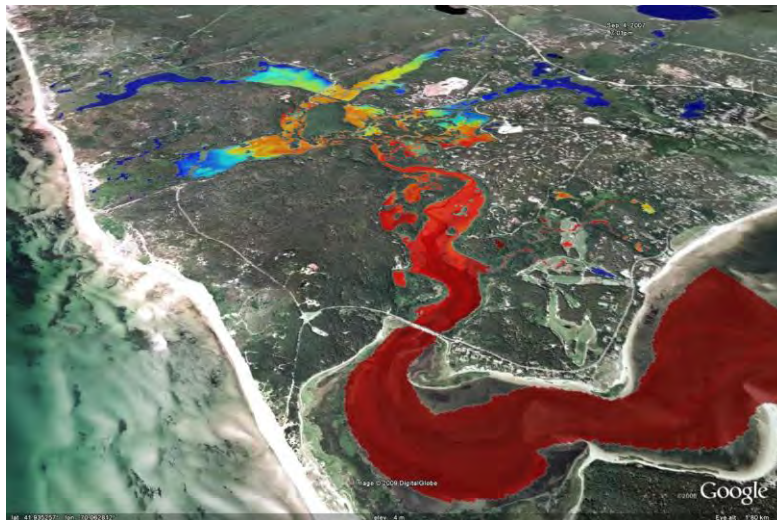


Herring River Hydrodynamic Modeling

Draft Modeling Report



Prepared For:

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June 2009

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DRAFT FINAL REPORT

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1.0 INTRODUCTION

The Herring River is the largest tidal river and estuary complex on Outer Cape Cod (Figure 1-1). It is a complex estuarine system that is forced by sea level fluctuations through a primary inlet connecting Herring River to Wellfleet Harbor. The inlet consists of a compound dike system at the Chequessett Neck Road crossing (Figure 1-2). The existing dike system consists of three 6-foot wide box culverts, each with an attached flow control structure. One culvert has an adjustable sluice gate, which is currently set to be partially open at 2 feet and allows bi-directional tidal flow. The remaining two culverts have tidal flap gates, which are designed to permit flow only during an ebbing (outgoing) tide. Therefore, limited seawater is allowed into the Herring River system. Subsequent constrictions exist at upstream crossings (e.g., High Toss Road, Pole Dike Road, Pamet Point Road, Route 6, etc.) throughout the system. The system is further complicated by the extensive existing and potential marsh plains, tributaries, significant area, surrounding infrastructure, and varying and multiple natural and anthropogenic constrictions that make up the Herring River system.

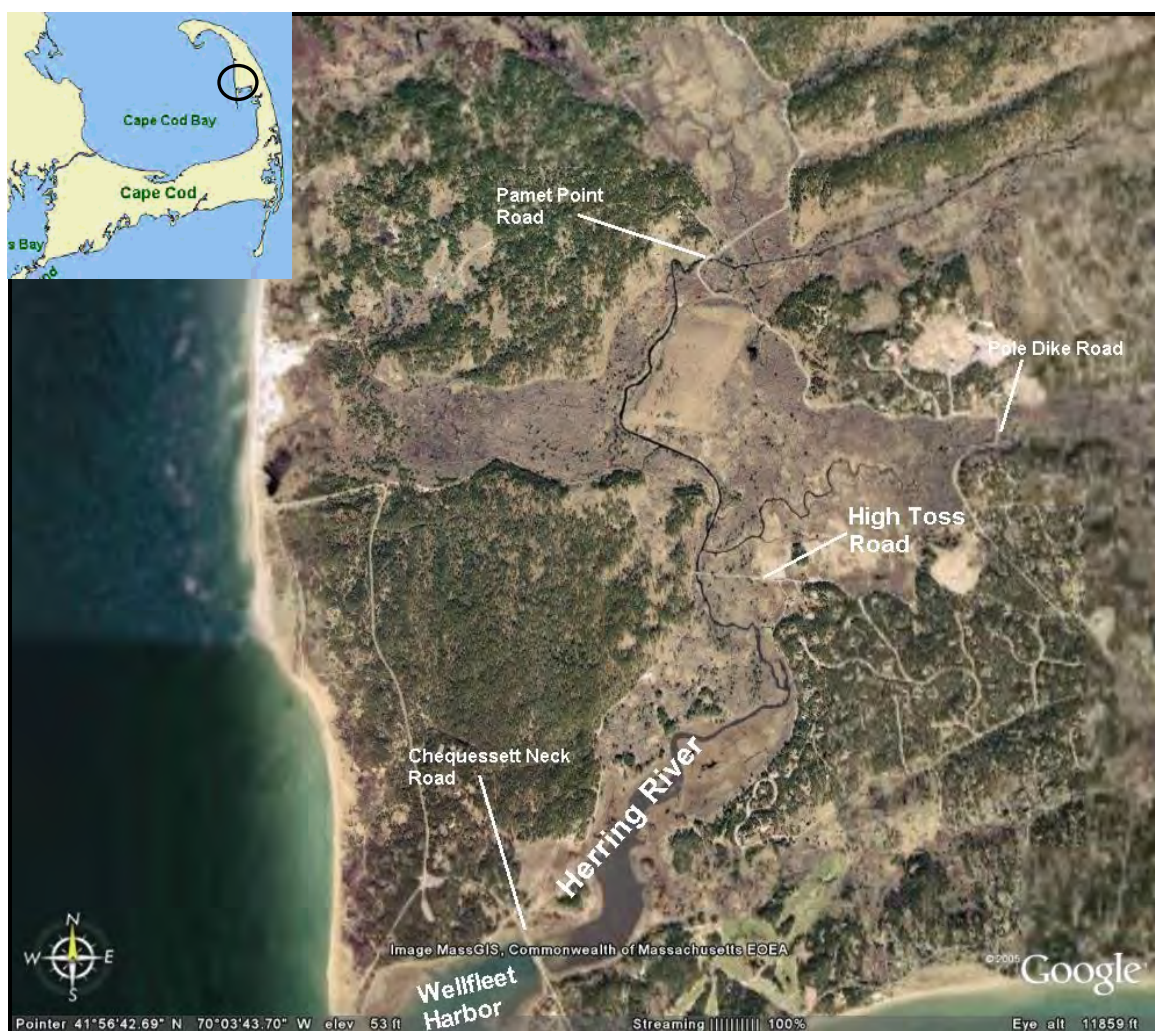


Figure 1-1. Herring River system (background image courtesy of Google Earth).



Figure 1-2. Dike structure at the Chequessett Neck Road crossing.

Prior to the dike construction in 1909, Herring River was connected to Wellfleet Harbor through a natural inlet at Chequessett Neck. The marsh system consisted of nearly 1,100 acres of thriving coastal wetlands, including a productive herring run, shellfishery, and salt marsh habitats. The dike construction, intended to control mosquitoes and create additional developable land area, significantly degraded the natural marsh ecosystem. Today, after 100 years of influence, as well as numerous other anthropogenic impacts (e.g., upstream culverts, railroad crossings, ditch creation, etc.), hundreds of acres of intertidal salt marsh has been converted to upland areas. This transition has eliminated habitat for estuarine flora and fauna. The Herring River Restoration Committee (HRRC), a multi-agency group appointed by the Cape Cod National Seashore and the Towns of Wellfleet and Truro, (2008) indicates that:

“...non-native salt-intolerant grasses and woody vegetation have increased, even in the river channel, restricting boating and anadromous fish habitat. Surface waters have been acidified by wetland drainage and the oxidation of sulfur in the diked salt marsh peat; high acidity leaches toxic metals from native clays to surface water; and in the summer, dissolved oxygen depletions are common due to reduced tidal flushing. All of this has caused fish kills in the river, while high acidity and periods of low oxygen have restricted fish access to mosquito breeding sites; thus, nuisance mosquitoes can still be abundant.”

The HRRC has recognized the environmental and socioeconomic benefits of restoring this tidally restricted and degraded wetland system, and is currently developing a

comprehensive restoration project/plan that is geared towards identification of restoration actions and adaptive management strategies that will improve the system through a monitored and adjustable approach.

As part of the restoration effort, the Town-appointed Herring River Technical Committee (HRTC) requested the development of a comprehensive hydrodynamic model that could be used to assess existing conditions within the estuarine system, as well as evaluate a range of alternatives and their potential impacts. The model was required to be sufficiently flexible to integrate with the adaptive management approach, capable of simulating the complexities of the Herring River system (e.g., marsh surface wetting and drying, a range of flow control structures, etc.), and be extendable to include potential additional technical items (e.g., sediment transport, water quality, etc.). Woods Hole Group, working with the Town of Wellfleet and the HRTC, was contracted by the Town of Wellfleet to identify and develop the hydrodynamic model for the Herring River system.

As discussed, the Herring River estuary system represents a significant floodplain (the largest estuary on outer Cape Cod), and through proper plan development has the potential to restore up to 1,000 acres of wetland area. The hydrodynamic modeling effort will be a major component of the restoration plan that will address numerous concerns associated with re-establishing increased tidal exchange, as well as provide the necessary information to design an appropriate system of dikes, culverts, and road crossings. Achieving these goals will require the development of a high quality hydrodynamic model that can provide, to a reasonable extent, the technical information needed to evaluate possible restoration alternatives for Herring River. The modeling development and execution follows a phased-approach, centered on key stopping points that allow for progress assessment and refocusing based on lessons learned from each prior task or model simulation. Since the model was developed to be a flexible, this allows for future inclusion of additional processes (e.g., sediment transport, etc.), adaptable alternative analysis, and capabilities to provide conceptual designs for the restoration plan.

The purpose of this report is to provide details on the first phases of the hydrodynamic model identification and development. It is expected that as the restoration plan continues to progress, the model will be used to assess additional alternatives and be extended to address additional physical processes, assess impacts, and provide visualizations of the proposed alternatives and their potential impacts. Therefore, the model presented herein will continue to be developed and refined in subsequent phases of the restoration project. This report outlines the scoping of the model, including goals and objectives and the data availability and requirements, the model selection, the model development, and the results from the simulation of an initial set of alternatives focusing on the Chequessett Neck Road dike. Specifically, the report is divided into the following primary sections:

- Section 2.0 presents the model scoping portion of the modeling program. This includes the goals and objectives of the modeling program, and the relationship to the overall restoration project goals and objectives. This section also presents a summary of all pertinent topographic, hydrographic, and hydrologic data that has been collected to date, including a brief summary of the existing literature related to Herring River. Additional data collection efforts that may further support the modeling program are also presented.

- Section 3.0 presents the requirements of the model, based on the goals and objectives of the model program, and potential models that may be used to simulate the complex processes of the Herring River estuary. This section also includes a “model matrix” that details a number of potential modeling candidates, with short descriptions of the limitations and capabilities. More detailed descriptions of the most suitable programs, addressing their particular abilities for simulating the unique combination of physical processes that occur within the Herring River Estuary, is also presented. Finally, Section 3.0 provides the final model selection, decided jointly by the HRTC and Woods Hole Group, as the most suitable modeling program or package for simulation of the complex physical processes of this estuarine system.
- Section 4.0 presents the proposed model approach, identifying the major tasks within the modeling scope and identifies key discussion points for evaluation of the modeling performance and progress. This allows for a flexible approach that can include the incorporation of new data, and/or a re-direction of the effort based on the results of the current modeling phase.
- Section 5.0 presents the development of the hydrodynamic and salinity model of the Herring River system. The purpose of the model is to aid in planning for tidal restoration of the estuary by predicting time dependent water levels, velocities, and salinity levels for various restoration alternatives. This section of the report describes how the model was calibrated and validated to existing conditions within the system, and then presents the use of the calibrated model to evaluate some initial alternatives. The report presents the results of a number of simulations with the estuary in its existing restricted state to aid in understanding of the current behavior of the system and to provide a baseline for comparison with future alternative simulations. These existing conditions simulations include: simulation of 1 and 100 year return period tidal floods, and simulation of 50 and 100 years of sea level rise.
- Section 6.0 presents the initial alternative simulations for evaluating changes at the Chequessett Neck Road dike. The alternatives included simulation of the Herring River system with the removal of all anthropogenic features (a representation of potential maximum restoration levels), optimization of the dike opening (ranging from a 10 meter (32 feet) opening to a 100 meter (330 feet) opening), assessment of the removal of the flood tidal shoal, and the potential impact of lowering the dike culvert inverts.
- Section 7.0 presents recommendations and next steps for further model development and marsh restoration alternatives.

2.0 MODEL SCOPING

2.1 MODEL GOALS AND OBJECTIVES

The overall goal of the Herring River Restoration Project is to create a productive environment that will sustain itself with improved water quality and a strengthened ecosystem by restoring tidal flow to the estuary. While it would be desirable to allow the Herring River Estuary to simply resume its previous natural state of un-impeded tidal flow, human and environmental constraints pose limitations on the extent to which the natural tidal flow can be restored. The success of the project will largely depend on the successful implementation of a comprehensive restoration plan, which addresses all the important issues related to those limitations. Hydrodynamic modeling will be central in developing this plan as it allows for the evaluation of specific questions about the potential changes to surface water flow, velocities, water surface elevation, and salinity levels within the estuary. Once constructed and properly calibrated, a hydrodynamic model can be applied to numerous alternative restoration scenarios with relative ease and expedience.

Restoring tidal flow to the Herring River is likely to increase both the tidal range and the salinity levels within the estuary. A hydrodynamic model can predict these increases and therefore help evaluate potential impacts to the habitat areas as well as property and low-lying structures. In addition, spatially-variable, time-dependent predictions of water surface elevation allow for assessment of flooding and drying, which will determine the extent of the inter-tidal zone and aid in identifying potential restoration zones and ponding or stagnation areas. Expanding the openings in the Herring River dike control structure may also have substantial effects on the current velocities within the system and especially near the dike, which could impact fish passage or result in significant changes in sedimentation patterns. With the increased tidal flow, saline water will also travel much further into the system than it has in the past century. Therefore, the numerical modeling can also directly provide information on the extent and concentration of salt within the estuary. Salinity levels can then be used to deduce the potential effects on water chemistry, groundwater levels, and flora and fauna. Understanding these changes will be necessary for successful restoration planning. Specifically, the modeling program should meet a number of well-defined goals addressing all pertinent limits on the restoration of tidal flow to the estuary. For example, some of these potential goals that have been initially developed include, but are not limited to:

- A restored water surface elevation not to exceed 9.1 feet NAVD 88, which corresponds to the 100-year flood elevation throughout most of the system
- A restored water surface elevation not to exceed 5.1 feet NAVD 88 in the Mill Creek tributary which has critical upland infrastructure (first floor elevations of local homes and the Chequessett Neck Country Club)
- Evaluate hydroperiod and percentage of time high tides wet the marsh in key areas
- Assess potential changes in the velocity regime and sedimentation patterns (near the dike specifically)
- Assess potential impacts to wells that are located within the floodplain (as presented in Table 2-1) to avoid saltwater intrusion
- Assess potential impacts to low-lying roads and structures (as presented in Table 2-2)
- Assess potential impacts to property, including upland flooding

- At a preliminary level, assess vegetation changes and vegetation die off due to saltwater intrusion
- Assess potential water ponding and stagnation
- Identify extent of Mean High Water (MHW), Mean High Water Spring (MHWS), and Annual High Water Spring (AHWS) for each alternative

Table 2-1. Low lying wells within the Herring River floodplain.

Map	Lot	Owner	Address	City	Zip	Northing	Easting
12	601	Sarah Rosenberg, Trustee	70 Way 672	Wellfleet	02667	4643912	412650
12	235	Judith Ellis	27 Way 672	Wellfleet	02667	4643844	412607
12	602	Gerald & Hila Feil	90 Way 672	Wellfleet	02667	4643782	412631
19	10	Martin Nieski	119 West Main St.	Dudley	01571	4643099	412700
19	66	Harvey F. Schwallie, Trustee	60 Mill Creek Lane	Wellfleet	02667	4642741	412998
19	68	Harriet D. Shapiro Trustee	80 Mill Creek Lane	Wellfleet	02667	4642831	413000
19	15	Sarah James	65 Way 60	Wellfleet	02667	4642958	413305
19	67	Katherine G. Gilmour, Trustee	70 Mill Creek Lane	Wellfleet	02667	4642792	412994

Although not all these objectives will be directly accomplished by the surface water model developed as part of this scope of work, the results of the surface water model will provide information and input needed to address these objectives. Additionally, another more general objective of the modeling effort will keep in mind that additional goals may be identified in the future. Therefore, the model should be flexible enough to allow for future integration of a sediment transport model, water quality model, and/or biological model. Some possible future goals may include:

- Direct simulation of sediment transport
- Direct simulation of water quality constituents
- Assess impacts to aquaculture beds due to historic presence of high fecal coliform bacteria
- Assess impacts on mosquito population

2.2 DATA AVAILABILITY

2.2.1 Previous Studies

Following an eel kill in the fall of 1980, which drew attention to the poor and declining water quality in the Herring River upstream of the newly (1974) re-built dike, a significant amount of literature has been generated documenting studies conducted within the area. Ecological and biogeochemical investigations have indicated the detrimental impact caused by diking and ditching of the river; and call for restoration of tides to the river in order to revitalize the ecosystem (Beskenis and Nuzzo, 1984; Portnoy, 1984; Portnoy et al., 1986; Portnoy and Giblin, 1997; Portnoy, 1999; Portnoy and Allen, 2006). However, the potential restoration also raised

concerns that the re-introduction of tides may cause saltwater intrusion into nearby wells. This led to studies of groundwater in the area, which to some extent have quelled these concerns (Fitterman et al., 1989; Fitterman and Dennehy, 1991; Martin, 2004; Masterson and Portnoy, 2005; Martin, 2007). Additionally, due to concerns of groundwater impacts and upland flooding in the Mill Creek region, a study of stream flow in Mill Creek was undertaken to examine options for a potential dike that may be required as part of the restoration plan (Nuttle, 1990). Sedimentation concerns, particularly stability of “the Gut” and potential effects on shellfish beds below the dike, have been addressed using historic evidence and existing hydrodynamic model results (Dougherty, 2004). Some of the above studies have used results provided by two previous hydrodynamic modeling efforts (Roman, 1987; Spaulding and Grilli, 2001). In the development of a more advanced model, as part of this scope of work, a brief summary of the two previous modeling efforts is presented below.

Table 2-2. Low lying structures within the Herring River floodplain.

	Map	Lot	Structure	Elevation (ft, NGVD 1929)
Mill Creek	19	81	CYCC fairways	≥2
	19	24	Basement floor	6.95
	19	10	Basement floor	5.04
	19	66	Garage floor	8.38
Lower basin	12	601	House Floor	7.6
			Cesspool Land Surface	5
			Basement floor	2.9
	12	602	House Floor	8.76
			Cesspool Land Surface	5.8
			Basement floor	1.57
Upper Pole Dike Creek	13	69.1	House Floor	6.16
			Septic Land Surface	15.7
	13	69	House Floor	9.01
	8	258	Restaurant floor	9.73
			Septic Land Surface	10.6
	14	26	Basement floor	7.22
	14	30	Garage basement floor	8.72
	14	34	Studio basement floor	6.51
	14	35	Basement floor	7.32
	8	234	Apt. first floor	8.24
	8	231	Garage slab	8.39
	8	232	Septic Land Surface	9.7
Barn first floor			9.96	
Roadways	High Toss Road			4.5
	Pole Dike Road			5.85
	Old County Road @ Bound Brook			3.86
	Bound Brook I. Road @ Herring River			5.62
	Old County Road @ Paradise Hollow			4.56

Twenty years ago, a group of researchers evaluated alternatives for restoration and management of the Herring River Ecosystem (Roman, 1987). In their report, they presented results of the first

hydrodynamic modeling effort for the estuary (Roman et al., 1995). This model used stage data collected both above and below the dike over a single semi-diurnal tidal cycle on three occasions (August 21, 1984; December 4, 1984; May 17, 1985) with three different sluice gate openings (0.51m, 1.3m, 0.61m) to calibrate a “tide-height” model for the estuary. The model was formulated using the principles of conservation of momentum and conservation of volume considering the area upstream of dike the as a reservoir with an instantaneously horizontal water surface. Momentum is conserved by balancing the horizontal pressure gradient across the dike with friction losses using Manning’s formulation, while volume is conserved assuming that surface area of the reservoir (which in turn determines the volume) can be represented by a piecewise linear function of the water surface elevation. Bottom profiles of the river channel in combination with marsh surface elevations were used to determine the required surface area function while Manning’s n was determined by calibration to the three data sets. Once calibrated, the model was applied to nine different cases over a semi-diurnal tide period with a typical mean tide range and different configurations of sluice gate and tide gate openings. In addition, simulations were conducted for four cases using conditions from the February 1978 storm representing a 100-year storm event.

The Roman et al. (1995) model results present high and low water elevation and tide range in the reservoir, and flood and ebb volumes. They also developed a one-dimensional salinity model for the Herring River in order to predict flushing time and tidally averaged salinity distribution within the estuary. This model was based on balancing the downstream advection of freshwater with the upstream turbulent diffusion of salt caused by tidal mixing and requires input of a diffusion coefficient and freshwater inflow. A freshwater inflow of $0.28\text{m}^3/\text{s}$ was used based on data collected between April and August 1985, and salinity data collected on August 23, 1984 was used to determine the diffusion coefficient. Once the diffusion coefficient was determined, the model was applied to the nine restoration alternatives modeled with the tide-height model.

More recently, an “inlet-basin” model similar to the previous tide-height model was developed (Spaulding and Grilli, 2001). Like the tide-height model (Roman et al., 1995), this model applies the principles of conservation of momentum and conservation of volume while considering the “basin” upstream of the dike as a simple reservoir. However, it is more elaborate in its formulation of bottom friction allowing for the inclusion of multiple inlet channels with variable cross-section, and allowing for the use of empirical coefficients to account for friction losses associated with jet formation within the inlets. A significant improvement over the previous model comes from the use of a more accurate surface area vs. elevation curve that was determined from topographic data collected by the NPS in 1999 combined with USGS provided 0, 3 and 6 m contours. This model was calibrated using a December 4, 1984 data set collected from the Roman et al. (1995) report, then validated using data collected May to November of 2000. Additional intensive data collections were conducted over the period of a single semi-diurnal tidal cycle on July 25, 2000 and September 27, 2000. These included measurements of depth and current velocities within the culverts under the dike and the culvert under High Toss Road, while salinity and temperature data were collected throughout the water column at a number of stations along the river. A salinity model was also developed, which is based on a steady-state salt balance similar to the Roman et al. (1995) model. This salinity model was calibrated and validated with the salinity measurements made on July 25, 2000 and September 27, 2000. These models were applied to the same restoration alternatives used by the previous model with inclusion of additional simulations that progressively increased the width of the inlet

at the river mouth. This model was later adapted and applied to additional alternatives that included a series of sluice gates at the Herring River dike (Spaulding and Grilli 2005). A more detailed review of the Spaulding and Grilli modeling effort can be found in the peer review conducted by Woods Hole Group (WHG, 2006). These modeling efforts provide a good initial evaluation of potential restoration options for Herring River. The model to be developed as part of this scope of work will further advance the hydrodynamic understanding throughout the entire Herring River estuarine system.

Overall, the previous modeling efforts demonstrated that larger openings in the dike would cause increases the mean tidal elevation and the tidal range. Increasing the opening will also increase the saltwater penetration distance. Using their model, Spaulding and Grilli found that increasing the opening in the dike to 30 m will allow fully unrestricted flow and that larger opening(s) would not substantially increase the tidal range within Herring River. While these models provide reasonable results for water surface elevation just above the dike, the results are less certain further upstream. The reservoir representation of the river neglects frictional effects upstream of the dike that will likely cause phase shift and attenuation of the tidal signal in the real system. Additionally, Spaulding and Grilli (2001) point out the difficulty in modeling systems with large flat marsh areas. In flat marsh plains, a small change in water surface elevation can result in a large change in the volume of water in the estuary. In order to accurately model this situation, it is imperative to have highly precise topographic data, especially within the range of the marsh surface elevation. For the tide-height and inlet-basin models, this constitutes creating a more accurate surface area versus elevation curve. However, it is not enough to consider only the flooded area of the estuary, its shape also has significant impact on the resulting water elevation and currents. Because it takes time to flood the marsh, a long narrow estuary will behave differently than a short wide one. Similarly, the one-dimensional salinity models give reasonable results for the steady-state conditions to which they are applied, but they should not be applied to conditions that are significantly different from those to which they were calibrated. In their formulation, they use a constant empirically determined longitudinal dispersion coefficient to account for mixing of the salt and fresh water. Actual dispersion, however, is highly dependent on velocity gradients and therefore, in a dynamic tidal system, its magnitude is both temporally and spatially variable. Variations in the volume of fresh water inflow, residual tidal currents, stagnation, and other factors all contribute to mixing within the estuary and can have significant effects on the resulting saltwater penetration. Considering the limitations of the earlier models, a more complex model is developed herein to provide improved accuracy in water surface elevation, tidal circulation, and salt concentration throughout the estuary. The model identified and developed in this report more precisely represents the geometry of the estuary (including its plan form); considers variable frictional effects throughout the estuary; allows for flooding, drying, and ponding of water; produces accurate current velocities and water surface elevations throughout the estuary; and properly represents the physics of mixing for a wide range of forcing conditions.

2.2.2 Existing Data

The data required for the development of this model are of two distinct types, topographic and hydrographic data are required to construct the model geometry, and hydrologic data is required for model forcing and proper calibration and verification to ensure the model will provide accurate predictions. Massachusetts Coastal Zone Management (MCZM) has provided recent

photogrammetric data (2007) for the Herring River area. These data provide high-resolution land surface elevation, including approximately 200,000 points within the estuary above the mean low water elevation. The photogrammetry will provide the necessary high resolution and precision required to accurately model the flooding and drying of the marsh surface. For the river and stream bottoms (below the lower limit of the photogrammetry data), two sources of data were used. Hydrographic data collected by the National Park Service (NPS) in 1999 and used by Spaulding and Grilli (2001) in their model, as well as supplementary data collected by NPS in 2008 were used to provide the depths within the creeks and streams of the Herring River estuary system. These data provide the necessary river bottom profiles for the model and in some areas, overlap the photogrammetry into the marsh. Figures 2-1 and 2-2 show the distribution and density of the topographic and hydrographic data sets, respectively. The blue dots in Figure 2-2 show the location of the 1999 creek elevation data, while the green dots show the location of the 2007 elevation data.

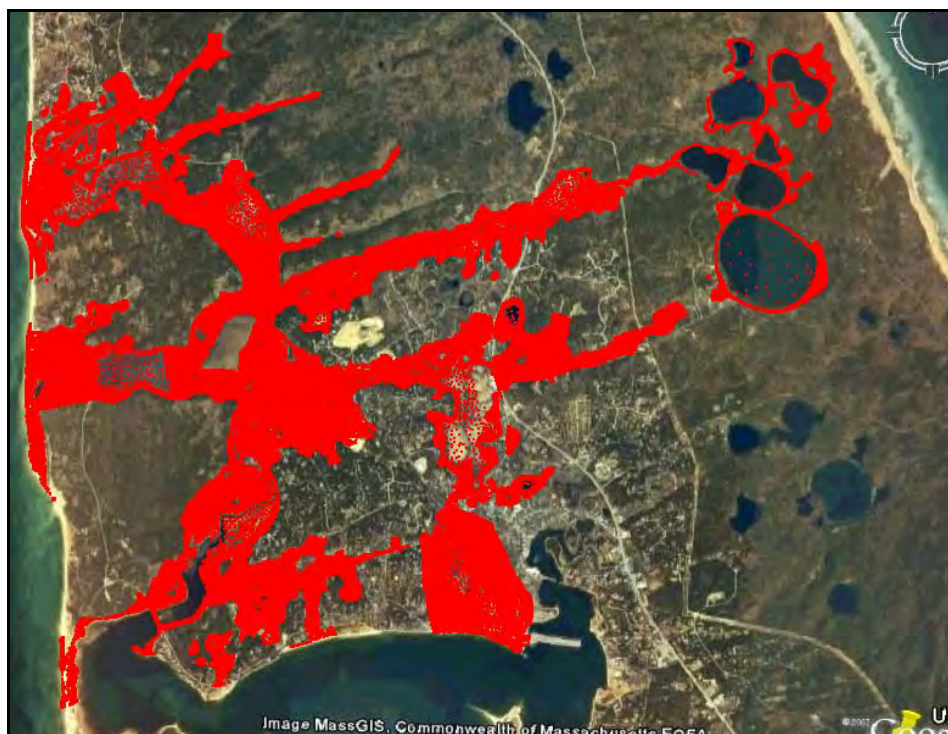


Figure 2-1. Land elevation data from MCZM photogrammetric survey collected in 2007 (background image courtesy of Google Earth).



Figure 2-2. Bathymetric data from 1998-1999 NPS total station surveys (blue dots) and 2008 NPS RTK Surveys (green dots) - (background image courtesy of Google Earth).

Once the model geometry is in place, the model is calibrated and verified with hydrologic data. This process is described in more detail in Chapter 4.0. In terms of hydrologic data, recent water surface elevation data at six (6) key locations were collected at 15-minute resolution from August 30 to October 4, 2007 by NPS (Figure 2-3). Additional water surface, salinity, and temperature data were also collected at 15-minute resolution at two (2) more locations between June 1 and October 26, 2007 (Figure 2-3). Subsets of these data can be used for both model calibration and verification. Other hydrologic data that was also used in model verification includes the data collected for the earlier modeling studies (Roman 1987, Spaulding and Grilli 2001). These data also allow for direct comparisons to the previous models. Table 2-3 shows a summary of all the hydrologic data observed within the Herring River System.

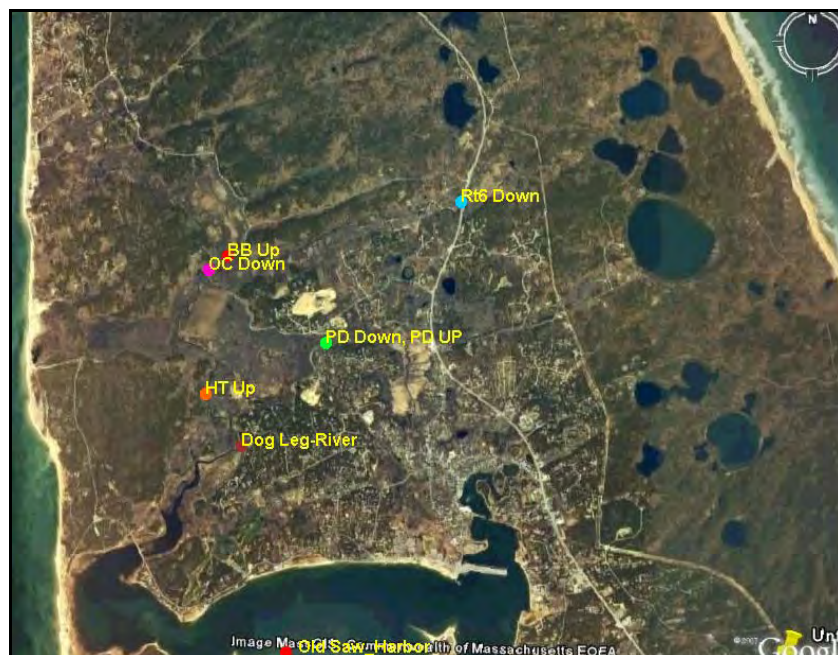


Figure 2-3. Hydrologic data gage locations (background image courtesy of Google Earth).

Table 2-3. Hydrologic data availability (X indicates no data available).

	Roman	Spaulding & Grilli				NPS		
Locations	12/4/84	8/12/99 8/27/99	-9/17/99 9/24/99	-9/29/99 10/13/99	-5/23/00 11/02/00	-6/1/07 8/30/07	-8/30/07 10/04/07	-10/04/07 10/26/07
Just Below Dike	wse*	X	X	X	X	X	X	X
Just Above Dike		X	X	X	X	X	X	X
1km Above Dike		X	X	X	X	X	X	X
Above Dike	X	wse*, temp*, salinity, pH			X	X	X	X
Below Dike	X	X	wse*, temp*, salinity, pH			X	X	X
BB,Up	X	X	X	X	X	X	wse*	X
PD, Down	X	X	X	X	X	X		X
Rt6,Down	X	X	X	X	X	X		X
OC, Down	X	X	X	X	X	X		X
PD, Up	X	X	X	X	X	X		X
HT, Up	X	X	X	X	X	X		X
Dog Leg-River	X	X	X	X	X	wse*, temp*, salinity, pH		
Old Saw_Harbor	X	X	X	X	X	wse*, temp*, salinity, pH		

* wse = water surface elevation, temp = temperature

2.3 DATA LIMITATIONS

A model, at best, is nothing more than a good approximation of its prototype. The better the data used to build it, the more potential it has to give accurate predictions. The elevation data available to construct this model comes from the combination of two different data sets. Where the photogrammetry data are comprehensive (Figure 2-1), the bathymetric data (Figure 2-2) are not nearly as dense. As such, the model domain will require some interpolation for certain bathymetric areas in the river, especially some of the upper reaches of the estuary. However, the amount of bathymetric data is sufficient to provide a reasonable model of the system, especially in the areas where the most significant changes are expected (e.g., lower Herring River, Mill Creek, etc.).

Water surface elevation data are available at eight (8) locations throughout the system, which will allow for comprehensive calibration and verification of the model with respect to this variable; however, there is limited velocity data and salinity data for this same time period. Therefore, the calibration to salinity and velocity will not be as robust as to water surface elevation.

Finally, although some limited vertical salinity data has been collected (over a single day), it is unclear the level of stratification that may exist within the system. Spalding and Grilli (2001) indicate the possible presence of a salt wedge, but the magnitude, distance, and persistence of the feature cannot be adequately determined from their limited data set. The level of stratification may be an important feature of the system that could influence circulation and mixing dynamics, and as such, vertical salinity data observations (during periods of low and high outflow) may want to be considered to more accurately assess salinity. If significant stratification is identified in the system, a three-dimensional model should be considered. The model developed for Herring River is flexible enough such that transitioning from a 2-D model to a 3-D model will be relatively seamless.

Overall, there is sufficient data to develop a reasonable two-dimensional hydrodynamic model of the system. No significant data gaps exist that would limit the model's calibration or potential performance. Model improvements can always be made with collection of additional data, model refinements, and expanded calibration, verification, and validation procedures; however, the data currently available will not limit the model's ability to function at the conceptual design level, for which it was intended.

2.4 FUTURE DATA RECOMMENDATIONS

Although no additional data was required to develop a reasonable two-dimensional hydrodynamic model, increased accuracy of the model could be obtained by specific data collection activities, such as:

- More recent bathymetric data (cross-sections) of the upper reaches of Herring River would provide for a more accurate representation of the geometry and reduce interpolation of the river bottom elevations in the upper reaches of the system. This additional data would likely result in limited improvement in the model; however, since the upper reaches of the system will experience limited tidal forcing, even for significant restoration cases.

- Cross-channel velocity profiles at a few critical locations within the estuary, simultaneously collected with water surface elevation, would provide more details related to the velocity distribution within the estuary, and allow for further model verification. Verifying the model to velocities would ensure confidence in the model's ability to produce accurate results for both water surface elevation and velocity. These velocity data would supplement the velocity profiles collected by Roman (1987).
- Observations of discharge at some of the key smaller tributaries throughout the Herring River system would provide improved freshwater discharge values for modeling of the system.
- As discussed in the previous section, vertical salinity observations should be considered to better define the river stratification and determine the potential need for a three-dimensional numerical model.
- Since a potential future goal of the modeling effort is to allow for the incorporation of water quality and/or sediment transport capabilities, the Town of Wellfleet and the MAC may desire to plan for future collection of data to support these models as well. This would include collection and analysis of sediment samples, water samples, and sediment flux observations at key constriction in the system.

3.0 MODEL SELECTION

3.1 MODEL REQUIREMENTS

A successful model aids in restoration planning by providing information needed to meet the goals detailed in Section 2.0. These goals require a model that incorporates the physics necessary to solve for water surface elevation, current velocities, and salinity, as well as potential future expansion into water quality and sediment transport. The model needs to be dynamic, capable of handling bi-directional flow, high resolution to identify important processes, and capable of being forced at its downstream boundary with an elevation time series, and at upstream boundaries with a freshwater, inflow discharge. Additionally, the Herring River estuary has a number of features that require specialized model abilities, including:

- In order to improve on previous modeling efforts, and due to the complex shape of the estuary, it will be necessary to build, at a minimum, a two dimensional (depth-integrated) model. The model selected should also have the ability to quickly adapt to three dimensions, if required by the physical processes being simulated.
- Herring River is bounded by a large relatively flat flood plain that used to be salt marsh prior to construction of the dike in 1908. With tidal restoration, it will once again become an active salt marsh and therefore must be included within the model domain. This flood plain has undergone substantial subsidence caused by the past century's dewatering of the marsh soils. This has created a unique topography that will be intermittently flooded and drained. Additionally, as the tide falls, depressions in the flood plain will likely trap water in a network of ponds that could have a significant impact on the overall circulation patterns throughout the estuary. To effectively model this situation the chosen model must be able to efficiently handle wetting and drying of the flood plain without error while having sufficient detail to resolve this ponding network.
- Variable vegetation and soil types throughout the estuary cause variable bottom friction. The successful model should be able to handle this by allowing for specification of variable bottom friction coefficients. Existing vegetation maps were used to determine bottom friction coefficients. Additionally, soil types, depth levels, and grain size information will be a requirement if the model is extended to simulate sediment transport throughout the system.
- Another important feature that must be considered when choosing a model for the Herring River is the flow control structures in the dike. The selected model must be able to simulate the control that these structures (flap gates, culverts, and sluice gates) have on flow within the estuary; and do so with consideration of the proper hydraulics involved, for example super-critical versus sub-critical flow through the sluice gate.

Therefore, in selecting the model best suited for the Herring River estuary, some of the key required features that should be considered are:

- At minimum, a 2-D depth integrated model
- At minimum, solves for water surface elevation, velocity, salinity
- Sufficient grid resolution and flexibility to represent important details
- Wetting and drying, and ponding
- Control structures (flap gates, culverts, sluice gates)
- Variable bottom friction
- Elevation and inflow specified boundary conditions

In addition, to meet the goal of flexibility for future model enhancements:

- Ability to expand to 3-D
- Sediment transport abilities
- Water quality abilities

3.2 MODEL MATRIX

There are numerous numerical, hydrodynamic models available, both commercial and non-commercial. A smaller subset of these meets the requirements for Herring River. Hydrodynamic models differ in many ways, all having their unique strengths and weaknesses. In order to help in choosing the most suitable model of this application, Table 3-1 presents a model matrix summarizing the features of a number of models that could be reasonably considered as candidates for use in developing the Herring River model. There are a number of additional hydrodynamic models that were considered (e.g., FVCOM); however, each of these had a significant limitation in respect to the requirements of the Herring River estuary system, or were still in development stages.

From this matrix of twelve (12) candidate models, Woods Hole Group has identified a short list of three (3) models (RMA/TABS, ADCIRC, and EFDC) that we believe to have the greatest potential for development of the Herring River estuary model. A brief description of these models is provided below.

RMA/TABS actually represents a suite of models. The models that would be applied for Herring River are: (1) RMA2 for simulating 2-D water surface elevation and horizontal velocity and, (2) RMA4 for simulating 2-D transport of conservative and/or non-conservative substances (i.e., salt). RMA2 computes a finite element solution of the depth-integrated, 2-D shallow water equations using a combination of triangular and quadrilateral finite elements. The solution uses linear basis functions for water surface elevation and quadratic basis functions for velocity requiring elements with mid-side nodes. The solution is implicit in time allowing for relatively large time steps in the solution. Subsequently, RMA4 uses the hydrodynamic solution provided by RMA2 to calculate a finite element solution of the 2-D advection-diffusion equation for the vertically well-mixed concentration of up to six (6) constituents. Unstructured finite element grids have the advantage of easily representing complex coastlines and allowing high grid resolution in areas where it is needed with easy transitions to lower grid resolution where it is not.

The ADvanced CIRculation model (ADCIRC) solves the shallow water equations for water surface elevation and velocity using a modified form of the continuity equation called the Generalized Wave Continuity Equation (GWCE). It employs the finite element method using

grid linear triangles and is explicit in time. ADCIRC can be run in either a 2-D depth integrated mode or 3-D mode. When run in the 3-D mode, an equation of state is simultaneously solved including salinity and temperature. ADCIRC's wetting and drying is accomplished by elemental elimination in which an element is considered dry and removed from computations when one of its nodes becomes dry. This method of wetting and drying can create unwanted channelization and blocking of flow and can be disadvantageous when applied to broad flat shallow areas. However, ADCIRC is also a modern code with an efficient matrix solver and an available multiple processor parallel version allowing for efficient simulations even with very large grids. Thus, an ADCIRC model with a large number of nodes and higher resolution may be able to explicitly resolve the micro-topographic features.

The Environmental Fluid Dynamics Code (EFDC) solves the shallow water equations for water surface elevation and velocity simultaneously with dynamically coupled transport of salt and temperature. The EFDC model is a two and three-dimensional model capable of simulating a diverse range of environment flow and transport problems. The model has been applied to studies of circulation, discharge dilution, water quality, TMDL, and sediment transport. EFDC is capable of predicting hydrodynamics and water quality in multiple dimensions and is a widely accepted EPA approved model. EFDC is also capable of simulating 21 water column state variables (e.g., nitrogen, algae, DO, phosphorus, carbon, etc.).

The EFDC model solves the three-dimensional, vertically hydrostatic, free surface, turbulent-averaged equations of motions for a variable-density fluid. The model includes dynamically coupled transport equations for turbulent kinetic energy, turbulent length scale, salinity and temperature. In addition, the EFDC model simulates cohesive and noncohesive sediment transport, eutrophication processes, both near field and far field dilution of discharges, and the transport and fate of toxic contaminants. The model is capable of simulating multiple size classes of cohesive and noncohesive sediments along with the associated deposition and resuspension processes and bed geomechanics. The transport of toxics in both the water and sediment phases is simulated. The model allows for the wetting and drying of shallow areas using a mass conservation scheme. EFDC also includes measures for simulating flow control structures and highly vegetated areas. An embedded single and multi-port buoyant jet module is also included for coupled near and far field mixing analyses.

The model matrix, presented in Table 3-1 provides a comparative tool for evaluating the various hydrodynamic models available to simulate the Herring River estuary. Key abilities and other factors that may be critical to success of the modeling effort, or are important from the HRTC's perspective, were evaluated through using this matrix. The matrix was ultimately used by Woods Hole Group and the HRTC to help guide the selection of the most appropriate model.

Table 3-1. Model selection matrix.

	RMA/TABS	EFDC	ADCIRC	ELCIRC	SELFE	MIKE	DELFT3D	ROMS/TOMS	ADH	TELEMAC	TRIM/unTRIM
2-D Hydrodynamics	yes (RMA2)	Yes	Yes	Yes	Yes	Yes (Mike 21)	Yes	Yes	Yes	TELEMAC-2D	Yes
3-D Hydrodynamics	yes (RMA10)	Yes	Yes	Yes	Yes	Yes (Mike 3)	Yes	Yes	Yes	TELEMAC-3D	Yes
Grid Type	Unstructured Triangles and Quadrilaterals	Curvilinear Orthogonal	Unstructured Triangles	Unstructured Orthogonal Triangles and Quadrilaterals	Unstructured Triangles and Quadrilaterals	Many different options	curvilinear, sigma-verticle	curvilinear Orthogonal, terrain following	Unstructured	Unstructured Triangles	Curvilinear Orthogonal / Unstructured Orthogonal
Equations/Numerics	SWE, Primitive Continuity, Finite Element(quadratic for velocity, linear for WSE), Implicit	SWE, finite volume or finite difference	SWE, GWCE, Finite Element(linear), Explicit, CFL limited	SWE, finite-volume/finite-difference Eulerian-Lagrangian algorithm, based on UNTRIM	SWE, finite-volume/finite-element Eulerian-Lagrangian algorithm, based on UNTRIM	Equations/Numerics are unclear, Finite Difference, Finite Volume, many different programs for different applications	SWE	primitive equations,	SWE or NS, Finite Elements with Adaptive Mesh Refinement	SWE(2D), N-S(3D), Finite Element Method	RANS, finite-volume/finite-difference Eulerian-Lagrangian algorithm
Wetting/Drying	Marsh Porosity or Elemental Elimination	Yes	Yes, Wet or Dry Elemental Elimination	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Salinity	Yes, RMA4,RMA11	Yes	yes, in 3-D mode	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Temperature	Yes, RMA4,RMA11	Yes	yes, in 3-D mode	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Water Quality	Yes, RMA4,RMA11	Yes	No	No	No	Yes	Yes	No	Yes	Yes	No
Sediment Transport	Yes, SED2D	Yes	Not readily available	No	No	Yes	Yes	Yes	Yes	Yes	No
Contaminant Transport	Yes, RMA4,RMA11	Yes	No	No	No	Yes	Yes	No	Yes	Yes	No
Particle Tracking	Yes, RMATRK	Yes	WHG developed	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
Boundary Conditions	Inflow, Elevation, Wind, Rainfall,Control Structures	Inflow, Elevation, Wind, Rainfall,Control Structures	Inflow, Elevation, Wind,Control Structures, Radiation Stress	Flux,Elevation,Wind,radiation,etc...	Flux,Elevation,Wind,radiation,etc...	Many	Many	atomospheric, tidal, river	Many	Many	Many
Data Assimilation	No	Yes	No	Yes	Yes	No	No	yes	Yes	N/A	N/A
Parallel	No	No	MPI	MPI	No	Coming in 2008	No	openMP, MPI	Yes	Yes	no
Pre-/Post-Processor	SMS	WHG developed	SMS, ADMAT	WHG developed	WHG developed	Has its own	Has its own	Yes	SMS, GMS	Has its own	has its own
Open Source	No	Yes	Yes	Yes	Yes	No	Not for Commercial	Yes	Yes	no	No
Language	Fortran	Fortran	Fortran	Fortran	Fortran	Not released	Not released	Fortran	Fortran and C	Not released	Not Released
Operating System	Windows	Windows/linux	Windows/linux	Linux	Linux	Windows	Windows/linux	linux	linux	linux, windows	N/A
Support	USACE-ERDC	TetraTech	email User Group	email user group	email user group	DHI	Delft Hydraulics	large usergroup	For USACE only	EDF, Sogreah	N/A
Peer Reviewed	Yes	Yes	Yes	some	Some	Yes	Yes	Yes	Yes	Yes	Yes
Documentation	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes, for users	Yes
Commercial Use	Yes	Yes	Yes	Yes	Yes	Yes	Yes	N/A	Yes	Yes	Yes
Upgradeable	Yes	Yes	Yes, frequent new versions	Yes	Yes	Yes	Yes	Yes	Yes	N/A	N/A
Training	Yes	No	Yes	No	No	Yes	Yes	N/A	No	N/A	N/A
Cost	\$3800 for SMS TABS Module	Free	\$ 5300 for SMS Module	Free	Free	\$30 to \$100k	\$100k	free	Free	~\$40k depends on modules	\$10 to \$20k
Comments	Marsh porosity is an advantage for wetting/drying where microtoporgaphy cannot be resolved	EPA supported model. Does everything including a number of control structures	mainly used for large region tidal circulation and hurricane storm surge, but has also been applied to rivers and estuaries with some success	seems like it has good poteitial, local mass conservation, relatively new code, requires "close to orthogonal" grid for accuracy, has not yet been used extensively	compared to ELCIRC: orthogonality not necessary, but local mass conservation not ensured by FE, minimum linear shape functions(compared to constant), has sigma-Z coordinates not just Z	has modular structure and can do many things, has good support, Closed Source, Unsure of solution methods	has modular structure and can do many things, many options for water quality and sediment,unfortunately commercial dedicated applications have fixed grid geometry, has good support, but must work closely with Delft Hydraulics	well developed large scale ocean circulation model with steep learning curve	Being developed at ERDC, works for supercritical, subcritical, and nonhydrostatic flow,good for modeling hydraulic structures, I.e. lock intake, still under development	there is a suite of programs associated with TELEMAC for doing just about anything for free surface flows, used extensively in France where it is developed	ELCIRC is based on this code, but free

3.3 FINAL MODEL SELECTION

The final model selection was made through a joint selection process with Woods Hole Group and the HRTC. The model selection focused on:

- the technical abilities of the model
- the ability of the model to meet the specific needs of the Herring River estuary system
- the ability of the model to provide visualizations and results to foster public understanding
- the flexible nature of the model to expand and adapt to future technical requirements

Based on the information presented herein, a discussion between stakeholders was conducted focusing on the pros and cons of the models on the “short list” (ADCIRC, RMA/TABS, EFDC). The discussion also raised specific technical questions related to the ability of these models, in addition to the numerous requirements defined above, including:

- The models ability to provide velocity data at various structure types (to deduce effect on fish passage), including sluices, weirs, and for a variety of flow types (sub and super critical, transition flow, etc.)
- The accuracy of the model at predicting water surface elevation, velocities, and salinity
- Ability to simulate additional physical processes, including wind, freshwater inflow, sea level rise, groundwater
- The reliability and methodology for simulation of sediment transport, including rates of erosion and deposition
- The ability to spatially variable bottom drag appropriate values will be chosen for different bottom conditions (higher drag for marsh grasses lower drag for sandy bottom).

After a detailed discussion, the Environmental Fluid Dynamics Code (EFDC) was selected because of its ability to meet all necessary requirements for the Herring River Estuary model, specifically the flexible nature of the model to handle future technical requirements and data integration as it is developed. A detailed explanation of the EFDC model is presented in the following section.

4.0 MODEL APPROACH

This section presents the overall model approach that was applied to develop the hydrodynamic model for the Herring River system. The approach identifies the major tasks within the model development, and provides key points for evaluation of the modeling performance and progress. This allows for a flexible approach that can include the incorporation of new data, and/or a re-direction of the effort based on the results of the current modeling phase. Therefore, the modeling approach is presented in phases, all of which follow the model selection task (Section 3.0) of the overall scope of services. The scope of services for this contract consist of the complete model development phases, including model calibration and validation, as well as the initial alternative simulations. As such, this report does not complete the entire Herring River modeling scope, as additional simulations, as well as simulation of additional physical processes, are likely to be developed in subsequent simulations.

4.1 PHASE I – MODEL CALIBRATION

Model calibration is the process by which adjustments are made to the model parameters to ensure the model appropriately simulates measured water surface elevation, salinity, and other observed parameters. This requires conducting a series of iterative model simulations to ensure the model is stable, and results compare favorably with measured data. Calibration can be a lengthy process involving hundreds of model simulations. Specifically, the model coefficients are adjusted (within acceptable ranges) until the modeled water surface elevation, salinity, and temperature closely approximates the measured field observations.

In practice, hydrodynamic models require input of some physical parameters that are unknown or only known within a reasonable range. For example, bottom friction parameters, because their formulations often account for a combination of frictional effects, leave some freedom in choosing an appropriate value. The calibration process involves “tuning” the model by systematically adjusting some unknown parameters, such as bottom friction, so that its results match reasonably well to a set of observed data. Horizontal eddy viscosity is another parameter that can reasonably lie within a significant range, however for different models its effect on results is considerably different due to the use of different numerical methods. In addition, different models have numerous different parameters that can be tuned to achieve better model results. To accomplish this, the observed data set must provide both the necessary information outside the model domain to drive the boundaries, while also providing information within the model domain for comparison to model output. Once a set of parameters has been found that produces the best possible fit to the data (and the selected parameters are reasonable compared to empirically defined values), the parameters are fixed for subsequent model simulations. The more extensive the calibration data set in terms of duration, spatial extent, and observed quantities, the better chance the model has at matching other independent sets of data and producing accurate predictions.

4.2 PHASE II – MODEL VALIDATION

Model validation is achieved by applying the calibrated model, with its fixed parameters, to one or more sets of observed data that are independent from the calibration data. Typically, sets of data for validation are collected at a different time and under conditions that differ from the calibration period. Results from validation simulations are quantitatively compared to the

observed data in the same way as in the calibration process. However, during validations there is no freedom to tune parameters. This demonstrates that the model performs reasonably under a different set of conditions. As such, validation simulations act as a check to see how well the model is capable of predicting accurate results under conditions that differ from calibration. A model must be satisfactorily verified before it can be used for forecasting (e.g., alternative simulations or different forcing conditions).

4.3 PHASE III – EXISTING CONDITIONS SIMULATIONS

Once the model has been calibrated and validated, further simulations using the unmodified model provide a better understanding of the behavior of the system over a broader range of forcing conditions. In addition to elucidating the present state of the Herring River estuary, these existing conditions simulations will also provide a baseline for comparison to proposed restoration alternatives in order to provide a gauge of the potential benefits and/or risks associated with different components of the restoration design.

4.3.1 Normal Tidal Conditions

While the calibration and validation results ensure the accuracy of the model, they represent the behavior of the system during specific time periods and may not be fully representative of the normal function of the estuary. Harmonic analysis of historically observed water surface elevation data from nearby NOAA tidal stations provides forcing conditions that include average spring and neap tide events. Evaluating the existing Herring River system under normal conditions will allow for assessment of the flushing characteristics, the overall hydrodynamics, and the regular volume flux that occurs between Wellfleet Harbor and the Herring River upstream of the Chequessett Neck Road Dike.

4.3.2 Storm Scenarios

Perhaps one of the biggest risks involved in restoring tidal flow the Herring River is the increased potential for upland flooding associated with storm events. Therefore, simulation of storm events is important. At a minimum, water levels representing storms with 1-year and 100-year return periods will be simulated for the system. Storm surge elevations were developed from historical records in the U.S. Army Corps of Engineers (USACE) New England Coastline Tidal Flood Survey of 1988, as well as the Federal Emergency Management Agency (FEMA) Flood Information Study for Wellfleet, MA. In addition, winds associated with large storms can have a significant impact on water levels, therefore wind forcing is included in these model scenarios. Understanding how the existing system responds to storm forcing, will be critical in assessing the potential impacts/risks that will be involved with each potential restoration scenario.

4.3.3 Sea Level Rise

The current degraded state of the Herring River estuary is the result of a century of anthropogenic modifications. With restoration, in another century, the state of the estuary may be significantly improved. However, progression into the 21st century and beyond, it is feasible that other long-term processes may have a significant effect on the state of the system. While the topic of accelerated sea level rise is still heavily debated, The Intergovernmental Panel on Climate Change (IPCC) has undergone a considerable effort to analyze and review the current

state of the knowledge and provide an estimated range of predicted sea level rise into the next century. Sea level rise estimates are evaluated from a number of sources, including NOAA and IPCC estimates. Ultimately the model will be simulated for a range of sea level rise predictions once the preferred alternative(s) are finalized, using estimates to help determine potential impacts of rising sea levels on future conditions within the Herring River system. In the current scope of work, model simulations are conducted for predicted sea level rise using projected 50-year and 100-year sea levels based on historical rates of measured sea level rise, which provide a reasonable median estimate of sea level rise predictions.

4.4 PHASE IV – INITIAL ALTERNATIVE SIMULATIONS

This contract for this scope of work called for the simulation of three alternatives. The focus of these alternatives is on changes to the Chequessett Neck Road dike. The alternatives simulated are presented in detail in section 6.0. Each alternative will be evaluated for three (3) tidal forcing events (average spring tide, 1-year tide forcing, and 100-year tide forcing), as well as analysis of an appreciable sea level rise (i.e., 50 years of average sea level rise or greater).

4.5 PHASE V – LEVEL 2 ALTERNATIVE SIMULATIONS

The Level 2 alternative simulations will consist of evaluation of new structural crossings and modifications to openings upstream of the Chequessett Neck Road dike. This may include expansion of culverts, instillation of flow control structures, a dike at Mill Creek, etc. Based on the results of the initial alternative simulations presented herein, the Level 2 alternative simulations may be modified and adjusted. Recommendations for subsequent model simulations and alternatives are presented in section 7.0.

5.0 MODEL DEVELOPMENT

This section describes the development of the hydrodynamic model for the Herring River estuary. In addition to describing the configuration, calibration, and validation of the model, this section presents the results of a number of simulations for the existing estuary conditions. These simulations, with the estuary in its existing restricted state, aid in developing an understanding of the current behavior of the system, and provide a baseline for comparison with alternative simulations. All data observations presented in this report were provided to Woods Hole Group by the HRTC. Therefore, the accuracy of the observations, corrections to the proper datum, instrument models, setups, and calibration, were not a component of this modeling study. Data provided was assumed to be correct and appropriate for model development of the Herring River system.

5.1 THE ENVIRONMENTAL FLUID DYNAMICS CODE

After careful consideration of a number of hydrodynamic, numerical models, the Environmental Fluid Dynamics Code (EFDC) (Hamrick, 1996), originally developed at the Virginia Institute of Marine Science (Hamrick, 1992), was selected as the most appropriate tool for modeling the Herring River estuary. EFDC has been applied to numerous aquatic systems including Chesapeake Bay (Hamrick, 1994), Mobile Bay in Alabama, Cape Fear River in North Carolina, and the Suwannee River in Florida. The model has been applied to studies of circulation, discharge dilution, water quality, TMDL, and sediment transport. EFDC is capable of predicting hydrodynamics and water quality in multiple dimensions and is a widely accepted EPA approved model. EFDC was chosen because of its ability to meet all necessary requirements for the Herring River Estuary model including the ability to simulate in two-dimensions time dependent water levels, current velocities, salinity, wetting and drying, and hydraulic control structures; and because it can be easily extended to simulate sediment transport and water quality parameters if needed in the future.

The EFDC model solves the three-dimensional, vertically hydrostatic, free surface, turbulent-averaged equations of motions for a variable-density fluid. The model includes dynamically coupled transport equations for turbulent kinetic energy, turbulent length scale, salinity and temperature. In addition, the EFDC model can simulate cohesive and noncohesive sediment transport, eutrophication processes, both near field and far field dilution of discharges, and the transport and fate of toxic contaminants. The model is capable of simulating multiple size classes of cohesive and noncohesive sediments along with the associated deposition and resuspension processes and bed geomechanics. The model allows for the wetting and drying of shallow areas using a mass conservative scheme. EFDC also includes measures for simulating flow control structures and highly vegetated areas.

To represent the geometry of a water body, the EFDC model uses Cartesian or curvilinear-orthogonal horizontal coordinates and, when applied in three dimensions, terrain-following vertical coordinates. A finite difference numerical scheme is employed within EFDC with two time levels and a procedure that splits the internal-external modes. The free-surface gravity wave or barotropic external mode is solved explicitly along with the simultaneous computation of a two-dimensional surface elevation field using an iterative conjugate gradient solver. Calculating the depth-integrated barotropic velocities using the newly computed surface elevation completes the external solution. Horizontal boundary condition options for the external mode solution include specifying the water surface

elevation, the characteristic of an incoming wave, free radiation of an outgoing wave, or the normal volumetric flux on arbitrary portions of the boundary.

The EFDC model implements a mass conservation solution scheme for the Eulerian transport equations, which is at the same time step or twice the time step of the momentum equation solution. The advective step of the transport solution uses either a central difference scheme or a hierarchy of positive definite upwind difference schemes. The horizontal diffusion step is explicit in time, while the vertical diffusion step is implicit. Horizontal boundary conditions include material or constituent inflow concentrations, which can be specified as being depth-dependant and both constant and time-variable. The EFDC model can be used to drive a number of external water quality models using internal linkage processing procedures described in Hamrick (1994).

5.2 MODEL CONFIGURATION

The development of the Herring River hydrodynamic model required configuration so that this particular application of EFDC would best approximate the form and function of the real system (i.e., the Herring River Estuary). Model configuration involves compiling observed data from the actual estuarine system into the format required for the execution of EFDC. This can be broken down into three major steps that are described below.

- First, topographic and bathymetric data are used to generate the computational grid on which EFDC's finite difference calculations are performed.
- Second, observed water levels, salinity, and atmospheric conditions are used to define boundary conditions for driving the model. The development of boundary conditions also includes the assignment of bottom friction for the various types of marsh substrate throughout the estuary system.
- Third, for this application, additional subroutines were applied within the EFDC code to dynamically compute discharge through three the different types of hydraulic control structures that are present in the estuary and have a significant effect on the hydraulics. Thus, the methods used to compute discharge through sluice gates, flapper gates, circular pipe culverts, and box culverts, and their parameterization in the Herring River hydrodynamic model are also described.

It is not critical that the reader be familiar with the concepts and/or equations presented in this section in order to understand the results of the Herring River model. This model configuration section presents details on the setup of the model, including the explicit steps followed to generate the grid, input boundary conditions, and determine flow dynamics at control structures. Following model setup, the governing equations are solved at each grid point through an iterative method. The model is then able to calculate the water surface elevation, velocity, and salinity at each time step. Once a certain level of accuracy is attained, the model advances to the next time step in the simulation and repeats the calculations. This methodology is continued until the model has simulated the entire time period of interest.

5.2.1 Grid Generation

After choosing the appropriate model, the first step in building the model is constructing the model grid. The grid is a digital abstraction of the prototype's geometry that provides the spatial discretization on which the model equations are solved. Different numerical methods require different types of grids, each having unique geometrical requirements. The grid building process involves using geo-referenced digital maps or aerial photos to define the model domain, then the grid is generated within this domain providing the desired degree of spatial resolution, and topographic data is incorporated by interpolation of elevation values to grid nodes or cells within the domain.

For EFDC, the computational grid defines the spatial domain on which EFDC performs its finite difference calculations. EFDC requires a structured grid, either Cartesian or curvilinear orthogonal, comprised of a number of grid cells on which model the variables are computed. The accuracy of the model is highly dependent on accurate representation of the form of the real system expressed through the model grid. While a curvilinear orthogonal grid is more difficult to implement than a strict Cartesian grid, the curvilinear option was chosen because of its increased flexibility, allowing grid boundaries to better follow natural irregular boundaries. The curvilinear orthogonal grid also allows gradual variation in horizontal resolutions, such that higher resolution areas can be defined in areas where greater detail is required. The model grid developed for the Herring River model is complex, and the generation of a satisfactory grid required modification of the typical methods used to generate computational grids for EFDC. To create the Herring River grid system, a method for 'nearly-orthogonal' grid generation (Akcelik et. al. 2001) was implemented and is described in detail within this section.

Topographic data from a number of sources were provided to Woods Hole Group for use in constructing the Herring River model grid. These data include: (1) a collection of RTK-GPS data, total station data, and map estimates collected in 1998-1999 used in the previous modeling study conducted by the University of Rhode Island for the National Park Service (Spaulding and Grilli, 2001) shown in Figure 5-1; (2) recent photogrammetric survey data provided by Massachusetts Coastal Zone Management shown in Figure 5-2; and (3) additional RTK-GPS river cross section data collected by the National Park Service in March 2008 in order to better define the bathymetry in the lower portion of the Herring River shown in Figure 5-3. Furthermore, the low-lying road survey conducted in December 2006 by Slade and Associates, Inc. was used to ensure that all roadways were accurately represented in the model grid and additional RTK-GPS "observation points" collected by the National Park Service in August 2008 were compared to the grid to verify its accuracy.



Figure 5-1. Bathymetric and topographic data collected in 1998-1999.



Figure 5-2. Photogrammetric data collected in 2007 and provided by Massachusetts Coastal Zone Management.



Figure 5-3. Additional RTK-GPS bathymetric cross sections collected by National Park Service in 2008.

All topographic and bathymetric data were checked for consistency, and when necessary converted to the Massachusetts Mainland State Plane NAD83 horizontal datum and NAVD88 vertical datum, then merged into a single data set. The 12 foot NAVD88 contour line from the merged data set was then used to roughly define the domain's boundaries in block rectangular fashion. Figure 5-4 shows the 12 foot NAVD88 contour in green and the rough block rectangular boundary used for initial grid generation in red.

The block rectangular boundary definition is required input for the EFDC companion grid generation code, GEFDC. Definition of the grid boundary as a block rectangular boundary is the first step in grid generation and is constrained such the number of "east" facing sides is equal to the number of "west" facing sides and the number of "north" facing sides is equal to the number of "south" facing sides. Figure 5-5 shows how the block rectangular boundary is represented in the logical domain where the orientation of the block rectangular boundary segments is more evident. Once the boundary of the domain has been approximated in this way grid points are distributed along the boundary with spacing that ultimately determines the resolution of the final grid.



Figure 5-4. 12 foot contour from merged elevation data (green) and block rectangular boundary definition in the physical domain (red).

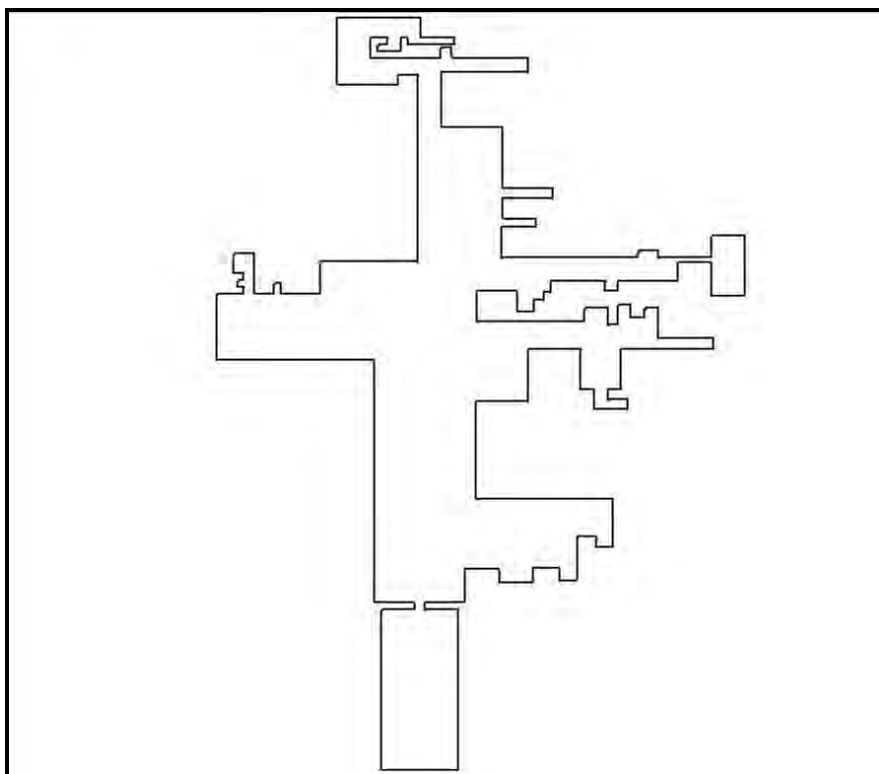


Figure 5-5. Block rectangular boundary definition in the logical domain.

Curvilinear-orthogonal grids are created by determining the computational domain coordinates (ξ, η) in the interior of the domain that correspond to the physical domain coordinates (x, y) . Given that physical Cartesian coordinates (x, y) are linear in nature, Equation 1 can be used to determine grid vertex points in the interior of the domain.

$$\frac{\partial}{\partial \xi} \left(f \frac{\partial x}{\partial \xi} \right) + \frac{\partial}{\partial \eta} \left(\frac{1}{f} \frac{\partial x}{\partial \eta} \right) = 0 \quad (1a)$$

$$\frac{\partial}{\partial \xi} \left(f \frac{\partial y}{\partial \xi} \right) + \frac{\partial}{\partial \eta} \left(\frac{1}{f} \frac{\partial y}{\partial \eta} \right) = 0 \quad (1b)$$

where

$$f = \frac{h_\eta}{h_\xi} \quad (2)$$

is the scale factor ratio, and the scale factors are described by

$$h_\xi = \sqrt{\left(\frac{\partial x}{\partial \xi} \right)^2 + \left(\frac{\partial y}{\partial \xi} \right)^2} \quad (3a)$$

$$h_\eta = \sqrt{\left(\frac{\partial x}{\partial \eta} \right)^2 + \left(\frac{\partial y}{\partial \eta} \right)^2} \quad (3b)$$

GEFDC uses the method of Ryskin and Leal (1983) to generate the horizontal model grid. In this method, f is allowed to vary throughout the domain as it is first calculated on the boundary then interpolated into the domain in an iterative fashion until a sufficiently converged solution for the grid is found. In areas where the scale factors iterate towards zero, the GEFDC code was modified to incorporate the method of (Akcelik et. al., 2001) for nearly orthogonal two-dimensional grid generation with aspect ratio control. Using this method, additional terms are added to Equation (1) which makes the local scale factors approximately the mean scale factor across each grid line. Equation (4) shows the additional inhomogeneous source terms that are added to Equation (1).

$$\frac{\partial}{\partial \xi} \left(f \frac{\partial x}{\partial \xi} \right) + \frac{\partial}{\partial \eta} \left(\frac{1}{f} \frac{\partial x}{\partial \eta} \right) + P_x(h_\xi) + Q_x(h_\eta) = 0 \quad (4a)$$

$$\frac{\partial}{\partial \xi} \left(f \frac{\partial y}{\partial \xi} \right) + \frac{\partial}{\partial \eta} \left(\frac{1}{f} \frac{\partial y}{\partial \eta} \right) + P_y(h_\xi) + Q_y(h_\eta) = 0 \quad (4b)$$

where

$$P(h_\xi) = c \left(h_\xi - \frac{\bar{h}_\xi^2}{h_\xi} \right) \quad (5a)$$

$$Q(h_\eta) = c \left(h_\eta - \frac{\bar{h}_\eta^2}{h_\eta} \right) \quad (5b)$$

$P(h_\xi)$ and $Q(h_\eta)$ favorably control the value of the scale factors and thus improve the aspect ratio of the resulting grid. They act as distributed forces repelling or attracting neighboring grid points, preventing grid lines from collapsing, when the local scale factor deviates from the mean scale factors $\bar{h}_\xi$ and $\bar{h}_\eta$ (computed on grid lines of constant η and constant ξ , respectively). The aspect ratio forcing constant, c , controls the strength of the forcing functions and can be varied throughout the domain. Also to improve grid quality, GEFDC was modified to allow the grid boundary points to “slide” along the boundary from their initial evenly distributed positions during the iterative grid generation process resulting in boundary point distribution that allows for better orthogonality of the generated grid.

After the grid was generated by this method, grid cells that lay outside the 12 foot contour were removed from the grid resulting in a better approximation of the upper bounds of the marsh surface. The resulting grid has 85,157 cells ranging in size from less than 10 ft wide near the Chequessett Road Dike to more than 100 ft wide in the kettle ponds. The grid has satisfactory orthogonality and aspect ratio, as well as smooth boundary point distribution and smooth resolution change. A histogram showing the deviation from orthogonality for the Herring River model grid is presented in Figure 5-6 and a color contour plot in Figure 5-7 shows how this deviation is distributed throughout the model domain. More than half of the grid cells have less than 1 degree deviation from orthogonality while more than 90% of the cells have less than 6 degrees deviation. Essentially, this means the Herring River grid is of sufficient quality and has more than adequate resolution.

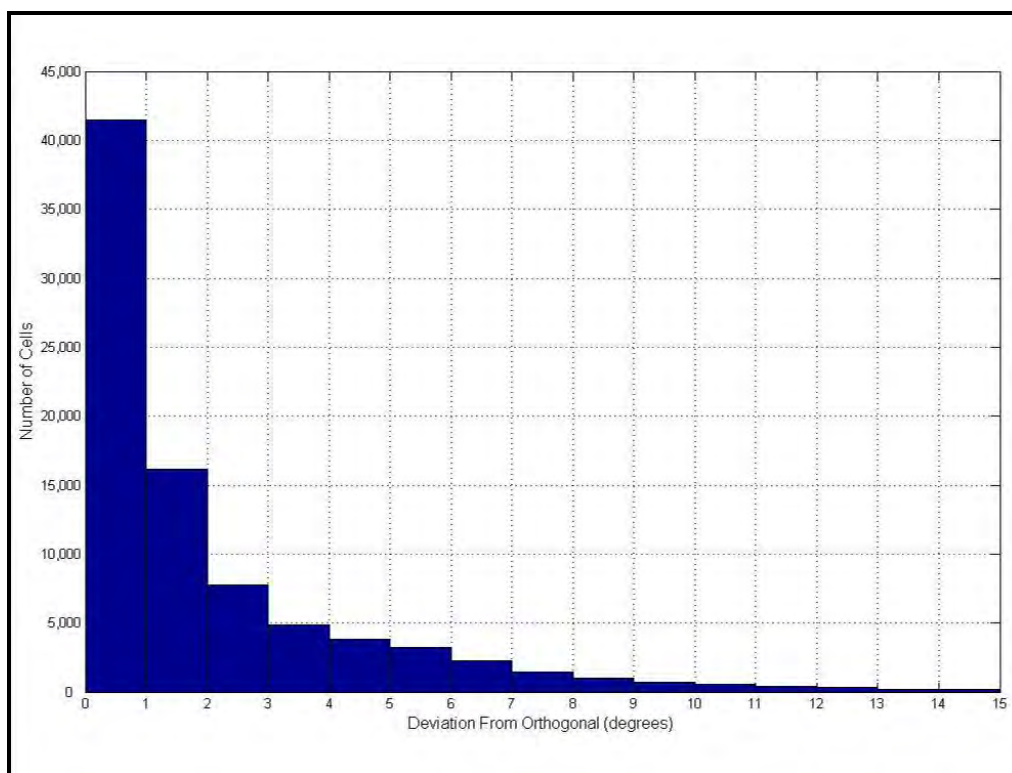


Figure 5-6. Histogram of grid cell deviation from orthogonality.

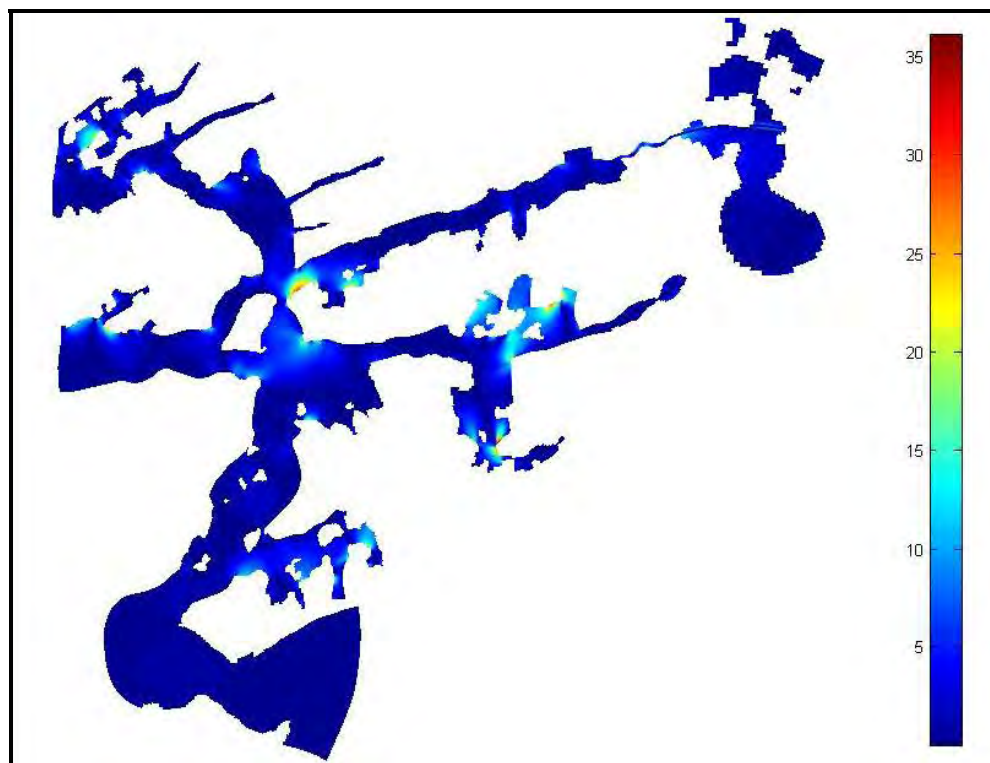


Figure 5-7. Color Contours showing deviation from orthogonality.

After the horizontal definition (2-D) of the grid was defined, the merged elevation data set was linearly interpolated to the model grid and the grid was carefully checked to ensure accurate elevation information. Bathymetric data were initially limited below the Chequessett Road Dike region known as “The Gut” (although this was subsequently updated – see section 6.0), and therefore, reasonable estimates to represent this region were utilized. Bathymetric data were also limited for most of the estuary’s upper reaches including Mill Creek, Bound Brook and the upper parts of Pole Dike Creek and the Herring River. In all these areas, cell elevations were interpolated between measured data points to best represent the channel and marsh plain. The final grid with colored elevation contours is presented in Figure 5-8, and a closer viewpoint of the Chequessett Road dike region is shown in Figure 5-9, to illustrate the detailed resolution in this region.

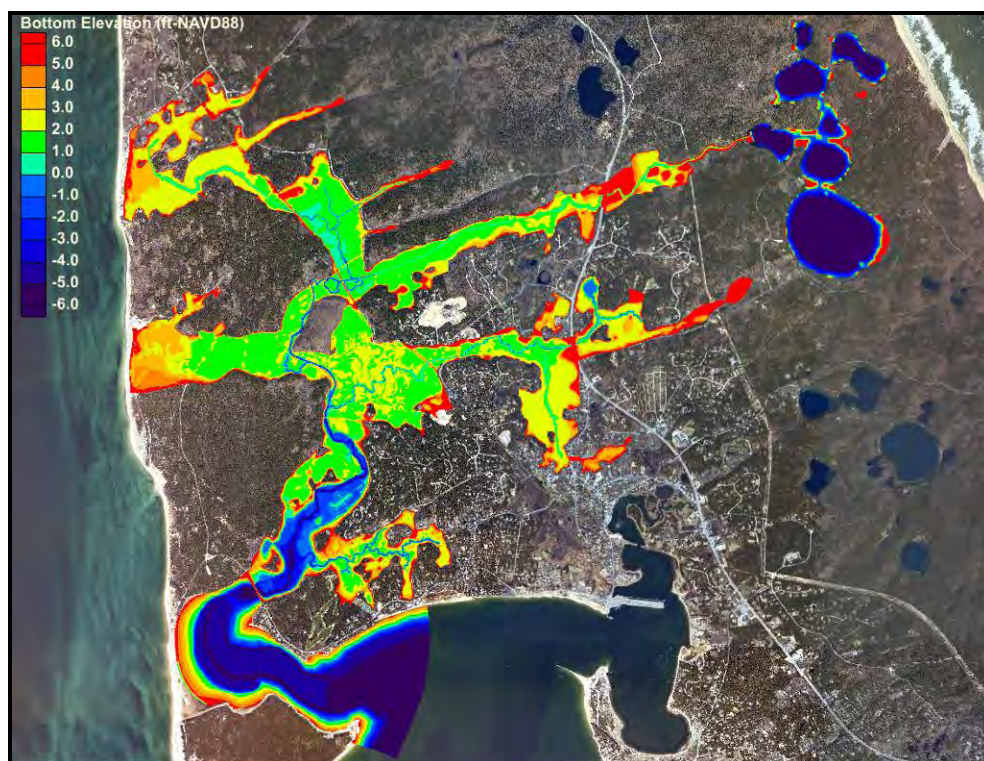


Figure 5-8. Herring River model grid bottom elevation contours.

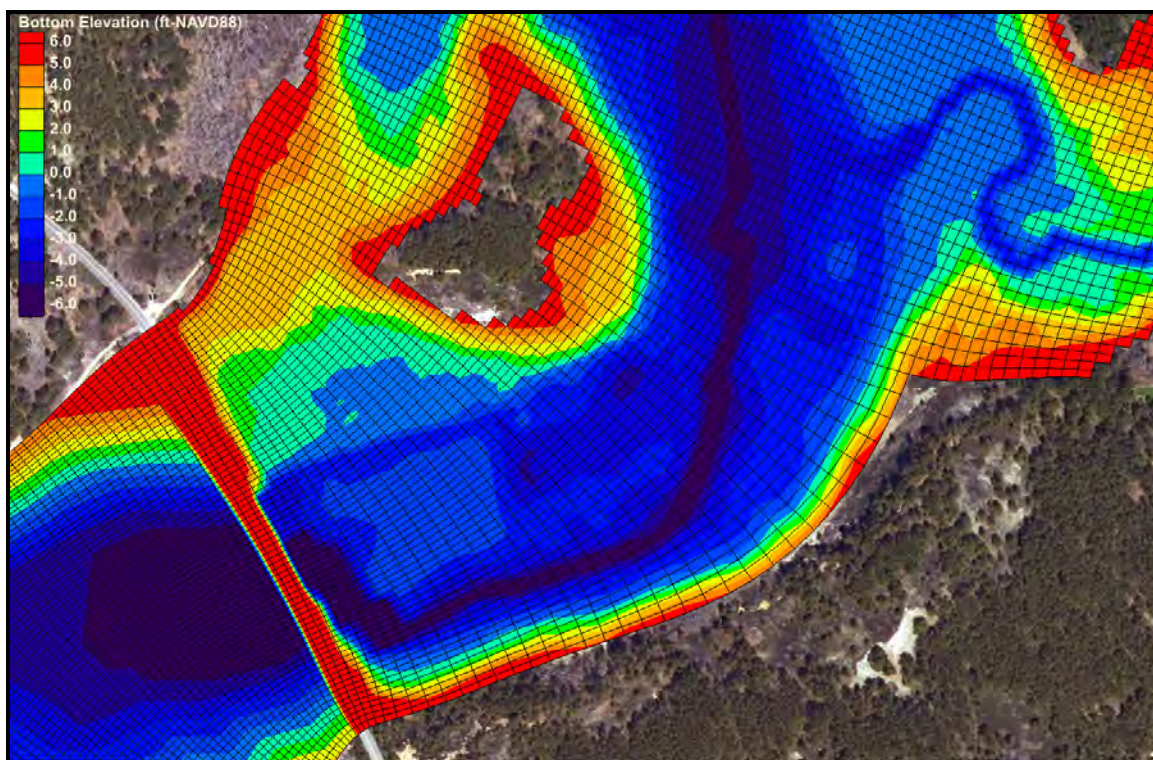


Figure 5-9. Detail of model grid showing bottom elevation contours and individual grid cells near the Chequessett Neck Road Dike.

5.2.2 Boundary Conditions

In order for the Herring River model to compute a hydrodynamic solution it is necessary to specify the model variables on the domain boundaries. Most of the model's boundary is considered to be a "land" boundary, which for the Herring River model was specified at an elevation of 12 ft NAVD88. This elevation provides the upper limit of expected water surface elevation during extreme storm events (100-year return period). At these land boundaries, water is constrained to flow only parallel to the boundary. The primary forcing for the model is provided by an open boundary at the southern end of the model domain in Wellfleet Harbor. At this location, time dependant water surface elevation and salt concentration is specified. Freshwater inflow volumetric flux is also specified in the model at three separate locations (Bound Brook, upper Herring River, and Pole Dike Creek) to simulate freshwater inflow into the estuary. The approximate location of these freshwater boundary conditions is shown in Figure 5-10. The red arrows show the freshwater influx locations, while the blue line shows the primary forcing boundary condition for water surface elevation and salinity. Figure 5-10 also shows the location of observation gauge stations in green. These stations are referenced throughout the report. All hydrologic data collected for this project was provided by various agencies that are part of the HRRC and provided to Woods Hole Group referenced to a vertical datum and corrected for atmospheric pressure.

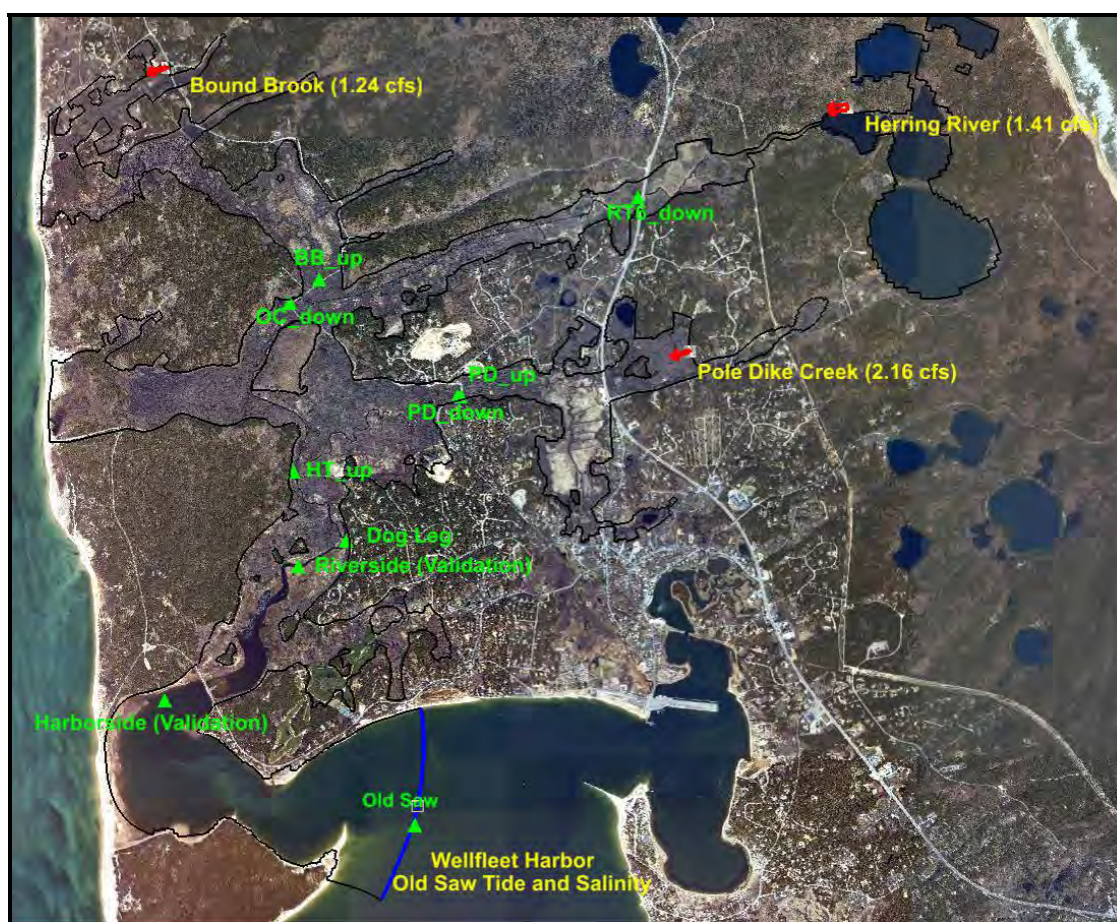


Figure 5-10. Locations of boundary conditions and observation gage locations.

Additional boundary conditions are applied over the entire model domain. These include atmospheric forcing and bottom frictional coefficients. For the Herring River model rainfall is input over the entire domain when appropriate and spatially variable bottom roughness length is specified and was varied to help achieve model calibration.

5.2.2.1 Tidal Forcing Data

Tidal data observed at the Old Saw location (Figure 5-10), including water surface elevation and salinity measurements for the time period between September 4, 2007 and October 4, 2007 are shown in Figure 5-11. These data were used to provide the tidal boundary conditions for the model calibration period. During this time period, the tide progress through an entire spring-neap-spring cycle representing a good range of conditions for model calibration. Figure 5-11 represents one specific example of the tide data specified at the Wellfleet Harbor boundary condition. Additional tidal boundary conditions were used for model validation and simulation of various conditions and return period storm events. The observed salinity data from September 4 to approximately September 8, 2007 indicate there was a measurement error for the initial part of the observations.

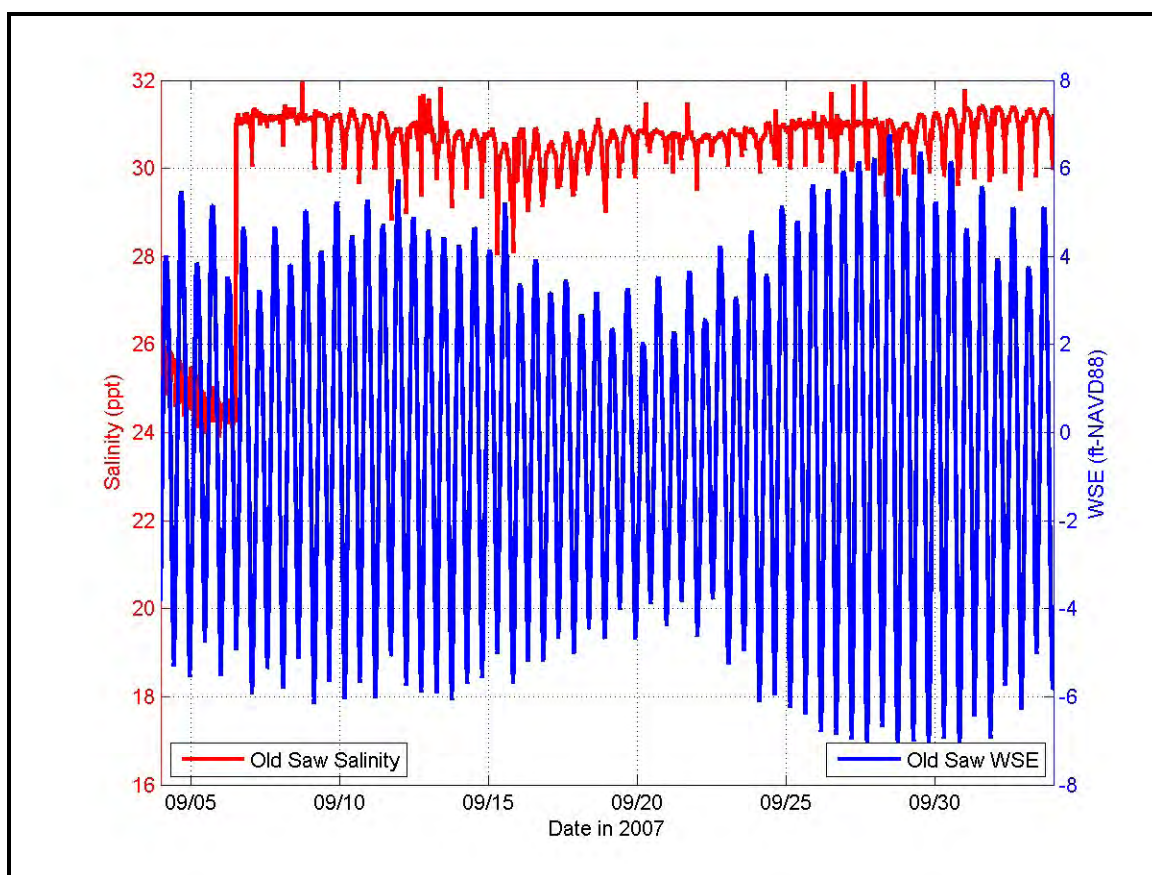


Figure 5-11. Water surface elevation and salinity at the Old Saw location during September 2007 (collected by NPS).

5.2.2.2 Freshwater Input

Freshwater input into the Herring River comes from a combination of rainfall, direct runoff, and groundwater flow. While the volume rate of freshwater input is known to be quite low, less than 11 cubic feet per second (Spaulding and Grilli, 2001; Roman 1987), actual fresh water input rates vary in time and spatial distribution, and are difficult to measure. Measured stream flow discharge data are not available for the river during the time periods used for model calibration and validation. However, estimates of freshwater inflow by sub-basin produced by a USGS groundwater model (Masterson, 2004) were used as a starting point for providing freshwater inflow to the model domain at the Bound Brook, Upper Herring River, and Pole Dike Creek locations shown in Figure 5-10. The USGS groundwater model data is presented in Figure 5-12.

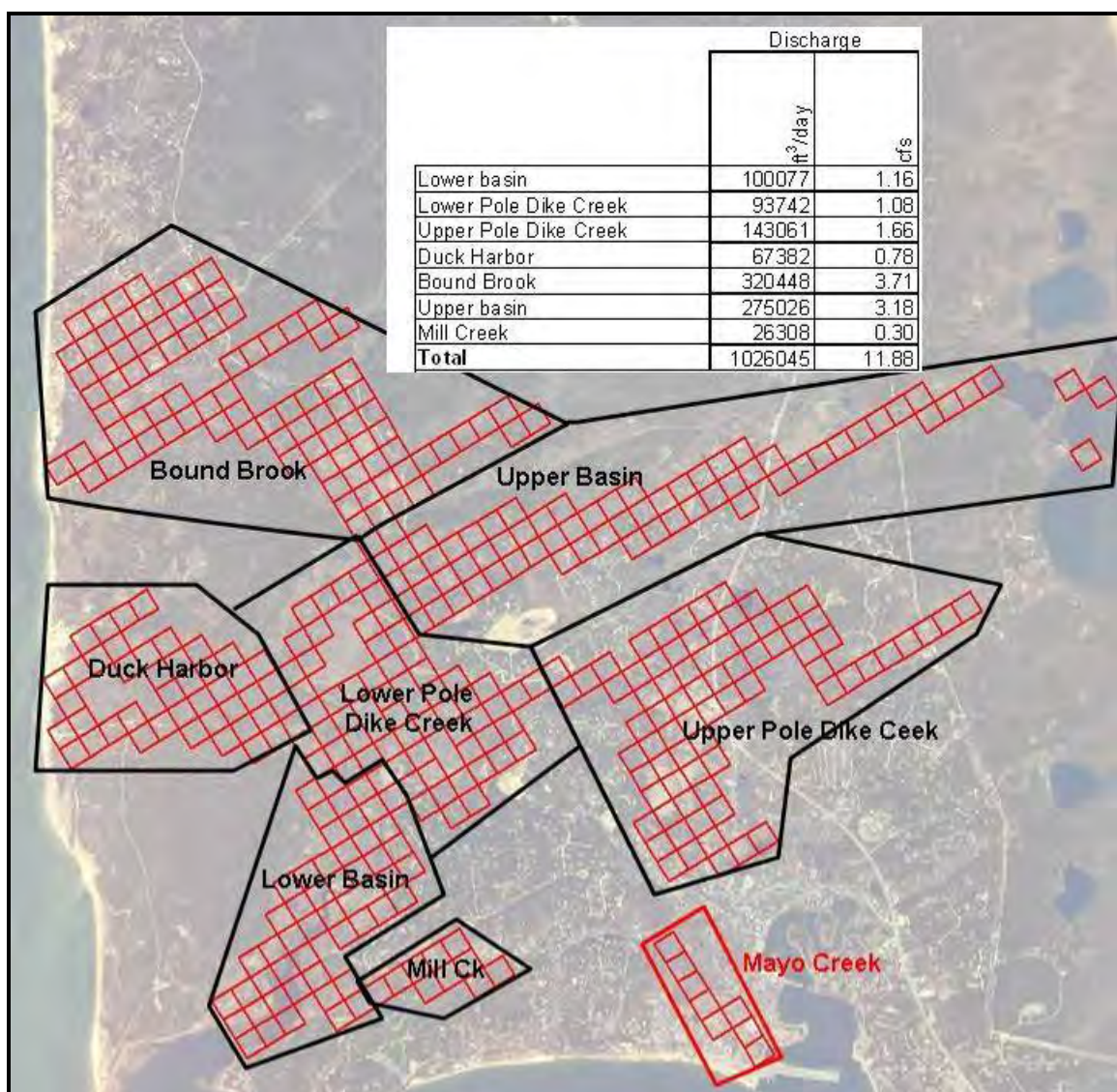


Figure 5-12. USGS groundwater model sub-basins and freshwater discharge by sub-basin.

Through sensitivity testing, the Herring River model was determined to be somewhat sensitive to the volumetric flux of freshwater inflow, particularly in the upper reaches of the estuary where the tidal influence is less significant. In order to match the observed water surface elevation, the freshwater inflow was varied within reasonable ranges at the three input locations. Ultimately, constant freshwater inflow was specified for model calibration and validation at the constant flow rates shown in Figure 5-10 (1.24 cfs for Bound Brook, 1.41 cfs for upper Herring River, and 2.16 cfs Pole Dike creek). These flow rates, while somewhat smaller, are reasonable upon comparison to output from the USGS groundwater model and information from the previous two modeling studies (Roman 1987, Spaulding and Grilli 2001). They also provide good agreement for observed water surface elevations for calibration of the Herring River model. These rates will ultimately vary given different environmental conditions; however, they are good approximations of the freshwater inflow during the typical time periods associated with the calibration and validation. As tides are restored to the estuary system, the relative contribution of freshwater input will become less important when compared to tidal influences, and therefore the model is less sensitive to freshwater inflow variations for the restoration alternative simulations.

5.2.2.3 Atmospheric Data

Bi-hourly precipitation data collected at the National Atmospheric Deposition Program (NADP) station MA01, as shown in Figure 5-13, was used to provide rainfall input to the model. Figure 5-14 shows the bi-hourly rainfall totals collected during September of 2007. Rainfall from this single location is assumed to fall uniformly over the entire model domain when applicable, which may not be an accurate assumption considering the significant land area that comprises the Herring River estuarine system.

5.2.2.4 Bottom Friction

EFDC uses the log law roughness length to parameterize bottom drag forces. Roughness length is a composite resistance including both form-drag and skin friction of the sediment and is a dynamic length rather than a mere geometric scale. EFDC utilizes a quadratic friction relationship to relate the bottom stress to the velocity in the bottom cell layer. The bottom stress ($\vec{\tau}_b$) is calculated as:

$$\frac{\vec{\tau}_b}{\rho} = C_d \sqrt{u^2 + v^2} (\vec{u} + \vec{v}) \quad (6)$$

Where C_d is a quadratic drag coefficient, $\vec{u}$ and $\vec{v}$ are the x and y components of the velocity, and ρ is the water density. Assuming that the bottom boundary layer exhibits a logarithmic velocity profile, the drag coefficient can be related to the dimensional roughness length (Z_0) by:

$$C_d = \left(k \frac{1}{\ln \left(\frac{H}{Z_0} \right)} \right)^2 \quad (7)$$

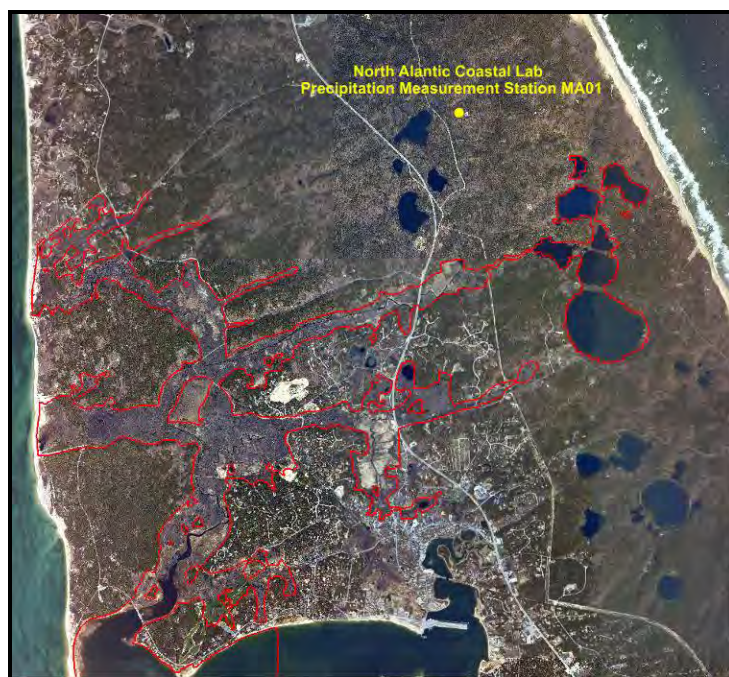


Figure 5-13. Location of rainfall observations used to specify rainfall levels within the model during calibration and validation time periods.

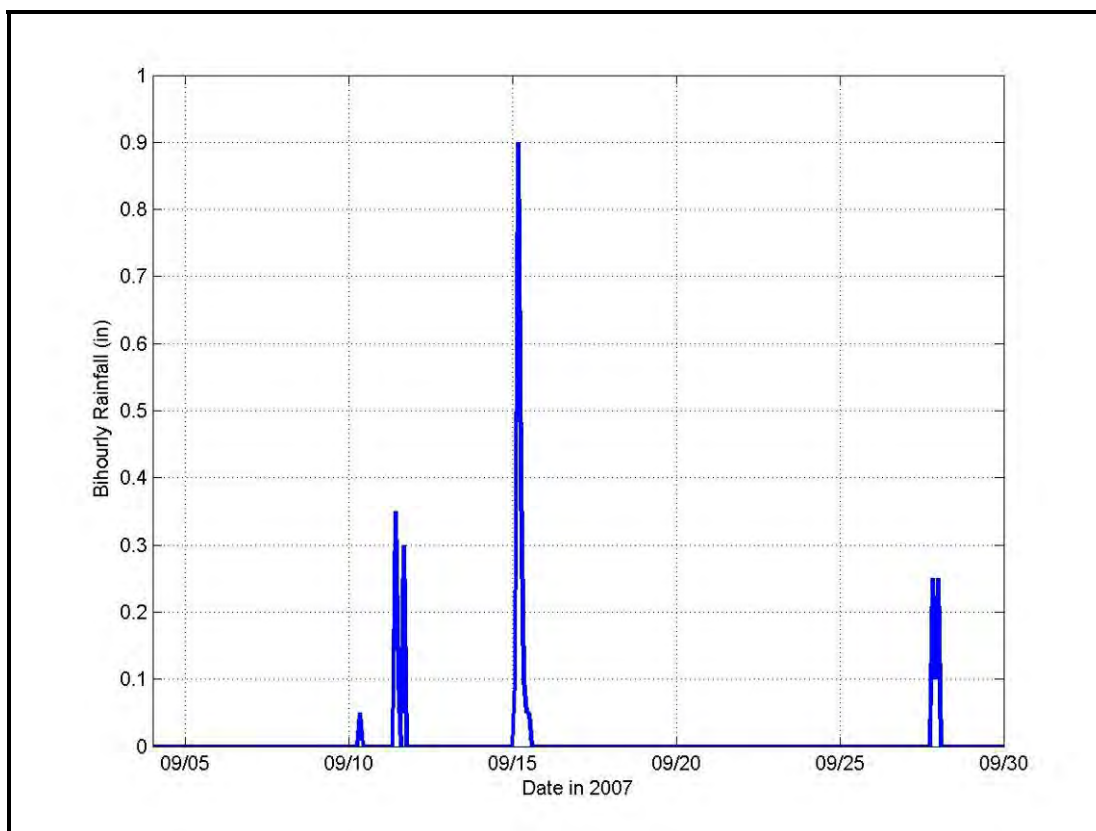


Figure 5-14. Bi-hourly rainfall collected at NADP station MA01 during September 2007.

Where k is the Von Karman constant and H is the depth of the flow. In EFDC values of Z_0 are specified by cell to control the bottom drag forces. Physically, Z_0 represents the distance from bottom within the bottom boundary layer at which the velocity appears to go to zero based on the logarithmic velocity profile.

Each grid cell may have a different value for roughness length. Physically, bottom drag forces depend on a number of phenomena that are difficult to characterize. These include bottom material type, growth of biota, and the amount of channel meander, which all contribute to the overall energy loss that are accounted for by the roughness length. Because the bottom friction parameterization in the model accounts for the effect of a number of different factors, it is difficult to determine a priori what values will provide the best results and thus it is typically used as the primary parameter for “tuning” the model to reproduce the observations during the model calibration process. For the Herring River model a value of 0.033 m was initially used throughout the model domain then local adjustments were made to the roughness length values in order to improve the model results to match observed data. Table 5-1 summarizes the roughness length values used for the final calibration of the Herring River model. These values are considered within the range of normal bottom friction values determined through empirical laboratory testing. However, the value used in the lower portion of Pole Dike Creek is relatively high. Observed data at the Pole Dike Creek gauge locations show the complete dampening of the tidal signal at this point in the estuary. This is likely due to the dense submerged aquatic vegetation (SAV) that exists in this creek. Observations conducted in 2008 indicated the creek to be almost impenetrable by canoe. Therefore, there are significant frictional and/or constriction influences in this portion of the estuarine system. In order to match the observed water surface elevations in this portion of the estuary, a higher roughness length value was specified for the Pole Dike Creek portion of the estuary.

Table 5-1. Bottom roughness length for the Herring River model.

Location Description	Roughness length (meters)
Herring River between Chequessett Neck Rd dike and High Toss Rd	0.025
Herring River between High Toss Rd. and Bound Brook Is. Rd.	0.05
Herring River between Bound Brook Is. Rd. and Route 6	0.06
Lower Pole Dike Creek	0.20
Remaining Model Domain	0.033

5.2.3 Control Structures

In addition to the culverts and gates at the Chequessett Neck Road dike, there are sixteen additional hydraulic control structures found within the Herring River estuary. Therefore, due to the high number, as well as varying type, of anthropogenic influences, EFDC was upgraded to specifically model these types of control structures. Additional subroutines were applied within the EFDC code, using well established methods to dynamically compute discharge through the control structures. Flow charts showing the decision tree and equations used by these subroutines for calculating flow through circular pipe culverts, sluice gates, and flapper gates are described in more detail throughout this section. The locations of the control structures within

the Herring River estuary are shown in Figure 5-15, with the control structures that are directly modeled labeled in blue. Control structures that were of less importance, specifically those located in the far upper reaches of the system, are shown in red and were not directly included in the model simulations. Circular pipe culverts that were installed throughout the system are of various lengths, pipe diameters, and construction materials.

5.2.3.1 Culverts

The characteristics of flow through culverts are complicated by many variables including the geometry, slope, size, roughness, approach, and headwater and tailwater conditions. Chow (1959) classified flow through a culvert into six flow types based on culvert slope, culvert height, headwater depth and tailwater depth and presents a method for approximating the flow. This method was simplified by assuming only hydraulically long culverts (limiting the number of flow types to four) and implemented in a subroutine within EFDC to dynamically calculate the flow through the culverts in the Herring River model. Figure 5-16 presents a flowchart which describes how the flow is determined within this subroutine. At each model time step the discharge through each culvert is determined and the appropriate volume is moved from the upstream culvert cell to the downstream culvert cell. The downstream and upstream end of each culvert, as well as the tailwater and headwater depths, change dynamically in the model since the tidal fluctuations result in bidirectional flow through each of the culverts.

For the Herring River model, culvert invert elevations and construction material detailed in the 2006 Slade and Associates Inc. low-lying road survey were used to provide input for the culverts in the model. Culvert lengths were estimated from aerial photographs since no measurements of each culvert were provided in the survey data. An initial typical value of 0.035 was used for Manning's n for each culvert. During model calibration some culvert roughness values were adjusted to provide better agreement between model results and observations. The final culvert parameters used in the calibrated model are summarized in Table 5-2.

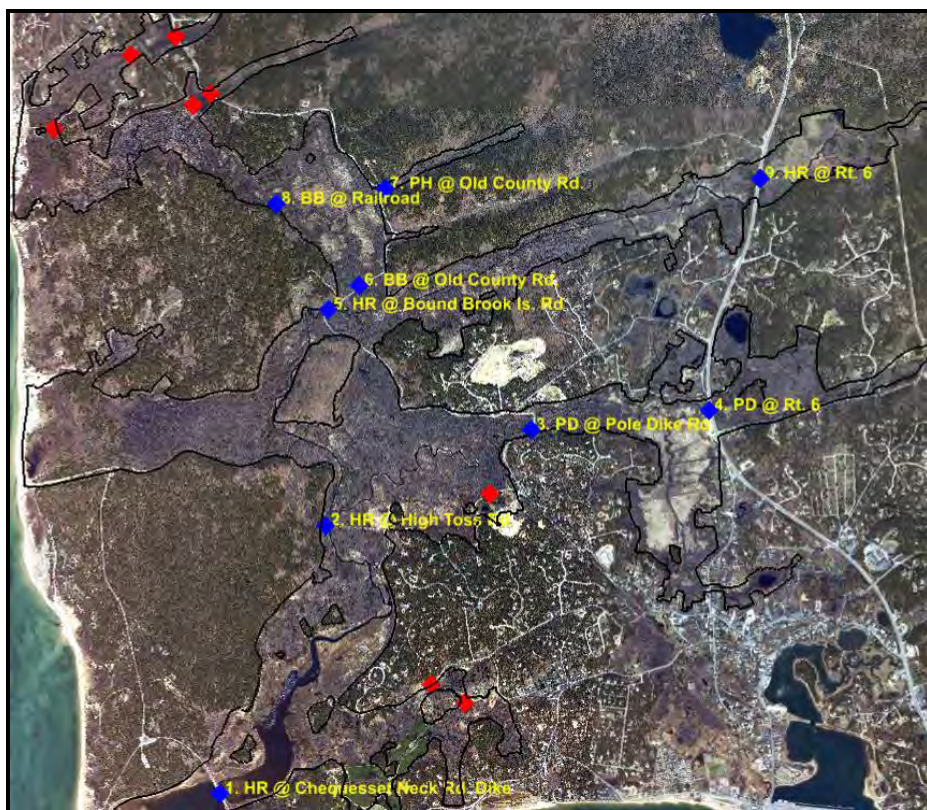


Figure 5-15. Location of culverts in the Herring River estuary. Locations presented in blue were dynamically simulated within the model, while those in red were treated as simple hydraulic connections.

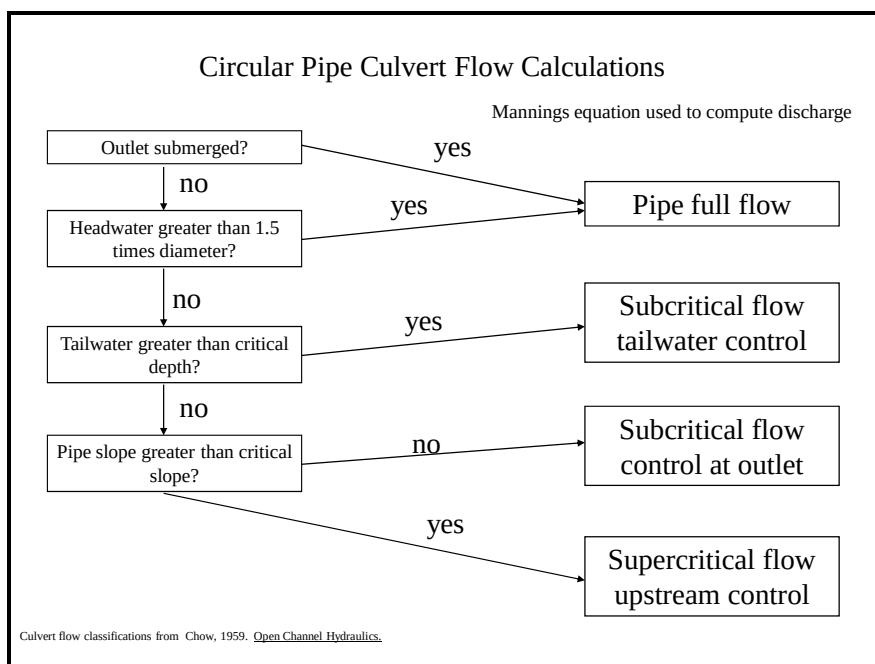


Figure 5-16. Flow chart for culvert flow type determination in the Herring River model.

Table 5-2. Herring River model circular pipe culvert details.

Location	Material	Length (ft.)	Dia. (ft)	Upstream Invert (ft - NAVD88)	Downstream Invert (ft - NAVD88)	Manning's n
Herring River @ High Toss Rd.	Concrete	39	5	-3.8	-4.3	0.035
Pole Dike Creek @ Pole Dike Rd.	Ceramic	46	3	-1	-1.3	0.035
Pole Dike Creek @ Rt. 6	Concrete	86	3	-0.44	-0.64	0.035
Herring River @ Bound Brook Is. Rd.	Concrete	46	5	-3.3	-3.5	0.04
Bound Brook @ Old County Rd.	Ceramic	36	2	-2.1	-2.3	0.08
Paradise Hollow @ Old County Rd.	Ceramic	49	0.5	-0.1	-0.75	0.035
Bound Brook @ Railroad	Corrug. Metal	82	2	-2.6	-2.6	0.035
Herring River @ Rt. 6	Concrete	82	7	-2.2	-2.2	0.035

5.2.3.2 Sluice Gates

Tidal flow from Wellfleet Harbor into the Herring River estuary is entirely controlled by a single sluice gate in the Chequessett Neck Road dike. An accurate model of the entire estuarine system in its existing state depends greatly on accurate representation of this single control structure. Therefore, a subroutine designed specifically for dynamically calculating the discharge through a sluice gate was incorporated into the EFDC code. Flow through a sluice gate can be characterized by two basic parameters: the nature of the flow (subcritical or supercritical flow) and the water depth (gate submerged or not). Figure 5-17 shows how the sluice gate flow is characterized in the model based on these parameters. When the water surface is effectively determined to be below the top of the gate, the gate is modeled as a broad crested weir. When the gate is submerged, the appropriate equation for either free sluice flow (super-critical) or submerged orifice flow (sub-critical) is applied. To determine the flow through the sluice gate at a given model time step the depth of water on the upstream side of the gate is compared to the depth on the downstream side of the gate. If the depth on the downstream side is less than 0.64 times the depth on the upstream side, the red colored path in Figure 5-17 is applied and the appropriate super-critical flow equations are used. If the ratio of downstream head to upstream head is greater than 0.68, the purple colored path is applied and the appropriate formula for subcritical flow (sub-critical weir or submerged orifice) is applied. Otherwise both discharges are computed and a weighted average of the two is used (the yellow path). The equations for calculating broad-crested weir flow are the same as those applied to barrier flow in SIMSYS2D (Leendertse 1987). For super-critical weir flow,

$$Q = C1 \times W \times \frac{2}{3} \times \sqrt{\frac{2}{3} G} \times H_{up}^{\frac{2}{3}} \quad (8)$$

for sub-critical weir flow,

$$Q = C2 \times W \times H_{dw} \times \sqrt{2G(H_{up} - H_{dw})} \quad (9)$$

where $C1$ and $C2$ are the super-critical and sub-critical discharge coefficients respectively, W is the width of the gate, G is acceleration due to gravity, and H_{up} and H_{dw} are the upstream and downstream depths above the gate bottom. The equations used to compute free sluice flow and submerged orifice flow are the same as those applied in the Brunner (2002). For free sluice flow,

$$Q = C3 \times W \times B \times \sqrt{2GH_{up}} \quad (10)$$

and for submerged orifice flow,

$$Q = C4 \times W \times B \times \sqrt{2G(H_{up} - H_{dw})} \quad (11)$$

where $C3$ and $C4$ are discharge coefficients for free sluice flow and submerged orifice flow respectively and B is the height of the gate opening. The invert elevation, gate opening, and discharge coefficients applied in the calibrated Herring River model are shown below in Table 5-3. The opening height refers to the distance between the bottom of the culvert and the bottom of the sluice, or in other words, the vertical distance of the opening.

Different values of the discharge coefficients were used during an ebb and flood flow to achieve the best agreement between model results and observations. A similar approach was taken by Spaulding and Grilli (2001) in their inlet-basin model, where less frictional loss was applied during a flooding tide. Considering that the modeled sluice gate must reproduce the observed flow in the real system, the sluice gate asymmetrically placed in the downstream end of a box culvert, and that flows observed during flood and ebb tides exhibit different set-down characteristics, the use of variable discharge coefficients should be acceptable and provides the best agreement between modeled results and observations.

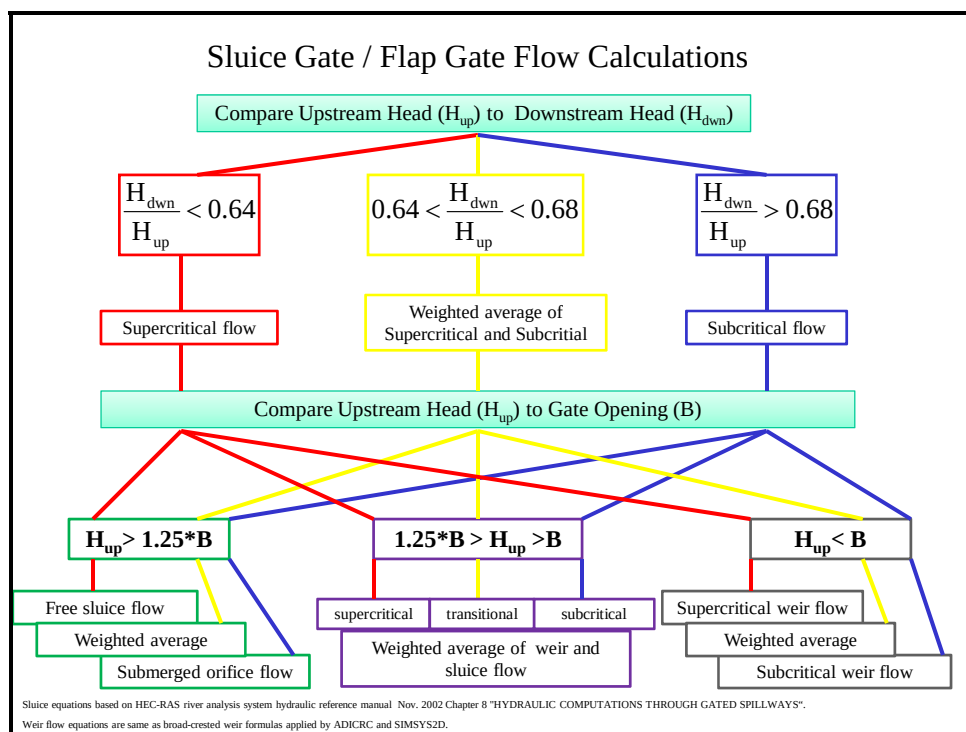


Figure 5-17. Flow chart for sluice and flap gate flow type determination for the Herring River model.

Table 5-3. Sluice gate invert elevation, opening height, and discharge coefficients.

Gate	Invert Elevation (ft-NAVD88)	Opening Height (ft)	C1	C2	C3	C4
Sluice Upstream	-3.38	1.67	1.375	1.375	1.4	1.35
Sluice Downstream	-3.38	1.67	1	1	0.6	0.8

5.2.3.3 Flap Gates

While a flooding tide is severely limited by the sluice gate, ebbing tide has significantly greater conveyance since the flap gates on the two neighboring box culverts open during an ebb flow. The flap gates are modeled like the sluice gate with a few modifications. The flap gates are modeled at all times as broad crested weirs with an additional head loss, HL , which reduces the depth H_{up} . The additional head loss varies depending on how much the flap is open. It is assumed, in order to open the flap some additional energy is required. As the flap opens, the

energy required to hold it open decreases, as the water level increases. The additional head loss is calculated by:

$$HL = HL_{\max} \times \left(1 - \frac{\frac{1}{2}(H_{up} + H_{dw})}{D_{HL}} \right) \quad (12)$$

The values used in the calibrated Herring River model are presented below in Table 5-4.

Table 5-4. Flap gate discharge coefficients and additional head loss parameters.

Gate	$HL_{\max}$ (ft)	D_{HL} (ft)	C1	C2
Flapper gates	1.97	2.9	1.375	1.375

5.3 MODEL CALIBRATION

Model calibration is the process in which model parameters are systematically adjusted through a range of acceptable values and results are examined using standard measures of error. Through a number of iterative simulations the configuration of model parameters that provides the best agreement between modeled variables and observed measurements is determined. Parameters adjusted during calibration of the Herring River model include: bottom roughness lengths, pipe culvert roughness coefficients (Manning's n), and sluice gate and flap gate discharge coefficients. The values presented in Tables 5-1 thru 5-4 are those used in the calibrated model. Additionally, because observed stream flow data were not available, reasonable constant flow rates for freshwater input into the upper reaches of the estuary were applied and adjusted during the calibration process to provide better agreement between observations and model results.

The Herring River model was calibrated to observations collected between September 5, 2007 and October 3, 2007 at seven locations throughout the estuary. These locations are shown in Figure 5-10 and descriptions of their locations are given in Table 5-5.

Table 5-5. Description of observation station locations.

Gauge Name	Location/Description
Dogleg	Approximately 5000 ft upstream of the Chequessett Neck Road Dike
HT_up	Immediately upstream of the culvert at High Toss Road
PD_down	Immediately downstream of the Pole Dike Creek culvert under Pole Dike road
PD_up	Immediately upstream of the Pole Dike Creek culvert under Pole Dike road
OC_down	Immediately downstream of the culvert on the main stem of the Herring River under Old County road
BB_up	Immediately upstream of the culvert on Bound Brook under Old County Road
Rt6_down	Immediately downstream of the culvert on the main stem of the Herring River under Route 6

The model performance is evaluated by comparing time series output from the model at specific observation points to the observed time series of both water surface elevation and salinity. The results are presented visually as time series plots and scatter plots, and absolute error of the model is quantified by calculating the bias and Root Mean Square Error (RMSE). Additionally, the most dominant tidal constituents (both amplitude and phase) as determined from tidal constituent analysis are compared.

The time series plots allow for a direct visual comparison of how the observed and modeled variables evolve through time and provide a “first glance” at the performance of the model. On the scatter plots, a point is plotted showing the modeled value for each discrete observed value. When the model has perfect agreement with the observed data, the points lie on a line with slope of unity (45 degrees) passing through the origin. The vertical distance between a point on the scatter plot and the line of perfect agreement represents the model error for that particular measurement. The overall error for a given observation time series is quantified in two ways:

$$Bias = \frac{\sum_{i=1}^n (p_{mod} - p_{obs})}{n} \quad (13)$$

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (p_{mod} - p_{obs})^2}{n}} \quad (14)$$

Where P_{mod} and P_{obs} are the modeled and observed values respectively and n is the number of discrete measurements in the time series. The bias provides a measure of how close on average the modeled results are to the observed data. A positive value indicates that the model is over-

predicting the observation while a negative value indicates that the model is under-predicting the observations; a bias of zero indicates that the on average over the time series the model reproduces the observations. The RMSE is an average of the magnitude of the error of each measurement in the time series. RMSE is always positive with smaller values indicating better model performance. Both the bias and RMSE are measures of absolute error having the same units of the measured quantity from which they are computed.

The U.S. EPA provides technical guidance on error statistic criteria for estuarine water quality models recommending that hydrodynamic variables (i.e. water surface elevation) have relative error less than 30% and transport variables (i.e. salinity) have relative error less than 25% (EPA, 1990). To compute relative error for water surface elevation the RMSE is normalized by the mean tidal range for Wellfleet Harbor of 9.74 ft, or a salinity value of 30 ppt.

5.3.1 Water Surface Elevation Calibration

This section presents the results of the calibrated Herring River model for water surface elevation. Visual comparison for the Dogleg and just upstream of the High Toss Road locations are shown in Figures 5-18 and 5-19, respectively. Figures 5-20 through 5-26 show scatter plots for all observation gauge locations, and include the bias and rmse error on the plot. Error measures for all observation gauge locations are summarized in Table 5-6. Overall the agreement is reasonable. The magnitude of the bias is less than 0.11 ft for all locations meaning that the calibration simulation reproduced average water levels that are within an inch or two of observed levels. RMSE is less than 0.4 ft for all locations indicating that on average the modeled water level is within a few inches of the observed level at any given time.

Table 5-6. Calibration water surface elevation error measures.

Gauge Location	Bias (ft)	RMSE (ft)	Relative Error (%)
Dogleg	-0.01	0.097	1.00
HT_up	0.05	0.102	1.05
PD_down	-0.11	0.150	1.54
PD_up	-0.06	0.133	1.37
OC_down	0.06	0.166	1.70
BB_up	0.08	0.357	3.67
Rt6_down	-0.10	0.179	1.84

At the Dogleg gauge results have a relative error less than 1% of the Wellfleet Harbor tidal range. Upstream of High Toss Road (HT_up), the error is nearly as small with a slight over-prediction of the peak water levels corresponding to rainfall events and a slight over-prediction of the overall tidal range. Downstream of Old County Road (OC_down) and upstream of Bound Brook (BB_up), the discrepancy in rainfall induced peaks and tidal range becomes more apparent, however the error is still quite small and well within recommended limits. For example, the peak that occurs after the September 15 rain event at the BB_up location that is over predicted by nearly one foot may be accounted for by the potentially oversimplified assumption of equally distributed rainfall amounts. Additionally, actual runoff rates into the main channels are potentially slowed by the resistance of the multitude of small streams and

ditches which feed the larger channels, infiltration into the soil, resistance to flow from vegetation, and evapotranspiration. However, since the main goal of the model is accurate prediction of tidal propagation, and not rainfall runoff directly, this is not a significant concern.

Generally, in the locations where tidal influence is apparent, the modeled water surface is characterized by a slight over prediction of the tidal range, which increases with distance upstream of the dike. In other words, the tidal range predicted by the model is slightly larger than the observed tidal range. This result can somewhat be expected when considering that the streams above High Toss road are quite narrow, in some places only a few meters wide. While the model grid resolution is quite high, it is not able to fully resolve the narrower streams and their meanders in the upper reaches of the estuary. Because of this and other less understood factors (e.g. resistance due to algal growth, micro-topography, storage in the marsh soils, SAV growth) in the upper reaches and marsh plains of the actual estuary, there is slightly greater tidal attenuation than that shown in the model. At the gauge locations above the realm of tidal influence, PD_down, PD_up, and RT_6 down the results are good, with average water levels within 0.1 ft of those observed.

Overall, the calibrated model reproduces the observed water levels within inches at all locations. The RMSE relative to the Wellfleet harbor tidal range is less than 3.7% for all gages and is well below the 30% limit recommended by the EPA for model calibration.

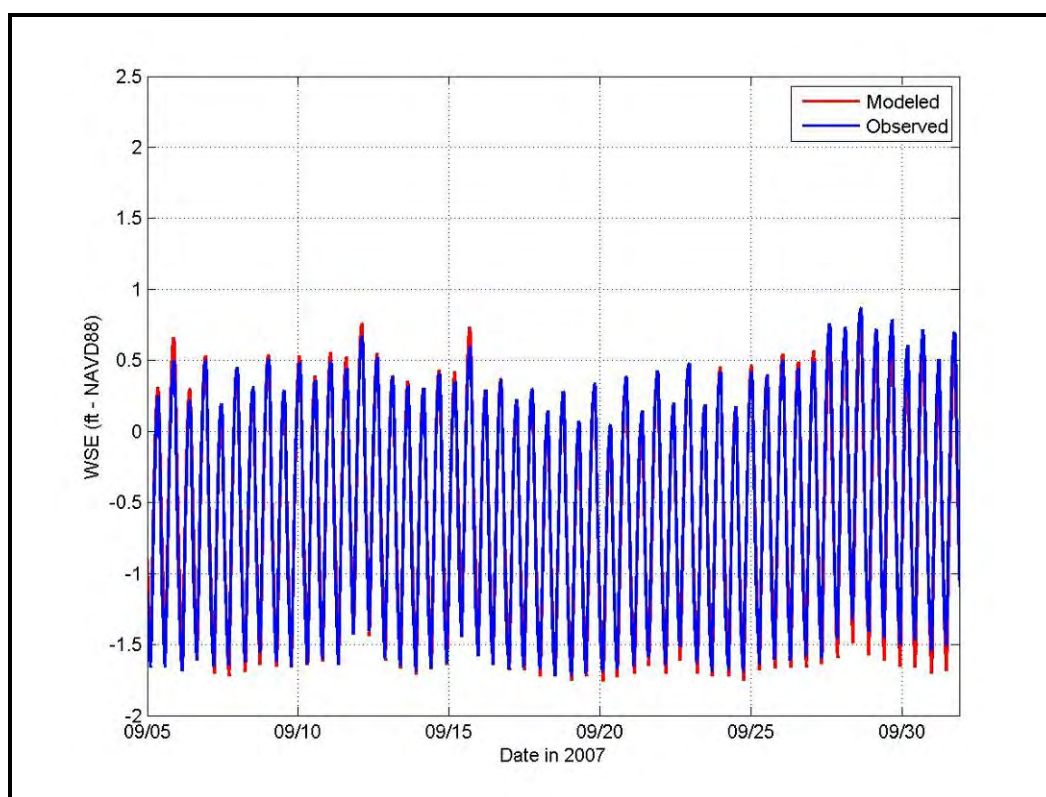


Figure 5-18. Dogleg water surface elevation comparison for modeled (red) and measured (blue) time series.

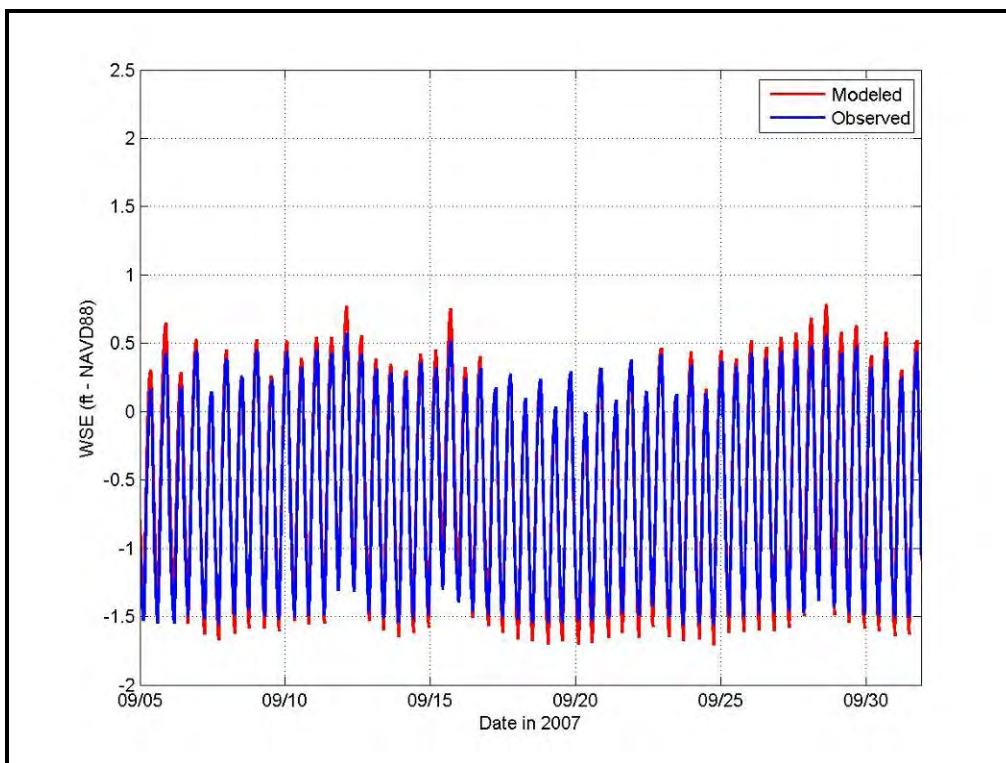


Figure 5-19. Upstream of High Toss Road (HT_up) water surface elevation comparison for modeled (red) and measured (blue) time series.

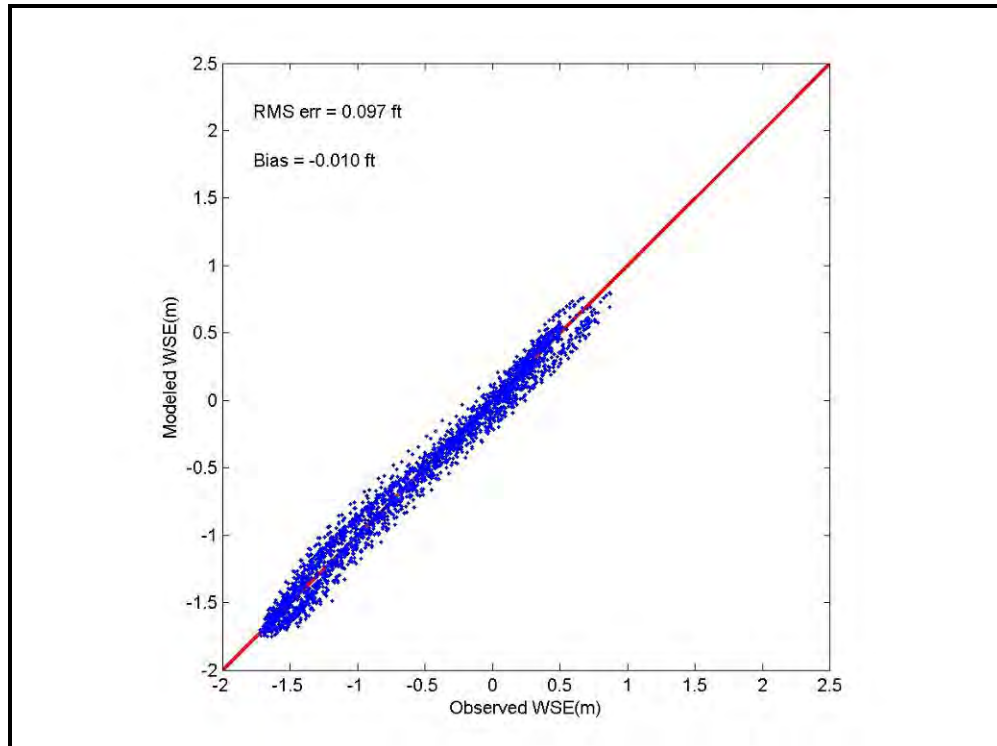


Figure 5-20. Dogleg water surface elevation scatter plot comparing modeled and measured water surface elevations.

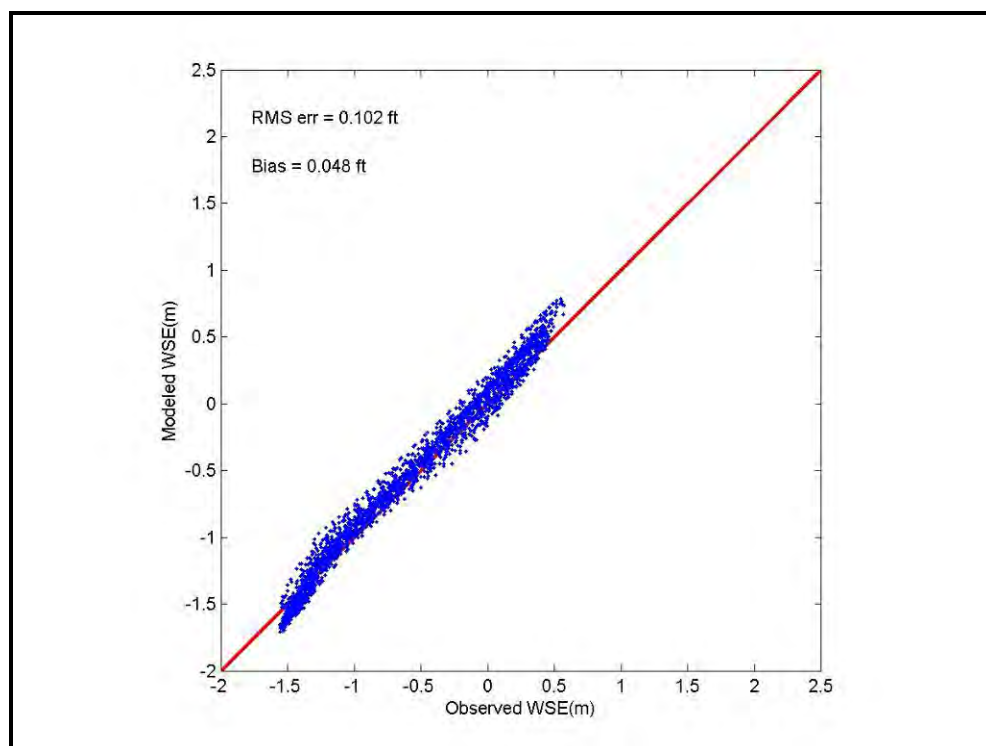


Figure 5-21. HT_up water surface elevation scatter plot comparing modeled and measured water surface elevations.

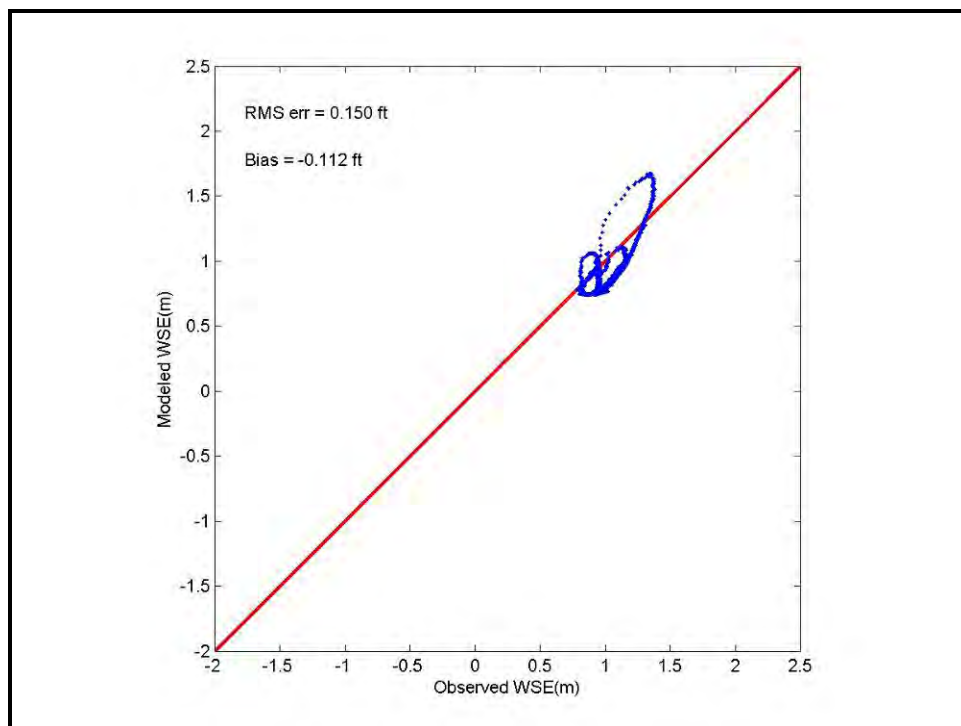


Figure 5-22. PD_down water surface elevation scatter plot comparing modeled and measured water surface elevations.

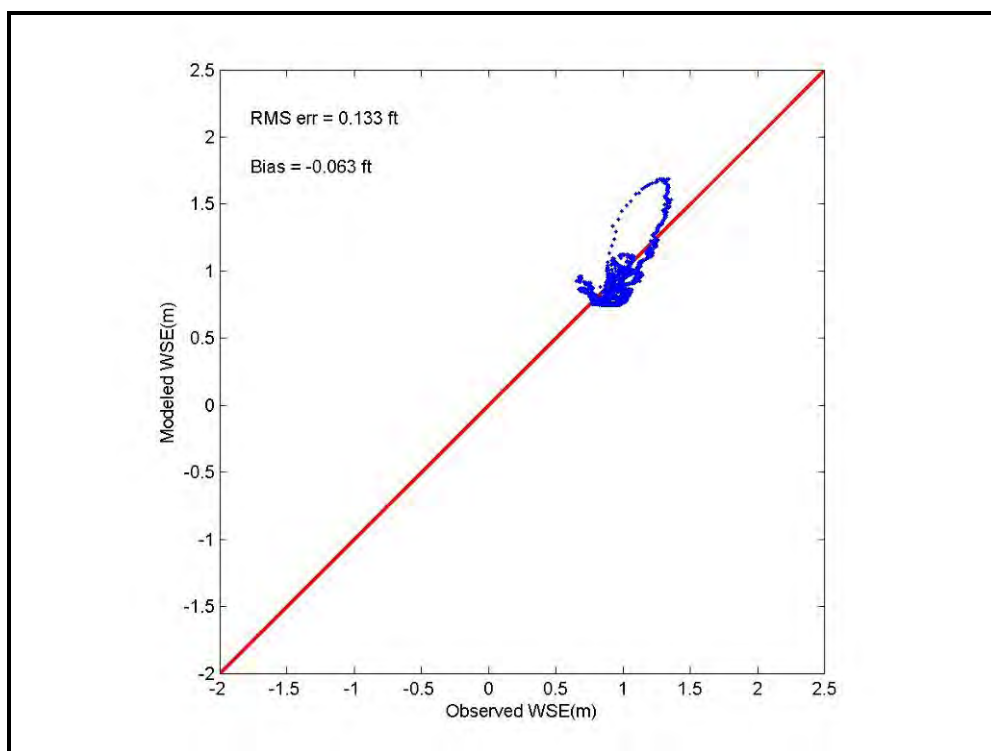


Figure 5-23. PD_up water surface elevation scatter plot comparing modeled and measured water surface elevations.

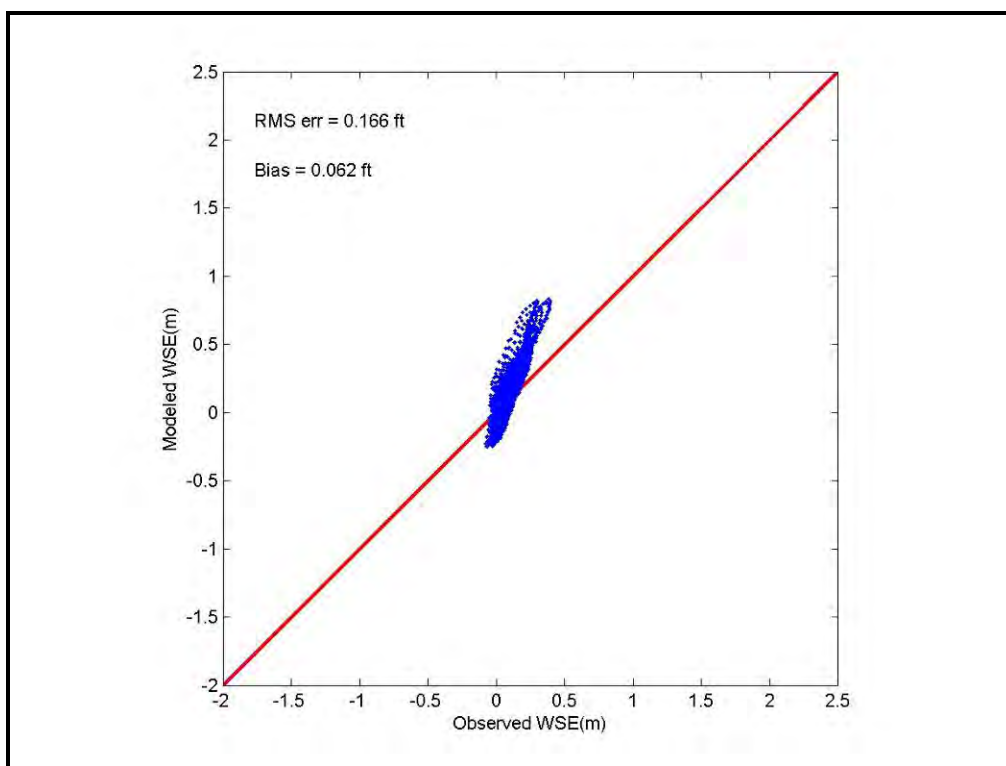


Figure 5-24. OC_down water surface elevation scatter plot comparing modeled and measured water surface elevations.

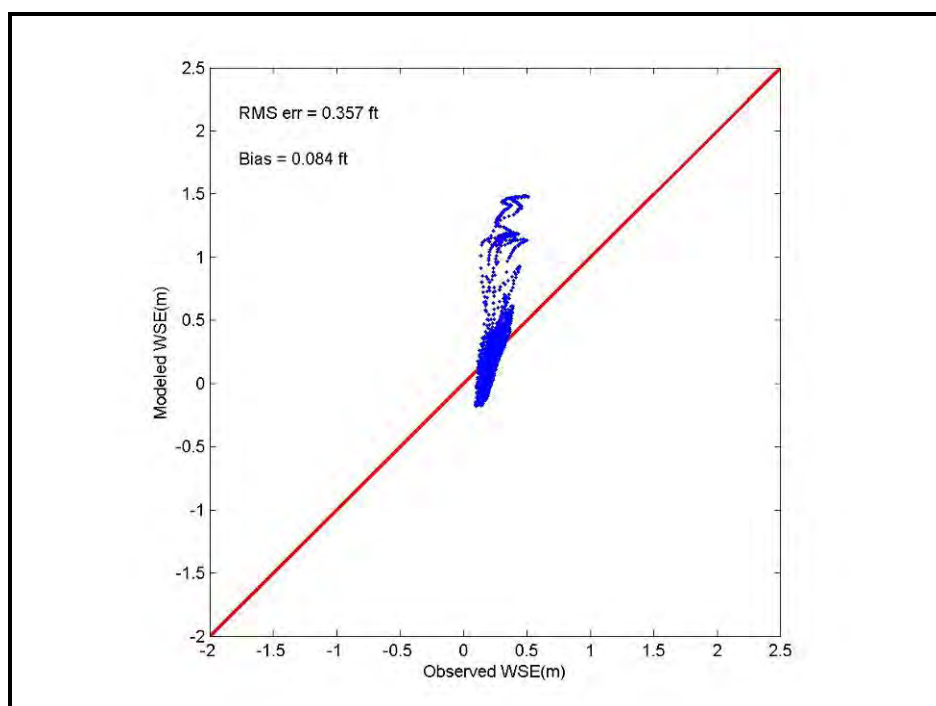


Figure 5-25. BB_up water surface elevation scatter plot comparing modeled and measured water surface elevations.

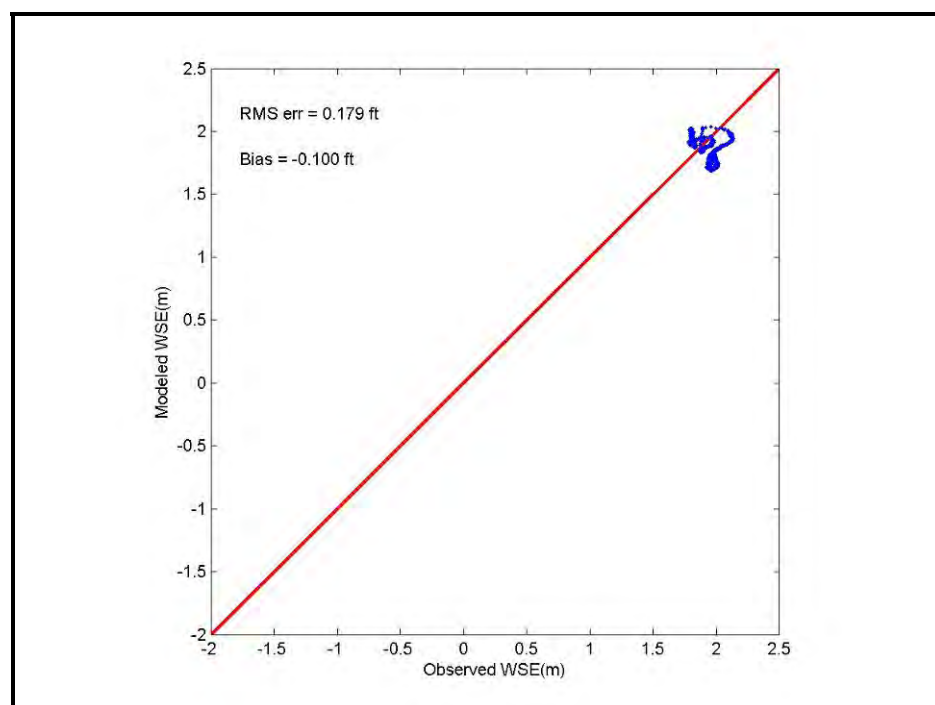


Figure 5-26. Rt6_down water surface elevation scatter plot comparing modeled and measured water surface elevations.

5.3.2 Tidal Constituent Calibration

To further verify the calibration of the Herring River model, model and data comparison are made for five dominant tidal constituents within the system. Table 5-7 presents the tidal constituent amplitude and relative phase for modeled and measured tides at the four locations that were primarily tidal. The constituent calibration resulted in satisfactory agreement between modeled and measured tides. Standard errors between the modeled and observed tides were less than 0.1 feet in all cases, and a majority of the time less than 0.03 feet. Tidal phase errors were also reasonable (less than 15-20 minutes in most cases) and were typically equal or less than the time step of the data observations (15 minutes). Ultimately simulating tidal components accurately is important for proper simulation of restoration alternatives since the restoration parameters are based on the tidal fluctuations and not episodic events (precipitation, wind, etc.).

Table 5-7. Tidal constituents for measure water surface elevation data and calibrated model output, with modeled error amplitudes and phases during the modeled calibration time period.

Measured Data										
Location	Constituent Amplitude (ft)					Phase (degrees)				
	M ₂	S ₂	N ₂	K ₁	O ₁	ΦM ₂	ΦS ₂	ΦN ₂	ΦK ₁	ΦO ₁
Dog Leg	0.96	0.12	0.16	0.08	0.09	39.60	88.78	36.00	198.44	173.16
HT_up	0.87	0.11	0.14	0.08	0.09	76.79	122.26	71.06	224.46	192.13
OC_down	0.09	0.02	0.02	0.02	0.03	162.23	181.96	137.20	296.19	248.38
BB_up	0.07	0.01	0.02	0.02	0.02	174.82	192.83	148.83	297.61	254.27
Modeled Calibration Data										
Location	Constituent Amplitude (ft)					Phase (degrees)				
	M ₂	S ₂	N ₂	K ₁	O ₁	ΦM ₂	ΦS ₂	ΦN ₂	ΦK ₁	ΦO ₁
Dog Leg	0.93	0.12	0.16	0.09	0.11	43.15	80.84	41.66	203.55	180.90
HT_up	0.89	0.11	0.15	0.09	0.11	46.43	85.00	45.65	207.98	184.38
OC_down	0.19	0.08	0.04	0.05	0.05	116.26	134.01	105.31	273.49	227.74
BB_up	0.17	0.02	0.04	0.05	0.05	129.10	123.62	113.88	285.33	222.07
Error										
Location	Constituent Amplitude (ft)					Phase Error (minutes)				
	M ₂	S ₂	N ₂	K ₁	O ₁	ΦM ₂	ΦS ₂	ΦN ₂	ΦK ₁	ΦO ₁
Dog Leg	0.02	0.00	0.01	-0.01	-0.01	-1.42	3.18	-2.26	-2.04	-3.10
HT_up	-0.02	-0.01	-0.01	-0.02	-0.02	12.14	14.90	10.16	6.59	3.10
OC_down	-0.10	-0.07	-0.02	-0.03	-0.03	18.39	19.18	12.76	9.08	8.26
BB_up	-0.10	-0.01	-0.02	-0.03	-0.03	18.29	27.68	13.98	4.91	12.88

5.3.3 Salinity Calibration

Salt penetration in the Herring River in its current restricted state is not normally observed above High Toss Road. However, at the Dogleg gauge location, salt concentration is observed to significantly vary during a tidal cycle. Salinity data was collected at the dogleg location for the calibration time period by the NPS. Figure 5-27 and Figure 5-28 show the time series comparison (between modeled and measured data) and scatter plot for salinity at the Dogleg

location, respectively. Because the Herring River model is a two-dimensional depth integrated model, the salinity computed by the model represents depth averaged salinity. Based on limited vertical observation data Spaulding and Grilli (2001) used depth-averaged salinity values to calibrate their hydrodynamic model. Although this data were not available for review, Spaulding and Grilli (2001) indicate that the water column was vertically well-mixed during their observation time period.

Lateral and longitudinal dispersion is computed within the numerical scheme of EFDC based on the physical processes that are simulated in the estuary system (as determined by the hydrodynamic model). The coefficients for dispersion are based on the work of Smagorinsky (1963). This eliminates the need for selection of dispersion coefficients for model simulation and more accurately represents dispersion since it is based solely on the physical processes that are being represented in the model.

The model slightly under-predicts the time-averaged salinity over the duration of the calibration period as indicated by the bias of -0.85 ppt. The RMSE of indicates that for discrete measurements, on average, the model computed salinity at the dogleg location is within 3.3 ppt of observed salinity. Overall the model is well calibrated for salinity with a relative error of 11% which is well below the EPA recommended value. Although relatively small, the 11% error is likely related to potential depth variations that may exist in the system during the calibration time frame. For example, the salinity observations were collected at a single depth level in the water column, while the model assumes a depth-averaged result. Although limited historical data (Spaulding and Grilli, 2001) seem to indicate the Herring River system is fairly well mixed, this may not have been the case during the calibration time period. Due to the flexible nature of the model selected, the model can be easily extended to 3-D if additional data reveals seasonal or temporary stratification or if increased resolution and processes handling is desired.

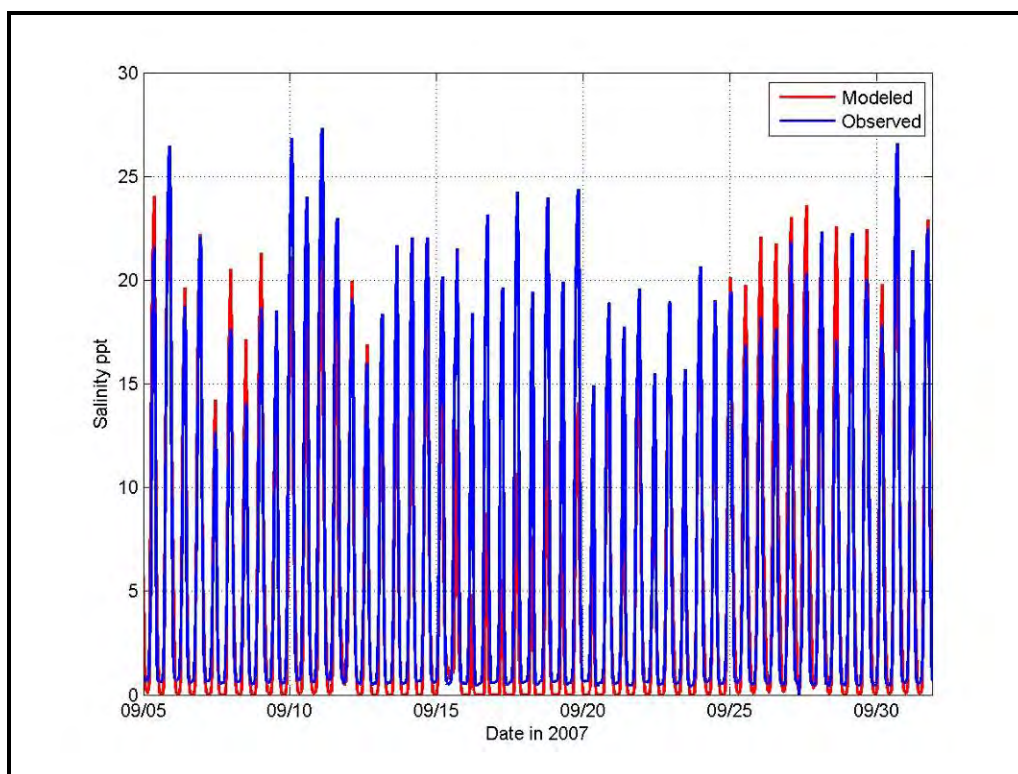


Figure 5-27. Dogleg salinity time series comparison between modeled (red) and measured (blue) salinity data.

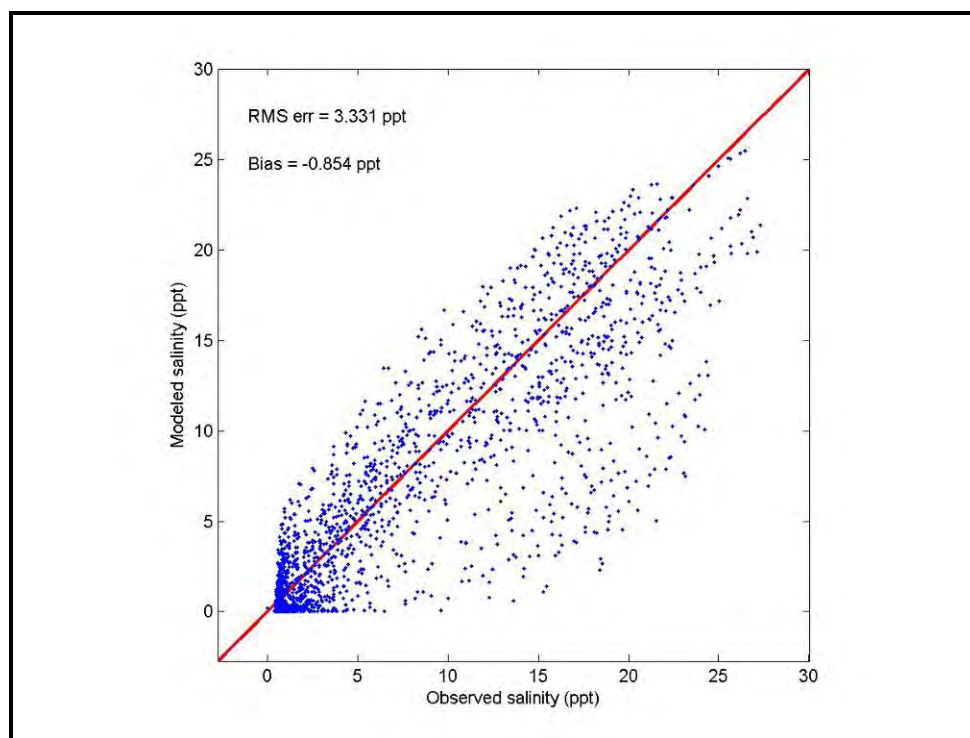


Figure 5-28. Dogleg salinity scatter plot comparing modeled and measured salinities.

5.4 MODEL VALIDATION

Prior to using a hydrodynamic model as a predictive tool it is common practice to validate a calibrated model to confirm the model's applicability to a reasonable range of conditions. Validation involves applying the calibrated model to set of observed data that are independent from the calibration data set without changing the model configuration or parameterization. Error statistics for model validation should meet the same guidelines as those applied to model calibration.

Observations of salinity and water levels collected for a previous modeling study (Spaulding and Grilli, 2001) shown in Figure 5-29 were used to validate the Herring River model. These data were collected at two locations, "Riverside" and "Harborside", shown in Figure 5-10. Data collected at the Harborside location were used to provide boundary conditions for the validation simulations. The impact of shoals and mudflats in "The Gut" is apparent when comparing the water surface elevation data collected at Old saw (Figure 5-11) and the water surface elevation time series at the Harborside location (Figure 5-29). The shape of the tidal observations, particularly at low tide, shows the effect on water levels as the lower portion of the tidal signal is limited by the Gut bathymetry. Therefore, it would be inappropriate to apply the Harborside boundary condition at the same open boundary used for model calibration, and for the validation simulation the grid was modified slightly to move the open boundary closer to the Harborside location. Beyond this necessary modification the model configuration and parameterization remains unchanged for everything upstream of the Chequessett Neck Road dike.

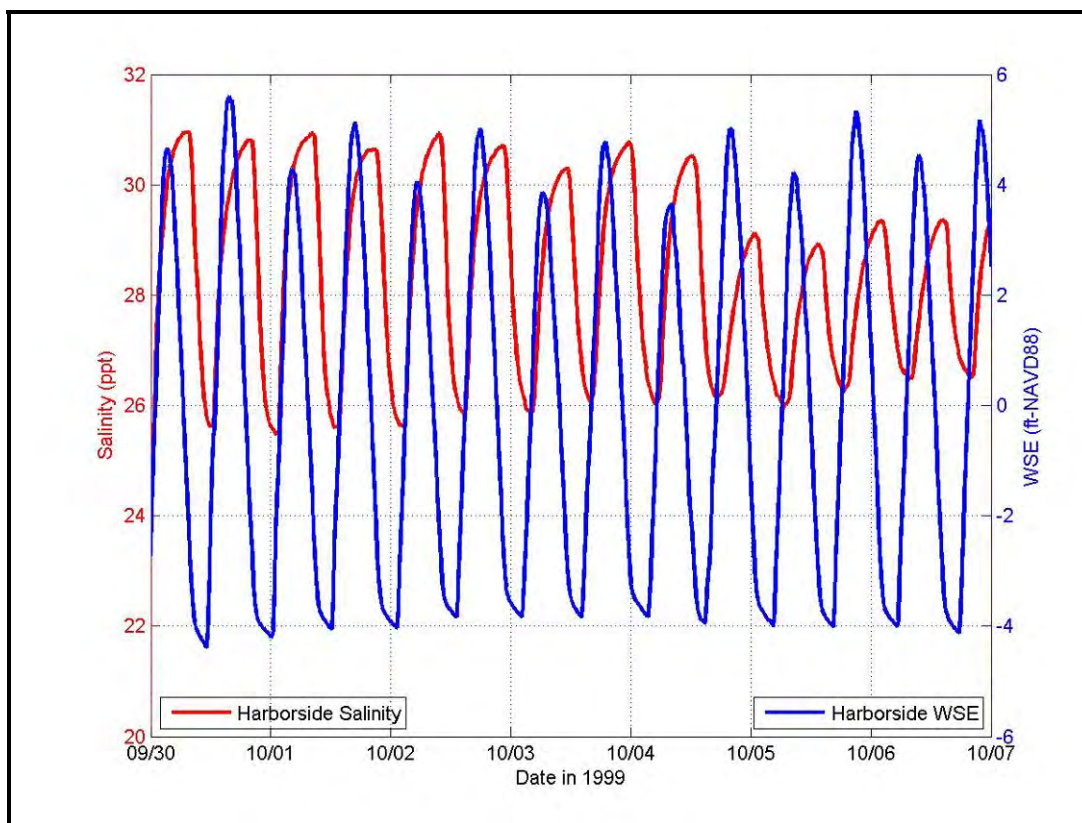


Figure 5-29. Water surface elevation and salinity boundary conditions used for model validation.

5.4.1 Water Surface Elevation Validation

Results for water surface elevation at the Riverside location are presented in Figure 5-30 and Figure 5-31. The agreement is reasonable with a slight (0.068 ft) under-prediction of the average water surface elevation and a RMSE of 0.136 ft. The relative error is 1.7% which is well below the recommend value.

5.4.2 Salinity Validation

Results for salinity at the Riverside location for the validation simulation are presented in Figure 5-32 and Figure 5-33. The results are again reasonable, with a model over prediction of the average salinity at the Riverside location by about 3.6 ppt; even though the modeled salinity peaks are lower than the observed salinity peaks. Overall, the relative error of 20 % is within the acceptable limits, and the peaks and troughs of the salinity values are reasonable captured. The increased error, when compared to the calibration case is likely due to (1) the slight offset in phasing between the modeled and measured salinities, (2) the potential inconsistencies when comparing depth averaged model results to observations collected at a single depth level, and (3) the potential limitation associated with assuming a well mixed estuary at this location due to limited data. Due to the flexible nature of the model selected, the model can be easily extended to 3-D if additional data reveals seasonal or temporary stratification or if increased resolution and processes handling is desired.

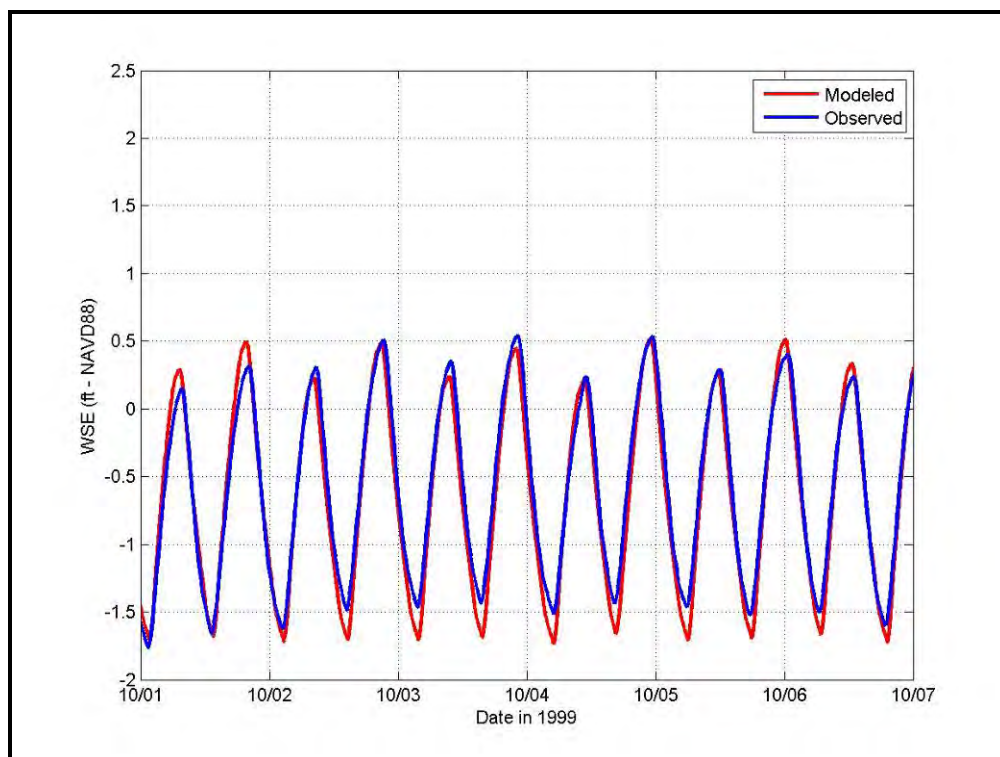


Figure 5-30. Riverside water surface elevation comparison for modeled (red) and measured (blue) time series.

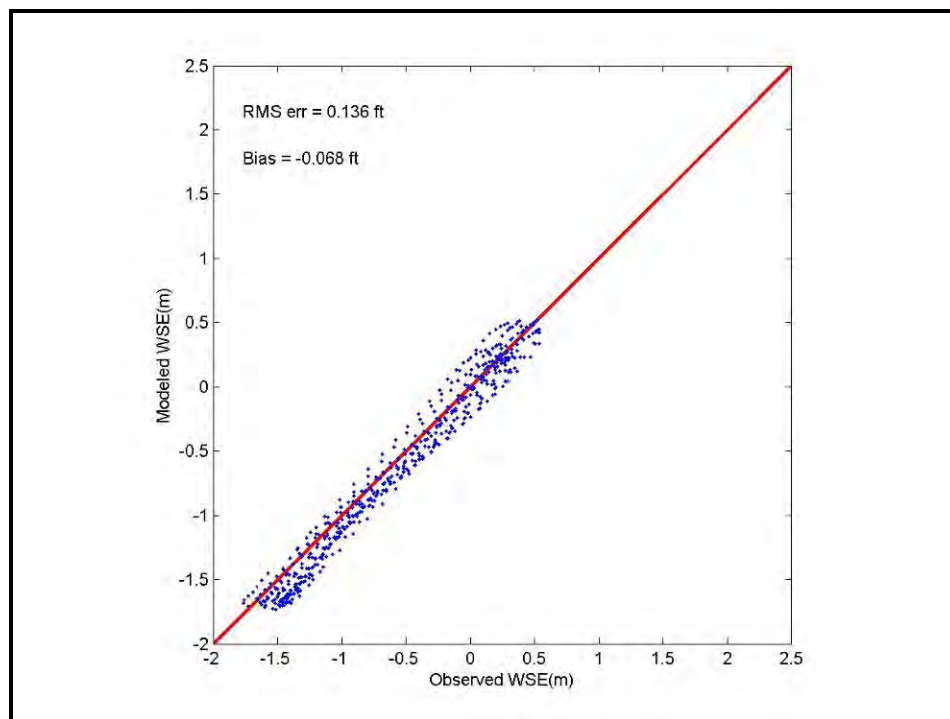


Figure 5-31. Riverside water surface elevation scatter plot comparing modeled and measured water surface elevations.

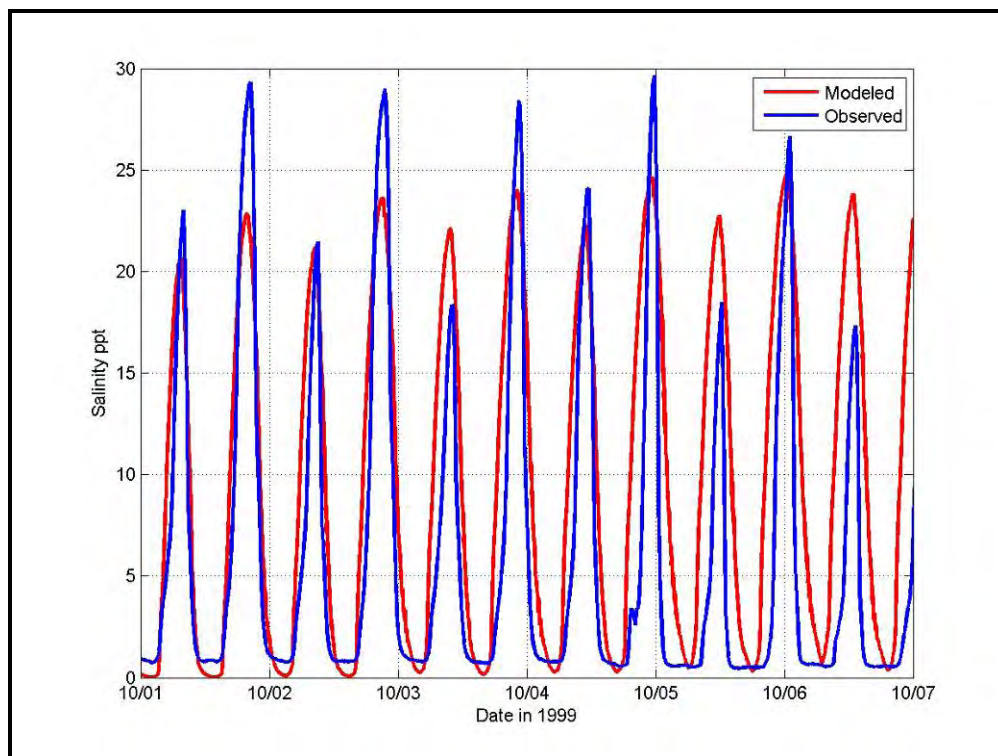


Figure 5-32. Riverside salinity time series comparison between modeled (red) and measured (blue) salinity data.

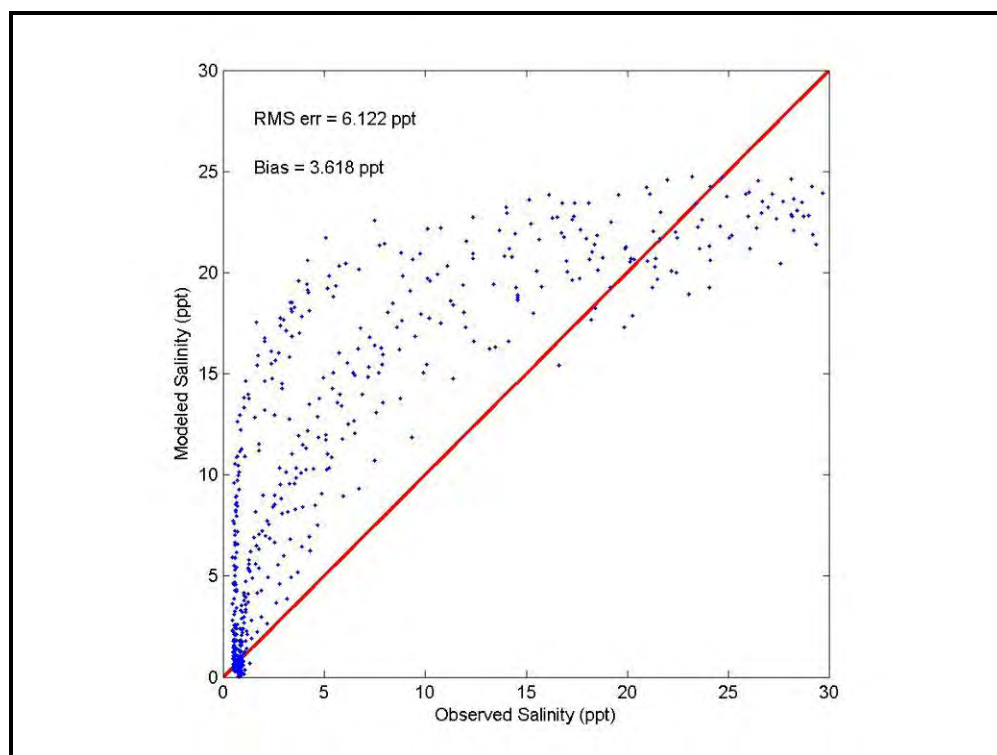


Figure 5-33. Riverside salinity scatter plot comparing modeled and measured salinities.

5.5 EXISTING CONDITIONS

The calibrated and validated model was further applied to simulate a number of hypothetical situations to aid in understanding the behavior of the Herring River estuary in its current restricted state. In addition to providing better understanding of the current system, these simulations will also be useful by providing a baseline for comparison to alternative simulations. For example, the impact of opening the Chequessett Neck Road dike on the potential storm surge signal throughout the estuary system can be evaluated. Storm events and forecasted sea level rise (SLR) scenarios are simulated, and the results presented in this section. Boundary conditions for these simulations were created by modifying the water levels observed at the Old Saw boundary conditions station to represent storm surge and/or long-term sea level rise increases. The baseline results used for comparison to the tidal flood and sea level rise simulations were taken from a simulations of normal tidal conditions which are identical to the calibration case without the inclusion of atmospheric forcing (wind, rainfall, etc.).

5.5.1 Tidal Flood Events

Tidal flood elevations (1-year and 100-year return periods) acquired from The New England Tidal Flood Profiles published by the U.S. Army Corp of Engineers (1988) were used to create a representative water surface elevation boundary condition for simulation of the tidal flood events. To create the boundary conditions, the water surface elevation data used for model calibration was modified by adding the surge (a sinusoidal peak with a 24 hour period) to the spring high tide that occurred on September 28th 2007. This produces three high tides that are higher than the normal conditions and represent the tidal flood conditions that might occur with

the passing of slow moving Nor'easter, somewhat like the conditions that occurred during the Northeast Blizzard of February 1978. The 1 year return period storm peaks at 6.9 feet NAVD88 and is just slightly higher than the spring high tide observed at noon on September 28th 2007. The 100-year return period storm peaks at 9.3 feet NAVD88 and is slightly less than the maximum high water level of 9.7 ft observed during the 1978 blizzard (Roman, 1987), indicating that the blizzard of 1978 was larger than a 100-year event for this location. The water surface elevation boundary conditions used for these simulations are shown in Figure 5-34.

A summary of the peak water surface elevations at the gauge locations shown in Figure 5-10 is provided in 5-8 and time series plots showing the water surface elevation at the various locations are given in Figure 5-35 thru Figure 5-41. After day 2.5 in the simulation, the storm has passed, and the time series for the return period storm (tidal flood) conditions matches the normal conditions (or baseline). The 1-year flood case is not clearly distinguishable from normal conditions. In fact, at the locations further up in the estuary (the Pole Dike gauges and the Rt6_up gauge) the tidal attenuation caused by the system produces results that are similar to spring tide conditions. The Rt6_up location is not affected by the storm surge for any of the flood scenarios since the current restrictions throughout the system render this location completely non-tidal. The return-period tidal flood simulations demonstrate the effectiveness of the Chequessett Neck Dike in reducing storm surge. For example, during the 100-year flood event, the greatest increase in peak elevation over the base line is 0.7 ft at the Dogleg location. This increase is only 27% of the increase that occurs due the return period storm at the boundary in Wellfleet Harbor. In other words, these results show a 63% reduction in storm surge height between Wellfleet Harbor and High Toss Road.

Table 5-8. Peak water surface elevation for return period tidal flood simulations (ft-NAVD88).

Location	Normal Conditions	1-Year Flood	100-Year Flood
Dogleg	0.75	0.80	1.44
HT_up	0.72	0.77	1.37
PD_down	0.79	0.79	0.95
PD_up	0.79	0.79	0.95
OC_down	0.65	0.71	1.33
BB_up	0.71	0.73	1.1
Rt6_down	NA	NA	NA

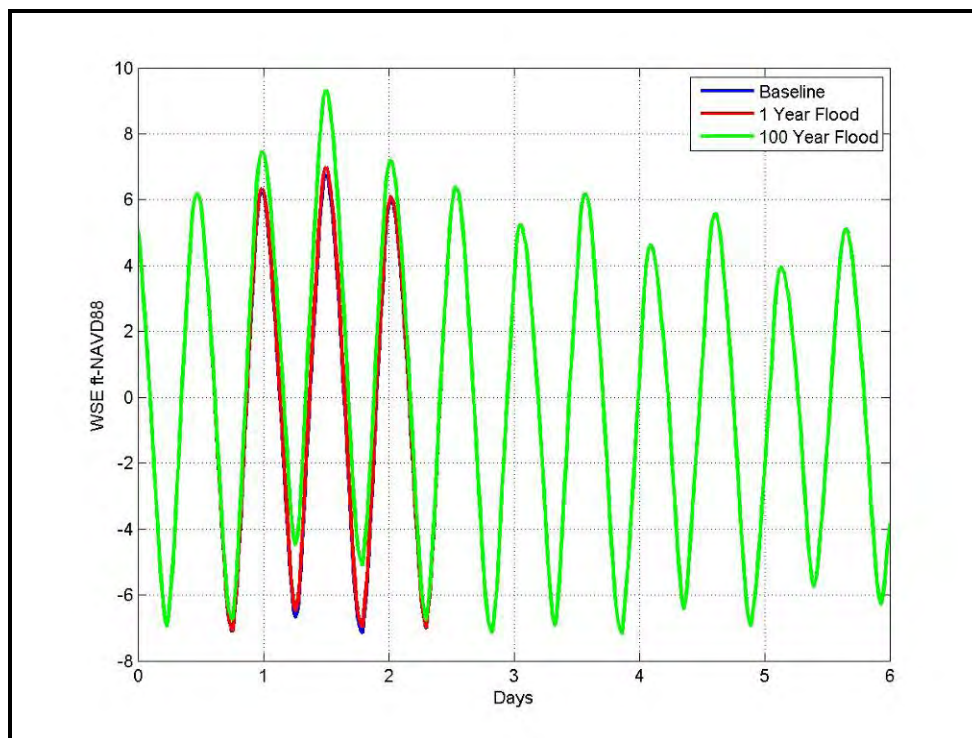


Figure 5-34. Open boundary water surface elevation boundary condition for return period storm simulations.

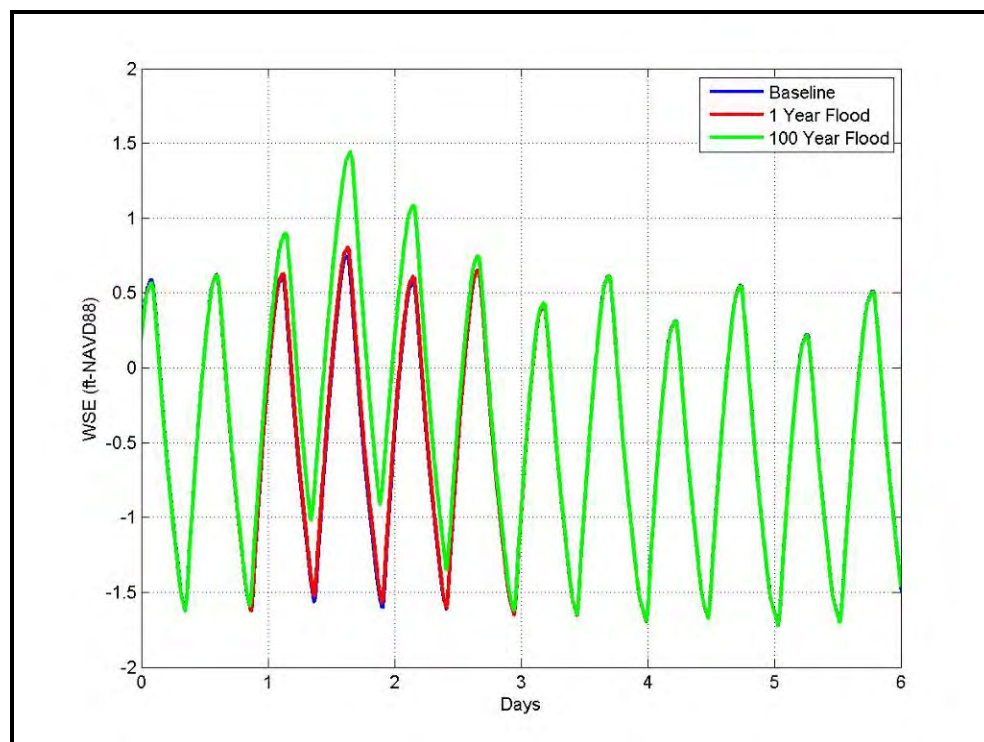


Figure 5-35. Tidal flood water surface elevation time series at Dogleg.

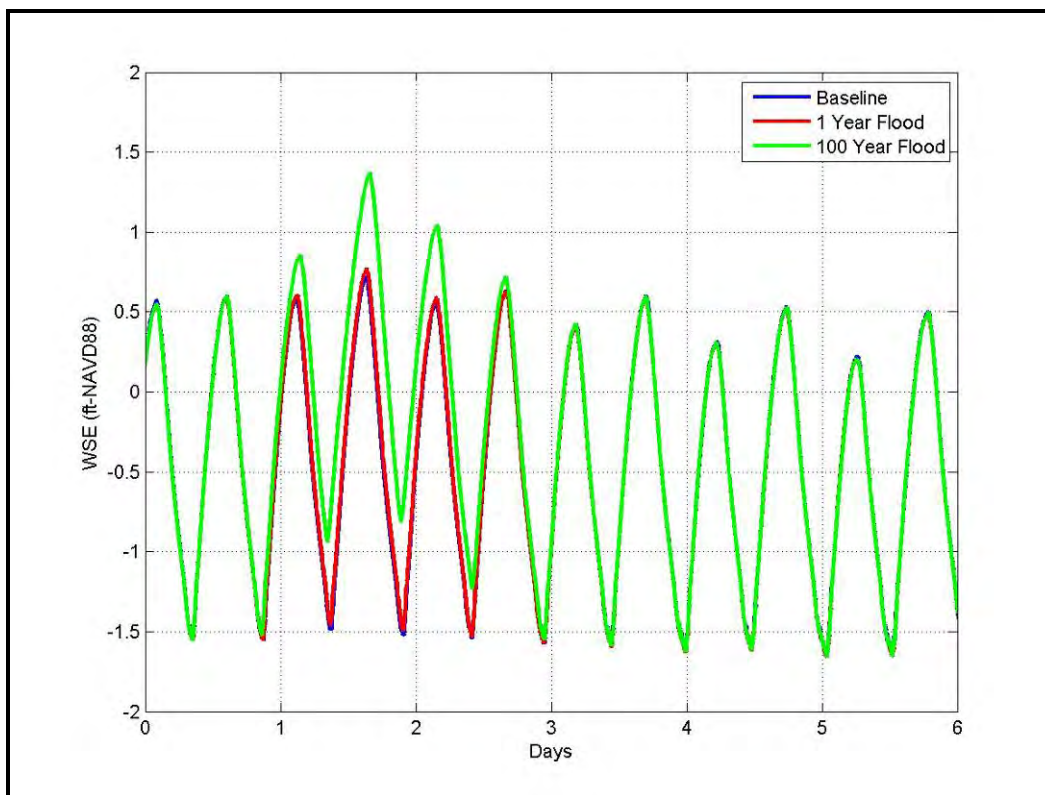


Figure 5-36. Tidal flood water surface elevation time series at HT_up.

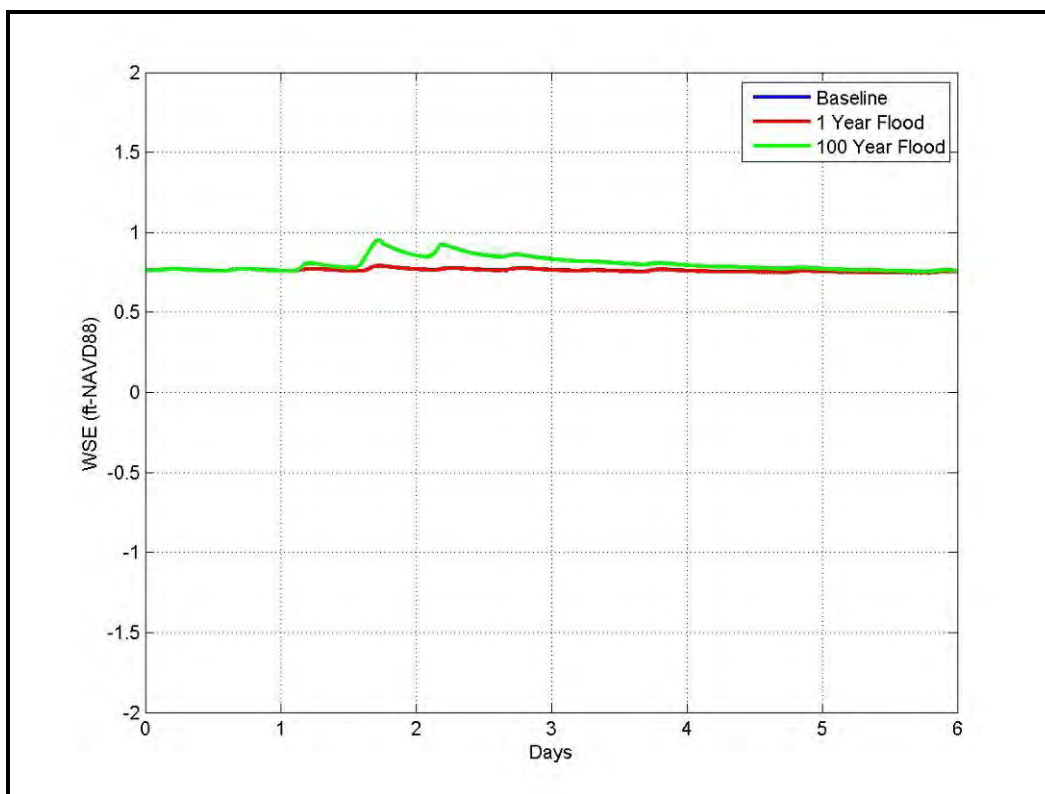


Figure 5-37. Tidal flood water surface elevation time series at PD_down.

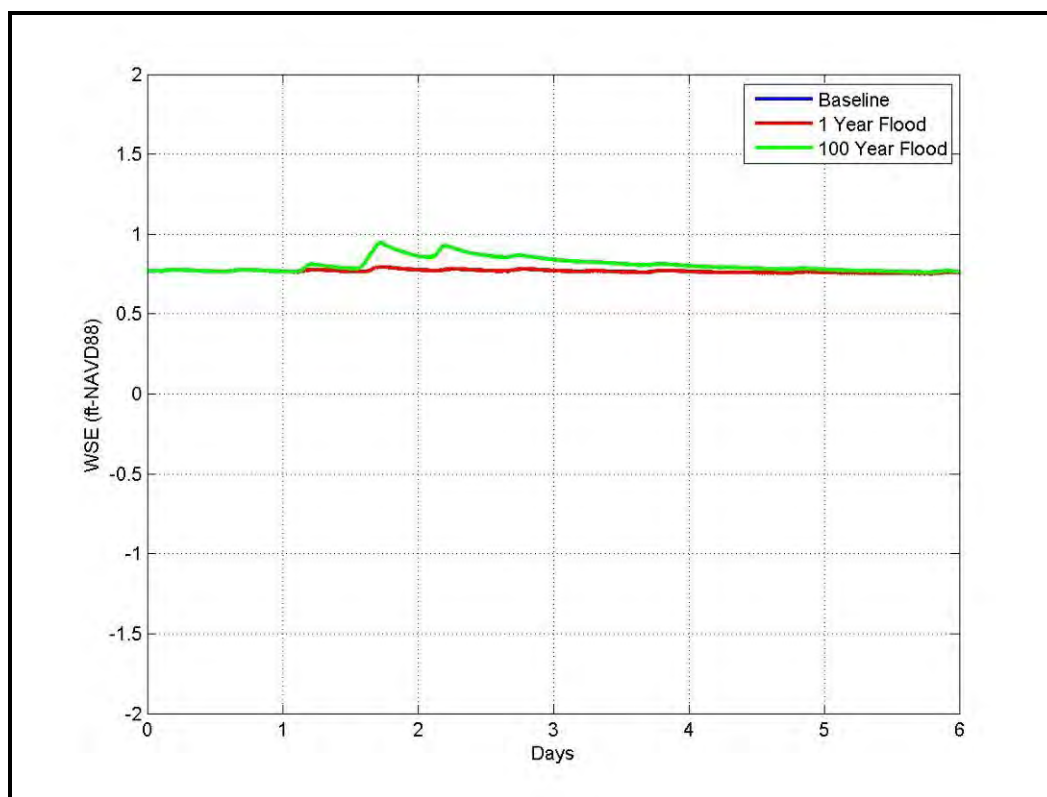


Figure 5-38. Tidal flood water surface elevation time series at PD_up.

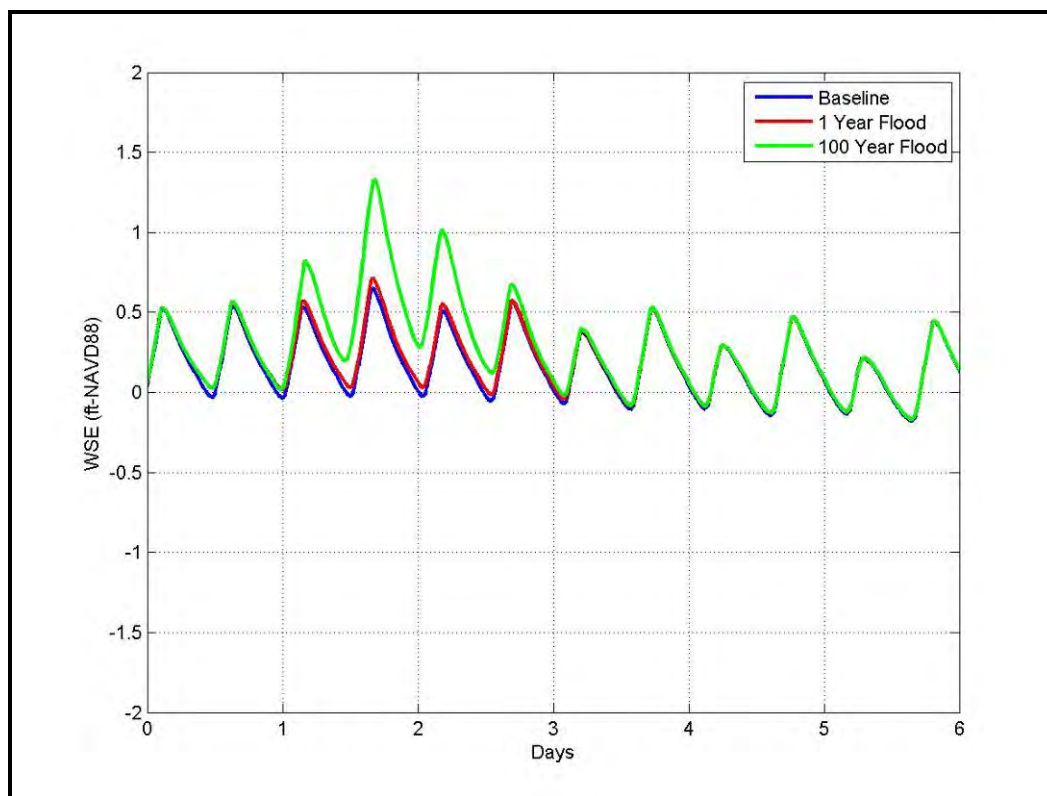


Figure 5-39. Tidal flood water surface elevation time series at OC_down.

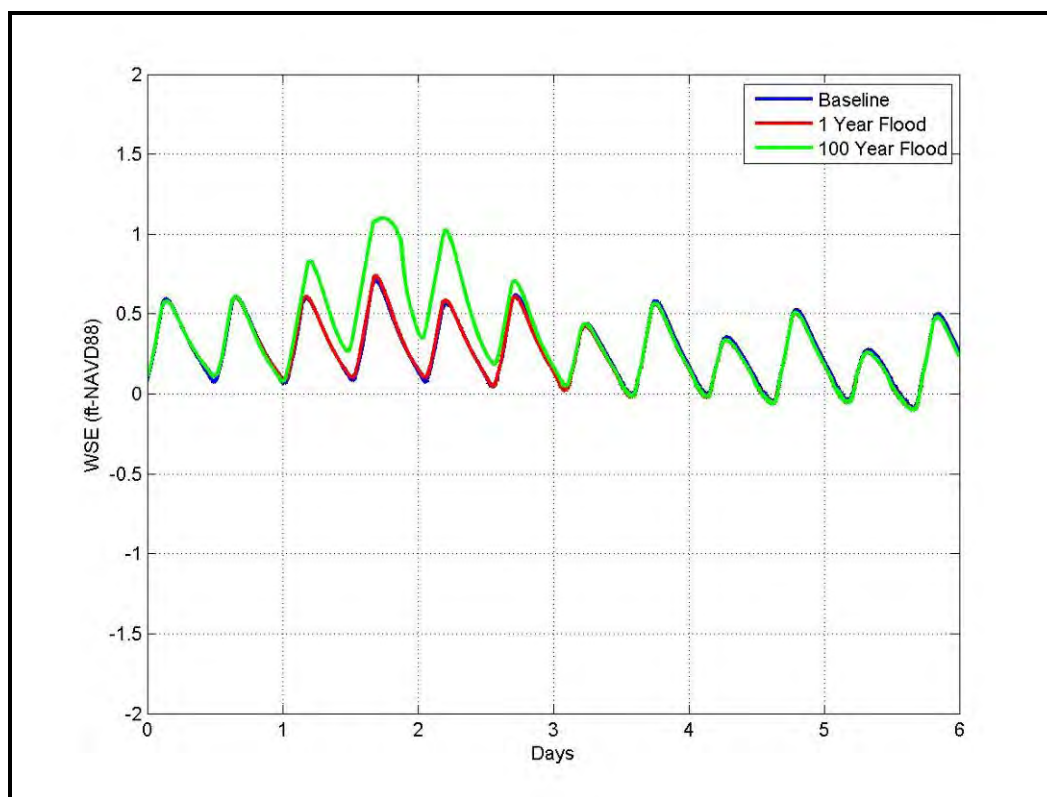


Figure 5-40. Tidal flood water surface elevation time series at BB_up.

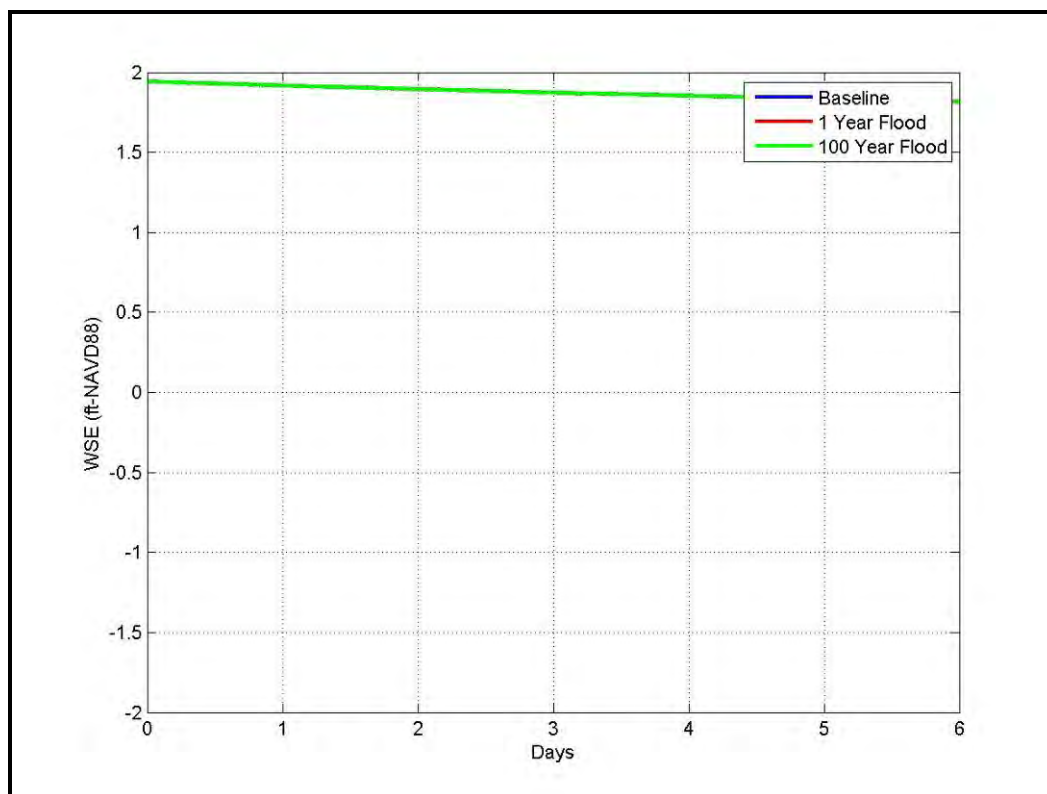


Figure 5-41. Tidal flood water surface elevation time series at Rt6_down.

Salinity results for the tidal flood simulations are summarized in 5-9 and time series plots for salinity at Dogleg and HT_up are shown in Figure 5-42 and Figure 5-43, respectively. The results for the 1-year return period flood show very little difference from normal conditions, while the 100-year return period flood indicates an increase at both locations. As for the water surface elevation, the dike significantly reduces salt water intrusion even during significant storm events. Results from the 100-year return period storm simulations shows minimal salt penetration north of High Toss Road.

Table 5-9. Peak salinity for tidal flood simulations (in ppt).

Location	Normal Conditions	1 Year Flood	100 Year Flood
Dogleg	24.0	24.4	27.35
HT_up	5.1	6.6	16.4

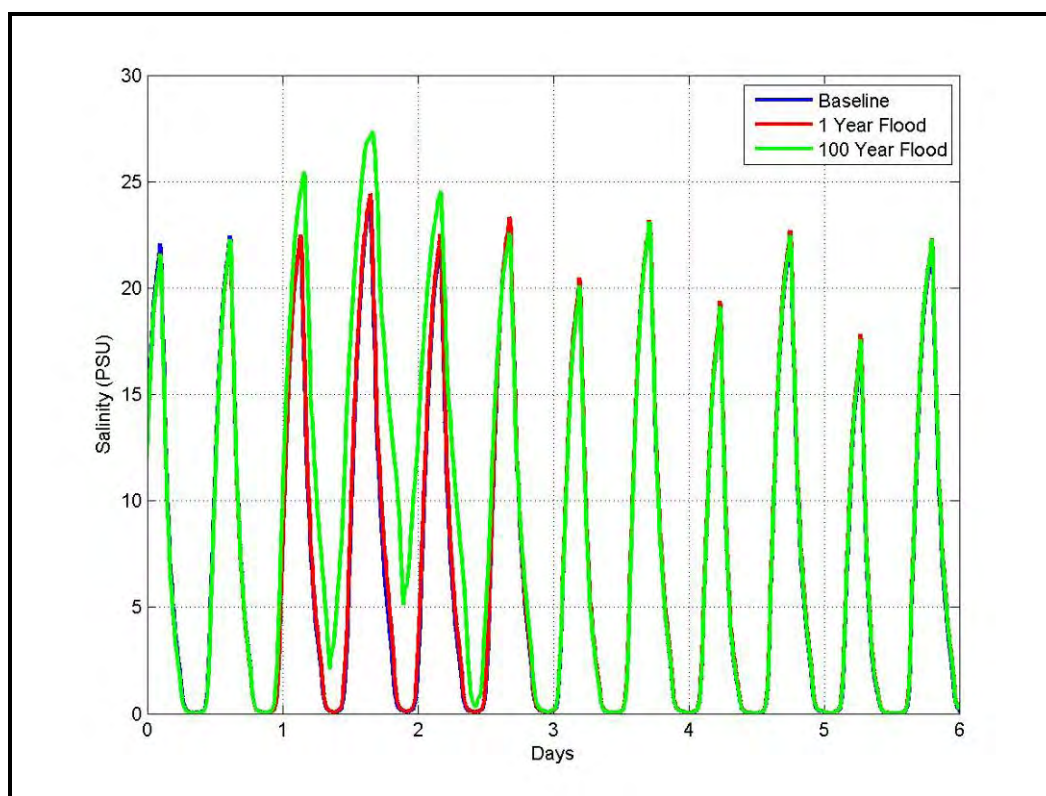


Figure 5-42. Tidal flood salinity time series at Dogleg.

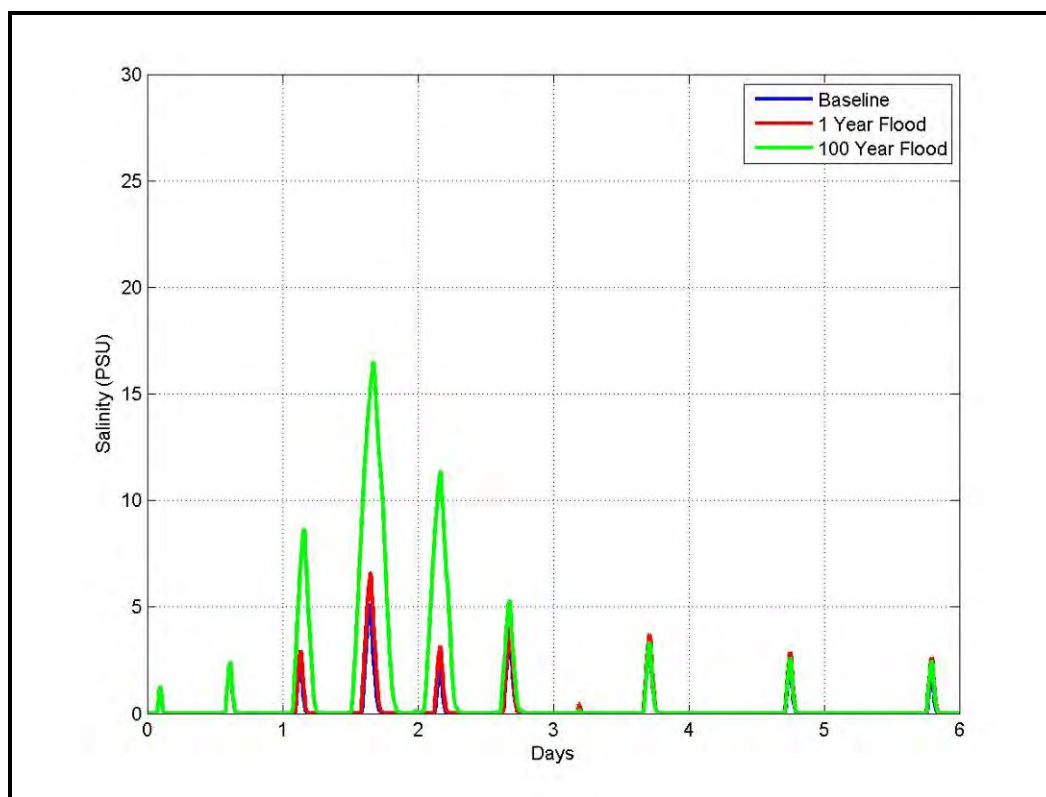


Figure 5-43. Tidal flood salinity time series at HT_up.

5.5.2 Sea Level Rise

Further simulations were conducted to provide an estimate of future predicted water levels and salinity in the Herring River over the next century as relative sea level increases. Specifically, simulations for 50 and 100 years of sea level rise were conducted assuming a constant, linear sea level rise rate of 2.63 mm/year over the next century. The rate of sea level rise is highly debated and varies significantly depending on the source. For these initial sea level rise simulations at Herring River, project rates are based on actual sea level rise observations to provide a reasonable first-order estimate of sea level rise. This rate is based on observations of monthly mean tide levels in Boston and is provided to the public by the National Oceanic and Atmospheric Administration's Center for Operational Oceanographic Products (<http://tidesandcurrents.noaa.gov/sltrends/sltrends.shtml>). The sea level rise simulations were created by simply increasing the water level boundary condition at the open boundary by the appropriate amount (i.e., 0.44 ft in 50 years and 0.88 ft in 100 years).

Since there is such a wide range of predicted sea level rise rates, as the model continues to be developed, additional sea-level rise scenarios will be conducted in order to simulate a more diverse range of projected rates of sea level rise. For example, increased rates of sea level rise will provide an upper bound of expected influence of future projections. Specifically, additional sea level rise scenarios will be developed and simulated once the alternative analysis narrows down the alternatives to a short list of preferred alternatives. In this manner, significant model simulation time will be conserved by running sea level rise simulations for only the alternatives that pass the normal tidal conditions screening for an effective restoration.

To quantify the effect of the sea level rise on water levels in the estuary, Mean High Water (MHW) and Mean Low Water (MLW) were computed over the six days of each of the three simulations (normal conditions or baseline, 50 years of sea level rise and 100 years of sea level rise) at each of the gauge locations shown in Figure 5-10. These values should not be considered representative of actual mean tidal data because of the short time period over which they were calculated; however they are useful for a relative comparison of water levels for these simulations. The sea level rise increases in MHW and MLW and are presented in Table 5-10 and Table 5-11, respectively.

Table 5-10. Increase in mean high water above normal conditions due to sea level rise (ft).

Location	50 year sea level rise	100 year sea level rise
Dogleg	0.05	0.35
HT_up	0.05	0.34
PD_down	0.04	0.09
PD_up	0.01	0.05
OC_down	0.06	0.34
BB_up	0.12	0.39
Rt6_down	NA	NA

Table 5-11. Increase in mean low water above normal conditions due to sea level rise (ft).

Location	50 year sea level rise	100 year sea level rise
Dogleg	0.04	0.35
HT_up	0.04	0.37
PD_down	0.06	0.07
PD_up	0.00	0.03
OC_down	0.07	0.15
BB_up	0.15	0.22
Rt6_down	NA	NA

This approach to predicting the effect of sea level rise is simplistic and does not consider a number of other factors that may also cause significant changes to tidal flows in the Herring River over the next century. For example, sea level rise may not cause a uniform increase in water levels over the entire tidal range, (i.e. the tidal range might change as the mean level increases). The sea level rise considered is only projects absolute (eustatic) sea level rise and does not include land/marsh based elevation changes (e.g., subsidence). For example, as water levels increase, vegetation will likely grow and cause marsh surfaces to accrete. It is also impossible to predict future anthropogenic activities or adaptive management activities. However, these sea level rise simulations are useful in increasing our understanding of the behavior of the Herring River Estuary system in its current restricted state, albeit not necessarily a perfect prediction for the future.

In general, the results are similar to what is seen in the tidal flood scenarios with the mean water levels analogous to the peak flood levels. The increases in water levels induced by the increase at the boundary are uniform throughout the time series, shown in Figure 5-44 through Figure 5-49. Since the change in water surface elevations was minimal for the 50 year sea level rise simulation, the time series for these results is not presented in the figures. After 50 years of sea level rise, with an increase of 0.44 ft in mean water levels in Wellfleet Harbor, there is hardly any change to water levels upstream of the dike. The greatest effect is observed in Bound Brook, where the increase in MHW is still only 30% of the sea level rise at the boundary. With 100 years of sea level rise, the change in sea levels upstream of the dike become a more apparent, but the maximum increase is still less than half the sea level rise in Wellfleet Harbor. Thus the dike, in its current state, provides significant attenuation of sea level rise, as it does for storm surge. For all cases, at the locations that currently do not experience significant tidal fluctuations under normal conditions (e.g., Pole Dike Creek and Route 6), the effect of rising sea level is insignificant. At the Rt6_down location there is no discernable difference in water level even after 100 years of sea level rise.

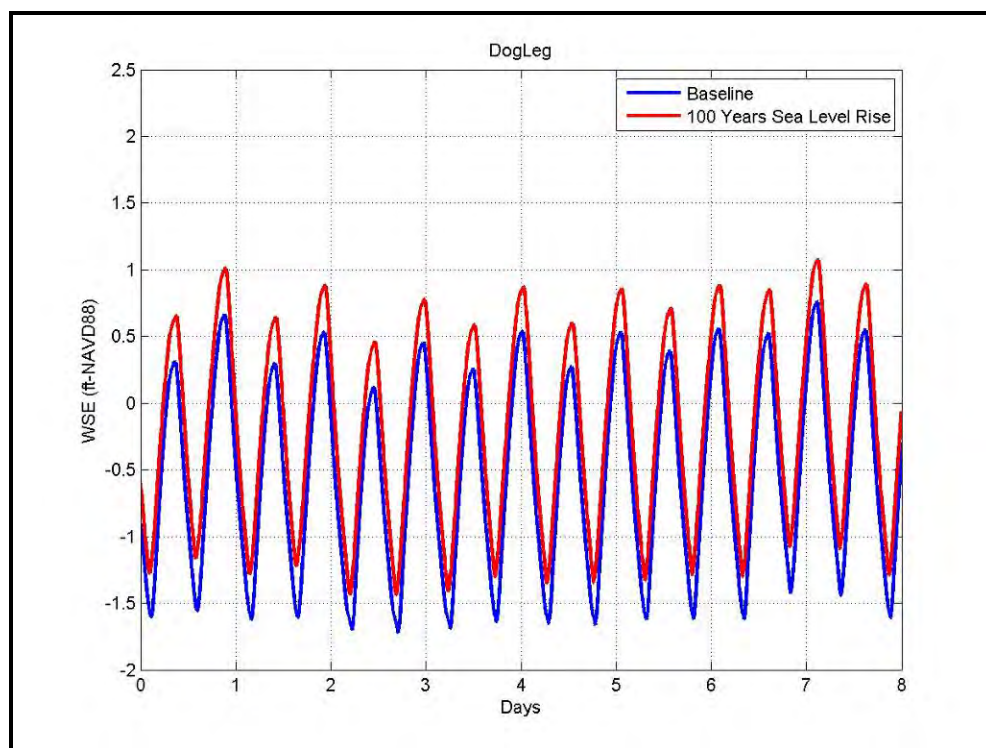


Figure 5-44. Sea level rise scenario for water surface elevation at Dogleg station. Normal conditions (baseline) compared to 100 years of projected sea level rise.

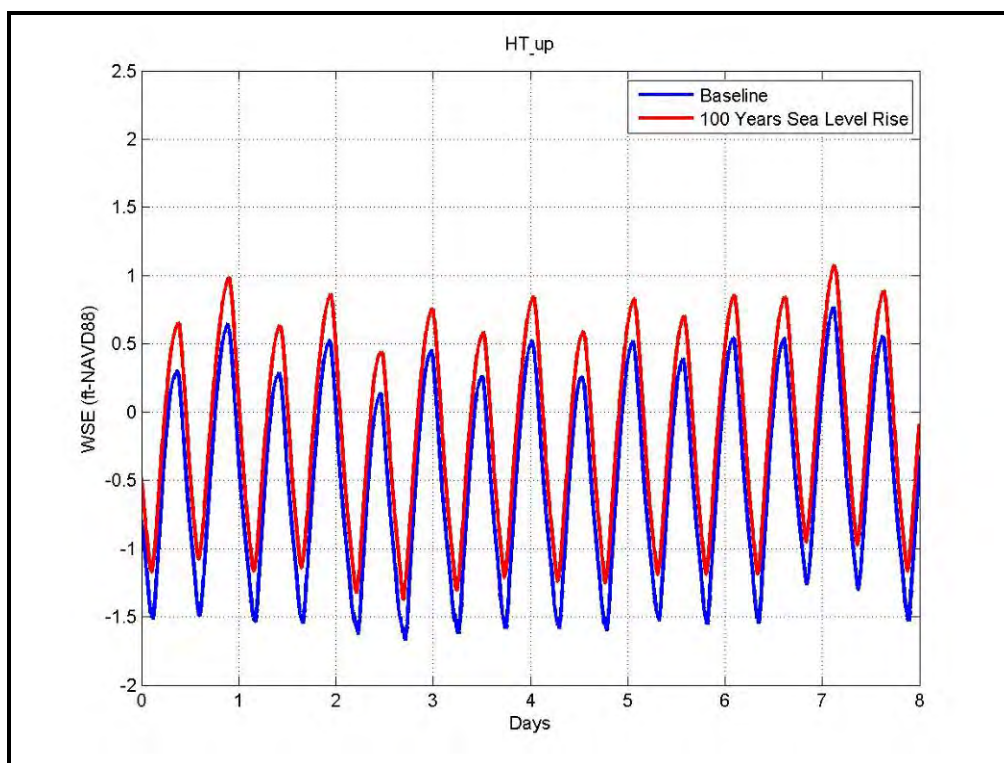


Figure 5-45. Sea level rise scenario for water surface elevation at HT_up station. Normal conditions (baseline) compared to 100 years of projected sea level rise.

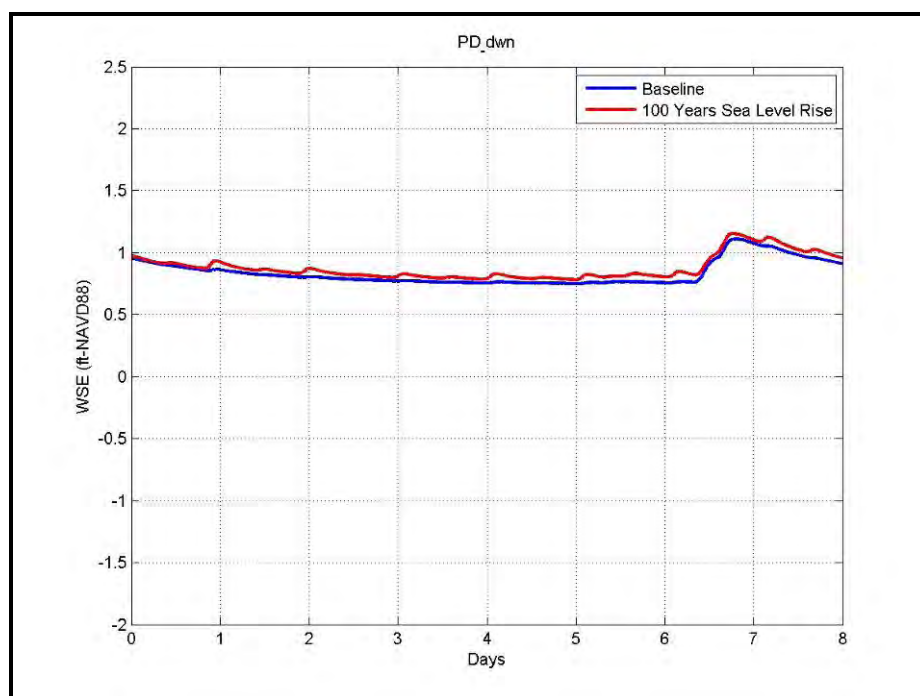


Figure 5-46. Sea level rise scenario for water surface elevation at PD_down station. Normal conditions (baseline) compared to 100 years of projected sea level rise.

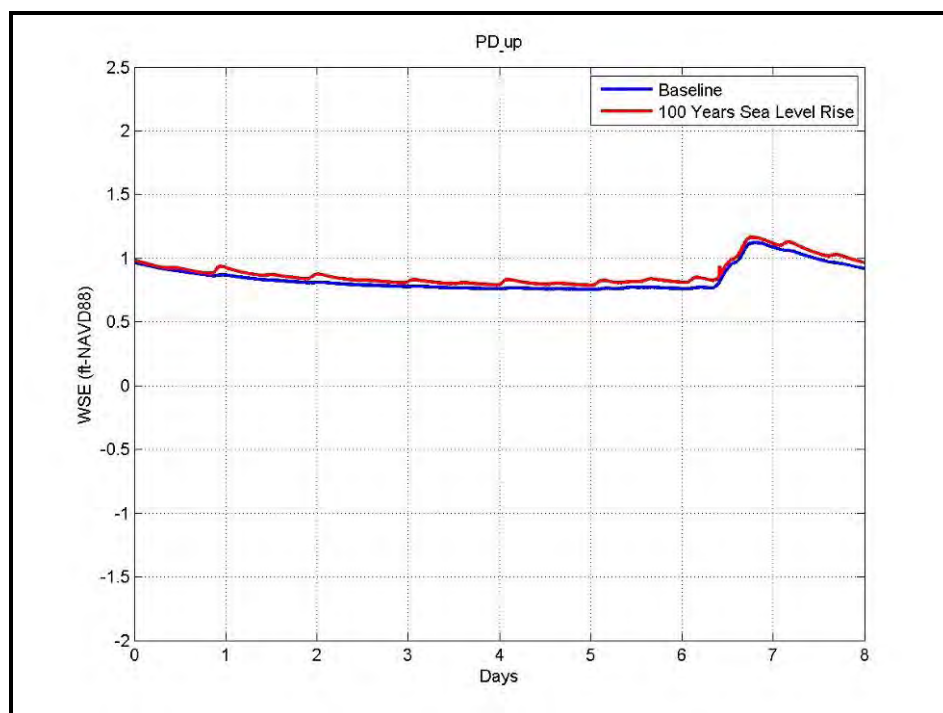


Figure 5-47. Sea level rise scenario for water surface elevation at PD_up station. Normal conditions (baseline) compared to 100 years of projected sea level rise.

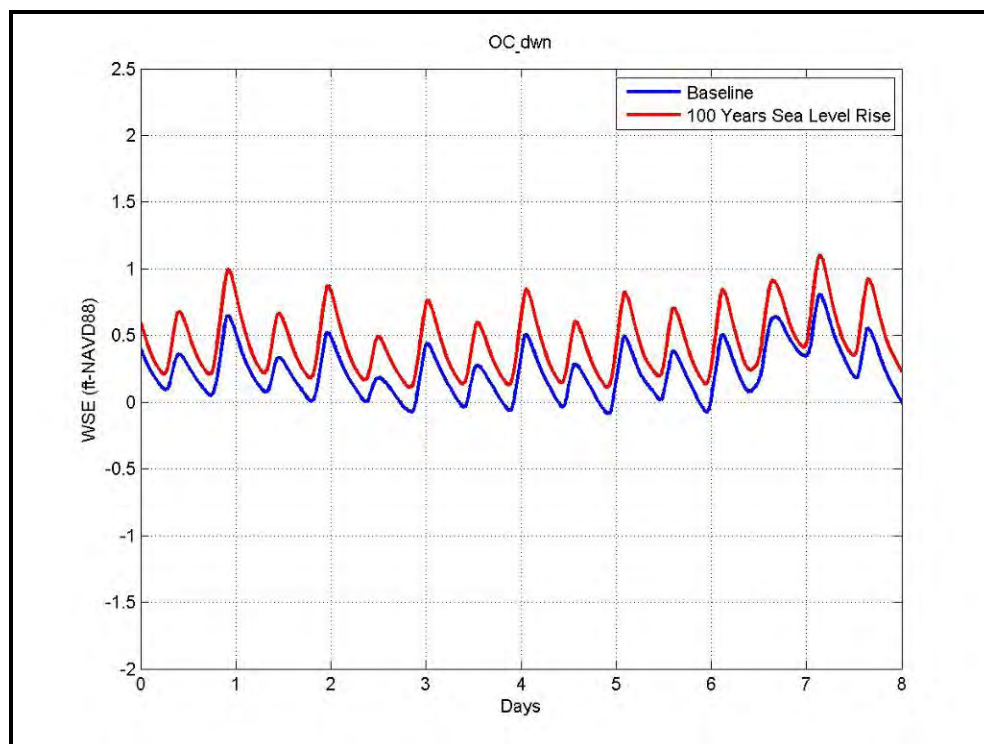


Figure 5-48. Sea level rise scenario for water surface elevation at OC_down station. Normal conditions (baseline) compared to 100 years of projected sea level rise.

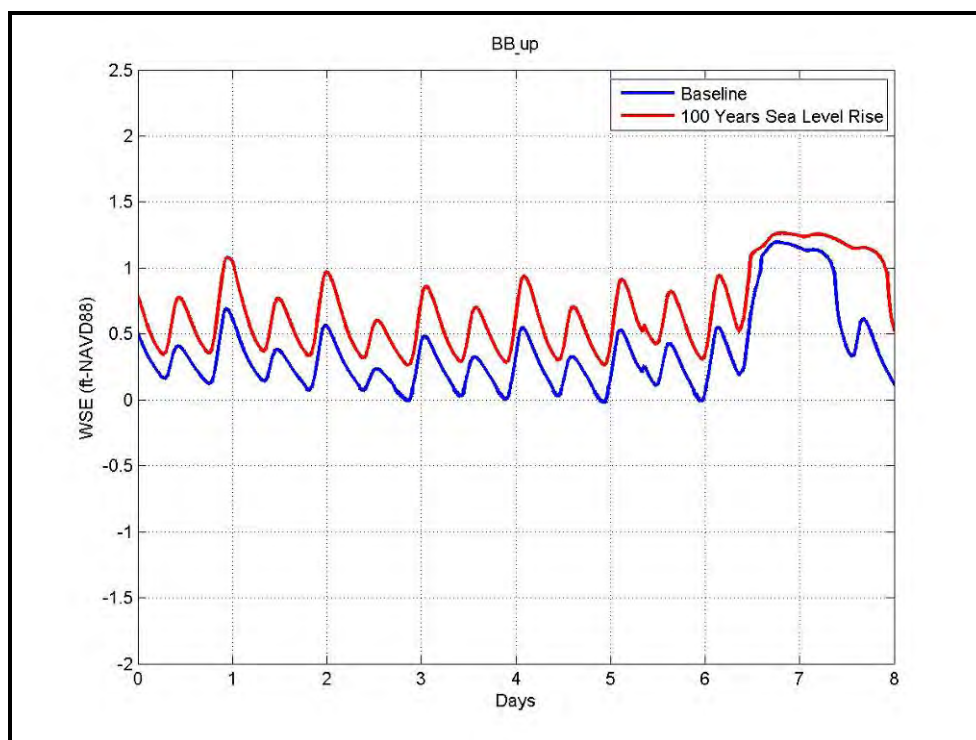


Figure 5-49. Sea level rise scenario for water surface elevation at BB_up station. Normal conditions (baseline) compared to 100 years of projected sea level rise.

In order to quantify the effect of sea level rise on salinity, an average over all the tidal cycles of the peak salinity was computed. These mean peak salinity values for each scenario at the Dogleg and HT_up gauges are presented in Table 5-12.

Table 5-12. Mean peak salinity values for sea level rise scenarios (ppt).

Location	Normal Conditions	50 Years SLR	100 Years SLR
Dogleg	22.52	22.62	25.09
HT_up	0.80	1.00	7.03

The results for salinity are similar to those for water levels. Fifty years of sea level rise does not significantly impact the salinity upstream if the dike, while doubling that increase does lead to a small increase in peak salinity values and slightly more salt penetration above High Toss road, as shown in Figure 5-50 and Figure 5-51.

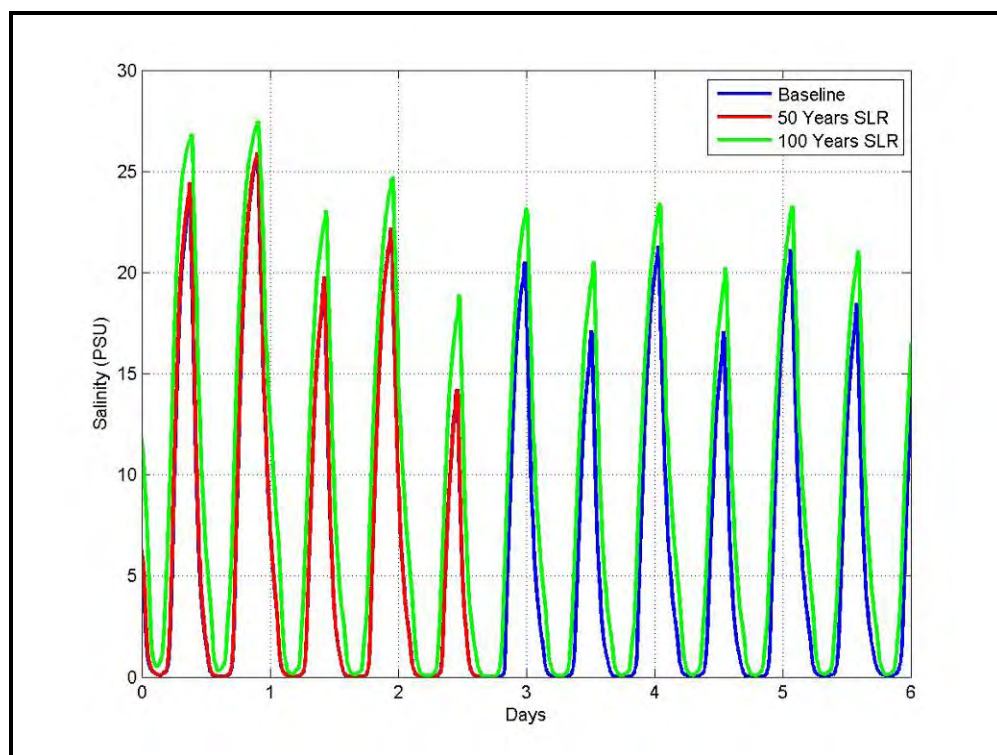


Figure 5-50. Sea level rise scenario for the salinity time series at Dogleg.

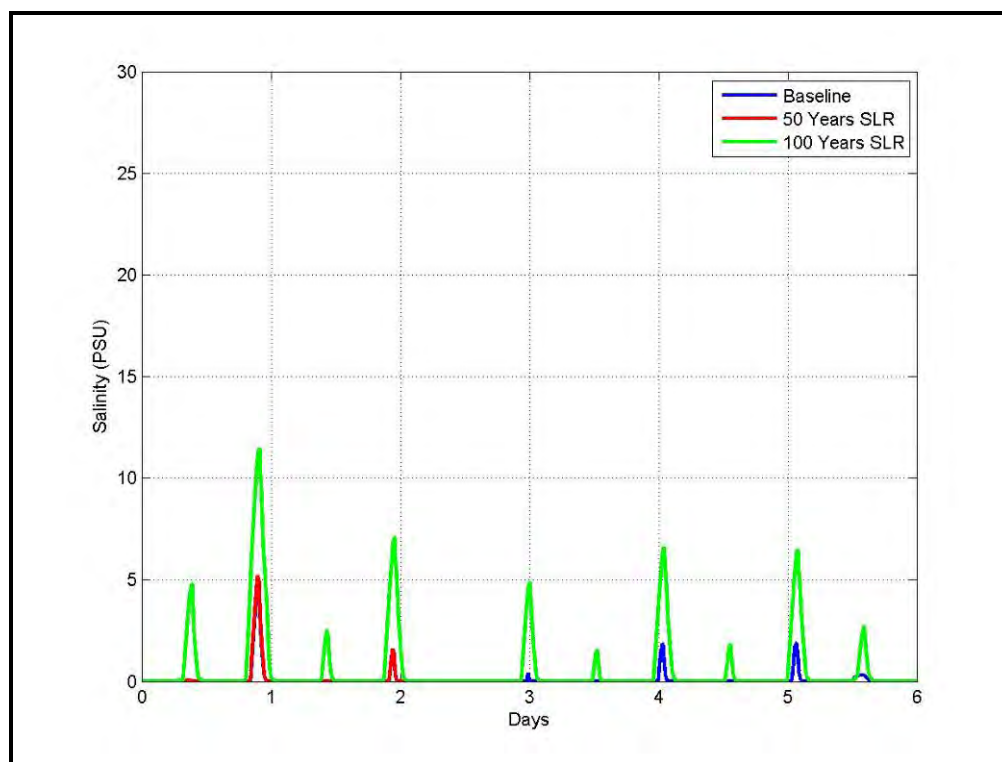


Figure 5-51. Sea level rise scenario for the salinity time series at HT_up.

5.6 MODEL ANIMATIONS AND VISUALIZATIONS

Model animations and visualizations are provided on the DVD entitled “Herring River Hydrodynamic Model – Model Animations and Visualizations, June 2009) that accompanies this report. The DVD includes a digital copy of the report, a digital copy of the final model presentation provided to the HRRC, animations of a number of different simulations, including a collection of representative animations of water surface and salinity for:

- Model calibration
- Model validation
- Storm and sea level rise simulations
- Alternative simulation scenarios as presented in the following section

Readme files are provided on the DVD to explain the files present, as well as provide instructions for special viewing options.

6.0 INITIAL ALTERNATIVE SIMULATIONS

This section provides results and discussion from a series of initial alternative simulations using the calibrated and validated hydrodynamic model of the Herring River System. The focus of these initial alternatives is related to evaluation of the opening at the Chequessett Neck Road dike, which is the primary restriction for the Herring River system. Specifically, the alternatives simulated include:

- 1) Simulation of a representative “natural” Herring River system through the removal of all anthropogenic features (e.g., culverts, dikes, railroad beds, etc.). As such, system was allowed to be fully open to tidal flow and allow relatively uninhibited exchange throughout the entire estuarine system. This alternative simulation could be considered a reasonable representation of the greatest restoration level that may be expected for a natural system (excluding natural and/or anthropogenic changes to the bathymetry/topography).
- 2) Simulation of a range of opening sizes replacing the existing dike at the Chequessett Neck Road crossing. These alternative simulations removed the existing dike and replaced them with box culverts of varying sizes, ranging from the current 10 meter opening to 100 meter opening. These alternatives also eliminated all flap gates and/or manual control from the culvert opening(s), which could be added in subsequent model alternatives. Therefore, full flow was allowed in both the flood and ebb direction through the dike. Ultimately, the results of these alternative simulations were used to determine the opening size that maximized the tidal range and salinity penetration in the lower portions of the Herring River estuary system (downstream of High Toss Road). The model results indicate that increasing the opening beyond an optimal size does produce diminishing returns in terms of tidal exchange and restoration potential.
- 3) Removal of the significant flood tidal shoal that has formed just upstream of the Chequessett Neck Road dike in response to the restriction imposed by the existing culverts in the dike. Proposed restoration alternatives with larger openings in the dike have raised concerns that this shoal may begin to impose a restriction, influencing tidal propagation farther up into the estuary as the current limiting restriction imposed by the dike’s culverts is removed. In order to assess any potential restriction the shoal may cause in the restored Herring River, a simulation was conducted with the flood tidal shoal removed.
- 4) An assessment of lowering the culvert inverts at Chequessett Neck. This alternative briefly examined if lowering the culvert inverts would significantly reduce tidal attenuation, improve tidal flux, or significantly impact water surface elevations or salinity levels. This alternative was also geared to determine if the local bathymetry was more of a limiting factor in the tidal attenuation.

6.1 REMOVAL OF ALL ANTHROPOGENIC STRUCTURES

Figure 6-1 presents the locations where anthropogenic features were removed from the model domain. This included the removal of all culverts, dikes, bridge crossings, and other anthropogenic structures that were directly restricting the tidal propagation in the system. The modification did not include changes that may have been made to the bathymetry/topography of the system through direct or indirect anthropogenic influences.

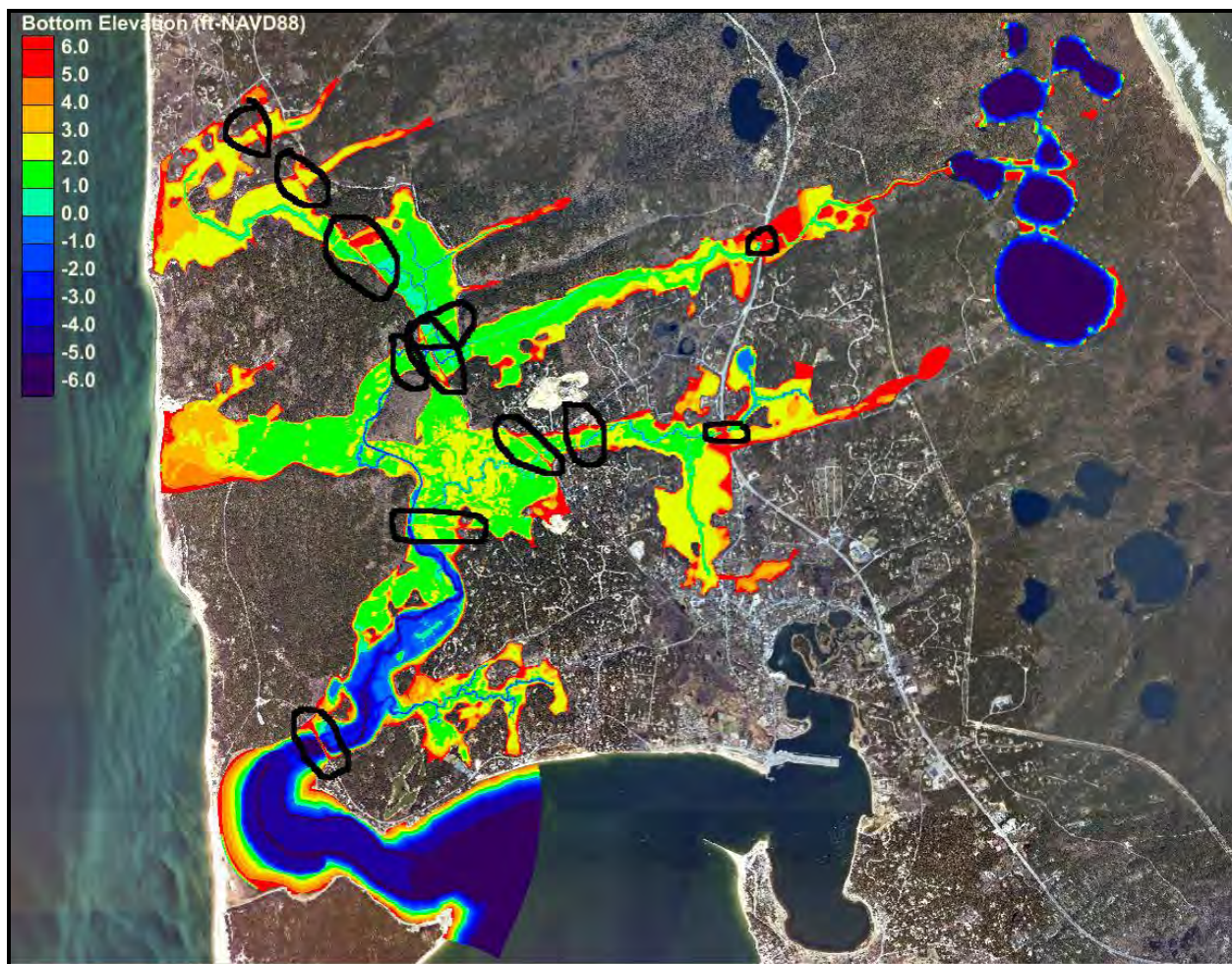


Figure 6-1. Locations within the model domain that were modified to represent removal of anthropogenic features.

At this initial stage, the model alternatives were simulated for normal conditions only in order to evaluate the potential feasibility of a restoration option. If the restoration alternative proves fruitful for normal conditions, then additional conditions (e.g., return period storms, sea level rise) can be simulated.

As an initial measure of the impact caused by the anthropogenic structures, the maximum water surface elevation level reached for existing conditions was compared to the maximum water surface elevation level reached for the simulation with anthropogenic structures removed. Figure

6-2 presents a comparison of the maximum water surface elevation for existing conditions (upper panel) and for the simulation that removed all anthropogenic structures (bottom panel).

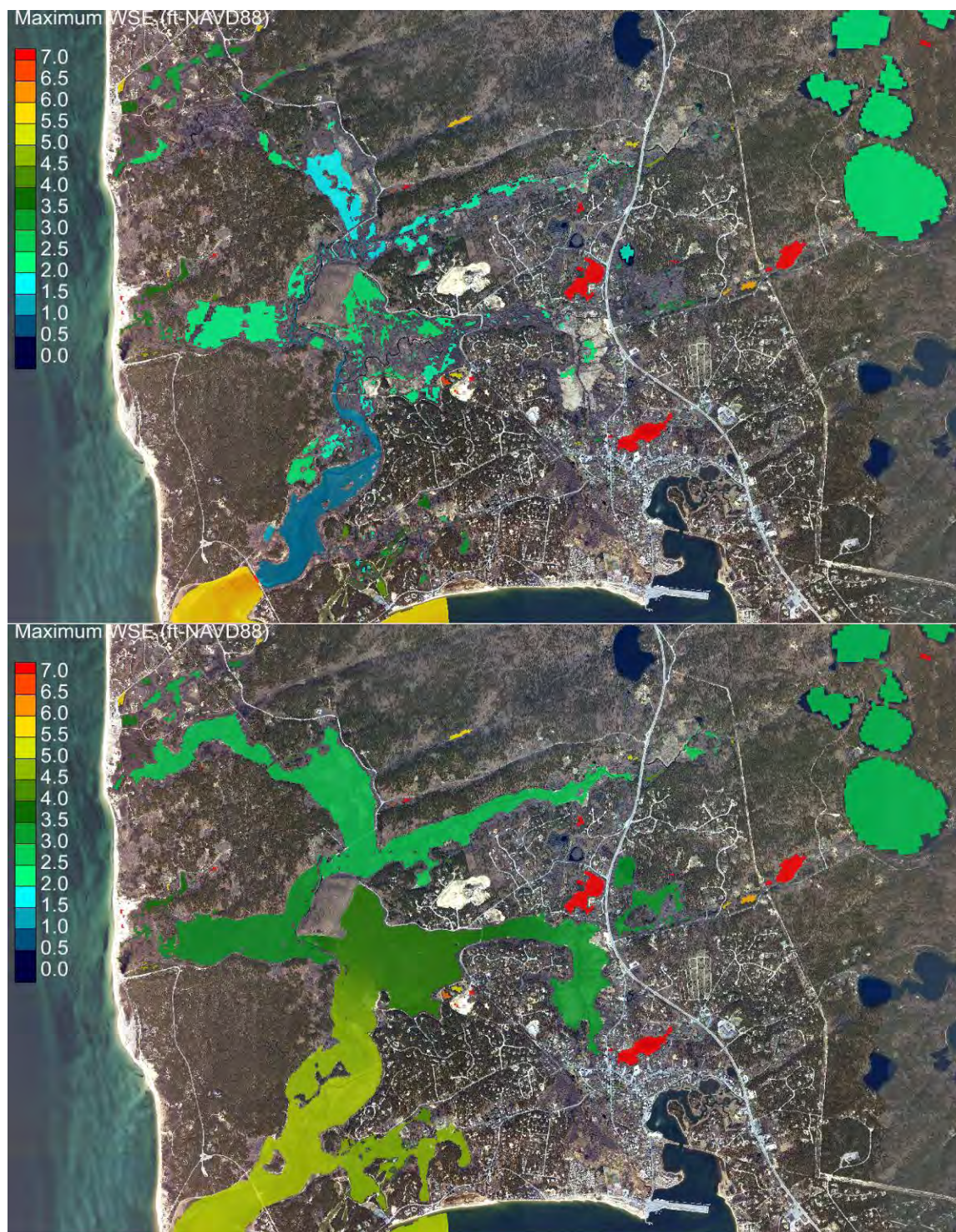


Figure 6-2. Maximum water surface elevation for existing conditions (upper panel) and for the no anthropogenic structures alternative (bottom panel). Color contours present water surface elevation in feet NAVD88.

Figure 6-3 presents a comparison of the fraction of the time the marsh surface is submerged during typical a tidal cycle for existing conditions (upper panel) and the no anthropogenic structure alternative (lower panel). Blue areas represent regions that do not ever get wet during normal tidal conditions, while red areas indicate areas that remain submerged throughout a normal tidal cycle.

Figure 6-4 presents a comparison of the maximum salinity levels for existing conditions (upper panel) and the no anthropogenic structure alternative (lower panel). Color contours range from freshwater (blue) to salinity levels equal to the Wellfleet Harbor salinity (30 ppt shown in red).

Significant increases in water level and salinity are clearly shown in Figure 6-2 and Figure 6-4 when comparing existing conditions to the fully open alternative. Maximum water levels increase by over 3.5 feet, while maximum salinities increase from freshwater to 30 ppt in numerous places. The spatial extent of the increases also illustrates the amount of historic marsh area that may have existed centuries ago. Similarly, the fraction of time the marsh plans remain submerged significantly increases throughout the region.

Although the fully open alternative is not likely a reasonable final solution given the upland infrastructure that has been developed over the last century, this alternative does provide a reasonable estimate of the maximum restoration potential for the Herring River system.

Additionally, specific tidal water elevations (e.g., MHW, MLW, tidal range) and time series comparison plots were determined for each of the restoration alternatives evaluated. This information is comprehensively presented in databases on the companion DVD to the report. Some specific water surface values and time series information for the fully open scenario are presented in the following section in order to ease comparison to various dike openings.

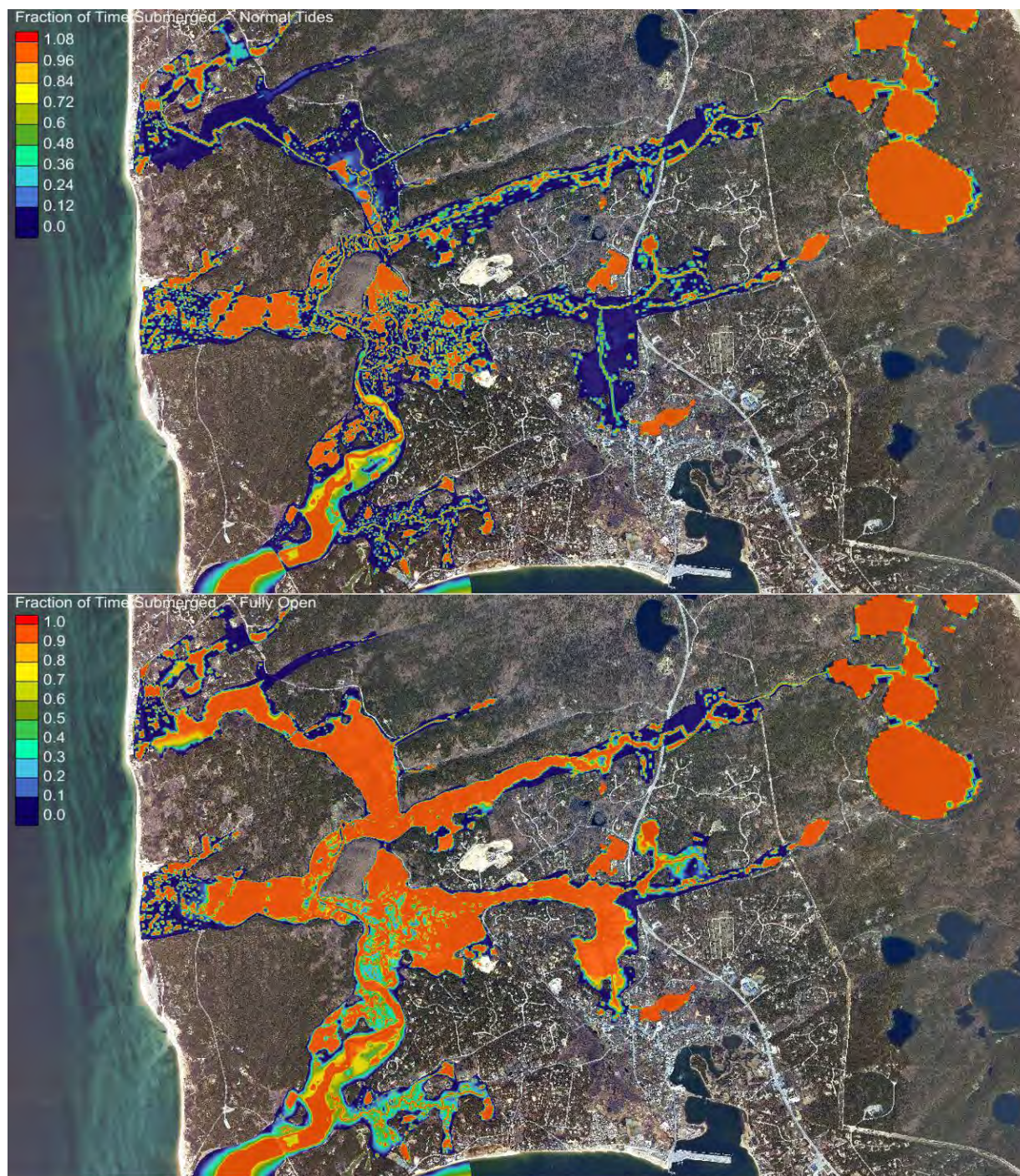


Figure 6-3. Fraction of time submerged during a typical tidal cycle for existing conditions (upper panel) and for the no anthropogenic structures alternative (bottom panel). Color contours present time submerged between 0% (blue) to 100% (red).

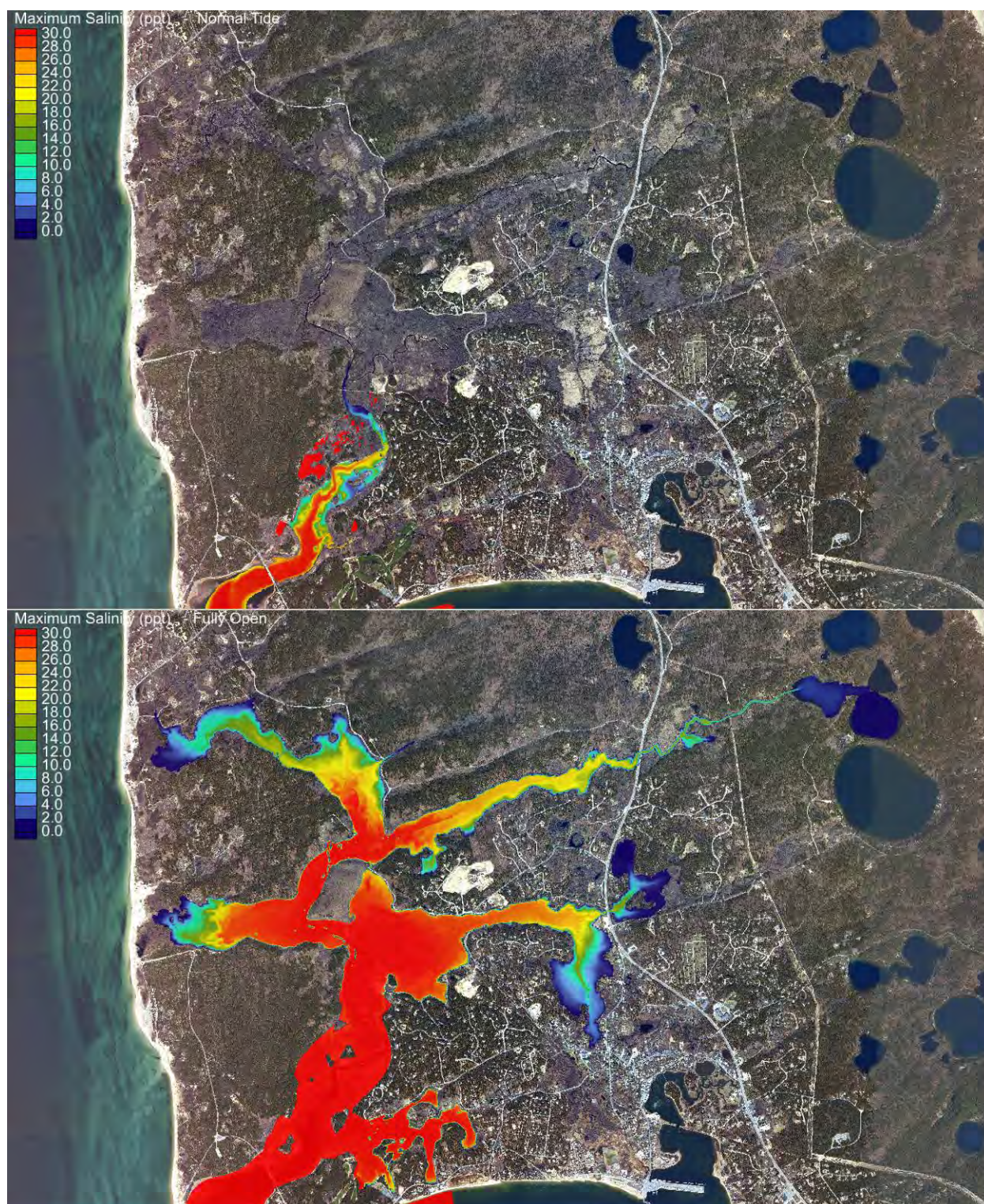


Figure 6-4. Maximum salinity levels for existing conditions simulation (upper panel) and for the no anthropogenic structures alternative (bottom panel). Color contours present the salinity level with blue being freshwater.

6.2 OPTIMIZATION OF THE CHEQUESSETT NECK ROAD OPENING

A critical part of the restoration success for Herring River will be based on the selection and optimization of the Chequessett Neck Road opening. As such, a wide range of potential openings, from 10 meters to 100 meters, were simulated in order to identify the optimal opening that will maximize restoration potential. Model results were evaluated in terms of both modeled water surface elevation and salinity levels. In these scenarios, no additional upstream changes were made to the Herring River estuary. So unlike the fully open scenario, which included the removal of all upstream anthropogenic features, these alternatives focus solely on the dike opening.

Table 6-1 provides detailed results of the tidal water levels and tidal range at selected locations throughout the Herring River estuary system for each of the simulated Chequessett Neck opening alternatives, as well as the fully open scenario presented in the previous section. For locations that are non-tidal, as indicated in Table 6-1, the MHW value represents the mean water surface elevation for the time period of the simulation. The locations where model comparisons were made are located within the major subembayment creeks throughout the system, as shown in Figure 6-5. There are three stations within each major subembayment creek. This data, along with additional information, is also presented within the databases presented on the DVD included with this report.



Figure 6-5. Station locations throughout the Herring River estuary system within all major subembayment creeks.

Table 6-1. Tidal water levels and tidal range at stations shown in Figure 6-5.

Location	Datum	Existing	10 m	20 m	30 m	40 m	50 m	60 m	100 m	Fully Open
Lower HR 1	MHW	0.53	2.87	3.71	3.97	4.05	4.17	4.22	4.52	4.22
	MLW	-1.83	-1.24	-1.84	-1.98	-2.00	-2.01	-2.02	-2.03	-1.94
	Range	2.36	4.11	5.55	5.95	6.05	6.18	6.24	6.55	6.16
Lower HR 2	MHW	0.39	2.81	3.58	3.78	3.86	3.96	4.03	4.34	4.18
	MLW	-1.80	-1.12	-1.39	-1.41	-1.38	-1.37	-1.36	-1.33	-1.19
	Range	2.19	3.93	4.98	5.19	5.24	5.32	5.39	5.67	5.37
Lower HR 3	MHW	0.40	2.75	3.48	3.73	3.83	3.89	3.91	4.19	3.99
	MLW	-1.68	-0.79	-0.30	-0.13	-0.05	-0.02	-0.01	0.04	0.10
	Range	2.08	3.54	3.77	3.86	3.88	3.91	3.92	4.15	3.89
Mill Cr 1	MHW	0.33	2.75	3.50	3.76	3.88	3.95	4.02	4.29	4.13
	MLW	-1.83	-1.23	-1.70	-1.80	-1.80	-1.80	-1.80	-1.80	-1.69
	Range	2.16	3.98	5.20	5.56	5.68	5.75	5.82	6.09	5.81
Mill Cr 2	MHW	0.15	2.65	3.39	3.62	3.71	3.77	3.86	4.11	3.94
	MLW	-0.92	-0.51	-0.57	-0.57	-0.57	-0.57	-0.57	-0.56	-0.58
	Range	1.07	3.17	3.95	4.18	4.28	4.34	4.43	4.67	4.52
Mill Cr 3	MHW	0.14	2.64	3.37	3.60	3.70	3.77	3.85	4.14	3.96
	MLW	-0.89	-0.50	-0.54	-0.54	-0.53	-0.53	-0.53	-0.52	-0.55
	Range	1.02	3.14	3.91	4.13	4.23	4.30	4.38	4.66	4.51
Mid HR 1	MHW	0.41	2.29	2.70	3.20	3.41	3.57	3.73	4.05	3.94
	MLW	-1.59	-0.42	0.92	2.31	2.64	2.77	2.84	3.04	0.12
	Range	2.00	2.71	1.79	0.89	0.77	0.80	0.90	1.02	3.82
Mid HR 2	MHW	0.43	2.16	2.49	2.85	3.06	3.20	3.30	3.58	3.13
	MLW	-0.24	0.80	1.67	2.54	2.76	2.86	2.92	3.08	2.13
	Range	0.67	1.36	0.81	0.31	0.29	0.34	0.39	0.50	1.00
Mid HR 3	MHW	0.55	2.04	2.43	2.78	2.96	3.08	3.17	3.39	2.76
	MLW	0.37	1.41	2.10	2.61	2.75	2.82	2.84	2.98	2.51
	Range	0.18	0.63	0.34	0.17	0.21	0.26	0.32	0.41	0.25
Upper PD 1	MHW	1.19	2.15	2.48	2.85	3.09	3.28	3.40	3.68	3.37
	MLW	1.12	2.10	2.39	2.56	2.72	2.81	2.86	3.02	2.60
	Range	0.07	0.06	0.09	0.29	0.37	0.47	0.54	0.66	0.77
Upper PD 2	MHW	1.38	2.23	2.60	2.88	3.02	3.13	3.20	3.45	2.96
	MLW	non-tidal	non-tidal	2.55	2.84	2.99	3.08	3.13	3.29	2.67
	Range	non-tidal	non-tidal	0.05	0.04	0.03	0.05	0.07	0.15	0.29
Upper PD 3	MHW	1.39	2.23	2.60	2.87	3.01	3.12	3.19	3.43	2.96
	MLW	1.31	2.18	2.55	2.84	2.98	3.07	3.12	3.28	2.67
	Range	non-tidal	non-tidal	0.05	0.03	0.03	0.05	0.07	0.15	0.29
Duck Harbor 1	MHW	0.42	2.13	2.44	2.81	3.02	3.16	3.26	3.53	3.03
	MLW	-0.25	0.79	1.67	2.52	2.74	2.84	2.89	3.05	2.14
	Range	0.67	1.34	0.78	0.29	0.28	0.32	0.37	0.47	0.89
Duck Harbor 2	MHW	1.82	2.00	2.34	2.77	2.97	3.11	3.21	3.47	2.96
	MLW	non-tidal	non-tidal	2.11	2.50	2.70	2.80	2.85	3.01	2.26
	Range	non-tidal	non-tidal	0.23	0.27	0.27	0.31	0.36	0.46	0.70
Duck Harbor 3	MHW	2.72	2.72	2.72	2.75	2.97	3.11	3.21	3.47	2.97
	MLW	non-tidal	non-tidal	non-tidal	non-tidal	2.72	2.81	2.86	3.02	2.73
	Range	non-tidal	non-tidal	non-tidal	non-tidal	0.25	0.30	0.35	0.46	0.24
Bound Brook 1	MHW	1.80	2.19	2.50	2.82	3.01	3.18	3.25	3.43	2.80
	MLW	non-tidal	non-tidal	2.38	2.61	2.75	2.79	2.86	2.99	2.48
	Range	non-tidal	non-tidal	0.12	0.20	0.27	0.39	0.38	0.44	0.32
Bound Brook 2	MHW	1.83	2.23	2.56	2.77	2.85	2.89	2.91	2.98	2.70
	MLW	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	2.54
	Range	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	0.16
Bound Brook 3	MHW	1.83	2.23	2.56	2.77	2.84	2.88	2.90	2.96	2.68
	MLW	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	2.55
	Range	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	0.13
Upper HR 1	MHW	0.60	1.99	2.44	2.76	2.99	3.10	3.14	3.32	2.72
	MLW	0.44	1.56	2.33	2.65	2.81	2.88	2.95	3.08	2.53
	Range	0.16	0.44	0.11	0.11	0.17	0.22	0.19	0.24	0.19
Upper HR 2	MHW	1.81	1.94	2.35	2.64	2.87	2.99	3.06	3.26	2.65
	MLW	non-tidal	non-tidal	non-tidal	non-tidal	2.77	2.85	2.89	3.04	2.50
	Range	non-tidal	non-tidal	non-tidal	non-tidal	0.10	0.14	0.17	0.22	0.16
Upper HR 3	MHW	2.10	2.17	2.39	2.54	2.66	2.75	2.85	3.01	2.56
	MLW	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	2.75	2.88	2.48
	Range	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	0.10	0.14	0.08

Water levels and range are also evaluated at specific locations throughout the marsh plain, as shown in Figure 6-6. Table 6-2 presents the MHW at these locations. The values in the tables can be used to assess the relative changes throughout the estuarine system.

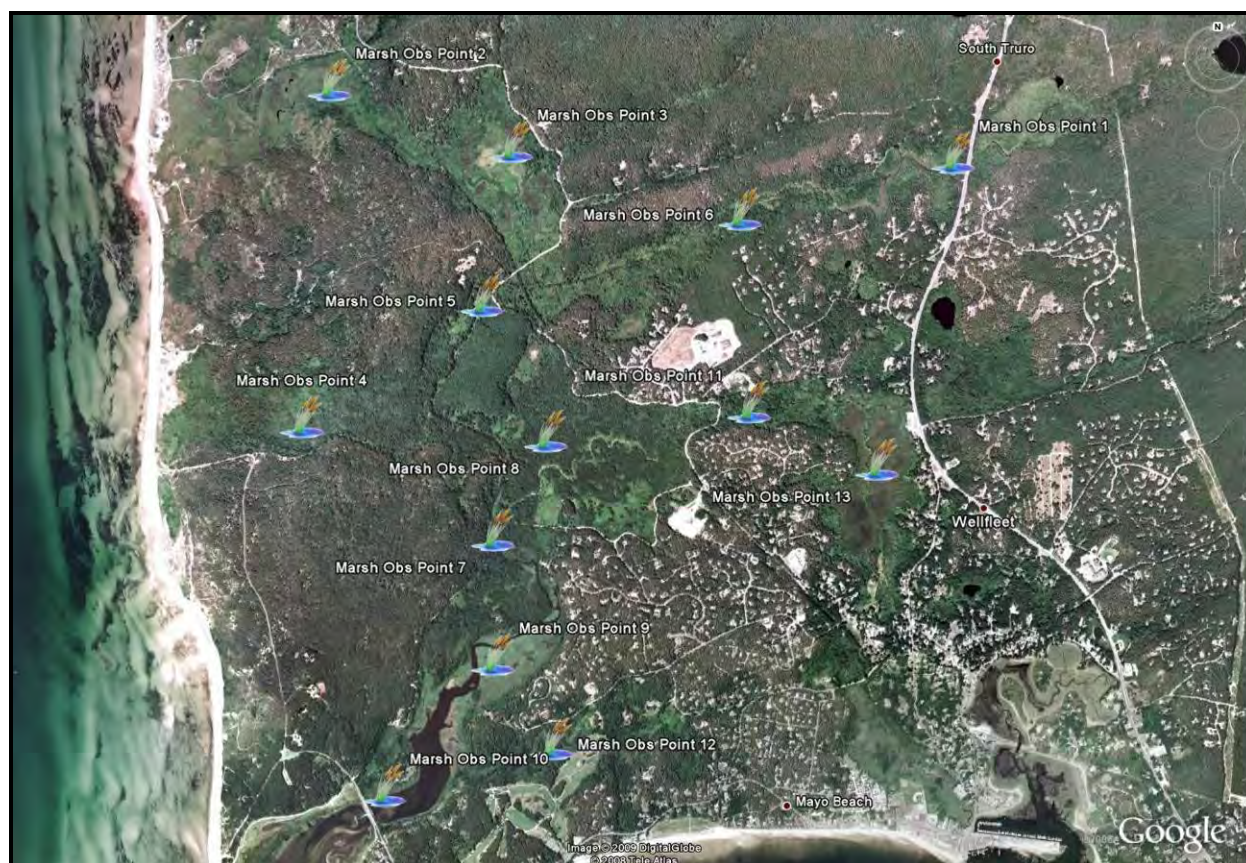


Figure 6-6. Station locations throughout the Herring River estuary system at selected marsh locations.

Table 6-2. Mean High Water Elevations at selected marsh stations shown in Figure 6-6.

Location	Existing	10 m	20 m	30 m	40 m	50 m	60 m	100 m	Fully Open
Marsh 1	dry	dry	dry	2.63	2.81	2.93	3.00	3.20	2.59
Marsh 2	dry	dry	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	non-tidal	2.66
Marsh 3	dry	non-tidal	2.39	2.69	2.88	2.99	3.07	3.26	2.68
Marsh 4	non-tidal	2.02	2.35	2.77	2.98	3.11	3.22	3.48	2.97
Marsh 5	non-tidal	2.02	2.38	2.72	2.90	3.02	3.10	3.33	2.70
Marsh 6	non-tidal	1.97	2.37	2.67	2.86	2.97	3.05	3.24	2.64
Marsh 7	0.39	2.71	3.42	3.67	3.76	3.83	3.86	4.13	3.93
Marsh 8	dry	dry	dry	2.84	3.06	3.23	3.36	3.72	3.39
Marsh 9	0.33	2.69	3.43	3.71	3.78	3.91	3.95	4.23	4.06
Marsh 10	0.28	2.79	3.59	3.78	4.09	3.91	4.13	4.43	4.08
Marsh 11	dry	2.22	2.58	2.84	2.99	3.10	3.17	3.42	3.14
Marsh 12	non-tidal	2.57	3.31	3.53	3.63	3.69	3.79	4.03	3.86
Marsh 13	non-tidal	non-tidal	2.60	2.86	2.99	3.10	3.18	3.41	2.95

From a water surface elevation perspective, the tidal attenuation between Wellfleet Harbor and Herring River is reduced the most when the dike opening is increased from existing conditions to 30 meters. Within the Lower Herring River and Mill Creek sub-basins, a 30 meter opening at the dike maximizes tidal exchange. Larger opening at the dike continue to increase the MHW level and tidal exchange; however, these increase result in less significant gains with diminishing returns. Figure 6-7 shows the water surface elevation (WSE) results at the lower Herring River station 1 location (just upstream of the dike) for all alternative cases. The figure also shows the river bottom elevation, the offshore tidal signal in Wellfleet Harbor, and the existing tidal range. MHW, MLW, and tidal range for the 30 meter opening are reasonably close to those produced by the 100 meter opening. For example, the tidal exchange achieved using a 100 meter opening is not significantly greater than a 30 meter opening. Additionally, the low tide level does not approach low tide in Wellfleet Harbor (Old saw) for any of the alternatives, including the fully open case due to the combined effect of the local dike bathymetry and the culvert inverts. The potential impact of lowering the culvert invert is further investigated in section 6.4 of this report.

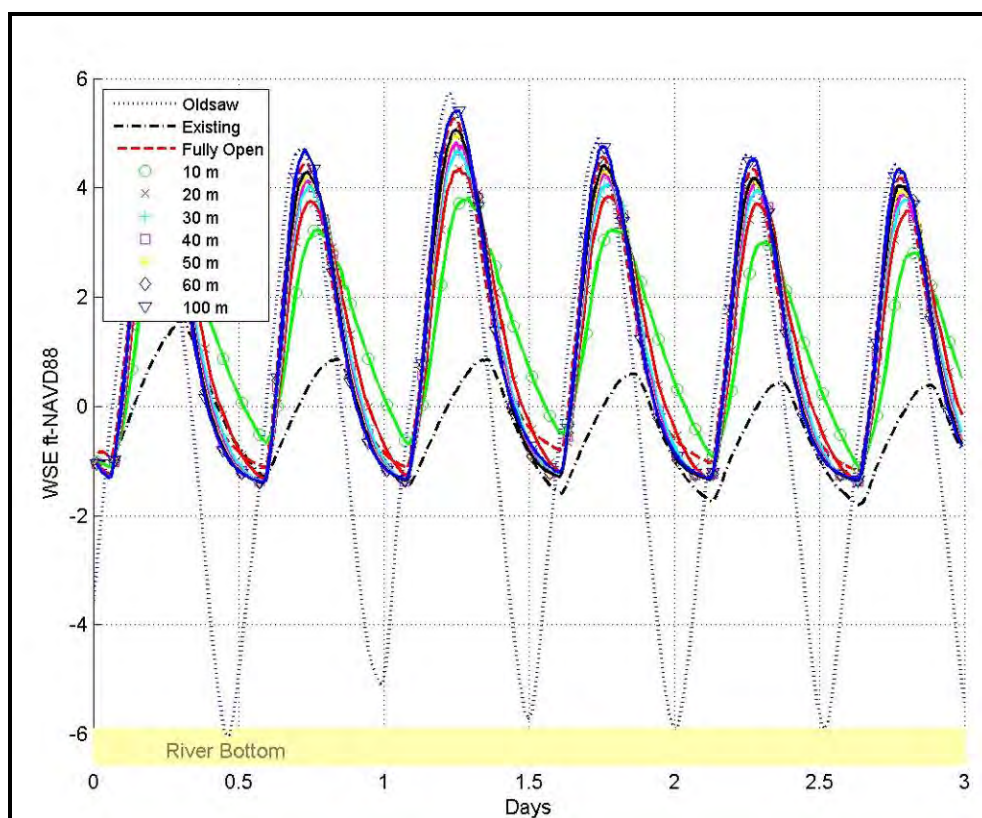


Figure 6-7. Water surface elevation (WSE) results at the Lower Herring River station 1 location for all alternative cases.

The potential optimal opening at Chequessett Neck Road was also evaluated from a salinity level perspective. While 30 meters appears to maximize the restoration potential from a water surface elevation perspective, a slightly larger opening maximizes the restoration potential from a salinity perspective. Figures 6-8 through 6-12 show the salinity concentrations throughout the Herring River system for dike opening sizes of 20 meters, 30 meters, 40 meters, 50 meters, and

60 meters, respectively. Based on visual observations of the results, as well as time series of salinity levels, a 50 meter opening best maximizes the salinity levels throughout the system. Increasing from a 50 meter opening to a 60 meter opening indicates minimal improvement in salinity levels. Therefore, openings beyond 50 meters indicate diminishing returns in terms of restoration potential.

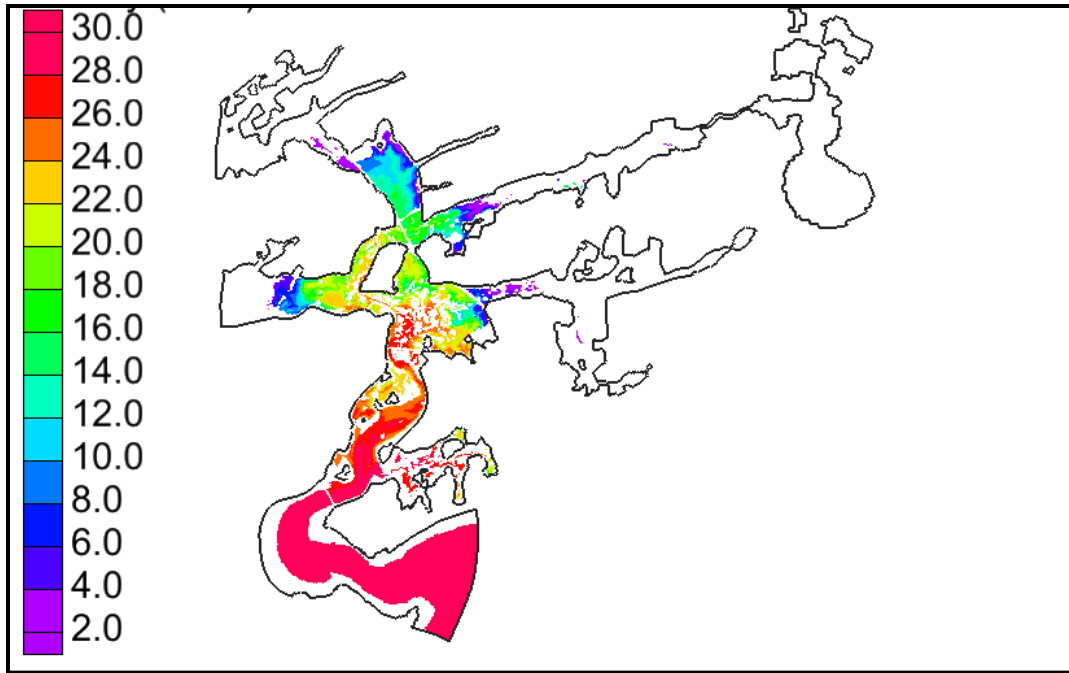


Figure 6-8. Salinity concentration throughout the Herring River system with an opening of 20 meters at the Chequessett Neck Road. Salinity contours are presented in ppt.

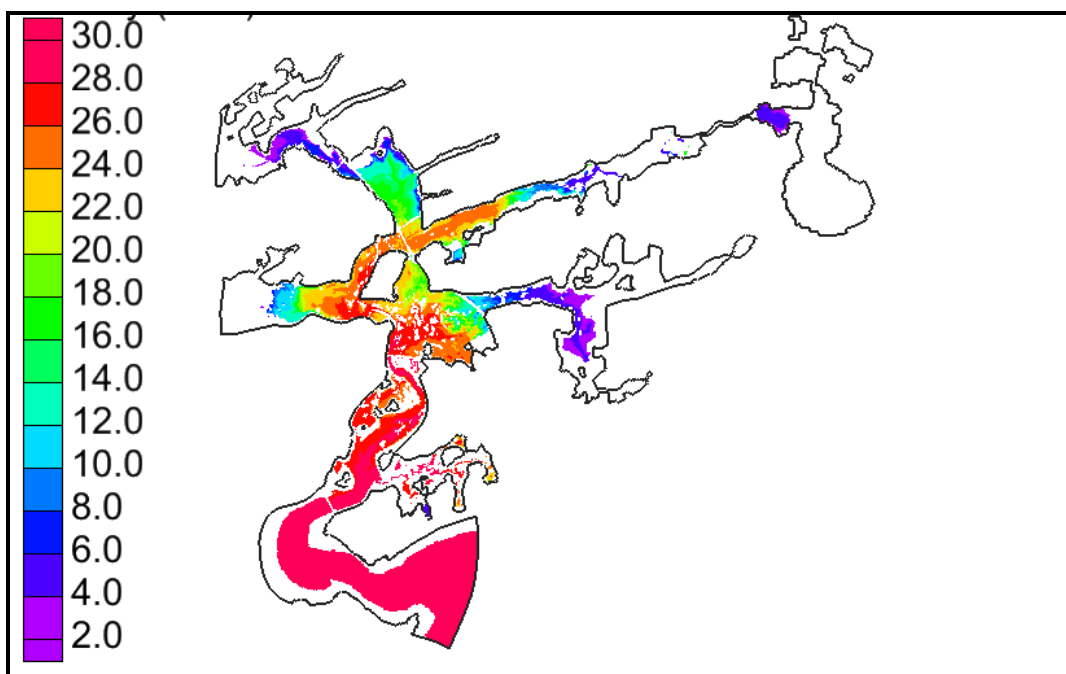


Figure 6-9. Salinity concentration throughout the Herring River system with an opening of 30 meters at the Chequessett Neck Road. Salinity contours are presented in ppt.

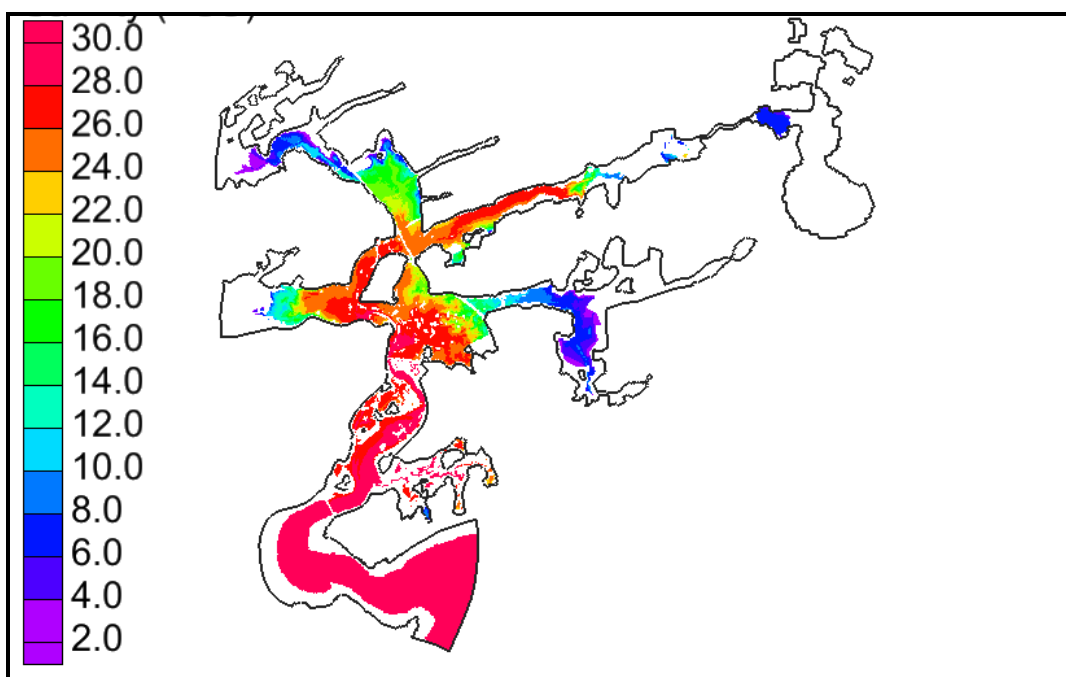


Figure 6-10. Salinity concentration throughout the Herring River system with an opening of 40 meters at the Chequessett Neck Road. Salinity contours are presented in ppt.

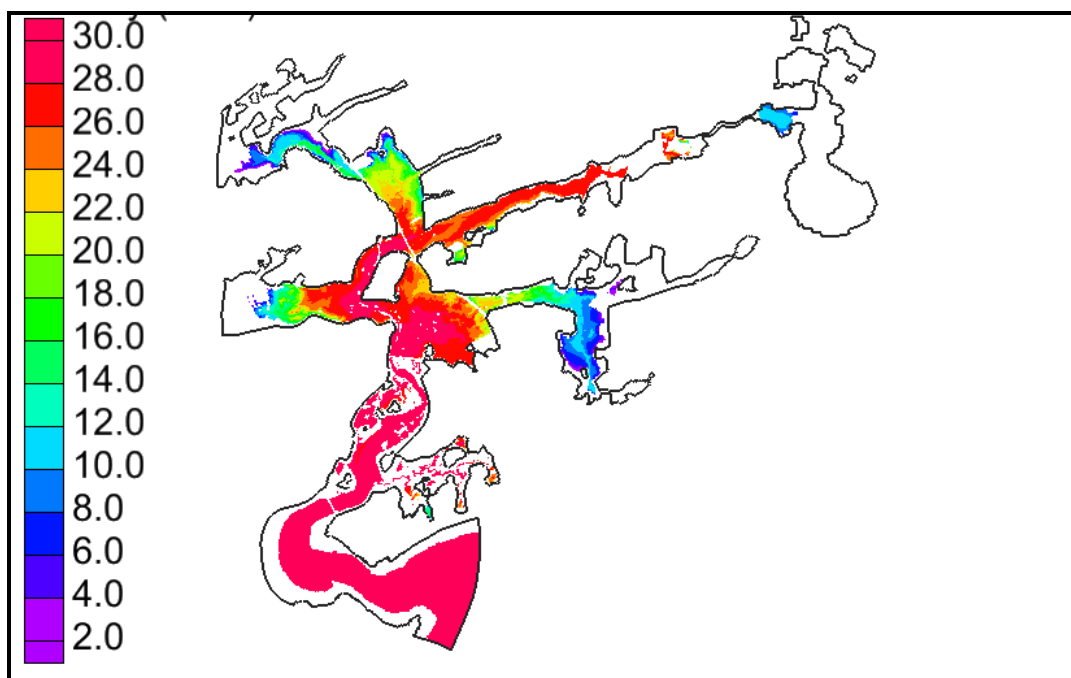


Figure 6-11. Salinity concentration throughout the Herring River system with an opening of 50 meters at the Chequessett Neck Road. Salinity contours are presented in ppt.

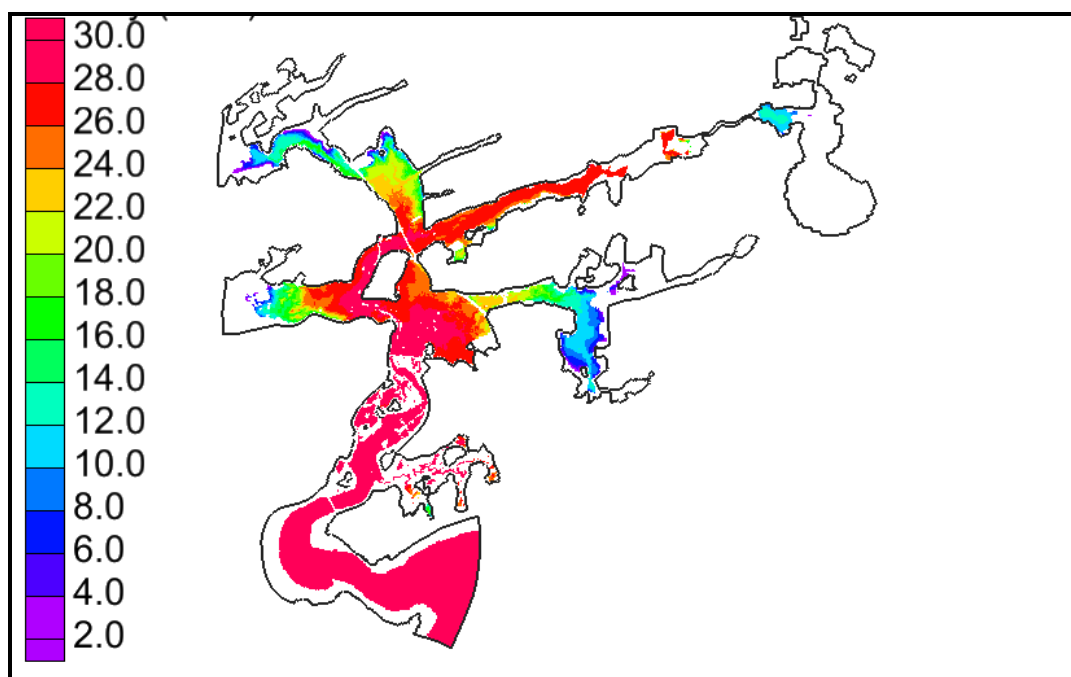


Figure 6-12. Salinity concentration throughout the Herring River system with an opening of 60 meters at the Chequessett Neck Road. Salinity contours are presented in ppt.

6.3 REMOVAL OF THE FLOOD TIDAL SHOAL

With the exception of the removed flood tidal shoal, this simulation was the same as the 30 meter dike opening simulation conducted as part of the second alternative. The section describes the changes made to the model grid, and the effect of the removal of the shoal on water levels and depth-averaged current velocities.

6.3.1 Grid Modification

Figure 6-13 shows the bathymetry of the Herring River EFDC model grid in its current unaltered state. The figure clearly shows the flood tidal shoal which has formed just upstream of the Chequessett Neck Road dike. There is a significant portion of this shoal which lies between -1.5 and -1.0 feet-NAVD88 and, during typical low tides, becomes exposed. The Herring River model grid was modified to remove this shoal by reducing the elevation of grid cells to an elevation of -3.4 feet-NAVD88. This elevation is same as the elevation of the culvert invert in the dike and is roughly the same as the surrounding river bottom. The resulting model grid bathymetry is shown in Figure 6-14. Removing the flood tidal shoal to this elevation in the actual river would require complete removal of approximately 6,000 cubic yards of sediment. The channel thalweg, which is located to the south of the flood tidal shoal and is approximately -6.0 feet NAVD88, may, like the shoal, be a response to the restriction imposed by the current configuration of the dike and shoal combination.

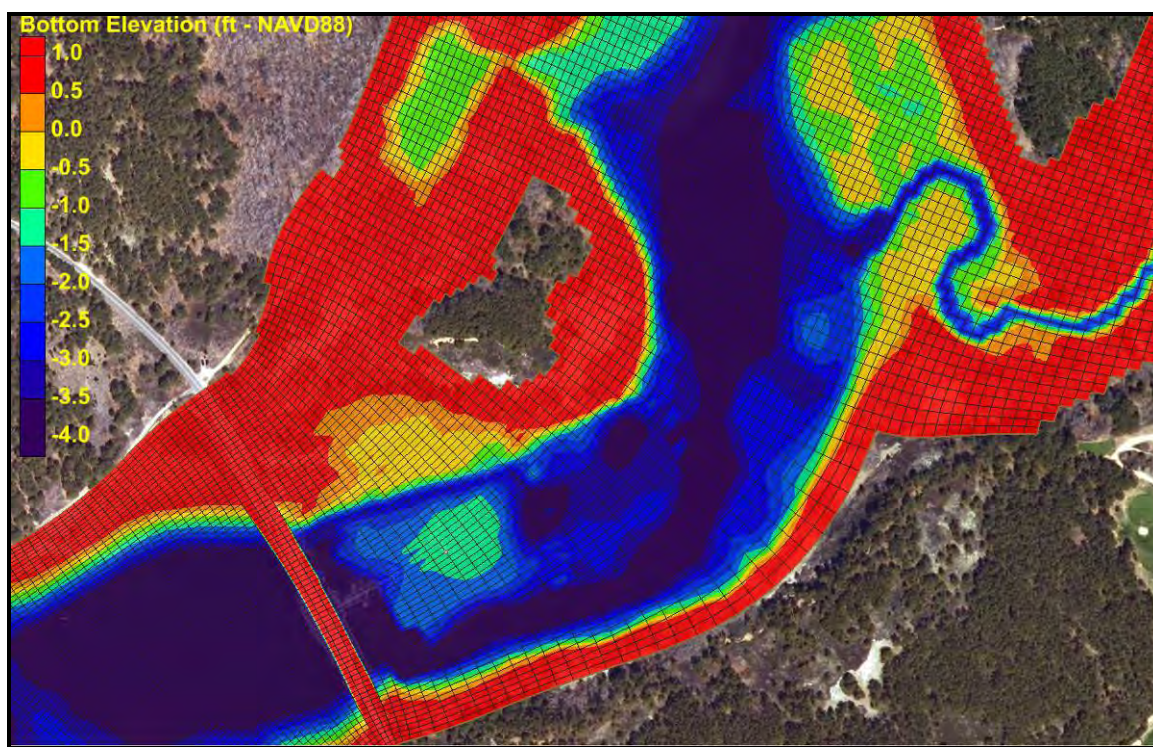


Figure 6-13. Current bathymetry of the Herring River model showing the flood tidal shoal.

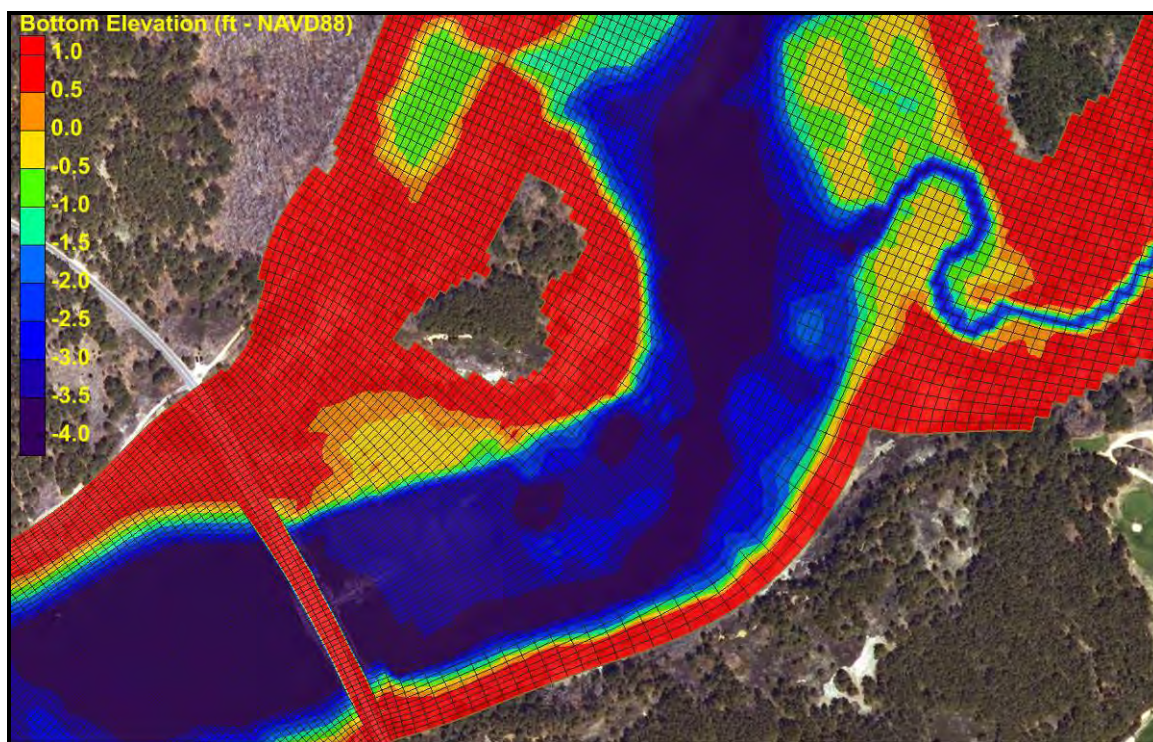


Figure 6-14. Bathymetry of the Herring River model with the flood tidal shoal removed.

6.3.2 Water Surface Elevation Results

In order to determine the effect of the flood tidal shoal on water levels in the proposed 30 meter dike opening restoration alternative, Figure 6-15 presents water levels taken from the model results at the Lower Herring River 2 observation location (Figure 6-5). This location is centrally located within the lower Herring River sub-basin and provides a representative sample of the water levels throughout lower sub-basin. Figure 6-15 shows the impact of the shoal removal on water surface elevations is minor. The low tide elevations are slightly lower (about 1 inch) with the shoal removed. This is evidence, that the shoal does provide some influential restriction to the ebbing tide particularly when the tide is low and the top of the shoal become visible. While this difference is quite small, it may be greater for larger dike openings. Besides the flood tidal shoal there are other factors which may limit low tide elevations upstream of Chequessett Neck Road. Potentially significant factors are the invert elevation of the culverts in the dike and bathymetry of “the Gut”.

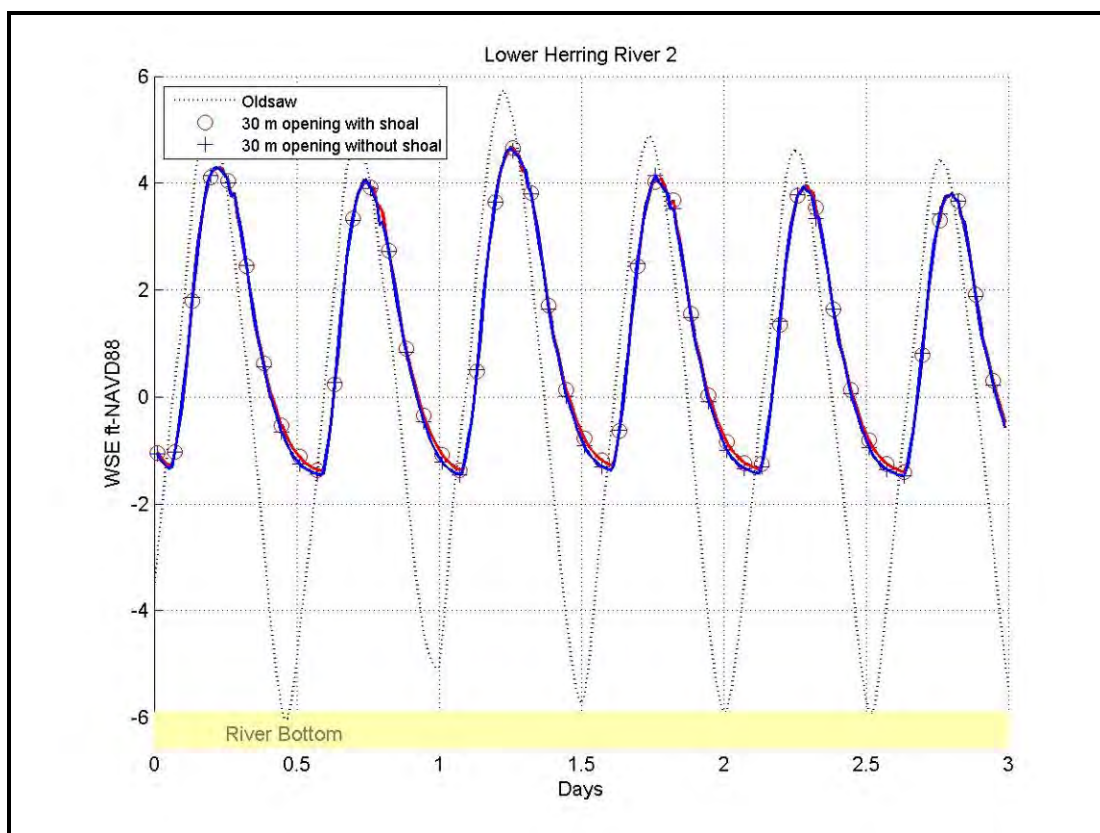


Figure 6-15 Hydrographs at the Lowering Herring River 2 observation station for 30 meter dike opening simulations with and without the flood tidal shoal.

6.3.3 Depth Averaged Velocity Results

While the effect on water levels caused by the shoal removal is minor, there is a more pronounced effect on current velocities in the area just upstream of Chequessett Neck Road. The greatest velocities in the area around the flood tidal shoal are observed during ebb flows when water is flowing out of all three culverts in the dike. Figure 6-16 shows the depth-averaged current velocity during ebb tide when the current speed is greatest for the 30 meter dike opening case with the flood tidal shoal present. Figure 6-17 shows a similar snapshot from the same time in the simulation with the shoal removed. Comparison of these two figures illustrates that the shoal causes the ebb current to be concentrated to the south of the shoal with current speeds over 3 feet per second. With the shoal removed, the maximum current speed is reduced by approximately half. While the maximum speed is greatly reduced by the removal of the shoal, the area with current speeds over 0.5 feet per second is greatly increased resulting in an overall greater total volume flux. This increased volume flux allows for the slightly greater tidal range shown in the lower sub-basin with the shoal removed.

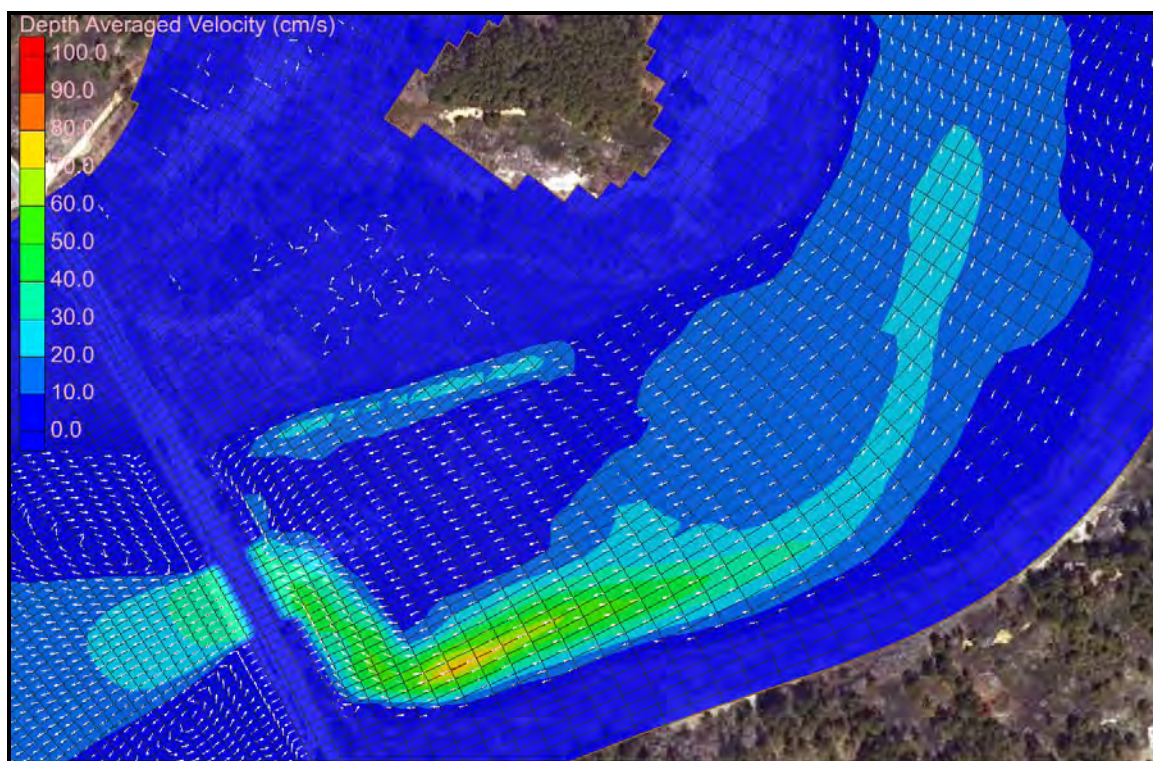


Figure 6-16. Depth averaged velocity contours and vectors during ebb for alternative with 30 meter dike opening.

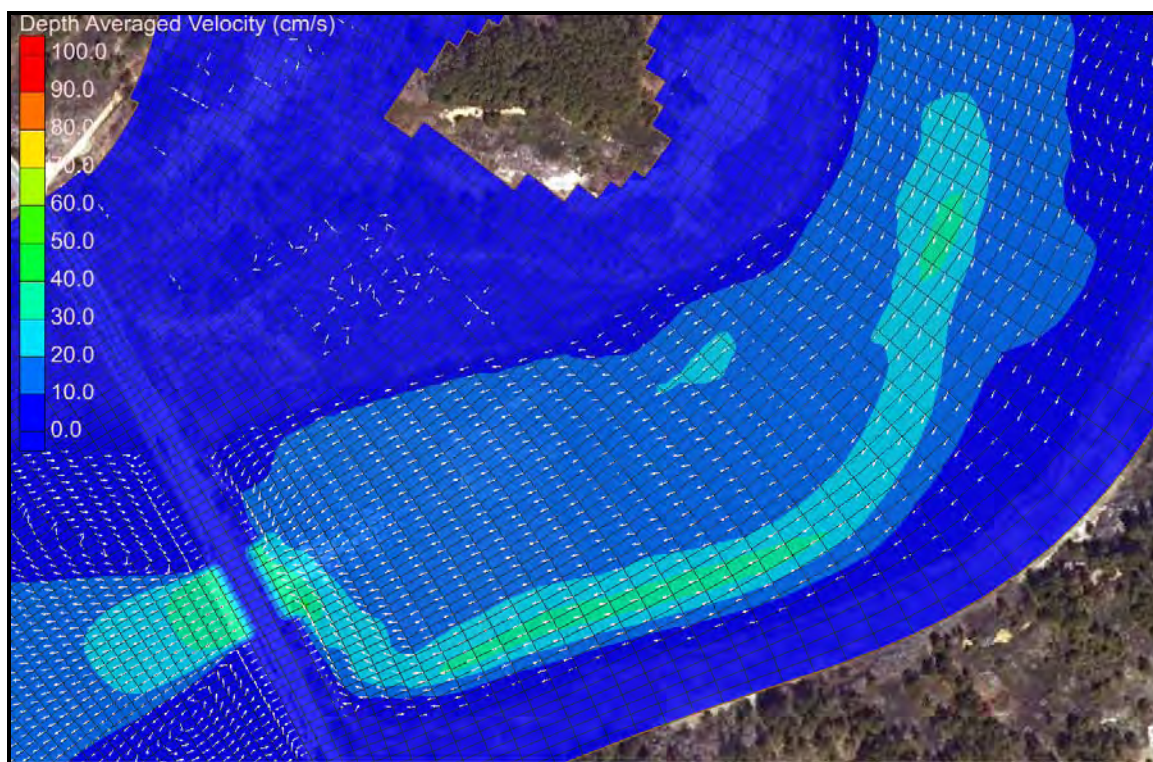


Figure 6-17. Depth averaged velocity contours and vectors during ebb for alternative 30 meter dike opening and flood tidal shoal removed.

6.4 LOWERING CULVERT INVERTS AT CHEQUESSETT NECK

As an initial test to evaluate the potential influence of modifying the culvert invert (specifically, lowering the invert) at the Chequessett Neck Road dike, two additional simulations were performed. The simulations were geared towards determining if the culvert invert alone (without natural or dredging adjustment to the bathymetry) would influence the water surface elevation. Updated Gut bathymetry (Section 6.5) and a 20 meter wide culvert opening were used for both simulations; however, in one simulation the existing culvert invert elevation of -3.4 feet NAVD88 was used while in the other the invert was lowered 1.5 feet to -4.9 feet NAVD88. The tops of the culverts were set sufficiently high in both simulations to prevent them from flowing completely full. The results for water surface elevation from these simulations in the lower Herring River sub-basin are compared in Figure 6-18. These results show that the existing culvert invert elevation is sufficiently low as to not pose a restriction of to low tide in the river. This also likely indicates that the bathymetry surrounding the dike tends to be the limiting factor for low tide levels in the river. Lowering the culvert does allow a greater volume of flow (slightly higher tides); however, without an adjustment to the bathymetry, the low water level does not decrease. In reality, it is likely that a lower culvert invert, combined with the increased volumetric flow, would cause scour and an eventual lowering of the river bed, which may result in a more significant change to the mean low water elevation.

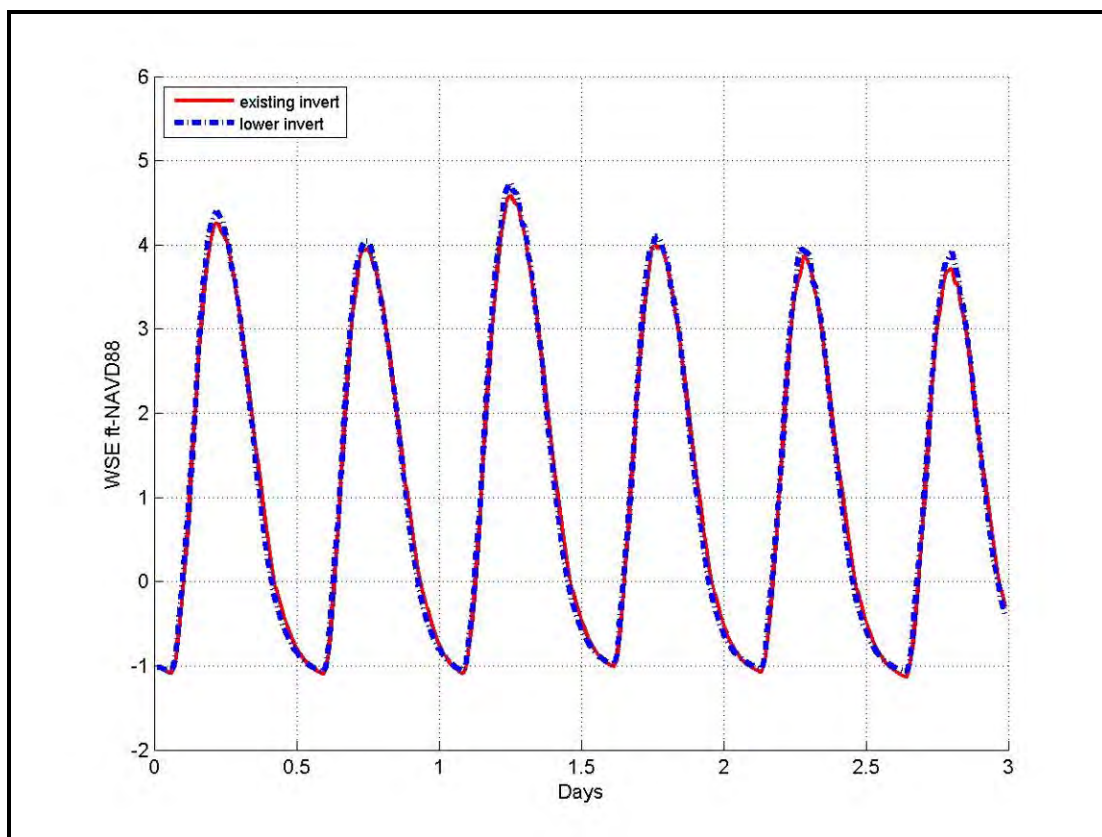


Figure 6-18. Time series of modeled WSE with existing Chequessett Neck Road dike culvert invert and with the invert lowered 1.5 ft.

6.5 UPDATED GUT BATHYMETRY

Following the calibration and validation of the Herrin River model, additional bathymetric data was collected in the Gut region to better define this region within the numerical model. After the acquisition of this new bathymetric data collected on February 26, 2009, modifications were made to the Herring River EFDC model. Since the addition of this new, more accurate data to the model may affect the results, the new elevation data were incorporated into the model grid and a number of simulations were re-run to quantify any effect this may have on model results. Simulations conducted using the updated bathymetric data include a simulation of existing conditions (to verify model calibration) and a simulation of the 30 meter wide dike culvert alternative (to determine if the bathymetric update impacts alternative simulations that contain more volumetric flux).

6.5.1 Bathymetric Data

Prior to the inclusion of the new Gut bathymetry in the Herring River hydrodynamic model, an approximate idealized bathymetry was used in the region. This initial estimate was based on outdated nautical charts of the area. Measurements indicate that there is an asymmetrical tidal signal downstream of the Chequessett Neck Road dike. This asymmetrical tide is characterized by a truncation of low tide portion just downstream of the Chequessett Neck Road Dike when compared to tidal signal at observed farther into the harbor at Old Saw. Ultimately, the bathymetry of the Gut limits the lower water level of the tidal signal. Therefore, accurate bathymetric data within the Gut was critical to ensure accurate model results. Figure 6-19 shows color contours of the idealized bathymetry initially used in the model, while Figure 6-20 shows the new measured Gut bathymetry. Additionally, Figure 6-21 presents hypsometric curves for the Gut area derived from both the estimated and measured bathymetries. In general, when compared to the new data from the recent survey, the initial estimate was fairly accurate in terms of the hypsometry of the region; however, the actual data provides a much more reliable surface for use in simulation of the model.

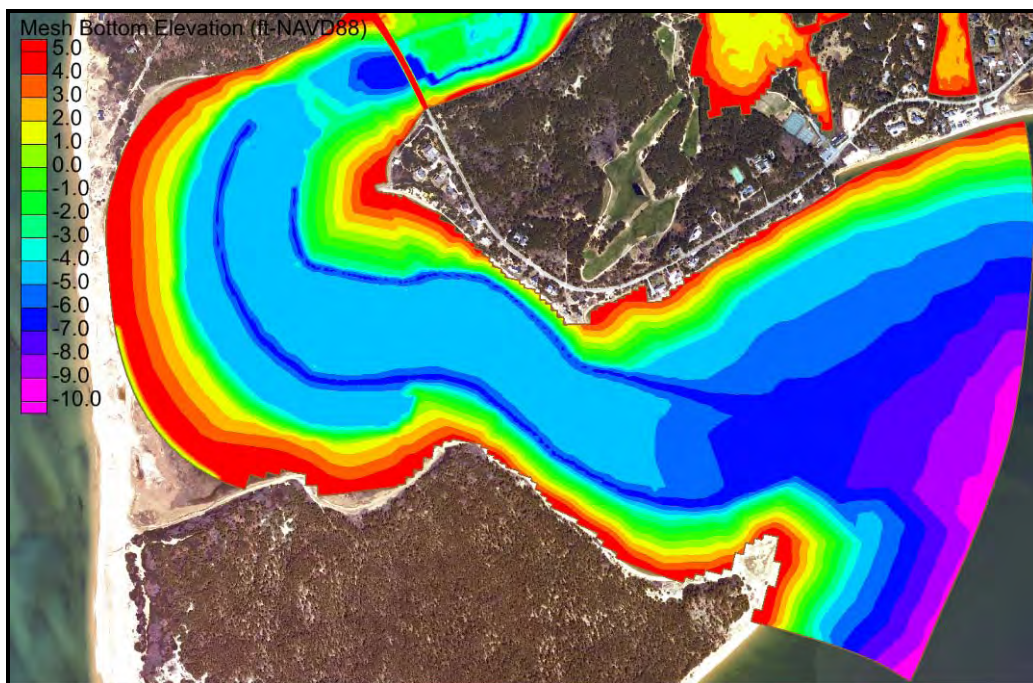


Figure 6-19. EFDC model grid showing idealized bottom elevation initially used in the Herring River model.

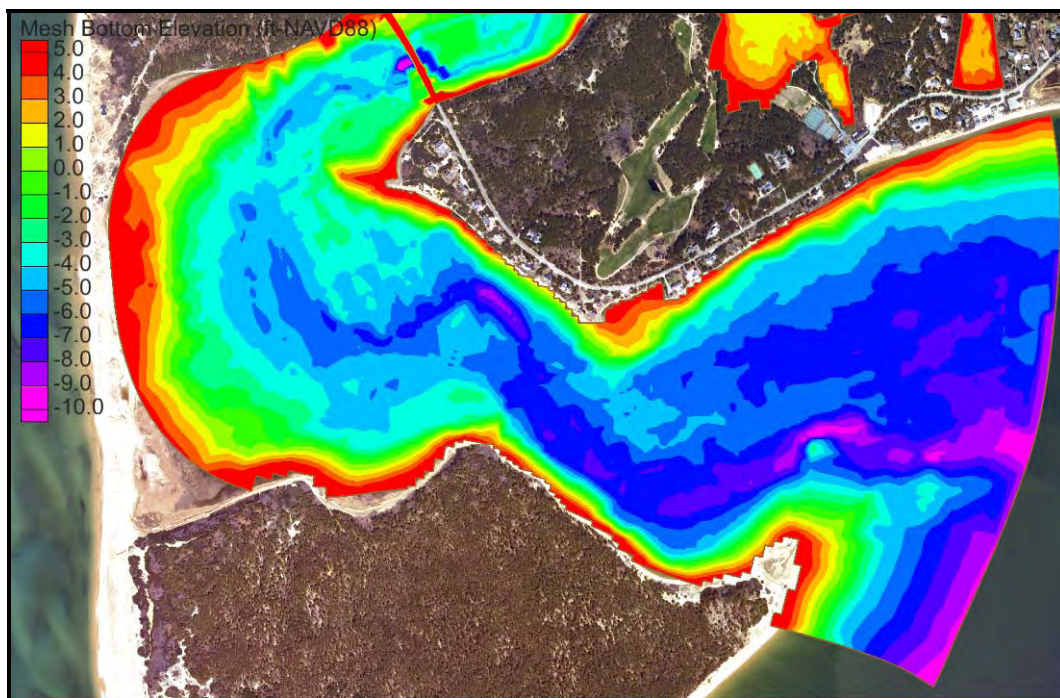


Figure 6-20. EFDC model grid showing actual bottom elevation contours from recent bathymetric survey and used in the updated Herring River model.

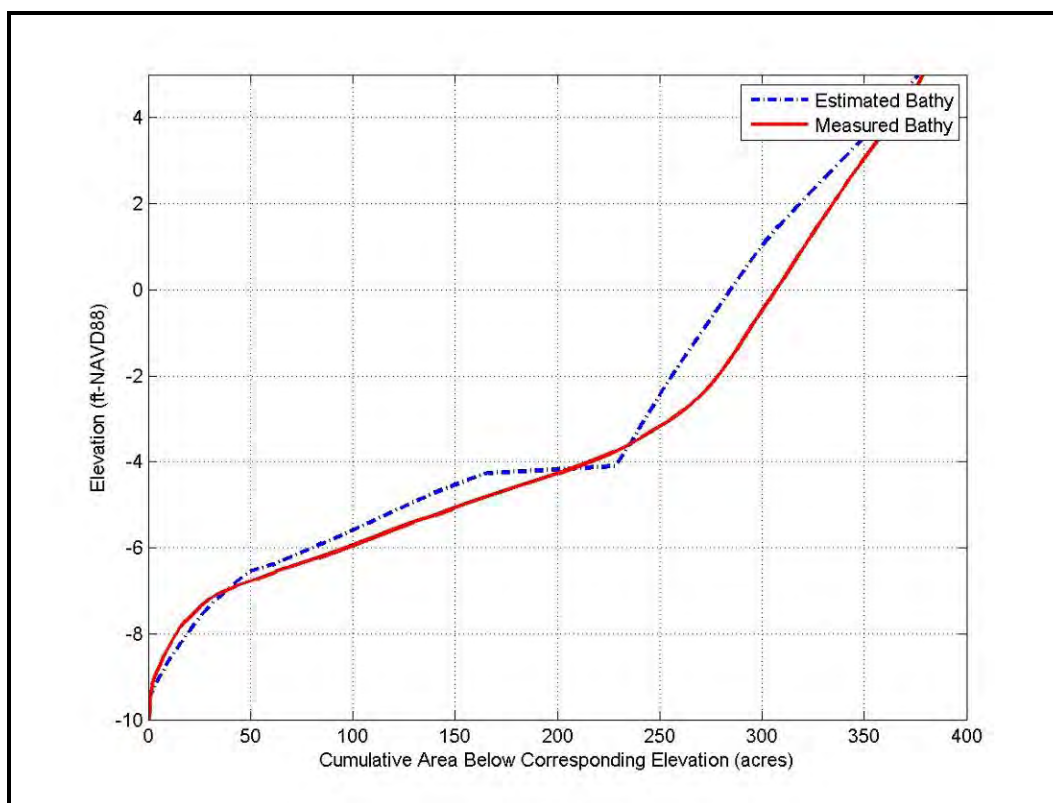


Figure 6-21. Hypsometric curves of estimated bathymetry and the measured bathymetry of the Gut.

6.5.2 Model Recalibration

In order to assess the potential impacts of the Gut bathymetry on the model calibration and verification, a portion of the Herring River model calibration was re-simulated with the updated model bathymetry. Representative time series plots showing a comparison of the observed water surface elevation and salinity data used in calibration of the Herring River hydrodynamic model are presented in Figures 6-22, 6-23, and 6-24. In general the inclusion of the new bathymetric data into the model did not have a significant effect on the model calibration. Figures 6-22 and 6-23 show time series plots of the water surface elevation comparing observed data to the modeled results (for both the original estimated bathymetry and the new measured bathymetry) at the dogleg and upstream of High Toss Road gauge locations. In these plots, all three lines are in good agreement and similar results were observed at all gauge locations. Figure 6-24 shows a time series of the salinity at the dogleg location, and the actual observed bathymetry within the Gut has resulted in an improved salinity level prediction. Ultimately, the model does not need to be re-calibrated based on the inclusion of the actual Gut bathymetry; however, the Gut bathymetry will be used in all subsequent model simulations and alternative evaluations to improve accuracy.

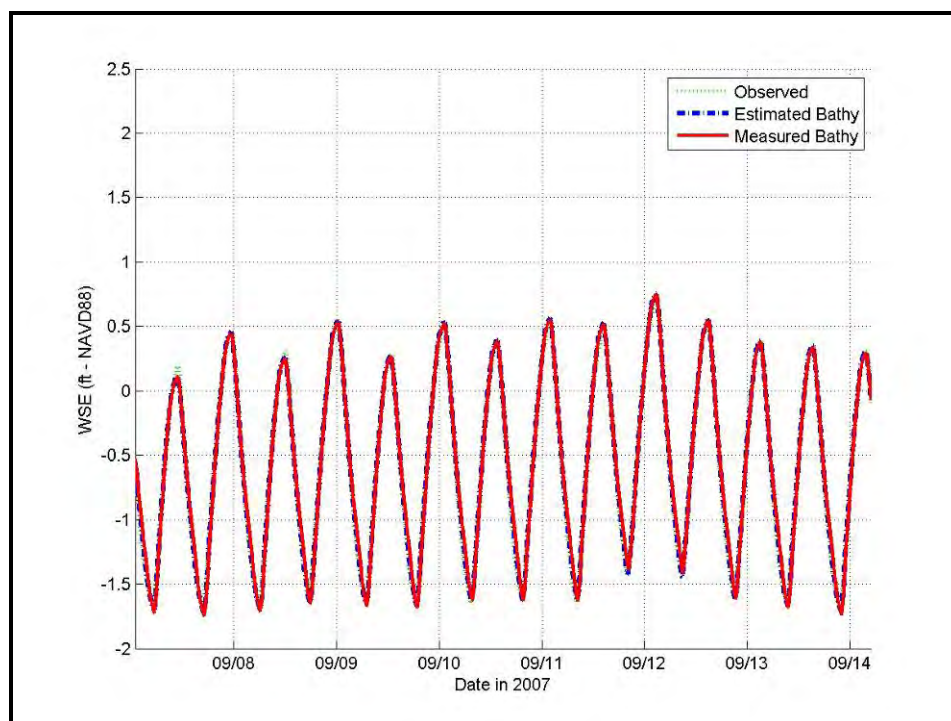


Figure 6-22. Time series showing observed WSE and modeled WSE with both estimated bathymetry and measured bathymetry at Dogleg.

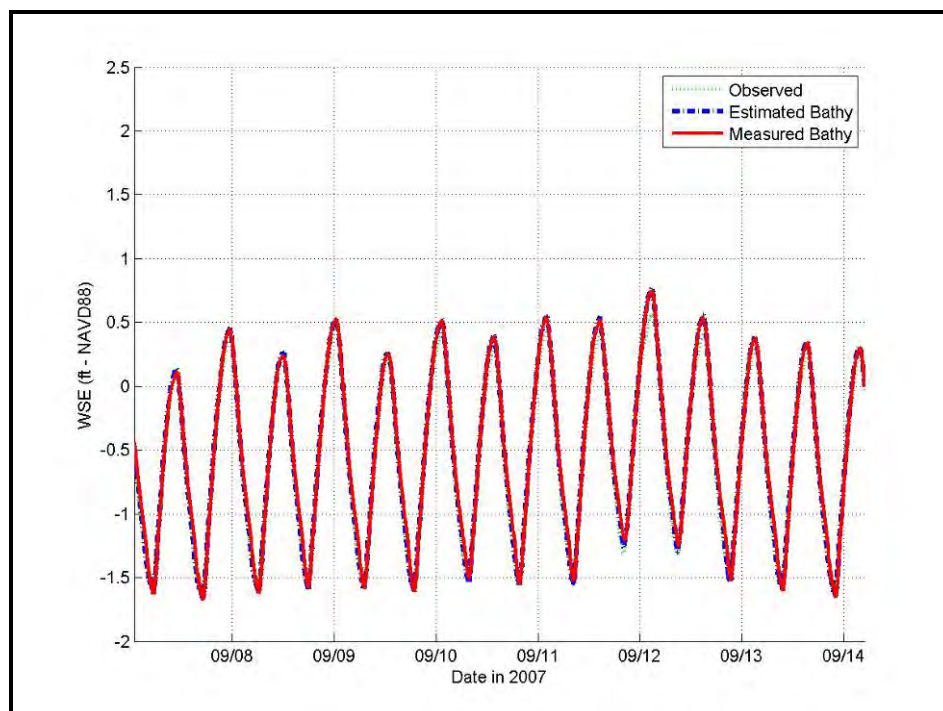


Figure 6-23. Time series showing observed WSE and modeled WSE with both estimated bathymetry and measured bathymetry upstream of High Toss Road.

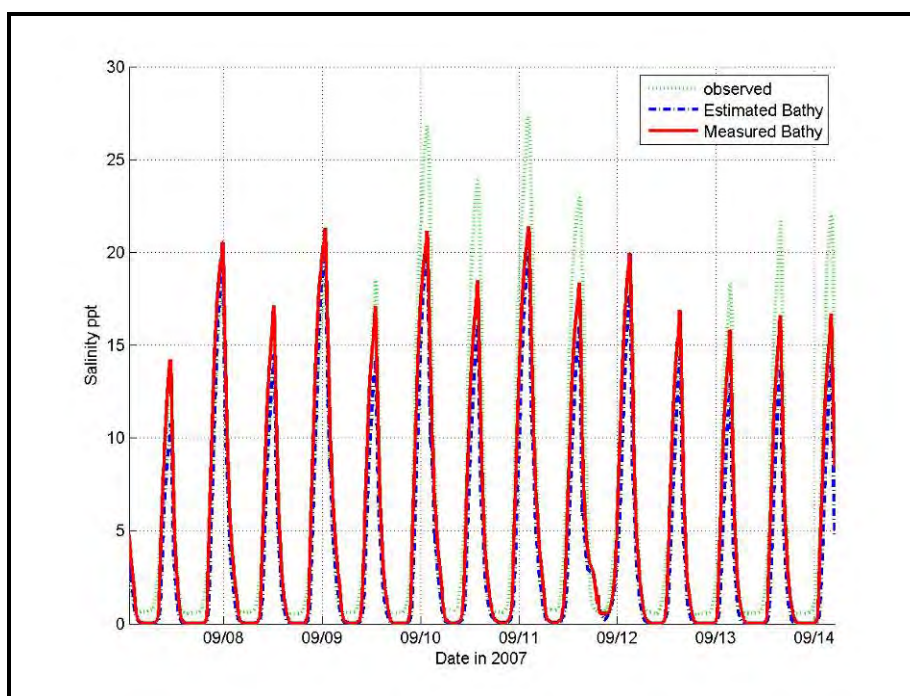


Figure 1-24. Time series showing observed salinity and modeled salinity for estimated bathymetry and measured bathymetry.

6.5.3 The Gut Bathymetry Impact on Alternatives

Currently, the combination of sluice and flapper gates in the Chequessett Neck Road Dike restricts the water levels upstream of the dike. With this restriction potentially removed, tides in the river may be limited by the bathymetry in the Gut. Thus, the actual measured Gut bathymetry may have a more significant effect on model results for alternative simulations with the dike restrictions removed since a greater volumetric flux will be exchanged.

As such, the alternative case with a 30 meter wide culvert opening in the Chequessett Neck Road dike was re-simulated with the updated model bathymetry to identify the potential influence of the actual Gut bathymetry on the restoration alternative simulations. Figure 6-25 shows a time series plot of the water surface elevation from the 30 meter alternative with the initial estimated bathymetry and with the updated bathymetry for a location in the lower Herring River sub-basin (approximately halfway between Chequessett Neck Road and High Toss Road). Assessment of the effect on water levels at this location is representative of the whole system. In general, updating the bathymetry tends to cause a small increase in the low tide elevations and a nearly negligible impact on the high tide elevations. This effect becomes less as the mean tide level increases farther upstream in the system (as shown in Figure 6-26 for Mill Creek). Therefore, the actual bathymetry in the Gut, combined with the proposed restoration alternatives, is slightly more efficient at transferring water. As mentioned, the actual Gut bathymetry will be used for all future scenario simulations to improve overall model accuracy.

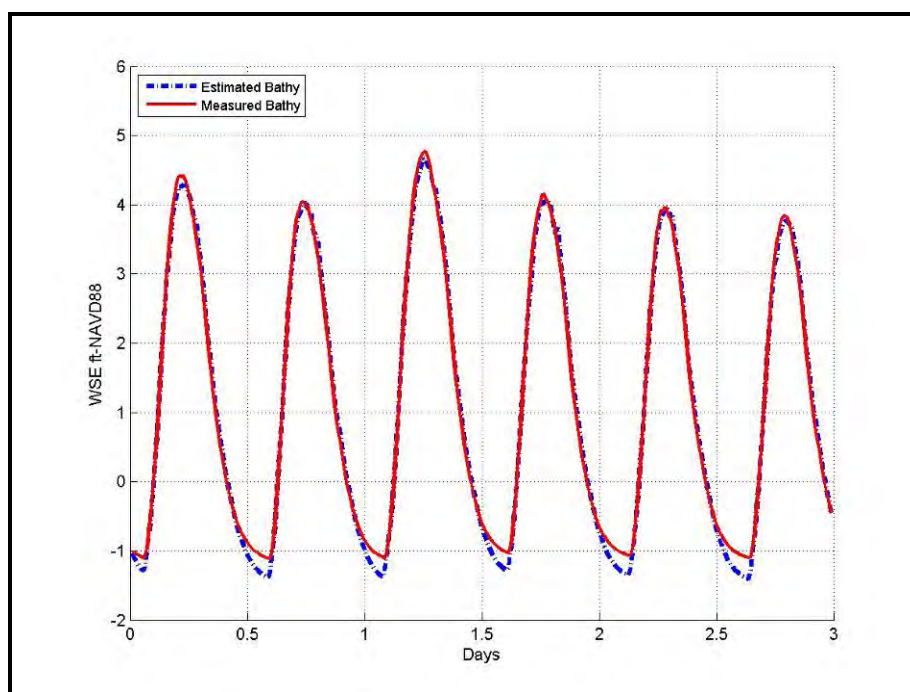


Figure 6-25. Time series of modeled WSE using estimated bathymetry and measured bathymetry in the Lower Herring River sub-basin.

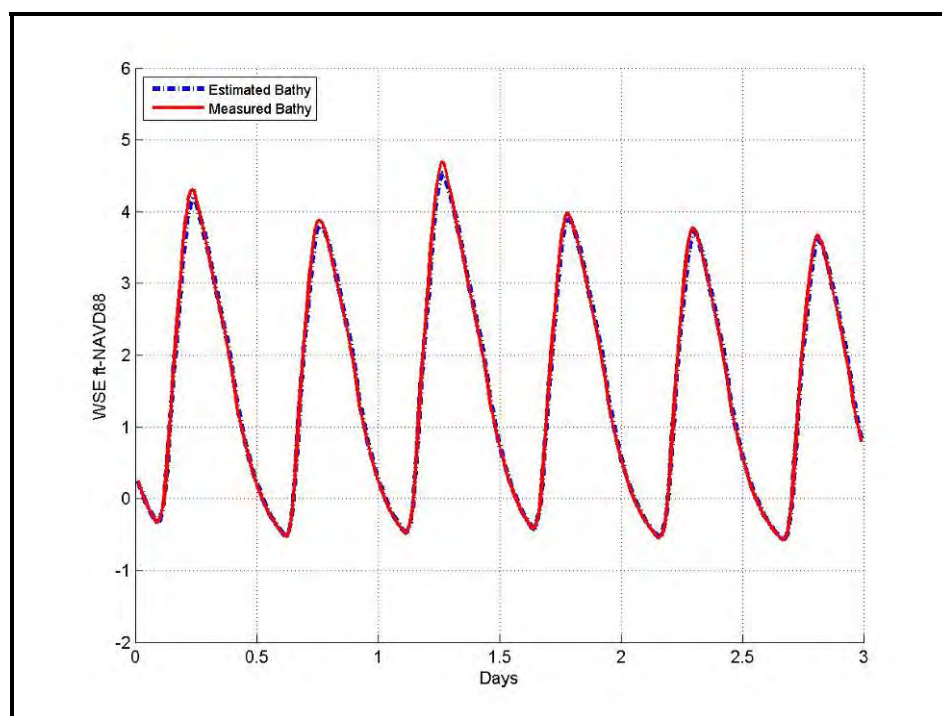


Figure 6-26. Time series of modeled WSE using estimated bathymetry and measured bathymetry in the Mill Creek sub-basin.

7.0 DISCUSSION AND NEXT STEPS

Based on the Woods Hole Group review of these initial alternative modeling simulations, as well continued discussions with the HRTC, key results of the alternative model simulations include:

- The tidal attenuation between Wellfleet Harbor and Herring River is reduced the most significant amount when the dike opening is increased from existing conditions to 30 meters. Within the Lower Herring River and Mill Creek sub-basins, a 30 meter opening at the dike maximizes tidal exchange. Larger opening at the dike continue to increase the MHW level and tidal exchange; however, these increase result in less significant gains with diminishing returns. MHW, MLW, and tidal range for the 30 meter opening are reasonably close to those produced by the 100 m opening. Therefore, from a purely water surface elevation and tidal exchange perspective, a 30 meter opening would reasonably maximize restoration.
- Salinity levels within the system increase substantially up to a dike opening of 50 meters. Openings greater than 50 meters have limited gains in salinity concentration levels and indicate diminishing returns. Therefore, although a 30 meter opening effectively maximizes water exchange, a larger opening is required to maximize salinity throughout the system.
- Dike opening widths greater than 10-20 meters cause overtopping of High Toss Road, which leads to reduced tidal range and increased water levels upstream of High Toss Road. When culvert widths at the dike become larger than 10 meters, water levels in the Lower Herring River sub-basin begin to overtop High Toss road on some high tides. Widths above 20 meters at the dike result in a significant volume of water flowing over High Toss Road on each flood tide. Figure 7-1 shows the color contours of the water surface elevation and velocity vectors at high tide during the simulation with a 60 meter opening at the dike. Water continues to flow upstream over the road even as the tide begins to fall causing a strongly asymmetrical tidal signal with a larger volume of water flooding over the road than ebbing over the road. This increases the low tide level and mean water level for all the basins upstream of High Toss Road. Figure 7-2 shows the discharge through the High Toss Road culvert for selected alternative simulations. This illustrates the strong asymmetry in the tide that occurs due to the overtopping of High Toss Road as there is a much larger volume of water ebbing than flooding through the culvert for alternative culvert widths greater than 10 meters. As such, the High Toss Road area is much more complicated than a single culvert constriction once the dike is opened a moderate amount. Future alternatives need to assess not only changes to the culvert, but also to the road itself.

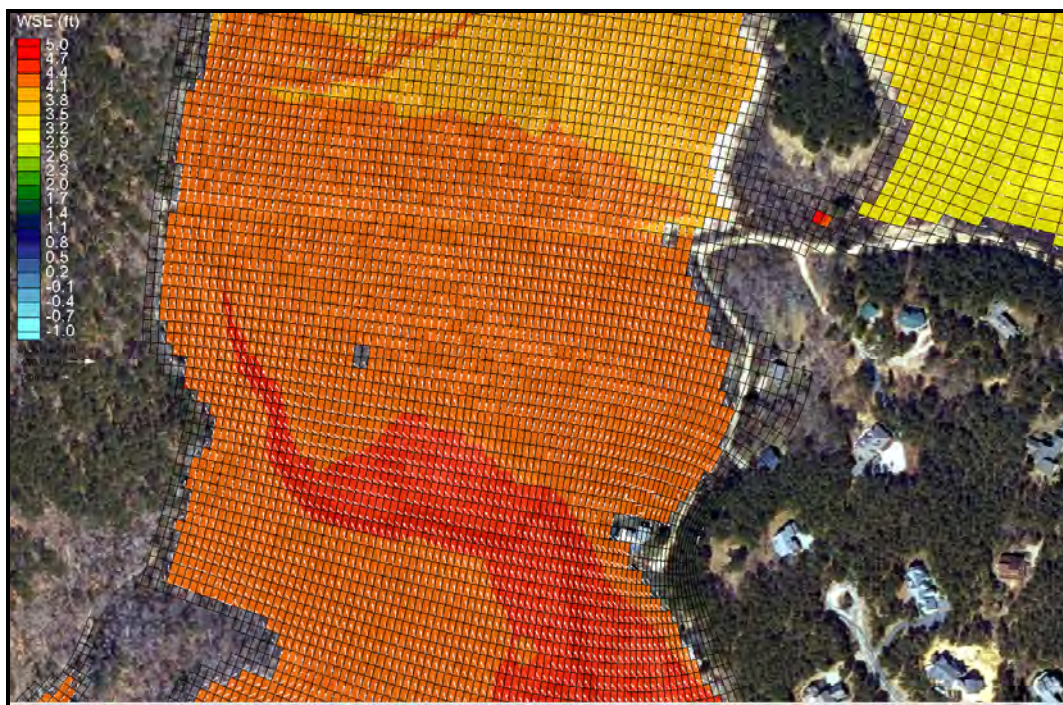


Figure 7-1. Water surface elevation and current velocity vectors at high tide for 60 m opening.

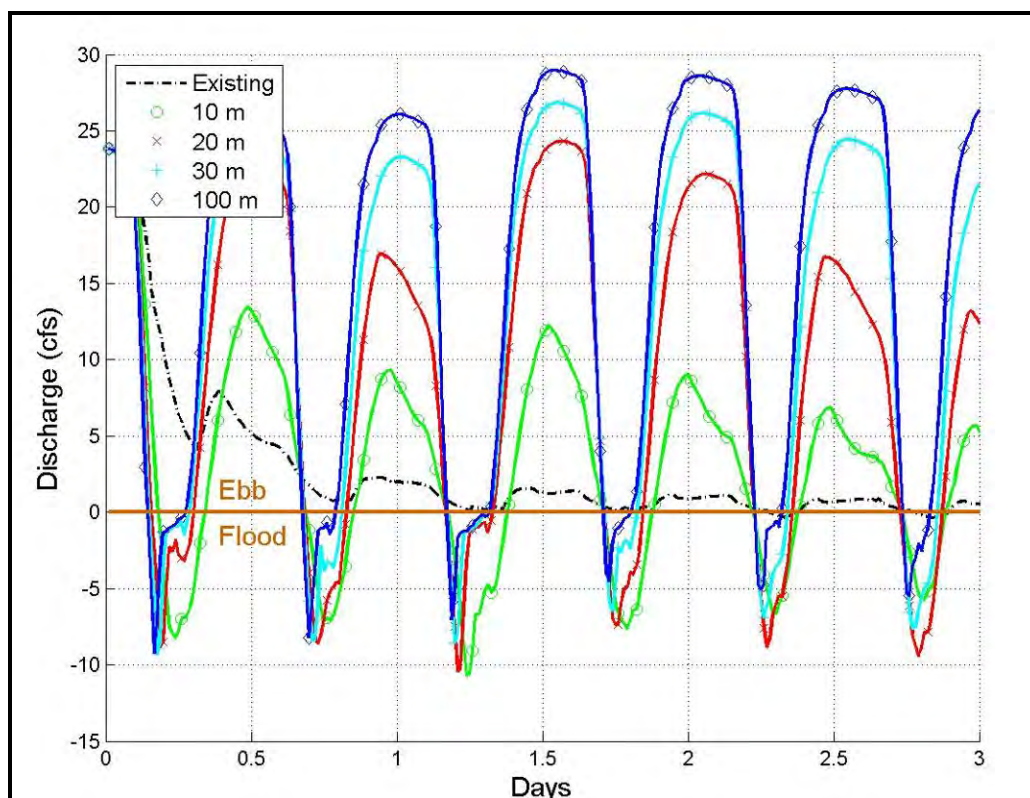


Figure 7-2. Discharge through the High Toss Road Culvert.

- The hypsometry of the Mid Herring River sub-basin contributes to substantial reduction in tidal range within this sub-basin. The results of the fully open case indicate that the natural geometry of the Herring River estuary causes a strong attenuation of the tidal signal. Even without the presence of the elevated High Toss Road causeway, an unrestricted tide propagating through the Herring River Estuary is significantly damped through the system. The natural “neck” across where High Toss Road was built poses a significant restriction to tidal flow. Just above this narrow section of the river the marsh plain expands extensively into the mid Herring River sub-basin, and also into the Upper Pole Dike creek sub-basin to the east and the Duck harbor sub-basin to the west. The effect in the upper sub-basins (i.e. Upper Herring River and Bound Brook) of the fully open simulation is a lower mean water level when compared to the 100 meter opening due to absence of the trapping effect of High Toss Road. The absence of additional restrictions at the railroad beds and Old County Road allow the tidal signal to propagate further into the Bound Brook and Upper Herring River sub-basins than it does during the other alternatives. Therefore, in a fully open scenario, tidal signals do appear in even the upper reaches of the estuary. This indicates that the fully open scenario, although not directly shown in the water surface elevation results, is the most effective at completely restoring tidal exchange throughout the entire system.
- Removal of the flood tidal shoal, either by natural process or by dredging, has minimal impact on the tidal attenuation throughout the system. However, changes to the shoal and the surrounding region do have a significant impact on tidal velocities and circulation characteristics near the Chequessett Neck Road area.
- Lowering the culvert invert at the dike did not significantly influence the results compared to the existing culvert invert. The surrounding bathymetry and river bed elevation may have a greater influence on the low water elevations than the culvert invert. Dredging and potential island building within the Herring River system should be considered as a potential restoration strategy for certain areas of the system.

Based on the results of these initial alternative simulations and the key results discussed above, Woods Hole Group recommends the following next steps:

- 1) ***Evaluation of the High Toss Road constriction*** - This evaluation would include a range of scenarios for both the High Toss Road culvert and roadway. Based on the alternatives simulated to date, the High Toss Road culvert and specifically the causeway/road, have a significant influence on the tidal processes in the mid and upper portions of the estuary. The complexity involved with the overtopping of the road, including the non-uniform overtopping on ebb versus flood, indicate the High Toss Road crossing is an important component of the overall restoration process. Due to the significant impact of not only the culvert, but also the road itself, the Town of Wellfleet should determine the potential options for High Toss Road (e.g., keep the road in place, replace with a walking bridge, remove completely, etc.) such that the High Toss Road alternatives can be fully developed.

- 2) ***Extension of the hydrodynamic model to simulate sediment transport*** – Based on the need to be able to predict potential changes to the Herring River system for potential restoration alternatives, as well as address potential concerns related to changes to the system, the HRTC should consider extending the hydrodynamic model to include sediment transport. Since the hydrodynamic modeling approach was designed to be flexible, sediment transport modeling is a relatively straightforward addition to the modeling effort. The sediment transport modeling will provide sediment flux values, prediction of areas of erosion, scour, accretion, and shoaling, and quantify rates of erosion or accretion throughout the system. These results can be used to identify the physical changes that will occur to the system for different alternatives, and can be used to provide answers to potential environmental impacts. For example, the sediment transport model extension would provide information on how ebb and flood tidal shoals may adjust, predict potential impacts on shellfish beds, and identify areas that may undergo significant scour that would need to be addressed during the engineering phase of the restoration project.

8.0 REFERENCES

- Akcelik, V., B. Jaramaz, and O. Ghattas. 2001. Nearly Orthogonal Two-Dimensional Grid Generation with Aspect Ratio Control. *J. Comp. Phys.* 171, 805-821.
- Beskenis, J.L. and R.M. Nuzzo. 1984. Herring River Survey, August 15-17, 1984. Massachusetts Division of Water Pollution Control (MDWPC), #13,700-38-50-7-84-C.R. 35 p.
- Brunner, G.W. 2002. HEC-RAS River Analysis System Hydraulic Reference Manual. Hydraulic Engineering Center, Davis CA.
- Chow, V. T. 1959. Open-Channel Hydraulics. The McGraw-Hill Book Company, Inc.
- Dougherty, Amy J. 2004. Sedimentation Concerns Associated with the Proposed Restoration of the Herring River Marsh, Wellfleet, MA. Report to Cape Cod National Seashore, Town of Wellfleet and Association of Women Geoscientists. 63 p.
- Fitterman, D.V., G.A. Brooks, and S.L. Snyder. 1989. Geophysical Investigation of Depth to Saltwater near the Herring River (Cape Cod National Seashore), Wellfleet, Massachusetts, U.S. Geological Survey Open-File Report 89-677.
- Fitterman, D.V., K.F. Denehy. 1991. Verification of Geophysically Determined Depths to Saltwater near the Herring River (Cape Cod National Seashore), Wellfleet, Massachusetts, U.S. Geological Survey Open-File Report 91-321.
- Hamrick, J. M. 1992. A Three-Dimensional Environmental Fluid Dynamics Computer Code: Theoretical and Computational Aspects. The College of William and Mary, Virginia Institute of Marine Science. Special Report 317, 63 pp.
- Hamrick, J. M. 1994. Linking Hydrodynamic and Diogeochemical Transport Models for Estuarine and Coastal Waters. *Estuarine and Coastal Modeling, Proceedings of the 3rd International Conference*, M. L. Spaulding *et al*, Eds., American Society of Civil Engineers, New York, 591-608.
- Hamrick, J.M. 1996. A User's Manual for the Environmental fluid Dynamics Computer Code. The College of William and Mary, Virginia Institute of Marine Science. Special Report 331, 185 pp.
- Leendertse, J. J. 1987. Aspects of SIMSYS2D A System for Two Dimensional Flow Computations. Prepared for USGS.
- Martin, L. 2004. Salt Marsh Restoration at Herring River: An Assessment of Potential Salt Water Intrusion in Areas Adjacent to Herring River and Mill Creek, Cape Cod National Seashore. Technical Report NPS/NRWRD/NRTR-2004/319.
- Martin, L. 2007. Identification of Private Wells Adjacent to the Herring River Flood Plain that Could be Affected by Restoration of Tidal Flow, Memorandum to John Portnoy September 4, 2007.
- Masterson, J.P. 2004. Simulated Interaction Between Freshwater and Saltwater and Effects of Ground-water Pumping and Sea-level change, Lower Cape Cod Aquifer System, Massachusetts: U.S. Geological Survey Scientific Investigations Report 2004-5014, 72 p.

- Masterson, J.P. and J.W. Portnoy. J.W. 2005. Potential Changes in Ground-water Flow and Their Effects on the Ecology and Water Resources of the Cape Cod National Seashore, Massachusetts, U.S. Geological Survey General Information Product 13 June 2005.
- Nuttle W.K. 1990. Extreme Values of Discharge for Mill Creek and Options to Control Flooding from the Herring River, Report to National Park Service Cape Cod National Seashore.
- Portnoy, J.W. 1984. Salt Marsh Diling and Nuisance Mosquito Production on Cape Cod, Massachusetts. *J. Amer. Mosq. Cont. Assoc.* 44:560-564.
- Portnoy, J.W., C.T. Roman, and M.A. Soukup. 1986. Hydrologic and Chemical Impacts of Diking and Drainage of a Small Estuary (Cape Cod National Seashore): Effects on Wildlife and Fisheries, Manuscript Submitted for Publication in the Proceedings of "A Symposium on Waterfowl and Wetlands Management in the Coastal Zone of the Atlantic Flyway." Wilmington, DE. September 16-19, 1986, 254-265.
- Portnoy, J.W. and A.E. Giblin. 1997. Effects of Historic Tidal Restrictions on Salt Marsh Sediment Chemistry, *Biogeochemistry*, 36:275-303.
- Portnoy, J.W. 1999. Salt Marsh Diking and Restoration: Biogeochemical Implications of Altered Wetland Hydrology, *Environmental Management*, 24:111-120.
- Portnoy, J.W. and J.R. Allen. 2006. Effects of Tidal Restrictions and Potential Benefits of Tidal Restoration on Fecal Coliform and Shellfish-water Quality, *J. Shellfish Research*, 25:609-617.
- Roman, C. T. 1987. An Evaluation of Alternatives for Estuarine Restoration Management: The Herring River Ecosystem (Cape Cod National Seashore). Technical Report. National Park Service Cooperative Research Unit, Rutgers University, New Brunswick, New Jersey
- Roman, C.T., R.W. Garvine, and J.W. Portnoy. 1995. Hydrologic Modeling as a Predictive Basis for Ecological Restoration of Salt Marshes, *Environmental Management*, 19:559-566.
- Ryskin, G. and L. G. Leal. 1983. Orthogonal Mapping. *J. Comp. Phys.*, 50, 71-100.
- Smagorinsky, J. 1963. General circulation experiments with the primitive equations, I. The basic experiment. *Mon. Weather Rev.*, 91, 99-164.
- Spaulding, M.L. and A. Grilli. 2001. Hydrodynamic and Salinity Modeling for Estuarine Habitat Restoration at Herring River, Wellfleet, Massachusetts. The University of Rhode Island, submitted to the National Park Service.
- Spaulding, M.L. and A. Grilli. 2005. Simulations of Wide Sluice Gate Restoration Option for Herring River. Report to NPS. 12 p.
- U.S. Army Corps of Engineers. 1988. Tidal Floo Profiles new England Coastline, September 1988.
- U.S. Environmental Protection Agency. 1990. Technical Guidance Manual for Performing Waste Load Allocations, Book III Estuaries, Part 2, Application of Estuarine Waste Load Allocation Models. EPA 823-R-92-003.

Woods Hole Group 2006, Review of “Hydrodynamic and salinity modeling for estuarine habitat restoration at Herring River, Wellfleet, Massachusetts” Final Report, Prepared for Conservation Law Foundation 13 p.



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