96	1.87	1.90	1.87	1.96	-	1.91	1.94
Change from Base[1]	0.00	-0.08	-0.06	-0.08	0.00	-	-0.04	-0.01
Percent Change[2]	0.00	-4.32	-3.17	-4.43	0.00	-	-2.25	-0.62
Normalized Change[3]	0.00	0.98	0.72	1.00	0.00	-	0.52	0.14
1999								
MWL (m)	1.85	1.78	1.79	1.77	1.85	-	1.84	1.84
Change from Base[1]	0.00	-0.07	-0.06	-0.08	0.00	-	-0.02	-0.01
Percent Change[2]	0.00	-4.17	-3.35	-4.76	0.00	-	-0.82	-0.54
Normalized Change[3]	0.00	0.88	0.71	1.00	0.00	-	0.18	0.12
1998								
MWL (m)	2.57	2.47	2.49	2.47	2.57	-	2.51	2.48
Change from Base[1]	0.00	-0.10	-0.08	-0.10	-0.01	-	-0.06	-0.10
Percent Change[2]	0.00	-4.13	-3.29	-4.05	-0.23	-	-2.43	-3.84
Normalized Change[3]	0.00	1.00	0.80	0.98	0.06	-	0.60	0.93
1986								
MWL (m)	3.84	3.82	3.79	3.83	3.57	-	3.89	3.75
Change from Base[1]	0.00	-0.02	-0.05	-0.02	-0.28	-	0.05	-0.09
Percent Change[2]	0.00	-0.65	-1.37	-0.50	-7.74	-	1.21	-2.40
Normalized Change[3]	0.00	0.09	0.19	0.07	1.00	-	-0.17	0.33

Notes:

1) Calculated by $d = WL_{S1_{\max}} - WL_{Si_{\max}}$

2) Calculated by $\dfrac{WL_{S1_{\max}} - WL_{Si_{\max}}}{WL_{S1_{\max}}} * 100$

3) Calculated by $\dfrac{WL_{S1_{\max}} - WL_{Si_{\max}}}{d_{\min}}$

To quantify the restoration potential of each scenario, the areal extent of various habitat types (subtidal, intertidal and supratidal or floodplain) is quantified and provided in Figure 15 and Table 13. These values are based upon representative reference values for MHHW and MLLW (Table 7) and the habitat definitions described previously. Inundation plots are provided in Figures 16 through 19 for scenario 2 as examples, and for all the scenarios in Appendix C. The distribution of water depth and aerial extent within each depth class are also provided for each scenario in Appendix C. In addition the volume of water exchanged through the breaches over the two flood tides of a spring tidal cycle is quantified and presented graphically in Figure 20.

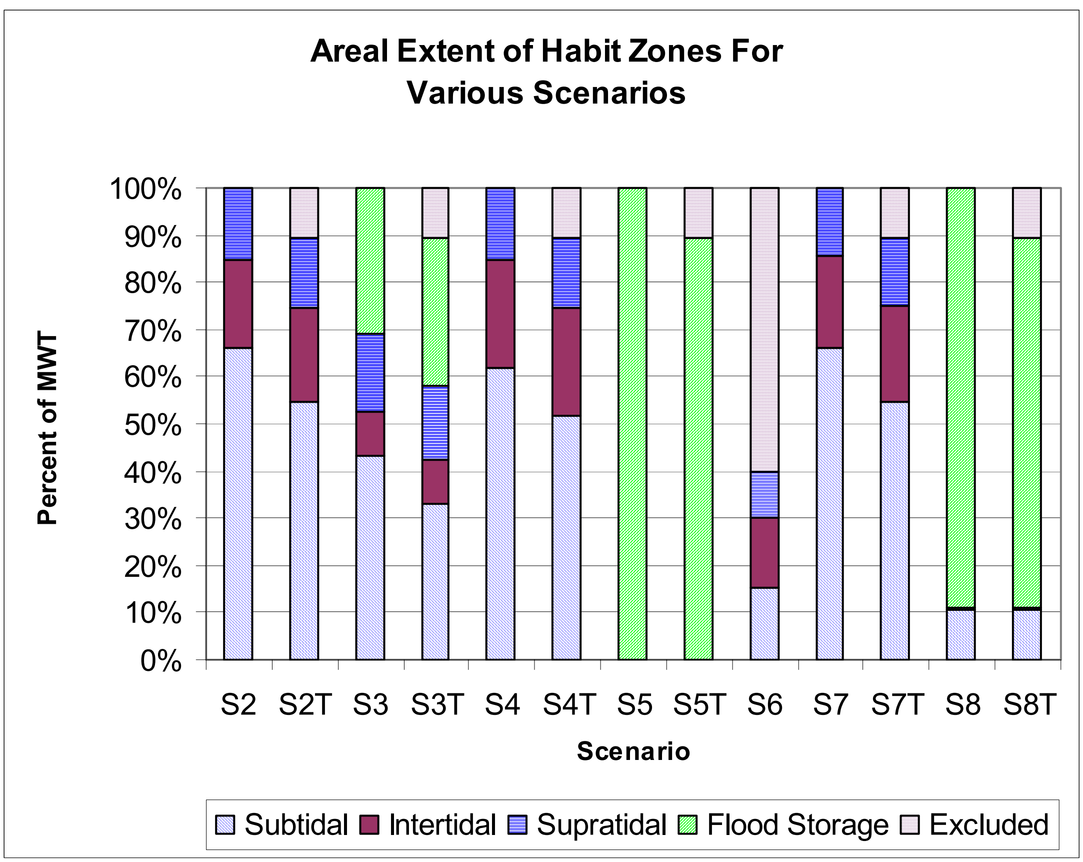


Figure 15. Areal extent of subtidal, intertidal, supratidal, flood storage and excluded, zones for each scenario considered.

Table 13. Areal extent in hectares of each of habitat zone created by each scenario.

Scenario	Subtidal	Intertidal	Supratidal	Flood Storage	Excluded
2	424	119	97	0	0
2 w/ tower levee	349	128	95	0	68
3	277	60	104	199	0
3 w/ tower levee	211	60	102	199	68
4	395	147	98	0	0
4 w/ tower levee	330	147	94	0	68
5	0	0	0	640	0
5 w/ tower levee	0	0	0	572	68
6	98	94	63	0	386
6 w/ tower levee	-	-	-	-	-
7	424	123	93	0	0
7 w/ tower levee	349	131	91	0	68
8	66	3	2	569	0
8 w/ tower levee	66	3	2	501	68

Scenario 2

Results from the scenario 2 simulations indicate that this management configuration provides benefits in both habitat enhancement and flood mitigation. Based upon inundation at the representative tides discussed previously, this scenario provides 97-ha of supratidal or seasonally inundated habitat, 119-ha of intertidal habitat and 424-ha of subtidal habitat. An inclusion of the tower levee to the habitat analysis reduces the amount of subtidal habitat to 349-ha. Figures 16 and 17 show the extent of tidal inundation for MLLW and MHHW respectively. Figures 18 and 19 show the extent of tidal inundation for MLLW and MHHW for scenario 2 with the inclusion of the tower levee. Refer to Table 13 for further details regarding habitat extent, and Appendix C for more detailed inundation statistics. Over a spring tidal cycle this scenario exchanges 16,080 cms with the surrounding network as shown in Figure 20.

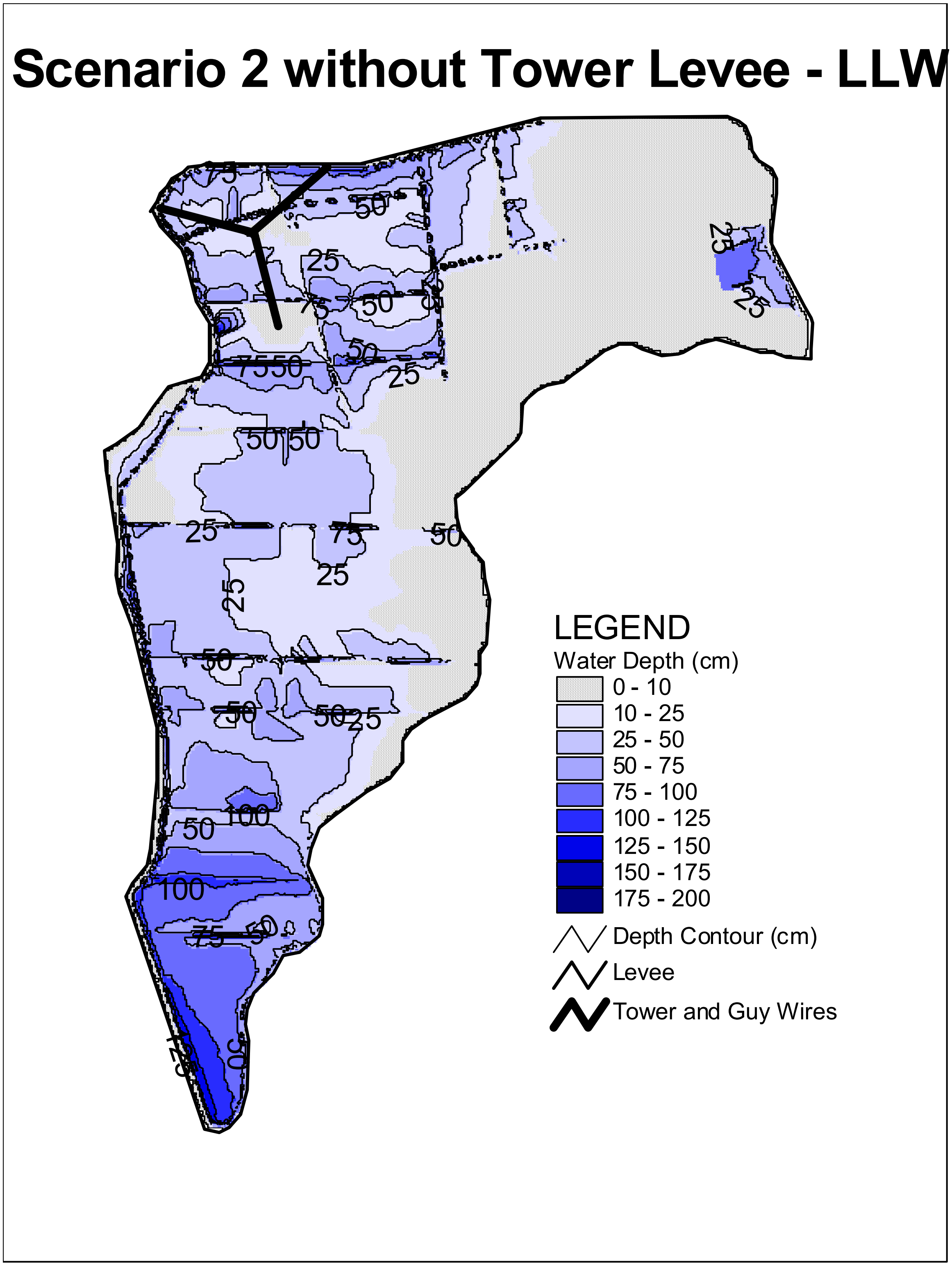


Figure 16. Inundation map of the McCormack-Williamson Tract at lower low water (LLW) based upon the model results for scenario 2 without the inclusion of the tower levee. Varying intensities of shading represent varying magnitudes of water depth (cm).

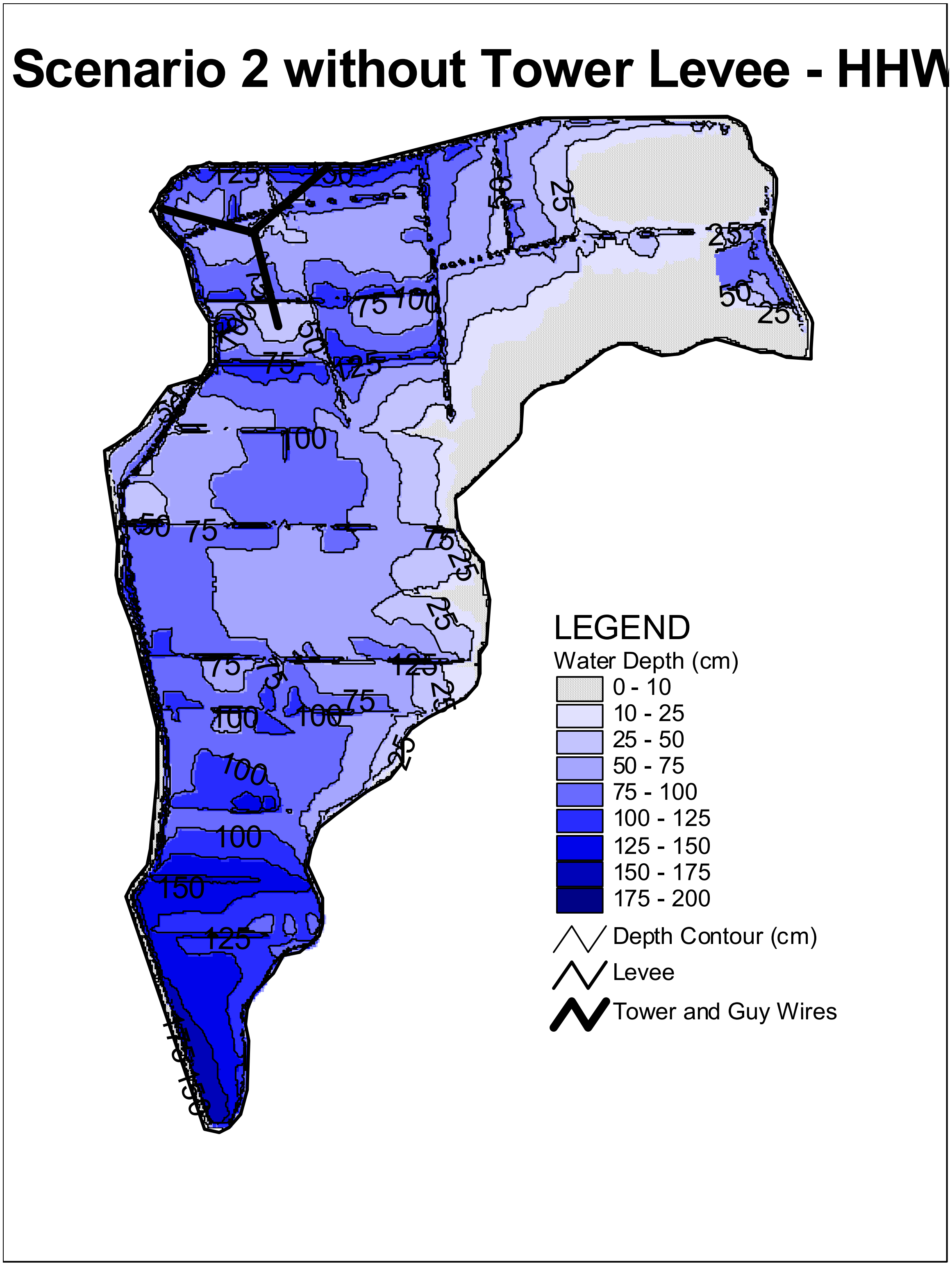


Figure 17. Inundation map of the McCormack-Williamson Tract at higher high water (HHW) based upon the model results for scenario 2 without the inclusion of the tower levee. Varying intensities of shading represent varying magnitudes of water depth (cm).

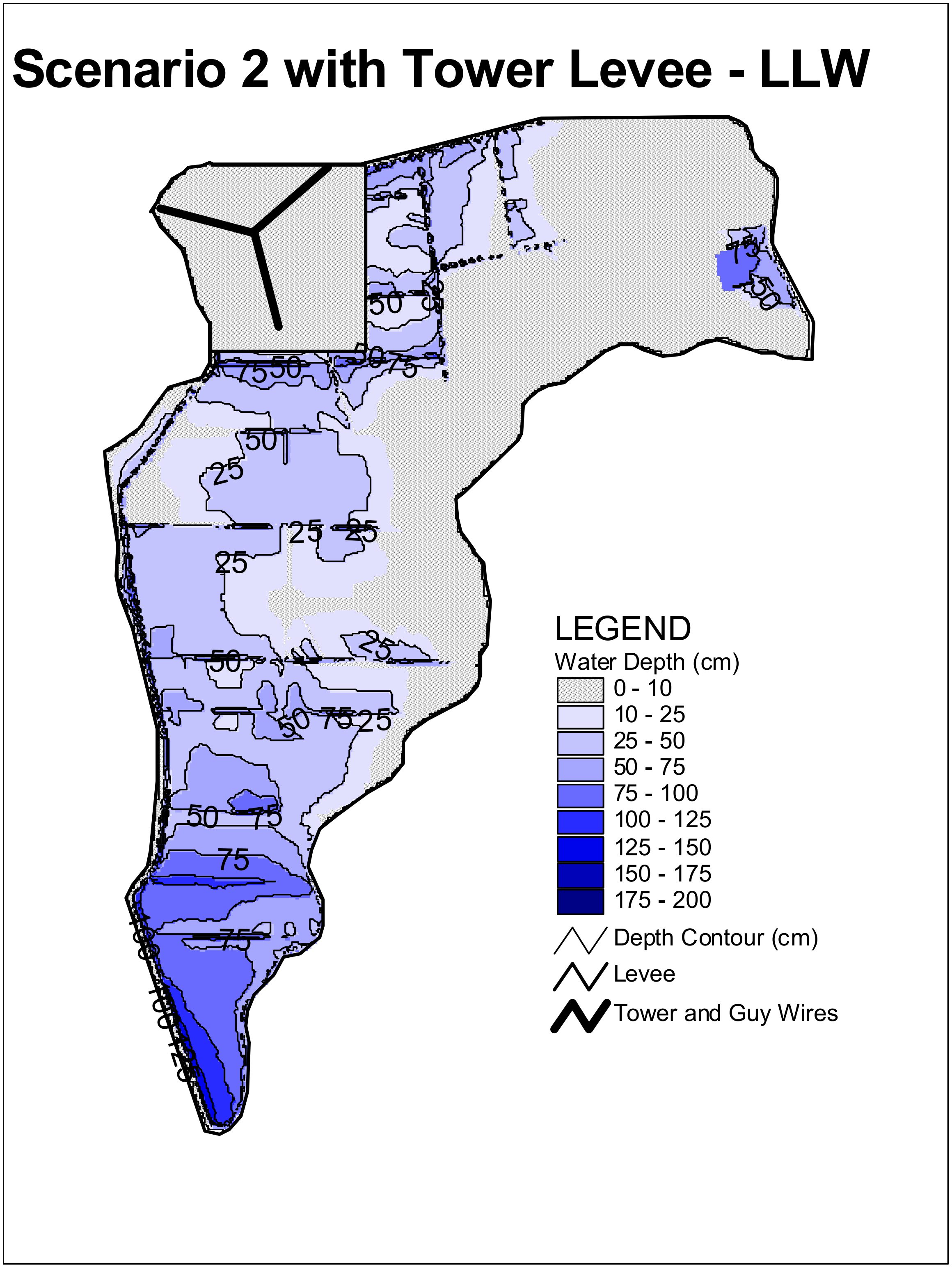


Figure 18. Inundation map of the McCormack-Williamson Tract at lower low water (LLW) based upon the model results for scenario 2 with the inclusion of the tower levee. Varying intensities of shading represent varying magnitudes of water depth (cm).

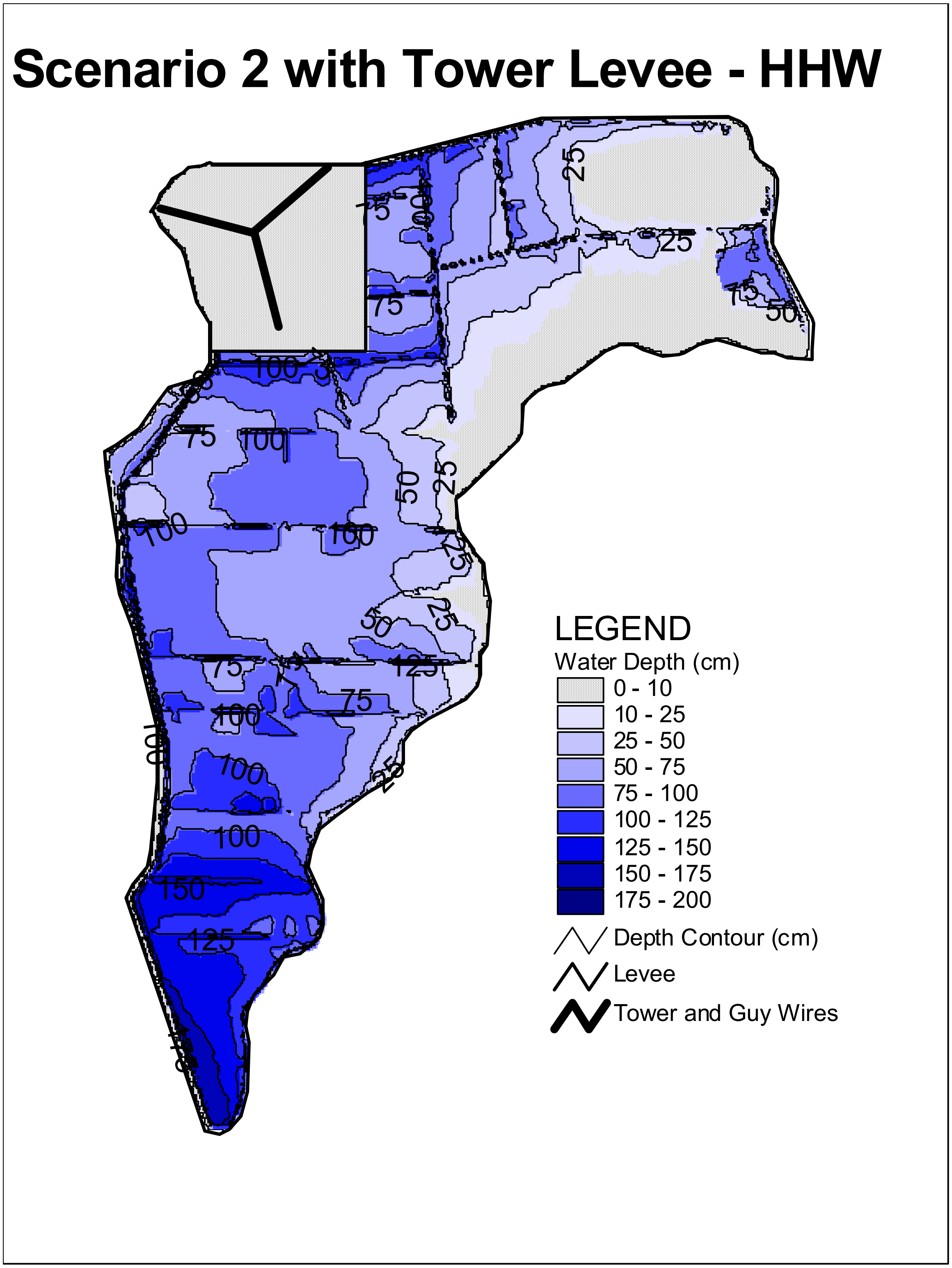


Figure 19. Inundation map of the McCormack-Williamson Tract at higher high water (HHW) based upon the model results for scenario 2 with the inclusion of the tower levee. Varying intensities of shading represent varying magnitudes of water depth (cm).

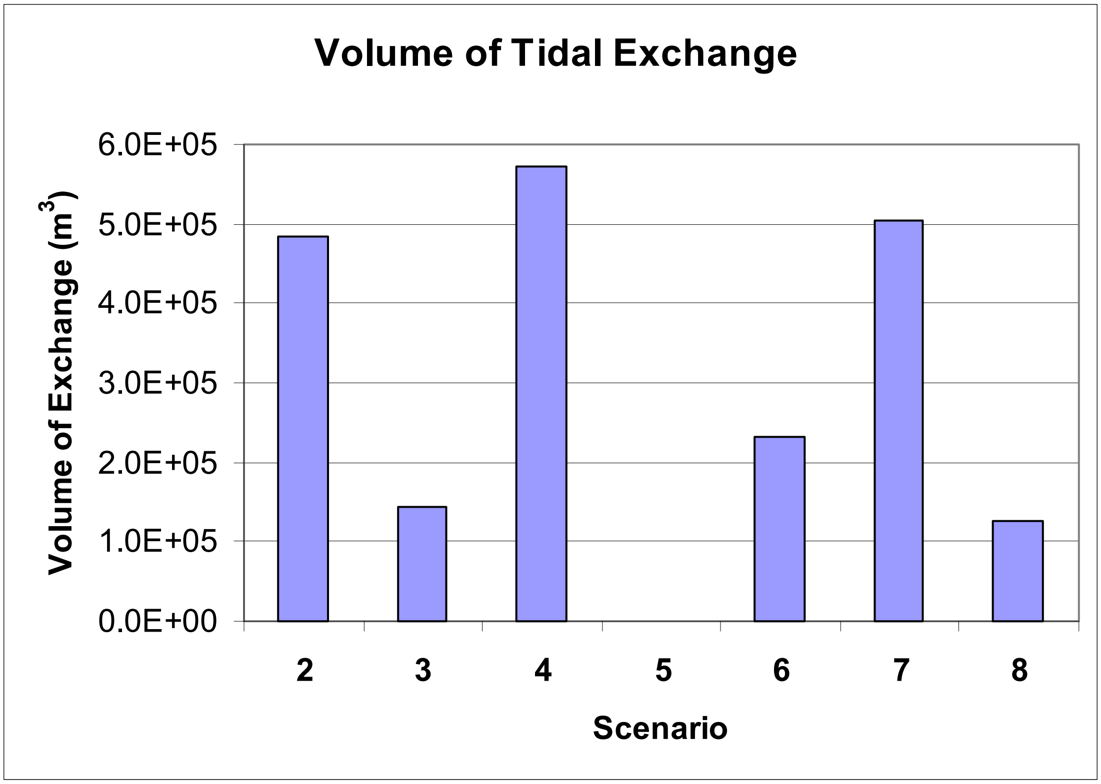


Figure 20. Comparison of volume of water exchanged with surrounding network during the two flood tides of a spring tidal cycle.

From a flood control perspective, Scenario 2 reduces maximum water levels (as compared to the as-is condition, Scenario 1) for all periods simulated. Stage reductions at Benson's Ferry (Table 9) range from 6 to 8 cm and from 3 to 11 cm at New Hope (Table 10). Reductions to the 1986 water levels were the smallest of the four periods simulated. The tower levee case simulation shows little difference in maximum water level reduction (Tables 11 and 12). Breach discharge results (Appendix C) show that breaches B1 and B2 act as inlets during flood pulses and breaches B3 and B4 acts as outlets. Further inspection shows that discharge through breach B1 only occurs during higher discharge periods.

Scenario 3

Simulation results and the GIS habitat analysis demonstrate that this scenario provides habitat enhancement and flood reduction in conjunction with continued agriculture on the southern portion (~200-ha) of the tract. This scenario without the tower levee provides 104-ha of supratidal, 60- ha of intertidal, and 277-ha subtidal habitat. Consideration of the tower levee changes the supratidal and intertidal values little but reduces the subtidal region to 211-ha. Over a spring tidal cycle this scenario exchanges 4,760 cms with the surrounding network.

During flood pulses, scenario 3 reduced peak water levels by 11 to 14 cm at Benson's Ferry and 6 to 9 cm at New Hope. The low cross levee (2.7 m) which partitions the tract is only overtopped during the 1986 pulse with discharge over the levee peaking near 275 cms. Breaches B1, B2 and B3 only convey flow during elevated water levels; acting as inlets, while breach B4 conveys a majority of the tidal exchange, in addition to acting as the outlet during flooding pulses. Inclusion of the tower levee does not alter flood reduction values at Benson's Ferry and only slightly (0.01 m) decreases the New Hope reductions observed in the non-tower levee condition.

Scenario 4

Scenario 4 provides the potential for 98-ha of supratidal, 147-ha of intertidal and 395-ha of subtidal habitat. While scenario 4 is very similar to scenario 2 (scenario 4 includes one 300 m breach along Snodgrass Slough not present in scenario 2) it provides 28-ha more intertidal habitat and 29-ha less subtidal habitat. An explanation for these changes lies in the ability of the breaches to convey tidal floodwaters (Figure 20). Over a spring tidal cycle this scenario exchanges 19,045 cms with the surrounding network. With a greater ability to exchange water with the surrounding hydraulic network and therefore drain the tract more effectively, LLW levels throughout the tract are lower than scenario 2, yielding a reduced subtidal region.

From a flood perspective, scenario 4 closely mimics the results of scenario 2 with flood reductions ranging from 6 to 8 cm at Benson's Ferry and 3 to 11 cm at New Hope. The breaches operate in a similar manner to that observed in Scenario 2 with breaches B1, B2 and B3 acting as inlets and breaches B4 and B5 acting as outlets. During the peak of the 1986 pulse, breach B3 acts as an outlet rather than an inlet as simulated in smaller events. Breach B1 only conveys flow during flooding events, and breaches B3, B4 and B5 convey the majority of tidal flows. For further details regarding the hydrodynamic or inundation results of this scenario refer to Appendix C.

Scenario 5

In this scenario the McCormack-Williamson Tract remains dry in most conditions with floodwaters accessing the weirs only in 1998 and 1986. Both weirs convey flow into the tract during the peak of the 1998 event, reducing maximum water levels by 0.01 m at both locations. In the 1986 event flood water levels were reduced by 0.09 m at Benson's Ferry and by 0.28 m at New Hope. In this pulse, discharge through the upper weir (W1) peaked below 130 cms, with the lower weir (W2) first acting as an inlet to the tract, and later conveying flow out of the tract. This scenario provides no habitat benefit beyond that offered through wildlife friendly farming practices, and no tidal exchange. Refer to Appendix C for more details regarding the results of the hydrodynamic simulations.

Scenario 6

The scenario is composed of a setback levee creating a 500 m wide zone along the Mokelumne River accessed by five breaches. Within this setback zone, model results suggest the potential for 63-ha of supratidal, 94-ha of intertidal and 98-ha of subtidal habitat. Over a spring tidal cycle this scenario exchanges 7,830 cms with the surrounding network. This configuration does reduce water levels during all years at Benson's Ferry, however it increases water levels by 6 cm at New Hope during the 1986 event, but reduces water levels in all other flood pulses. The tower levee condition was not applied to this scenario because the setback levee was assumed not to overtop.

Scenario 7

The entire levee along the Mokelumne River is removed in this scenario resulting in 93-ha of supratidal, 123-ha of intertidal and 424-ha of subtidal habitat. An inclusion of the tower levee into the analysis yields 91-ha of supratidal, 131-ha of intertidal and 349-ha of subtidal habitat. Over a spring tidal cycle this scenario exchanges 16,843 cms with the surrounding network. When compared at Benson's Ferry, scenario 7 ranks highest for each period simulated, reducing water levels from 18 to 27 cm with the largest reduction seen in 1998 and the smallest in 1986. However at New Hope, this scenario reduces maximum water levels for the three smaller storms (2000, 1999, 1998), but raises stage by 4 cm in the 1986 simulation. Incorporation of the tower levee to the scenario yields very little change to the peak reduction values at both locations.

Scenario 8

In this scenario a 70-ha region in the lower McCormack-Williamson Tract is open to tidal inundation with a 2.4 m high east-west trending levee isolating the northern portion of the tract (Figure 12). This levee is overtopped and the flood storage utilized in both 1998 and 1986 reducing maximum water levels at New Hope by a tenth of a meter. During the large pulse of 1998, floodwaters inundated all areas of the tract below one meter in elevation. All three breaches convey tidal flows, while a majority of tidal exchange occurs through breaches B2 and B3. Due to the depth of the tidally inundated area, supratidal and intertidal habitat is limited to the perimeter of the tract where levees elevate the ground surface, with a majority of this region (66.5 ha) providing solely subtidal habitat. While the portion retained north of the low levee would provide some floodplain habitat in larger less frequent events (10 yr + recurrence interval storms) little seasonally inundated floodplain habitat is offered. Over a spring tidal cycle this scenario exchanges 4,200 cms with the surrounding network as shown in Figure 20.

Scenario Summary

A comparison of the potential habitat extent, flood mitigation benefits, and tidal exchange of the scenarios is instructive and is provided graphically in Figures 14, 15 and 20. When compared to other scenarios analyzed, scenario 3 provides the potential for the largest area of supratidal habitat at 104-ha (16 % of the tract area), while scenario 4 provides the largest area (147=ha, 23% of the tract area) of intertidal habitat. Scenarios 2 and 7, tie for top ranking in the subtidal habitat category, with both yielding 424-ha (66 % of the tract area) of this habitat type. When considered with the tower levee, these rankings remain unchanged, however the areal extent values for the supratidal and subtidal habitat zones do change. When ranked by tidal exchange, scenario 4 provides the largest volume of water exchanged (19,045 cms) with scenarios 7 and 2 following (16,843 cms and 16,080 cms respectively).

Scenario 7 provides the largest flood peak reduction when observed at Benson's Ferry for all years, however water level reduction at New Hope is not as straightforward. Scenario 5 reduces flood levels at new Hope by the largest amount in 1986, however it has little to no effect in the smaller magnitude storms. Scenario 8 reduces water levels in 1998 and 1986, when the low cross levee is overtopped, however has little effect in smaller magnitude, more frequent events (1999, 2000). Scenarios 2, 3 and 4 all provide flood peak reduction for all years, with the maximum benefit observed in 1998 for all three scenarios.

General Summary

The results of this study suggest that scenarios, which encourage exchange with the surrounding hydraulic network, achieve a larger intertidal habitat zone (scenarios 2, 4, and 7). Configurations with more limited tidal exchange (scenario 3) show reduced amounts of this habitat type. While scenario 7 would appear to provide the largest amount of tidal exchange, due to the removal of the Mokelumne River levee, the natural levee and elevated land surface along this side of the tract limit tidal exchange. An inspection of Figure 20 illustrates that scenario 4 with breaches located along Dead Horse Cut and Snodgrass Slough provides a larger amount of tidal exchange with a smaller amount of levee removal. The extent of the supratidal zone does vary, however differences are small between scenarios, with major reductions only observed when portions of the tract in a suitable elevation range for this habitat zone are excluded from annual inundation, as in scenarios 6 and 8. Subtidal habitat is easily obtained, and model simulation results show that its areal extent can be reduced through the careful placement of low levees isolating regions of the tract from frequent (daily) inundation (scenario 3, 6, and tower levee cases).

Flood benefits vary, based upon the location of interest. Maximum stage reductions at Benson's Ferry are gained by configurations with levee breaches at locations further upstream along the Mokelumne River (scenarios 3, 6 and 7), and maximum results achieved with larger breaches in these locations (scenario 7). Presumably breaches located further upstream act to reduce the "bottleneck" effect, which the tract levees create. Scenarios that maximize stage reductions at Benson's Ferry, show smaller reductions at New Hope, and in some cases actually increase water level maximum values (scenarios 6 and 7), although these increases are small. Scenarios, which reserve a portion of the tract for flood storage (3, 5 and 8), do provide flood mitigation, however the peak reductions only occur in larger magnitude events. Reduction values for scenario 3 at New Hope are less than those achieved with scenario 4 when compared across the full range of flows simulated in this study (Figure 14).

DISCUSSION AND CONCLUSIONS

The application of the Cosumnes-Mokelumne-North Delta Model to the investigation of ecological and flood mitigation benefit of various McCormack-Williamson Tract management options provides meaningful insight into the complex hydraulic nature of the northern Sacramento San Joaquin Delta, in addition to the opportunities for habitat enhancement in this region. Hydrodynamic model results demonstrate that the model is able to simulate the hydraulics of this complex network. While model results in most cases agree well with observed gage data, caution should be used in interpreting the results of the study. This study is not meant to be predictive. Rather it is intended to show trends, and provide a basis for the evaluation of habitat potential and flood mitigation benefit, and allow comparison between various scenario configurations, across flood events of varying magnitudes.

The habitat zone methodology utilized allows the comparison of the various scenarios based upon the potential range of various habitat zones; yet again caution should be used when applying the results in a predictive manner. The areal extent values calculated in this analysis are based upon mean values, which represent average conditions for the period that the statistics summarize. The resulting distribution of habitat types will rely upon actual tidal water levels that vary significantly based upon inter-seasonal and inter-annual variations in regional hydrologic conditions, in addition to water resource facility operations. In addition, the results rely upon the output from a one-dimensional model, which makes a number of assumptions to simplify the hydrodynamics of the problem.

Water movement through the McCormack-Williamson Tract will certainly not be one-dimensional, with the resulting inundation pattern responding in a more complex manner than described by this study.

Model results suggest that restoring tidal freshwater marsh habitat within the McCormack-Williamson Tract would have minimal impacts upon flood stages in a variety of flood magnitudes including rare, large events. Each scenario, except for scenario 5, provides the potential for a mosaic of habitat types, as well as providing some flood mitigation benefit in at least one of the time periods studied. Which scenario is the best depends on the objectives of the combined flood control and habitat enhancement effort. Not surprisingly, the optimum management configuration is different depending upon the desired result. Management decisions and further research reached regarding the benefit, or lack thereof in the creation of subtidal-shallow water habitat, are needed to determine whether it is necessary to partition the tract reducing the potential area of subtidal habitat.

An important result of this study is that in many cases restoration and flood mitigation can be compatible. Most scenarios evaluated in this study provided positive values for ecological and flood control benefits through the range of flood magnitudes investigated. Breach width, depth, and location will exert a large control upon the regional hydraulics, as well as the resulting inundation pattern upon the tract, and therefore the areal extent of the different habitat types. Furthermore, many factors in addition to the flooding frequency, will contribute to the development of the various habitat types in a restored

McCormack-Williamson Tract. These include sediment deposition and bio-geomorphic evolution of the tract, seed sources of both desired and exotic vegetation types, founder effects, and variations in hydrologic conditions and water resource management.

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