Appendix 3-4
March 2013 Technical Memorandum BP-1 on Bournes Pond Inlet Opening Evaluations
March 12, 2013

To: Town of Falmouth, MA

Copy to: File; Project Team

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Subject: Wastewater and Nutrient Management Services
Bournes Pond Inlet Opening Evaluations, Preliminary Design, and Nitrogen Removal Benefits

1 BACKGROUND AND PURPOSE OF MEMO

This memorandum is prepared to summarize preliminary design evaluations for an enlarged inlet to Bournes Pond and a new bridge to go over that inlet. It is also developed to summarize the nitrogen removal benefits of a larger inlet as determined by water-quality modeling. GHD is assisted by Applied Coastal Research and Engineering (ACRE) who is providing hydrodynamic and water-quality modeling to determine the optimized size of the proposed new inlet and identify the nitrogen removal benefits of a larger opening; and BETA Group Inc. who is providing preliminary design information on the proposed new bridge and roadway changes.

The existing inlet to Bournes Pond and the bridge that crosses that inlet is illustrated in Figure 1-1; it can also be found in Appendix BP-1-1 on Sheet 1 of 5.

2 BACKGROUND ON THE INLET OPENING EVALUATION

Evaluations by the Massachusetts Estuaries Project (MEP) documented water quality problems in Bournes Pond in the MEP Technical Report entitled “Linked Watershed-Embayment Model to Determine Critical Nitrogen Loading Thresholds for Great/Perch Pond, Green Pond, and Bournes Pond, Falmouth, Massachusetts”, Final Report April 2005. These evaluations documented impaired water quality in Bournes Pond and identified that the pond currently exceeds its nitrogen loading threshold. The evaluations identified several ways that the nitrogen threshold could be met in the future; and the opening of the pond inlet was identified as a possible way to increase tidal flushing to the pond to help meet the threshold (their discussion related to this topic starts on page 197 of the Technical Report).

Based on the MEP evaluation, Massachusetts Department of Environmental Protection (MassDEP) developed Total Maximum Daily Loads (TMDLs) for Bournes Pond in their report titled “Great, Green, and Bournes Pond Embayment Systems, Total Maximum Daily Loads for Total Nitrogen (Report #96-TMDL-6 Control #181” dated April 2006.

Based on the MEP evaluations and the approved TMDL, the Town of Falmouth initiated a Comprehensive Wastewater Management Planning Project in 2007 to investigate cost-effective methods (among other goals) to meet the nitrogen TMDL for Bournes Pond. Evaluations as part of the CWMP project resulted in a
Technical Memorandum dated March 10, 2008 from GHD to the Town of Falmouth to evaluate the benefits of enlarging the Bournes Pond Inlet; and the Town decided to pursue inlet opening of Bournes Pond as a non-traditional method to meet the TMDL. This is documented in the Draft CWMP/DEIR document filed with the MEPA Office of Energy and Environmental Affairs (EEA) as part of EEA Project No. 14154.

This technical memorandum summarizes the preliminary design evaluations and nitrogen removal benefits for the inlet opening, and is the Town’s next step to proceed with this non-traditional wastewater and nutrient management project.

3 PROJECT TEAM AND SCOPE

The Falmouth Project Team is comprised of the following members who attended many project meetings and assisted in the decision-making that resulted in this technical memorandum:

- Ray Jack—Falmouth DPW Director
- Jerry Potamis, P.E.—Falmouth Wastewater Superintendent
- Erik Turkington—Falmouth Water Quality Management Committee (WQMC) Chairman
- Virginia Valiela—Falmouth WQMC
- Peter McConarty, P.E., PLS—Falmouth Town Engineer
- GHD
- ACRE
- BETA Group

During the first project meeting the following project scope was decided:

- Complete hydrodynamic inlet optimization
  - Additional elevation data collection
  - Updated hydrodynamic model development
  - Model runs to explore optimal sizes/positions of inlet opening
- Evaluate alternative inlet openings
- Summarize preliminary design items
- Summarize findings in a technical memorandum

The project team was later asked to evaluate the benefits of the inlet opening through water quality modeling and cost comparison to conventional wastewater management nitrogen removal.

4 TECHNICAL EVALUATIONS AND SELECTION OF PREFERRED NEW INLET OPENING

The hydrodynamic modeling by ACRE started at the beginning of the project to:

- Collect additional elevation data at the inlet;
update the hydrodynamic model; and
complete several model runs to optimize size and positions of the inlet opening.

These hydrodynamic evaluations are briefly summarized later in this memo, and summarized in detail in the attached Appendix BP-2.

The evaluations of alternative inlet opening started at the first Project Team Meeting and extended through two subsequent meetings as optimal size calculations were completed and the features of each alternative were considered. The four scenarios evaluated include:

• Scenario 1: Replace the existing bridge with a new single-span bridge.
• Scenario 2: Replace the existing bridge with a double-span bridge.
• Scenario 3: Add one or more precast concrete culverts to the east of the existing bridge to increase the opening size.
• Scenario 4: Replace the existing bridge with a group of precast concrete culverts

Each of these scenarios is described below with findings of the elimination (screening) of two of the scenarios.

4.1 Scenario 1—Single Span Bridge

Under this scenario the existing bridge would be removed and replaced with a 97 foot single span structure. This proposed bridge would likely be comprised of pre-stressed, concrete 1400 New England Bulb Tees supporting a reinforced concrete deck and bituminous pavement. The bridge would be founded on timber pile supported abutments with the westerly abutment positioned within the footprint of the existing west abutment. The bridge would provide for a 90 foot average clear channel width with no intermediate piers. Reconstruction and extension of the west jetty, construction of a new east jetty, and armoring of the channel bottom under, and 20 feet each side, of the bridge would be required (See Appendix BP-1-1, Sheet 2 of 5).

As the proposed superstructure would be approximately 3 feet deeper than that of the existing bridge, the profile of Menauhant Road would need to be raised necessitating the reconstruction of approximately 200 feet of Menauhant Road on either side of the bridge and the installation of retaining walls along the north side of the road to prevent filling of the adjacent environmental resource areas.

This Scenario was anticipated to have the highest capital costs due to the increase in roadway profile additional structures within the barrier beach system, extent of roadway reconstruction, increase in bridge length and superstructure depth, and as a result was screened out for further evaluation.

4.2 Scenario 2—Two Span Bridge

Under this scenario the existing bridge would be removed and replaced with a two span structure having two equal 50 foot spans. The proposed bridge would likely be comprised of 21” deep pre-stressed, concrete deck beams supporting a reinforced concrete deck and bituminous pavement. The bridge would be founded on timber pile supported abutments and center pier with the westerly abutment positioned within the footprint of the existing west abutment. The bridge would provide for two 45 foot average clear channels with an overall average width of 93 feet. Reconstruction and extension of the west jetty, construction of a new east jetty, and
armoring of the channel bottom under, and 20 feet each side, of the bridge would be required (See Appendix BP-1-1, Sheet 3 of 5).

The proposed superstructure would be approximately the same depth as the existing bridge thereby allowing the profile of Menauhant Road to be maintained and limiting the reconstruction of Menauhant road to approximately 100 feet either side of the bridge.

This Scenario was accepted for further evaluation.

4.3 Scenario 3—Existing Bridge with Adjacent Box Culverts

Under this scenario the existing bridge would be retained and two precast culvert box culverts installed to the east of the existing bridge and channel. Each culvert opening is likely to be 19 feet wide by 12 feet high and supporting a standard roadway section Hot Mix Asphalt or HMA top, intermediate and base courses, subbase and gravel borrow) over the top of the culvert. Due to concerns over the site’s location and exposure to ocean storm events the culverts would be anchored to a concrete base slab supported by timber piles (See Appendix BP-1-1, Sheet 4 of 5).

This Scenario would provide for a 100 foot average clear channel but would require armoring extending into the channel for the protection of the interface between the culverts and existing bridge. This would result in two distinct channel openings passing under Menauhant Road. Reconstruction and extension of the west jetty and construction of a new east jetty would also be required.

With this Scenario the profile of Menauhant Road would be maintained and thereby limiting the reconstruction of Menauhant road to approximately 100 feet either side of the bridge.

Due to the double channel opening separated by a “land mass”, shoaling was a major concern for this system. Additionally, a wider overall inlet width would be required to maintain the same tidal prism, further exacerbating shoaling and creating inlet stability concerns. This scenario was screened out for further evaluation.

4.4 Scenario 4—Multiple Box Culverts

Under this scenario the existing bridge would be removed and replaced by five (5) butted precast concrete box culverts. Each culvert opening is likely to be 19 feet wide by 12 feet high and supporting a standard roadway section (HMA top, intermediate and base courses, sub-base and gravel borrow) over the top of the culverts. Due to concerns over the site’s location and exposure to ocean storm events the culverts would be anchored to a concrete base slab supported by timber piles (See Appendix BP-1-1, Sheet 5 of 5).

This scenario would provide for an approximate 100 foot average clear channel with the reconstruction and extension of the west jetty and construction of a new east jetty. With this Scenario the profile of Menauhant Road would be maintained and thereby limiting the reconstruction of Menauhant road to approximately 100 feet either side of the bridge.

This Scenario was accepted for further evaluation.
4.5 Detailed Evaluation of Scenarios 2 and 4

Additional evaluation of Scenarios 2 and 4 proceeded with the following items noted for both:

- The elevation of the existing bridge’s low chord will be maintained which will maintain the existing 3’ 3” freeboard.
- The roadway cross section (illustrated in Sheets 3 and 5 of Appendix BP-1-1) would be increased from 32-feet for 34-feet to meet minimum AASHTO design criteria.
- As a result, the roadway section will consist of the following:
  - Two 11-foot wide travel lanes
  - Two 2-foot offsets
  - One 5-foot sidewalk on the south side
  - MassDOT approval and crash-tested metal bridge railing on both sides of the bridge

Costs were developed as summarized below.

Estimated quantities for the major components of each scenario were determined and unit prices applied to arrive at an estimate of probable construction cost. Due to the conceptual development of the scenarios a 25% contingency was included to account for unforeseen field conditions, potential further design developments, and the general nature of the work.

The following table provides the estimate of probable construction costs including the estimated costs for the permitting, design and construction engineering/oversight:

Table 4-1 Capital Cost Comparison for Scenarios 2 and 4

<table>
<thead>
<tr>
<th>Estimate of Probable Construction Costs</th>
<th>Scenario 2 (Two-Span Bridge) ($)</th>
<th>Scenario 4 (Multiple Culverts) ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge and Road Work</td>
<td>2,500,000</td>
<td>2,600,000</td>
</tr>
<tr>
<td>Jetty Modifications and Armoring</td>
<td>800,000</td>
<td>720,000</td>
</tr>
<tr>
<td>Dredging and Beach Nourishment</td>
<td>75,000</td>
<td>80,000</td>
</tr>
<tr>
<td>Permitting and Special Study Allowance</td>
<td>300,000</td>
<td>300,000</td>
</tr>
<tr>
<td>Design</td>
<td>400,000</td>
<td>400,000</td>
</tr>
<tr>
<td>Fiscal, Legal, and Engineering During Construction</td>
<td>520,000</td>
<td>520,000</td>
</tr>
<tr>
<td>Contingency (25%)</td>
<td>920,000</td>
<td>930,000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>5,520,000</strong></td>
<td><strong>5,550,000</strong></td>
</tr>
</tbody>
</table>
In addition, each scenario was evaluated for operation and maintenance costs.

### Table 4-2  O&M Cost Comparison for Scenarios 2 and 4

<table>
<thead>
<tr>
<th>Maintenance Dredging/Beach Nourishment</th>
<th>Scenario 2 (Two-Span Bridge) ($)</th>
<th>Scenario 4 (Multiple Culverts) ($)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bridge/Culvert Maintenance^1</td>
<td>30,000</td>
<td>25,000</td>
</tr>
<tr>
<td>Jetty Maintenance^2</td>
<td>5,000</td>
<td>6,000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>41,000</strong></td>
<td><strong>74,000</strong></td>
</tr>
</tbody>
</table>

Notes:
1. Bridge superstructure replacement annualized over 45 years, and roadway repaving annualized over 15 years.
2. Periodic jetty maintenance has been annualized.
3. Assumes 1,000 cubic yards of dredging every two years, where additional culvert clean-out will be required after major storms (once every 5 years).

After review of the costs and additional considerations of Scenarios 2 and 4, the Project Team selected Scenario 2 as the Preferred Alternative. Two major concerns for Scenario 4 include difficult long-term maintenance of the channel within five butted precast concrete box outlets, and concern about protection of multiple vertical walls within the channel.

On December 6th, the four scenarios and the findings of the preliminary design evaluations were presented to the Town’s WQMC who is overseeing the Town’s Wastewater and Nutrient Management projects. The WQMC voted unanimously to proceed with Scenario 2.

### 4.6 Construction Sequence and Design Summary

In order to construct the proposed improvements, Menauhant Road would be closed to all vehicular and pedestrian traffic. Vehicular traffic would be detoured via Central Street, Route 28 and Acapesket Road. It is anticipated that the inlet opening would require approximately 18-24 months to construct and would require the following general sequence of construction:

1. Close Menauhant Road to traffic and institute detour
2. Install temporary water line / utility bridge
3. Demolish existing bridge superstructure
4. Install steel sheeting cofferdam and dewater east side of project site
5. Demolish existing abutment / abandon existing piles
6. Excavate and drive timber piles for support of proposed easterly substructures
7. Form and place concrete for the proposed easterly substructure elements
8. Dredge and construct east jetty and channel; place beach-compatible material as beach/dune nourishment on Menauhant Beach east of inlet
9. Place scour protect on the bottom of the new side east channel
10. Remove ends of cofferdam to open the new east side of the channel and reinstall to close off west side of channel
11. Dewater west side of project site
12. Demolish existing west abutment / abandon existing piles
13. Excavate and drive timber piles for proposed westerly substructures
14. Form and place concrete for the proposed westerly substructure elements
15. Dredge and reconstruct west jetty and channel; place beach-compatible material as beach/dune
   nourishment on Menauhant Beach west of inlet
16. Remove cofferdam
17. Install proposed superstructure and/or precast culvert units
18. Form and place concrete deck slab, abutment backwalls and approach slabs (as appropriate)
19. Form and place sidewalk, safety curb, highway guardrail transitions and install bridge railing
20. Install permanent water line and remove temporary water line / utility bridge
21. Reconstruct approach roadway
22. Pave roadway and bridge, stripe, and open to traffic
23. Complete site restoration and clean-up

The bridge design basis would include:

- Design in accordance with the 2009 MassDOT Working Draft LRFD Bridge Manual and AASHTO
  LRFD Specifications, 6th edition for HL-93 Live load
- Project to be bid as a local project subject to review by MassDOT’s Bridge Engineer as required by
  M.G.L. Chapter 85, §35
- Two-span structure consisting of two equal 50 foot spans providing an average channel width of
  approximately 93 feet
  - Superstructure: precast concrete deck beams with a concrete deck
  - Substructure: timber pile supported cast-in-place abutments and center pier
- Existing bridge low chord elevation will be maintained
- Roadway section increased to 34 feet consisting of: two 11-foot travel lanes; two 2-foot offsets; one
  5-foot sidewalk on the south side; and metal bridge railings

4.7 Environmental Permitting Overview and Recommended Strategy

Widening of Bourne’s Pond Inlet is similar to many estuarine and salt marsh restoration projects that have
been permitted in Massachusetts. However, the primary purpose of this project is slightly different, as it
focuses specifically on water quality improvements needed to address the federal Clean Water Act, at the
same time it is restoring marine and benthic habitats. The primary water quality improvement purpose of the
project potentially can create concerns with the environmental regulatory agencies, as many of the
regulatory agencies involved do not routinely consider the implications of the Clean Water Act and the
related TMDL process as part of their decision-making process. Therefore, it will be critical that discussions
with regulatory agencies include a background component to ensure that the environmental concerns related
to the barrier beach system are considered relative to the overall ecological improvements within the estuary,
as well as the Town’s requirements to meet the TMDL for Bourne’s Pond. Following sections of the technical
memorandum provide some of that background.
These discussions with the regulatory community have been initiated. The Executive Office of Energy and Environmental Affairs (EEA) has established a Nutrient Management Technical Workgroup to (among other goals) facilitate the environmental review process on non-traditional nutrient management projects such as the Bournes Pond Inlet Opening project. Nathan Weeks of GHD and Alison Leschen of WBNERR are members of this group along with senior members of MassDEP, Cape Cod Commission, Division of Marine Fisheries, Coastal Zone Management, MEPA, and EEA. This group has been discussing this project and the needed permitting components to gain permit approval.

A successful pre-application meeting (or series of meetings) with key regulatory staff will be needed.

To provide support for inlet widening at Bournes Pond, initial submissions to MEPA and the U.S. Army Corps of Engineers should include the following:

- Engineering analysis of inlet optimization and preliminary design of the proposed inlet opening
- MEP analysis of water quality improvements associated with opening inlet
- Alternatives analysis of various design options to meet optimized cross-section
- Conceptual design of alternatives
- MEP report for Great, Green, and Bournes Ponds
- TMDL for Great, Green, and Bournes Ponds

Following official submission of an Expanded ENF to MEPA, it remains unclear what additional studies, if any, will be required through the state permitting process. Typically, a project of this scale would require a mandatory Environmental Impact Report (EIR); however, this stipulation often has been waived for restoration projects. It is likely that additional work will be required to ensure environmental concerns are addressed including analysis of potential increased flooding due to widened inlet.

After the MEPA process (and Development of Regional Impact process with the Cape Cod Commission, if necessary) is concluded, the following permit applications will be required:

- Notice of Intent (Falmouth Conservation Commission and MassDEP)
- 401 Water Quality Certification (MassDEP)
- Chapter 91 License (MassDEP)
- 404 Permit (U.S. Army Corps of Engineers)
- Coastal Zone Consistency (Massachusetts CZM)
- MESA Permit (MA Natural Heritage and Endangered Species Program)

Due to the unique nature of the Bournes Pond Inlet widening, it is not possible to determine the precise duration of the environmental permitting process. It is anticipated that the environmental permitting effort will likely require approximately three (3) years to complete.
5 ADDITIONAL DETAILED INFORMATION ON THE EXISTING BRIDGE, INLET, AND COASTAL PROCESSES IN THIS AREA

5.1. Existing Inlet Channel and Bridge

The existing channel opening of Bournes Pond Inlet is trapezoidal in shape with a bottom width of 30 feet and armored sides sloping back at a 1:1.5 slope. The resulting opening is spanned by a 50 foot long single span bridge carrying Menauhant Road over the inlet channel. The bridge was constructed in 1984 and consists of 21” deep precast concrete deck beams supporting a 24 foot wide roadway; 12” safety curb on the north side; and a 4’-4” wide sidewalk on the south side. A 1’-4” wide concrete parapet with aluminum handrails extend along both sides of the bridge. The bridge is supported by cast-in-place concrete abutments founded on 12” diameter timber piles. The bridge also provides support for a 12” diameter insulated waterline along the south fascia of the bridge. The existing bridge provides approximately 3’-3” of freeboard at mean high tide (See Appendix BP-1-1, Sheet 1 of 5).

The bridge was inspected by the MassDOT Highway Division on June 7, 2012 and was found to be in fair condition with a Deck, Superstructure and Substructure Rating of 5. The bridge is owned and maintained by the Town of Falmouth.

5.2. Previous Inlet Work and Historical Coastal Processes

Understanding the natural and anthropogenic processes that have led to the existing conditions of the Bournes Pond Inlet system is critical for developing a successful design to improve the inlet system, without causing adverse impacts. Perhaps the single most important consideration for tidal inlet design is to ensure that an inlet is ‘stable’, where the destructive forces attempting to close the inlet (e.g. the sediment movement along the fronting barrier beach system) are balanced by the restorative forces attempting to scour a deeper inlet channel (e.g. the inlet current velocities). Since inlet stability is partially governed by longshore coastal sediment transport, understanding the regional long-term shoreline change and littoral movement of sand is critical for evaluating stability of the Bournes Pond Inlet. In general, the observed longshore transport rates along the south coast of Falmouth are relatively low, primarily as a result of the quiescent wave environment of Nantucket and Vineyard Sounds. Although the amount of sand moving along the coast is small, the tidal prism through Bournes Pond Inlet also is relatively small, due to a tide-range of only about 1.5 feet. Since the construction of the existing jetty system in 1985 at the entrance, the inlet has generally reached equilibrium, where the tidal velocities through the main channels are sufficient to prevent significant shoaling. Recent annual dredging of these inlets has only been on the order of 1,000 cubic yards per year (personal communication with Wayne Jaedtke, Barnstable County Dredge, 2004).

In addition, it appears that the south coast of Falmouth (between Falmouth Harbor and the west entrance to Waquoit Bay) has generally equilibrated to changes in local coastal sediment transport caused by the construction of shoreline armoring. Extensive armoring of the Falmouth shoreline began in the late 1800s and early 1900s with construction of the railroad to Woods Hole, the old stone dock, the Falmouth Harbor jetties, and the Waquoit Bay east jetties. This shoreline armoring continued through the mid-1900s with the construction of stone groin fields, which often replaced existing wooden structures. Along the shoreline to the west of Bournes Pond entrance, these wooden and stone structures were constructed to protect Menauhant Road and waterfront dwellings. In 1985, Bournes Pond Inlet was relocated from the western portion of the beach to a location in the middle of the beach, as recommended by the Final Environmental Impact Report.
(Weston & Sampson, 1981). This inlet relocation project was performed because the historical inlet was unstable and infilled frequently, leading to water quality problems within Bournes Pond. However, at the time, data did not exist to optimize the inlet system for water quality improvements.

For the tidal inlet at Bournes Pond, the influence of shoreline change and the related longshore sediment transport rates directly influence the stability of the existing inlet system. One portion of the Falmouth south coast that has not reached equilibrium is immediately to the west of the existing Bournes Pond Inlet system, in the vicinity of the historical pre-1985 inlet. The “hot spot” erosion in the vicinity of Bournes Pond likely represents adjustment of the shoreline following placement of the relatively recent jetties, as seen in Figure 4-1. The previous inlet was located where the remnant road bridge still exists to the south of Menauhant Road. Repositioning of the jetties caused a shift in the beach form, as sediments accreted in the vicinity of the newly constructed jetties. The area of the historical ebb shoal (seaward of the previous inlet) shows substantial erosion since 1938, with annual recession rates in excess of 3 feet per year (Figure 4-1).

With the exception of the highly erosional area in the vicinity of the pre-1985 inlet, the long-term (1938 to 2004) shoreline change shows a relatively stable shoreline compared to many other areas of the Falmouth south coast. Although this area represents a low-lying barrier beach system, the overall stability indicates that the beach receives longshore sediments that are transported by wave action and that the area that is highly erosional likely is related solely to repositioning of the inlet within the system.
Current management practices at Bournes Pond Inlet consists of periodic dredging to maintain the channel in between the jetties, as well as through the shallow ebb shoal offshore of the inlet. Recently, this inlet has been dredged on a semi-annual basis; however, the impounded sediment volumes are small (generally less than 1,000 cubic yards). The sand dredged from Bournes Pond Inlet has typically been placed along the beach west of the inlet. This placement location represents the most highly eroded areas adjacent to the inlet, as illustrated in Figure 4-1; however, passing inlet sediment to downdrift shorelines (beaches to the east of the inlet) would supply these areas with needed littoral sediments as well. This was initially proposed as part of the Final EIR (Weston & Sampson, 1981) for the original Bournes Pond Inlet relocation project, but was never implemented.

More recently, a dune nourishment program was attempted in the vicinity of the pre-1985 inlet to maintain the shoreline that is out of equilibrium. Approximately 18,500 cubic yards of beach compatible sediment were placed along the beach and dune system to the west of the Bournes Pond entrance (Fields, 2009). Figure 4-2 illustrates the observed shoreline change between 2004 (prior to the nourishment) and 2009 (after placement), which indicates that the fill volume placed was not able to maintain the shoreline. The proposed inlet widening also includes a 25-foot lengthening of the west jetty to improve stability of the western section of Menauhant Beach. This increase in structure ‘footprint’ may require shortening of another structure as mitigation; a process that has been accepted by regulatory agencies at nearby Rushy Marsh to offset potential adverse impacts of the project.

Figure 4-2  Historical Shoreline Change Between 2004 and 2009
After the completion of the beach and dune nourishment west of Bournes Pond Inlet in spring of 2009, natural physical processes began to erode and transport the nourishment material eastward within the active littoral zone. The erosion of nourishment material from the beach raised concerns about where the material was being transported and potential shoaling impacts upon Bournes Pond Inlet. In the summer of 2010 the US Army Corps of Engineers conducted a LiDAR survey along the south coast of Massachusetts which provided detailed elevation data east and west of Bournes Pond. As part of the Bournes Pond Inlet widening evaluation, a detailed hydrographic survey was conducted on October 17, 2012, which covered the area immediately offshore and the lower portions of Bournes Pond. The two elevation datasets provided an opportunity to evaluate what changes had occurred to the ebb and flood shoals and the tidal channel into Bournes Pond. Bathymetric changes for the nearshore area of Bournes Pond were analyzed. LiDAR data from 2010 and hydrographic survey data from 2012 were used to create two bathymetric surface models, a method utilized to compare seafloor topography when datasets are composed of unevenly spaced points. The results illustrate changes on the seafloor surface by showing accretion with positive cell values and erosion with negative values, as shown in Figure 4-3. The figure demonstrates the substantial deposition of material on the ebb shoal (seaward of the inlet) and shoaling of the main tidal channel along the western edge of the flood shoal (landward of the inlet) within Bournes Pond. The deposition of material on the shoal has raised the surface elevation on average 2 to 3 feet with higher deposition rates seen in the eastern shadow of the ebb shoal. The main tidal channel within the pond has experienced significant shoaling, up to 4.5 feet along the channel, which raises concerns about the impacts to tidal flushing, water quality within the Pond, and ongoing maintenance of the inlet channel. The total volume of accretion on the ebb shoal is approximately 17,000 cy, which corresponds very closely with the nourishment volume of 18,500 cy that was initially placed on the beach west of the inlet.

Figure 4-3 Bathymetry Change Between 2010 and 2012

While beach nourishment can be an effective form of shore protection, it is not clear that the placement at Menauhant Beach performed as expected. An example of a more successful local project is the placement of approximately 120,000 cubic yards of beach nourishment was placed along the Falmouth Heights
shoreline in 1957 as part of the navigation improvement project for Falmouth Harbor (U.S. Army Corps of Engineers, 1964). Based on shoreline change data, much of this material can still be found on the beach between the Falmouth Heights bluffs and Little Pond inlet, where the shoreline has shown accretion between 1938 and 2004. If designed properly, both dune restoration and beach nourishment projects can be constructed in a manner that will not affect dredging frequency and/or stability of the existing tidal inlets to Falmouth’s south shore ponds.

It is anticipated that the future inlet widening project will utilize the lessons learned from previous local projects to ensure that dredged material is placed in the appropriate location to ensure (a) it helps maintain the regional shoreline position to the maximum extent practicable, (b) it does not exacerbate shoaling within the Bournes Pond Inlet channel, and (c) littoral sediments continue to bypass the inlet (either by natural or anthropogenic means) to supply down-drift shorelines. These goals may require that the Town rethink sediment management practices based on the updated information regarding local sediment transport patterns.

5.3. Hydrodynamic Evaluations for Inlet Size Optimization

5.3.1. Tidal Hydrodynamics

The hydrodynamic evaluation of widening Bournes Pond Inlet initially began as part of the MEP analysis (Howes, et al., 2005). A widened inlet was run as a stand-alone alternative and in conjunction with a scenario that included wastewater load removal in the lower watershed. The preliminary inlet widening alternative doubled the existing inlet size (the current jettied inlet is 50-feet wide) to 100-feet in width to examine the influence of increased tidal exchange on water quality within in the Pond. This MEP analysis did not include any optimization of the inlet size to control flow velocities, minimize shoaling, or minimize inlet maintenance requirements. The inlet size was selected based on the inlet size at nearby Green Pond which has minimal tidal attenuation throughout the system. The present hydrodynamic analysis of the inlet was undertaken to evaluate alternatives for widening the existing inlet for improved water quality within the Pond; while optimizing the size to minimize shoaling within the inlet and channels to minimize future maintenance requirements. This optimized inlet would represent the “best-case” stable inlet system that could be effectively maintained to provide upstream water quality improvements via reductions in Total Nitrogen concentrations with the estuary. The current hydrodynamic analysis of the inlet was undertaken to evaluate alternatives for widening the current inlet to improve water quality within the pond; while optimizing the inlet size to minimize shoaling in the inlet and adjoining tidal channels to minimize future maintenance requirements.

The original MEP model was developed from data collected in 1999 and; therefore, required updating in order to reflect current physical conditions within Bournes Pond. To update the basis of the model, water surface elevation measurements along with a hydrographic survey of the lower Pond and Vineyard Sound were collected in fall of 2012. The measurements provide greater detail to refine and characterize the physical conditions within the Pond. The refinements were necessary to increase the resolution of the finite element grid over the evolving flood and ebb shoals in addition to the area around the inlet to provide a basis for the evaluation of the proposed bridge structures across the widen inlet. After the finite element grid for the hydrodynamic model was updated, the model was recalibrated using the 2012 water surface elevation
measurements. The recalibration procedures required the revised hydrodynamic model to meet the same calibration tolerances as the original MEP Model.

Using the updated hydrodynamic model, the analysis to determine the appropriate inlet width began with evaluating a series of increasing inlet widths. The goal of the evaluation was to increase the tidal prism entering the pond to maximize tidal flushing of nutrients, while optimizing the average maximal peak velocities through the inlet channel to ensure cross sectional stability. Figure 1 shows a representation of the initial inlet sizing runs that determined the maximum inlet width before the inlet became unstable and would require significant maintenance to sustain the tidal flow between the Pond and Sound. The analysis determined that an inlet width of approximately 90-feet would increase tidal flushing and would maintain the inlet cross-sections under typical tidal conditions. The determination of optimal cross-sectional area allowed the four (4) bridge/culvert alternatives to be refined and evaluated. A full discussion of the bridge selection process and criteria are in the previous sections of this memorandum. The preferred alternative from the selection process was the Double Span Bridge over the widened inlet.

The hydrodynamic model was updated to reflect the bridge pier and abutment configuration for the double span alternative. The model was then re-run over a series of inlet widths to examine the sensitivity of the tidal velocities through the inlet relative to minor changes in inlet width and the resulting influence on tidal prism volumes entering the pond. The sensitivity analysis confirmed that the 90-foot bridge opening with two (2) 45-foot spans, a 3-foot wide pier centered in the channel, and vertically faced bridge abutments would maximize flushing, while balancing inlet velocities to minimize maintenance dredging requirements. The proposed configuration increases the tidal prism by approximately eight percent and the reduced mean system volume by one and a half percent. Increasing the tidal prism and reducing mean volume relative to existing conditions will reduce the nutrient concentrations within the pond. A full evaluation of the total nitrogen reductions as a result of this widened inlet scenario will be submitted in a separate technical memorandum. Hydrodynamic model results for existing and improved inlet conditions are presented in Figure 5-1. In the top plot, tide attenuation is apparent by the higher elevation of the low tides, and also by the time delay of the tide signal inside the pond. In the bottom plot, tidal attenuation is dramatically reduced for the proposed double span 90-foot wide inlet, to the point where there is little difference between the range and phase of both tide signals. The reduction in tidal attenuation demonstrates that the proposed inlet width has maximized the potential flushing within Bournes Pond.

The hydrodynamic evaluation identified several advantages with the proposed Double Span Bridge configuration:

- Minimal structural influence on tidal movement through the inlet;
- 7.5 percent increase tidal prism;
- 1.5 percent decrease in mean tidal volume;
- Optimized channel width to reduce the ongoing maintenance requirements of the inlet;
- The natural bottom substantially reduces the complexity of maintaining the inlet; and,
- Natural channel bottom (as opposed to a culvert) will allow natural variation in bottom contours.

These advantages are further illustrated on the following Figures 5-1 and 5-2.
Figure 5-1 Average Maximum Peak Velocities for Flood and Ebb Tides Through the Inlet to Bournes Pond

Inlet velocities that are greater than the sediment movement threshold indicate a stable inlet channel that can be generally maintained by tidal currents.
6 SUMMARY OF EVALUATIONS TO ESTIMATE THE NITROGEN REMOVAL BENEFITS OF A LARGER INLET

6.1. Introduction

The main purpose of developing a preliminary design of a larger inlet is to provide greater tidal flushing and the associated improvements in water quality. The Massachusetts Estuaries Project (MEP) water quality model for Bournes Pond was used to evaluate the estimated water quality improvements and to estimate the effective load of nitrogen that is removed from the system (“Inlet Opening Effective Nitrogen Load Removal”) with the larger inlet. Costs were then evaluated to allow a comparison of the costs for the new bridge and larger opening, and the costs for removal of the same amount of nitrogen with wastewater collection and treatment. The following sections of this memorandum summarize these evaluations to estimate the nitrogen removal estimates of a larger inlet and the beneficial cost comparison.
6.2. Water Quality Modeling Evaluations

Coastal engineers from Applied Coastal Research and Engineering in association with Brian Howes of UMass Dartmouth School of Marine Science and Technology (SMAST) completed water quality modeling of the larger inlet as described in Section 5 of the attached Appendix BP-1-2. They utilized the following water quality modeling evaluation steps:

- The MEP water quality modeling programs that Massachusetts DEP utilized as the basis of the Total Maximum Daily Loads (TMDLs) for Bournes Pond were updated for the evaluations. Updates included new inlet depth measurements and finer grid spacing in the hydrodynamic model as discussed earlier in this memorandum.
- The existing nitrogen loading conditions were modeled with the existing inlet opening. This model run closely matched the existing average total nitrogen concentration at the Bournes Pond sentinel station for Bournes Pond. This concentration is 0.578 mg/L.
- The existing nitrogen loading conditions were modeled with the recommended larger opening of the 2-span bridge. This model run produced a projected average total nitrogen concentration of 0.555 mg/L at the sentinel station. This value may seem to be a small reduction compared to the existing nitrogen concentration of 0.578 but it is very significant when compared to the TMDL threshold concentration of 0.45 mg/L which takes into account the background nitrogen that comes into the estuary from Vineyard Sound with the allowable increase. This indicates a lowering of the needed nitrogen concentration reduction from 0.128 mg/L to 0.105 mg/L relative to the TMDL threshold concentration.
- The "Inlet Opening Effective Nitrogen Load Reduction" value was then estimated by re-running the model with the existing inlet opening; and reducing the watershed loads from the existing loading to a condition where the sentinel station TN concentration matched the nitrogen concentration that was achieved with the larger inlet (0.555 mg/L) provided by the double-span bridge. The existing nitrogen load of 9.61 kg/day was reduced to 4.14 kg/day to match the concentration at the sentinel station. This load reduction of 5.47 kg/day is a 56.9-percent reduction. This 56.9-percent reduction equates to a reduction of 1,995 kg/year within the watershed.

6.3. Cost Comparison Evaluations

The capital cost comparison evaluations using conservative assumptions for wastewater nitrogen are summarized below:

- The total capital cost of the new opening with double-span bridge is $5,520,000 as summarized earlier in this Technical Memorandum.
- The capital cost of removing 1,995 kg/yr of nitrogen with wastewater collection, advanced treatment, and treated water recharge outside of the watershed is estimated at 12,830,000 based on the following factors:
  - Wastewater system to be developed for 273 houses based upon:
    - 1,995 kg/yr divided by 7.3 kg/yr/house based on 151 gallons of water consumption/day/house at a wastewater nitrogen concentration of 35 mg/L. This flow and
nitrogen concentration basis is from page 37 of the MEP Technical Report (excerpt attached in Appendix BP-1-3).

- Estimated cost of $47,000 per house based on the capital cost incurred for the New Silver Beach Wastewater Project to sewer 231 properties in 2007 scaled to 2013 costs. Information on the costs and number of properties is attached in Appendix BP-1-3.

This cost comparison indicates that the cost to increase the inlet opening and remove 1,995 kg/yr of nitrogen is approximately 43 percent of the costs to provide conventional wastewater management to this area. Stating this comparison in another way, removing the 1,995 kg/yr of nitrogen with wastewater management would be approximately 2.3 times more expensive than opening the inlet.

There are many assumptions used in the cost comparison summarized above. They are believed to be conservative assumptions that are intended to not overstate the cost savings with the inlet opening.

There is a second method to estimate the number of homes that would need to be sewer to remove the 1,995 kg/yr load which indicates an even greater cost savings for the inlet opening. The Massachusetts Estuaries Project (MEP) used additional factors in their nitrogen load calculations as indicated on page 33 of the Technical Report (attached in Appendix BP-1-3) as summarized below:

- The MEP assumed that 25 percent of the nitrogen discharged from each home is removed in the leach field as documented at MassDEP’s Alternative Septic System Test Center at MMR. This means that the nitrogen concentration from the septic system is reduced from 35 mg/L to 26.25 mg/L.
- The MEP assumed that only 90 percent of the water consumption at each home becomes wastewater due to non-potable water uses such as irrigation and other outdoor uses. This means that the wastewater flow transporting the 26.25 mg/L of nitrogen is 136 gallons per day instead of 151 gallons per day.

These two factors indicate a daily household nitrogen load of 4.9 kg/yr per house; therefor, there is a need to sewer 407 houses to remove the 1,995 kg of wastewater nitrogen that is actually entering the estuary.

The cost to sewer 407 houses at $47,000/house would be $19,130,000 which would be over 3.4 times the cost of the inlet opening.

The two methods used above indicate that the cost savings is very significant.
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LEGEND

TOWN OF FALMOUTH, MA
WASTEWATER AND NUTRIENT MANAGEMENT
EXISTING BOURNES POND INLET

NOT TO SCALE

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www.ghd.com
Appendix BP-1-1

Sheets 1 through 5 Illustrating Sketches of:

1. Existing Bridge
2. Single-Span Bridge
3. Double-Span Bridge
4. Existing Bridge with Culverts Added
5. Replacement of the Existing Bridge with Five Culverts
Appendix BP-1-2
Hydrodynamic and Water Quality Analysis for Widening Bournes Pond Inlet
1. INTRODUCTION

The hydrodynamic evaluation of widening Bournes Pond Inlet initially began as part of the MEP analysis (Howes, et al., 2005). A widened inlet was run as a stand-alone alternative and in conjunction with a scenario that included wastewater load removal in the lower watershed. The preliminary inlet widening alternative doubled the existing inlet size (the current jetted inlet is 50-feet wide) to 100-feet in width to examine the influence of increased tidal exchange on water quality within the Pond. This MEP analysis did not include any optimization of the inlet size to control flow velocities, minimize shoaling, or minimize inlet maintenance requirements. The inlet size was selected based on the inlet size at nearby Green Pond which has minimal tidal attenuation throughout the system. The MEP analysis showed that doubling the inlet width would increase the average tide prism (amount of water exchanged through the inlet) by over 20% and decrease nitrogen concentrations by approximately 0.07 mg/L at the Threshold Station, midway up the main portion of the estuary. The present hydrodynamic analysis of the inlet was undertaken to evaluate alternatives for widening the existing inlet for improved water quality within the Pond; while optimizing the size to minimize shoaling within the inlet and channels to minimize future maintenance requirements. This optimized inlet would represent the “best-case” stable inlet system that could be effectively maintained to provide upstream water quality improvements via reductions in Total Nitrogen concentrations with the estuary.

The original hydrodynamic modeling analysis was based upon bathymetry and tide data collected in 1999. Since this previous analysis was directed at estuarine-wide nutrient management, the evaluation of the tidal inlet widening was only evaluated using a generalized approach, based upon the data available. More recent (2009) regional topography/bathymetry data collection has been performed by the U.S. Army Corps of Engineers. In addition, an updated bathymetry survey, combined with tidal data collection offshore and within Bournes Pond, was conducted to determine contemporary conditions. In this manner, an updated hydrodynamic model calibrated to 2012 conditions could be utilized as the baseline for assessing inlet widening options.

In a similar manner to the hydrodynamics, an updated evaluation of local coastal processes also was performed to ensure that recent alterations to the beach system have not affected the performance of the inlet. For example, shoaling within the inlet “throat” as a result of longshore sediment transport and regional beach erosion can alter the efficiency of the tidal inlet. Understanding the natural and anthropogenic processes that have led to the existing conditions of the Bournes Pond inlet system is critical for developing a successful design to improve the inlet system, without causing adverse impacts. Perhaps the single most important consideration for tidal inlet design is to ensure that an inlet is ‘stable’, where the destructive forces attempting to close the inlet (e.g. the sediment movement along the fronting barrier beach system) are balanced by the restorative forces attempting to scour a deeper inlet channel (e.g. the inlet current velocities). Since inlet stability is partially governed by longshore coastal sediment transport, understanding the regional long-term shoreline change and littoral movement of sand is critical for evaluating stability of the Bournes Pond inlet.
2. HISTORIC SETTING, SHORELINE AND BATHYMETRIC CHANGE

In general, the observed longshore transport rates along the south coast of Falmouth are relatively low, primarily as a result of the quiescent wave environment of Nantucket and Vineyard Sounds. Although the amount of sand moving along the coast is small, the tidal prism through Bournes Pond Inlet also is relatively small, due to a tide range of only about 1.5 feet. Since the construction of the existing jetty system in 1985 at the entrance, the inlet has generally reached equilibrium, where the tidal velocities through the main channels are sufficient to prevent significant shoaling. Recent annual dredging of these inlets has only been on the order of 1,000 cubic yards per year (personal communication with Wayne Jaedke, Barnstable County Dredge, 2004).

In addition, it appears that the south coast of Falmouth (between Falmouth Harbor and the west entrance to Waquoit Bay) has generally equilibrated to changes in local coastal sediment transport caused by the construction of shoreline armor. Extensive armor of the Falmouth shoreline began in the late 1800s and early 1900s with construction of the railroad to Woods Hole, the old stone dock, the Falmouth Harbor jetties, and the Waquoit Bay east jetties. This shoreline armor continued through the mid-1900s with the construction of stone groin fields, which often replaced existing wooden structures. Along the shoreline to the west of Bournes Pond entrance, these wooden and stone structures were constructed to protect Menauhant Road and waterfront dwellings (Figures 2-1 and 2-2). The remnants of wooden groins and bulkheads can be found along much of Falmouth's south coast (Figure 2-3).

In 1985, Bournes Pond inlet was relocated from the western portion of the beach to a location in the middle of the beach, as recommended by the Final Environmental Impact Report (Weston & Sampson, 1981). This inlet relocation project was performed because the historical inlet was unstable and infilled frequently, leading to water quality problems within Bournes Pond. However, at the time, data did not exist to optimize the inlet system for water quality improvements.

As shown in Figures 2-1, 2-2, and 2-4, hurricanes can have a significant impact on both the shoreline and the inlets. Due to the relatively quiescent wave and tide regime within this region, the impact of infrequent storms, primarily a result of storm surge, can be dramatic. According to historic flooding information (U.S. Army Corps of Engineers, 1988), the storm surge level in the Green Pond area was 11 feet NGVD29 (more than 10 feet above mean sea level). Due to this elevated water level, the series of low-lying barrier beaches that separate Nantucket Sound from the coastal ponds were overtopped, often carrying beach sediment into the estuaries. These infrequent storms can reshape the shoreline in ways that would require many years or decades under the typical wave, wind, and tide regime of the Falmouth south coast. During the twentieth century, the severe hurricanes influencing the Falmouth shoreline include the hurricanes of 1938, 1944, and 1954, as well as Hurricane Bob in 1991. Of these storms, the Hurricane of 1944 had the largest storm surge along the south shore of Falmouth (U.S. Army Corps of Engineers, 1988).
Figure 2-1. Photograph of the Wellsmere Inn in Maravista immediately after the 1944 Hurricane. Note the wood bulkheads and concrete seawall utilized to armor the shoreline.

Figure 2-2. Photograph of Menauhant Road in Maravista immediately after the 1944 Hurricane. Note the stone revetment armoring the roadway.
More recently, the passage of Hurricane Sandy in 2012 created substantially elevated tidal conditions for several days (as shown by the tidal data collection effort described later in this report), where the peak storm surge was approximately 3.5 feet above normal water levels. Figures 2-5 illustrate the wave overtopping and storm surge during Hurricane Sandy, which showed a flood level that was between a 5-year and 10-year storm event. In general, the type of storm conditions experienced during Hurricane Sandy also occurs during severe Nor'easters that influence water levels in Nantucket Sound.

Figure 2-3. Photograph of the shoreline west of Menauhant Beach taken in 2004 showing remnants of a timber bulkhead and groin, as well as more recent stone structures.
Figure 2-4. Photograph of the old Great Pond bridge immediately after the 1944 Hurricane. The photograph shows that storm overwash eroded the roadway and approach ramps to the bridge. Note the Great Pond jetty at the left side of the photograph.

Figure 2-5. Photograph of Bournes Pond inlet during Hurricane Sandy on October 29, 2012, where the jetties are completely submerged. During this storm event the dune west of the inlet was completely eroded.
2.1 HISTORICAL SHORELINE CHANGE

Shoreline change maps can effectively be used to evaluate the effects of long-term coastal processes. In addition, shoreline change maps also can indicate the effects of short-term changes that often occur as the result of anthropogenic (e.g. development of extensive shore protection structures) or natural (e.g. inlet migration) processes. Prior to developing conclusions and/or management recommendations that depend on shoreline change estimates, it is critical to understand potential errors and uncertainties associated with this type of analysis. Understanding the limitations of shoreline change data is critical for developing appropriate management strategies for shorelines and inlets in areas with relatively low shoreline migration rates, such as Falmouth’s south coast.

When determining shoreline position change, all data contain inherent errors associated with field and laboratory compilation procedures. These errors should be quantified to gage the significance of measurements used for research/engineering applications and management decisions. Table 2-1 summarizes estimates of potential error associated with shoreline data sets used for this study. Because individual errors are considered to represent standard deviations, root-mean square error estimates are calculated as a realistic assessment of combined potential error. Using these estimates, the total root mean square (RMS) estimate for the 1938 to 2009 time period is ±30.5 feet, or approximately 0.4 feet per year and for 2004 to 2009 is ±20 feet, or approximately 4.0 feet per year.

Shoreline change was evaluated for this study during the time period from 1938 to 2004. Change calculations were made at 20-meter intervals along the coast in the vicinity of Menauhant Beach, MA using the Automated Shoreline Analysis Program (ASAP) for ArcGIS 9.0. Shore-normal transects were developed using average shoreline angles determined at each analysis point. All transects used for determining change rates were visually inspected to ensure suitability for analysis and shoreline structure avoidance.

For the tidal inlet at Bournes Pond, the influence of shoreline change and the related longshore sediment transport rates directly influence the stability of the existing inlet system. One portion of the Falmouth south coast that has not reached equilibrium is immediately to the west of the existing Bournes Pond inlet system, in the vicinity of the historical pre-1985 inlet. The “hot spot” erosion in the vicinity of Bournes Pond likely represents adjustment of the shoreline following placement of the relatively recent jetties, as seen in Figure 2-6. The previous inlet was located where the remnant road bridge still exists to the south of Menauhant Road. Repositioning of the jetties caused a shift in the beach form, as sediments accreted in the vicinity of the newly constructed jetties. The area of the historical ebb shoal (seaward of the previous inlet) shows substantial erosion since 1938, with annual recession rates in excess of 3 feet per year (Figure 2-6).

With the exception of the highly erosional area in the vicinity of the pre-1985 inlet, the long-term (1938 to 2004) shoreline change shows a relatively stable shoreline compared to many other areas of the Falmouth south coast. Although this area represents a low-lying barrier beach system, the overall stability indicates that the beach receives longshore sediments that are transported by wave action and that the area that is highly erosional likely is related solely to repositioning of the inlet within the system.
Table 2-1. Estimates of Potential Error Associated with Shoreline Position Surveys.

<table>
<thead>
<tr>
<th>Cartographic Errors (1938)</th>
<th>Map Scale 1:10,000</th>
</tr>
</thead>
<tbody>
<tr>
<td>Inaccurate location of control points on map relative to true field location</td>
<td>±16 ft</td>
</tr>
<tr>
<td>Placement of shoreline on map</td>
<td>±10 ft</td>
</tr>
<tr>
<td>Line width representing shoreline</td>
<td>±3 ft</td>
</tr>
<tr>
<td>Digitizer error</td>
<td>±3 ft</td>
</tr>
<tr>
<td>Operator error</td>
<td></td>
</tr>
<tr>
<td><strong>Historical Aerial Surveys (1938)</strong></td>
<td></td>
</tr>
<tr>
<td>Delineating high-water shoreline position</td>
<td>±16 ft</td>
</tr>
<tr>
<td><strong>Orthophotography (2009)</strong></td>
<td></td>
</tr>
<tr>
<td>Delineating high-water shoreline position</td>
<td>±10 ft</td>
</tr>
<tr>
<td>Position of measured points</td>
<td>±10 ft</td>
</tr>
<tr>
<td><strong>GPS Surveys (2004)</strong></td>
<td></td>
</tr>
<tr>
<td>Delineating high-water shoreline position</td>
<td>±3 to ±10 ft</td>
</tr>
<tr>
<td>Position of measured points</td>
<td>±3 to ±10 ft</td>
</tr>
</tbody>
</table>

Table 2-2. RMS Uncertainty to April 2009

<table>
<thead>
<tr>
<th></th>
<th>± feet</th>
<th>± feet/year</th>
</tr>
</thead>
<tbody>
<tr>
<td>July 1938</td>
<td>30.5</td>
<td>0.4</td>
</tr>
<tr>
<td>April 2004</td>
<td>20.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

Current management practices at Bournes Pond inlet consists of periodic dredging to maintain the channel in between the jetties, as well as through the shallow ebb shoal offshore of the inlet. Recently, this inlet has been dredged on a semi-annual basis; however, the impounded sediment volumes are small (generally less than 1,000 cubic yards). The sand dredged from Bournes Pond inlet has typically been placed along the beach west of the inlet. This placement location represents the most highly eroded areas adjacent to the inlet, as illustrated in Figure 2-6; however, passing inlet sediment to downdrift shorelines (beaches to the east of the inlet) would supply these areas with needed littoral sediments as well. This was initially proposed as part of the Final EIR (Weston & Sampson, 1981) for the original Bournes Pond inlet relocation project, but was never implemented.

More recently, a dune nourishment program was attempted in the vicinity of the pre-1985 inlet to maintain the shoreline that is out of equilibrium. Approximately 18,500 cubic yards of beach compatible sediment were placed along the beach and dune system to the west of the Bournes Pond entrance (Fields, 2009). Figure 2-7 illustrates the observed shoreline change between 2004 (prior to the nourishment) and 2009 (after placement), which indicates that the fill volume placed was not able to maintain the shoreline.
Figure 2-6: Historical shoreline change between 1938 and 2004.
2.2 BATHYMETRIC CHANGE

After the completion of the beach and dune nourishment west of Bournes Pond Inlet in spring of 2009, natural physical processes began to erode and transport the nourishment material eastward within the active littoral zone. The erosion of nourishment material from the beach raised concerns about where the material was being transported and potential shoaling impacts upon Bournes Pond Inlet. In the summer of 2010 the USACE conducted a LiDAR survey along the south coast of Massachusetts which provided detailed elevation data east and west of Bournes Pond. As part of the Bournes Pond inlet widening evaluation, a detailed hydrographic survey was conducted on October 17, 2012, which covered the area immediately offshore and the lower portions of Bournes Pond. The two elevation datasets provided an opportunity to evaluate what changes had occurred to the ebb and flood shoals and the tidal channel into Bournes Pond. Bathymetric changes for the nearshore area of Bournes Pond were analyzed. LiDAR data from 2010 and hydrographic survey data from 2012 were used to create two bathymetric surface models, a method utilized to compare seafloor topography when datasets are composed of unevenly spaced points. The results illustrate changes on the seafloor surface by showing accretion with positive cell values and erosion with negative values, as shown in Figure 2-8. The figure demonstrates the substantial deposition of material on the ebb shoal (seaward of the inlet) and shoaling of the main tidal channel along the western edge of the flood shoal (landward of the inlet) within Bournes Pond. The deposition of material on the
shoal has raised the surface elevation on average 2 to 3 feet with higher deposition rates seen in the eastern shadow of the ebb shoal. The main tidal channel within the pond has experienced significant shoaling, up to 4.5 feet along the channel, which raises concerns about the impacts to tidal flushing, water quality within the Pond, and ongoing maintenance of the inlet channel. The total volume of accretion on the ebb shoal is approximately 17,000 cy, which corresponds very closely with the nourishment volume of 18,500 cy that was initially placed on the beach west of the inlet.

![Bathymetry Change](image)

Figure 2-8. Bathymetry change between 2010 and 2012.

While beach nourishment can be an effective form of shore protection, it is not clear that the placement at Menauhant Beach performed as expected. An example of a more successful local project is the placement of approximately 120,000 cubic yards of beach nourishment was placed along the Falmouth Heights shoreline in 1957 as part of the navigation improvement project for Falmouth Harbor (U.S. Army Corps of Engineers, 1964). Based on shoreline change data, much of this material can still be found on the beach between the Falmouth Heights bluffs and Little Pond inlet, where the shoreline has shown accretion between 1938 and 2004. If designed properly, both dune restoration and beach nourishment projects can be constructed in a manner that will not affect dredging frequency and/or stability of the existing tidal inlets to Falmouth's south shore ponds.
It is anticipated that the future inlet widening project will utilize the lessons learned from previous local projects to ensure that dredged material is placed in the appropriate location to ensure (a) it helps maintain the regional shoreline position to the maximum extent practicable, (b) it does not exacerbate shoaling within the Bournes Pond inlet channel, and (c) littoral sediments continue to bypass the inlet (either by natural or anthropogenic means) to supply downdrift shorelines. These goals may require that the Town rethink sediment management practices based on the updated information regarding local sediment transport patterns.
3. FIELD DATA COLLECTION AND ANALYSIS

To form a thorough understanding of the estuarine tidal processes governing the flow in and out of Bourne Pond and its influence upon water quality within the Pond, a significant amount of data was required. The initial basis for the evaluation began with the data collected (tides, currents, bathymetry, and water quality parameters) as part of the MEP analysis for Bournes Pond in 1999. To supplement and update the previous datasets, tidal water surface elevations and hydrographic data were collected in the fall of 2012. Both sets of data were utilized to revise the hydrodynamic model to provide a basis for evaluating the impacts associated with the proposed inlet widening for the mitigation of nutrient loading to the estuarine system. Accurate numerical modeling of system hydrodynamics is dependent upon measured conditions within the estuary for two important reasons:

- To provide ‘real’ observations of hydrodynamic behavior to calibrate and validate the model results
- To define accurately the system geometry and boundary conditions for the numerical model

A discussion and analysis of the original data collected as part of the MEP project is contained in the Linked Watershed-Embayment Model to Determine Critical Nitrogen Loading Thresholds for Great/Perch Pond, Green Pond and Bournes Pond, Falmouth, Massachusetts Report (April 2005). The 2012 data was collected as part of the inlet widening analysis and was required to update the bathymetry within the hydrodynamic model and examine changes in tidal propagation that have occurred since the original data collection in 1999.

3.1 BATHYMETRY

A detailed survey of the bathymetry was conducted in lower reaches of Bournes Pond and extended to the area immediately offshore of the pond in Vineyard Sound. The survey was designed to collect bathymetric measurements across the flood and ebb shoals of the system, as well as through the inlet throat. Due to the higher tidal and wave-induced current velocities in these regions, it is anticipated that these areas are subjected to the largest variations. In addition, the previous hydrodynamic modeling effort depended on relatively sparse bathymetry collected in 1999. The updated data set provides enhanced detail of the highly dynamic regions through the throat of the channel and the series of shoals both seaward and landward of the inlet. The updated survey transects were most dense in the vicinity of flow constrictions and in the vicinity of channel 'bends', where the greatest variability in bottom bathymetry was expected. The hydrographic survey was conducted on October 17, 2012.

Survey transects were spaced 25 feet apart augmented by perpendicular cross-tie lines spaced 100 feet apart to allow statistical analysis of data uncertainty. Transects spaced 100 feet apart were occupied in portions of the Pond further from the causeway. Background imagery including USGS quadrangles and orthophotos obtained from MassGIS were imported to the navigation and acquisition software to aid real-time analysis and quality control. All data have been projected to the Massachusetts Mainland State Plane grid (NAD83, US Foot). Processed data have been provided as NAVD88 elevations (US Foot).

The bathymetric data acquisition system consisted of a laptop computer running HYPACK hydrographic survey software, the serially-interfaced Trimble DGPS and an Odom Hydrographic, Inc. CV-100 precision echo sounder interfaced via ethernet. Ping rates ranged from 5 to 15 Hz, depending on the depth. An 8-degree beamwidth 208-kHz transducer was
used to acquire soundings. The echo sounder transducer was mounted to the rail of the survey vessel amidships using a high strength adjustable boom. The DGPS antenna was attached to the top of the transducer boom, eliminating the need to correct for horizontal offsets. The transducer depth below the water surface (draft) was checked and recorded at the start and end of each day.

Echo sounder accuracy was checked at the start and end the survey by comparing acoustic water depth measurements to absolute water depths (ranges) obtained using the “bar check” method, in which a metal plate is lowered beneath the echo sounder’s transducer to several known distances (e.g., 5, 10 and 20 ft) below the water’s surface. “Bar-check” calibrations were consistently accurate to within 0.1 foot. Acoustic soundings were also compared to depth measurements made using a survey staff (0.01 ft increments). These comparisons also confirmed accuracy to within 0.1 foot. Additional calibrations were conducted in both inshore and offshore portions of the survey area by collecting water column profiles of sound velocity. Sound velocity was determined based on measurements of temperature and salinity made using a YSI, Inc. Model 22 water quality meter. Sound velocity profiles were calculated using the Chen equation.

Bathymetric data were processed using the HYPACK Processor Module. Dense aquatic vegetation and shoals of small fish were present in survey records and were removed from records to the greatest extent possible. Soundings were then adjusted to NAVD88 elevations using tide data collected in Bournes Pond and offshore in Vineyard Sound.

A figure of the survey measurements is shown in Figure 3-1. Corrected NAVD88 elevations ranged from -0.64 feet to -19.8 feet (average = -5.6). Data were statistically evaluated to document accuracy and uncertainty according to U. S. Army Corps of Engineers specifications (US ACOE, 2002). The ACOE 95th percentile confidence interval requirement for bias is +/- 0.2 feet and “resultant elevation/depth accuracy” is +/- 0.5 feet in depth less than 15 feet and +/- 1.0 foot in depths from 15 to 40 feet. Cross-tie statistics for 327 comparisons yielded a bias of -0.012 feet and a 95% confidence interval uncertainty of 0.37 feet. This analysis documents compliance with ACOE Performance Standards for Navigation and Dredging Support Surveys.

In addition to the hydrographic data, the USACE conducted a LiDAR survey along the south coast of Massachusetts in the summer of 2010. The LiDAR data provided detailed elevation data east and west of Bournes Pond which was incorporated into the hydrodynamic model.
3.2 WATER ELEVATION MEASUREMENTS AND ANALYSIS

Changes in water surface elevation were measured using internal recording tide gages. These tide gages were installed on fixed platforms (pier pilings) to record changes in water pressure over time. Variations in the water surface can be due to tides, wind set-up, or other low frequency oscillations of the sea surface. The tide gages were installed in two (2) locations as shown in Figure 3-2. Since the goal was to update the data and evaluate changes in attenuation caused by changes in the system, a gage was deployed in Vineyard Sound and another within Bournes Pond. The gages were deployed October 16, 2012 and recovered November 6, 2012. Data records span the passage of Hurricane Sandy which can be seen in the water surface elevation records for both gages.

The tide gages used for the study consisted of Brancker XR-420 instruments. Data collections parameters were set for 10-minute intervals, with each 10-minute observation resulting from a 16-second burst of measurements that are averaged for each observation. Each of these instruments use strain gage transducers to sense variations in pressure, with resolution of 0.001% full scale and a pressure accuracy of 0.01% full scale. Each gage was calibrated prior to installation to assure accuracy.
Once the data were downloaded from each instrument, the water pressure readings were corrected for variations in atmospheric pressure. Hourly atmospheric readings were obtained from the NOAA meteorological station in Buzzards Bay, MA (Buzzards Bay, Station BUZM3), interpolated to 10-minute intervals, and subtracted from the pressure readings, resulting in water pressure above the instrument. Further, a (constant) water density value of 1025 kg/m³ was applied to the readings to convert from pressure units (psi) to head units (for example, feet of water above the tide gage). All of the tide gages were surveyed using MTS RTK network to determine horizontal and vertical positions; these survey values were used to adjust the water
surface to a known vertical datum, North American Vertical Datum of 1988 (NAVD88). The result from each gage is a time series representing the variations in water surface elevation relative to NAVD88. Figure 3-3 presents the water levels at each gage location.

Analyses of the tide and bathymetric data provided insight into the hydrodynamic characteristics of the system. Harmonic analysis of the tidal time series produced tidal amplitude and phase of the major tidal constituents, and provided assessments of hydrodynamic ‘efficiency’ of each segment within the system in terms of tidal attenuation. This analysis also yielded an assessment of the relative influence of non-tidal, or residual, processes (such as wind forcing) on the hydrodynamic characteristics of each system.

Figure 3-3 shows the tidal elevation for the period October 16 through November 6, 2012 within Bournes Pond and offshore in Vineyard Sound. The curves have a predominant semi-diurnal (twice-a-day) variation. The passage of Hurricane Sandy can be clearly seen on October 29, which had a peak surge elevation of 4.2 feet NAVD88.

![Water elevation variations as measured in Bournes Pond.](image)

Figure 3-3.

To better quantify the changes to the tide through the inlet into the Pond, the standard tide datums were computed from the 11-day record prior to the passage of Hurricane Sandy. The computed datums are presented in Table 3-1. For most NOAA tide stations, these datums are computed using approximately 19 years of tide data, the definition of a tidal epoch. For this study, a significantly shorter time span of data was available; however, these datums still provide a useful comparison of tidal dynamics within the system. The Mean Higher High Water (MHHW) and Mean Lower Low Water (MLLW) levels represent the mean of the daily highest and lowest water levels. The Mean High Water (MHW) and Mean Low Water (MLW) levels
represent the mean of all the high and low tides of a record, respectively. The Mean Tide Level (MTL) is simply the mean of MHW and MLW. The tides in Bournes Pond are semi-diurnal, meaning that there are typically two tide cycles in a day. There is usually a small variation in the level of the two daily tides. This variation can be seen in the differences between the MHHW and MHW, as well as the MLLW and MLW levels.

<table>
<thead>
<tr>
<th>Location</th>
<th>Max Tide</th>
<th>MHHW</th>
<th>MHW</th>
<th>MTL</th>
<th>MLW</th>
<th>MLLW</th>
<th>Min Tide</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vineyard Sound</td>
<td>2.2</td>
<td>1.5</td>
<td>1.2</td>
<td>0.2</td>
<td>-0.8</td>
<td>-1.0</td>
<td>-1.4</td>
</tr>
<tr>
<td>Bournes Pond</td>
<td>2.1</td>
<td>1.4</td>
<td>1.1</td>
<td>0.2</td>
<td>-0.7</td>
<td>-0.8</td>
<td>-1.1</td>
</tr>
</tbody>
</table>

Harmonic analyses were performed on the time series from each gage location. Harmonic analysis is a mathematical procedure that fits sinusoidal functions of known frequency to the measured signal. The amplitudes and phase of 23 known tidal constituents result from this procedure. Table 3-2 presents the amplitudes of the four largest tidal constituents. The $M_2$, or the familiar twice-a-day lunar semi-diurnal, tide is the strongest contributor to the signal with an amplitude of 0.64 feet in Bournes Pond. The range of the $M_2$ tide is twice the amplitude, or about 1.3 feet. The diurnal (once daily) tide constituent, $K_1$ (solar), has an amplitude of approximately 0.27 feet and account for the higher high tide followed by the lower low tide seen in Figure 3-3. The $M_4$ and $M_6$ constituents are higher frequency harmonics of the $M_2$ lunar tide (twice and three times the frequency of the $M_2$, respectively) and result from frictional dissipation of the $M_2$ tide in shallow water. The observed astronomical tide is therefore the sum of several individual tidal constituents, with a particular amplitude, frequency, and phase.

<table>
<thead>
<tr>
<th>Location</th>
<th>$M_2$</th>
<th>$M_4$</th>
<th>$M_6$</th>
<th>$K_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vineyard Sound</td>
<td>0.66</td>
<td>0.16</td>
<td>0.05</td>
<td>0.28</td>
</tr>
<tr>
<td>Bournes Pond</td>
<td>0.64</td>
<td>0.10</td>
<td>0.04</td>
<td>0.27</td>
</tr>
</tbody>
</table>

The loss of amplitude together with increasing phase delay as the distance increases from Vineyard Sound is defined as tidal attenuation. Tidal attenuation can be a useful indicator of frictional damping progressing upstream. Attenuation of the tidal signal is caused by the geomorphology of the nearshore region (shoals), as well as flow restrictions caused by the inlet and tidal channels within an estuary. Channel restrictions (e.g., bridge abutments, narrow channels, shoaling) and shallow bents in the pond are the primary factors which influence tidal damping. A visual comparison of the tide gages in Figure 3-4 shows the increase in tidal lag, but not a significant amount of tidal attenuation. The system does not have considerable loss of tidal energy as the tidal signal propagates along the length of the system.
Figure 3-4. Water elevation variations as measured at Bournes Pond, between October 20-22, 2012.

Table 3-3 presents the phase delay (in other words, the travel time required for the tidal wave to propagate throughout the system) of the $M_2$ tide between Vineyard Sound and Bournes Pond. The delay increases as the tide propagates upstream due to increasing energy losses and the distance traveled between the two measurement points. There is approximately a 58 minute delay from offshore through the inlet and into the lower reaches of the pond.

<table>
<thead>
<tr>
<th>Location</th>
<th>Delay (minutes)</th>
<th>Distance (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vineyard Sound</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Bournes Pond</td>
<td>58.2</td>
<td>4,750</td>
</tr>
</tbody>
</table>

The tide data were further evaluated to determine the importance of tidal versus non-tidal processes to changes in water surface elevation. Non-tidal processes include wind forcing (set-up or set-down) within the system, as well as sub-tidal oscillations of the water surface. Variations in water surface elevation can also be affected by freshwater flow into the system, if these volumes are relatively large compared to tidal flow. The results of an analysis to determine the energy distribution (or variance) of the original water elevation time series for the system is presented in Table 3-4 compared to the energy content of the astronomical tidal signal (re-created by summing the contributions from the 23 constituents determined by the harmonic analysis). Subtracting the tidal signal from the original elevation time series resulted with the non-tidal, or residual, portion of the water elevation changes. The energy of the non-
tidal signal is compared to the tidal signal, and yields a quantitative measure of how important these non-tidal physical processes are relative to hydrodynamic circulation within the estuary. The large percentage of non-tidal energy is due to the passage of Hurricane Sandy during the gage deployment. Figure 3-5 shows the comparison of the measured tide at the Vineyard Sound station, with the computed astronomical tide resulting from the harmonic analysis, and the resulting non-tidal residual. Table 3-4 and the bottom plot within Figure 3-5 shows that outside the passage of Hurricane Sandy, that the percentage contribution of tidal energy is relatively unchanged throughout the system. This shows that freshwater inflow and meteorological effects have a negligible impact on the system. The data indicates that local effects due to winds and other non-tidal processes are minimal, and the system is largely driven by tidal energy.

<table>
<thead>
<tr>
<th>Location</th>
<th>Total Variance</th>
<th>Total</th>
<th>Tidal</th>
<th>Non-tidal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vineyard Sound</td>
<td>0.767</td>
<td>100</td>
<td>68.2</td>
<td>31.8</td>
</tr>
<tr>
<td>Bournes Pond</td>
<td>0.736</td>
<td>100</td>
<td>65.9</td>
<td>34.1</td>
</tr>
</tbody>
</table>

Figure 3-5. Results of the harmonic analysis and the separation of the tidal from the non-tidal, or residual, signal measured in Vineyard Sound.
4. HYDRODYNAMIC MODELING

The hydrodynamic evaluation of widening Bournes Pond Inlet initially began as part of the MEP analysis (Howes, et al., 2005). A widened inlet was run as a stand-alone alternative and in conjunction with a scenario that included wastewater load removal in the lower watershed. The preliminary inlet widening alternative doubled the existing inlet size to 100-feet in width to examine the influence of increased tidal exchange on water quality within the Pond. This MEP analysis did not include any optimization of the inlet size to control flow velocities, minimize shoaling, or minimize inlet maintenance requirements. The inlet size was selected based on the inlet size at nearby Green Pond which has minimal tidal attenuation throughout the system. The present hydrodynamic analysis of the inlet was undertaken to evaluate alternatives for widening the existing inlet for improved water quality within the Pond; while optimizing the size to minimize shoaling within the inlet and channels to minimize future maintenance requirements. This optimized inlet would represent the “best-case” stable inlet system that could be effectively maintained to provide upstream water quality improvements via reductions in Total Nitrogen (TN) concentrations with the estuary. The current hydrodynamic analysis of the inlet was undertaken to evaluate alternatives for widening the current inlet to improve water quality within the pond; while optimizing the inlet size to minimize shoaling in the inlet and adjoining tidal channels to minimize future maintenance requirements.

The original MEP model was developed from data collected in 1999 and, therefore, required updating in order reflect current physical conditions within Bournes Pond. To update the basis of the model, water surface elevation measurements along with a hydrographic survey of the lower Pond and Vineyard Sound were collected in fall of 2012 (see Chapter 3 for details of the data collection program). The measurements provide greater detail to refine and characterize the physical conditions within the Pond. The refinements were necessary to increase the resolution of the finite element grid over the evolving flood and ebb shoals in addition to the area around the inlet to provide a basis for the evaluation of the proposed bridge structures across the widen inlet. After the finite element grid for the hydrodynamic model was updated, the model was recalibrated using the 2012 water surface elevation measurements. The recalibration procedures required the revised hydrodynamic model to meet the same calibration tolerances as the original MEP Model.

The hydrodynamic study was performed to understand the tidal dynamics of Bournes Pond and the influence the inlet width has upon the water quality within the system. The tidal hydrodynamics were simulated using the RMA-2 model developed by the Resource Management Associates. It is a two-dimensional, depth-averaged finite element model, capable of simulating transient hydrodynamics on a flexible mesh. Once the model is recalibrated and the existing conditions numerical model is developed, the numerical modeling tool can be used to accurately simulate inlet alternatives.

4.1 Model Theory

The analysis of the Bournes Pond estuarine system utilized a numerical computer model to evaluate tidal hydrodynamics. The particular model employed was the RMA-2 model developed by Resource Management Associates (King, 1990). It is a two-dimensional, depth-averaged finite element model, capable of simulating transient hydrodynamics. Finite element models are well-suited to modeling riverine and estuarine areas with complex shoreline and bathymetric contours, and also allow for greater density of computational elements to be applied in areas of interest in the model domain. RMA-2 is widely accepted and tested for analyses of estuaries and rivers.
RMA-2 is a finite element model designed for simulating one- and two-dimensional depth-averaged hydrodynamic systems. The dependent variables are velocity and water depth, and the equations solved are the depth-averaged Navier-Stokes equations. Reynolds assumptions are incorporated as an eddy viscosity effect to represent turbulent energy losses. Other terms in the governing equations permit friction losses (approximated either by a Chezy or Manning formulation), Coriolis effects, and surface wind stresses. All the coefficients associated with these terms may vary from element to element. The model utilizes quadrilaterals and triangles to represent the prototype system. Element boundaries may either be curved or straight.

The time dependence of the governing equations is incorporated within the solution technique needed to solve the set of simultaneous equations. This technique is implicit; therefore, unconditionally stable. Once the equations are solved, corrections to the initial estimate of velocity and water elevation are employed, and the equations are re-solved until the convergence criterion is met.

In its original form, RMA-2 was developed by William Norton and Ian King under contract with the U.S. Army Corps of Engineers (Norton et al., 1973). Further development included the introduction of one-dimensional elements, state-of-the-art pre- and post-processing data programs, and the use of elements with curved borders. Graphic pre- and post-processing routines are supplied by Aquaveo through a software package called the Surface-water Modeling System or SMS. SMS is a front- and back-end software package that allows the user to easily modify model parameters (such as geometry, element coefficients, and boundary conditions), as well as view the model results and download specific data types. While the RMA model is essentially used without cost or constraint, the SMS software package requires site licensing for use.

4.2 MODEL SETUP

There are three main steps required to update and implement the hydrodynamic model:

- Grid generation
- Boundary condition specification
- Calibration

The extent of the revised finite element mesh was the same as the original MEP model, the boundaries were generated using digital aerial photographs from the MassGIS online orthophoto database. Shorelines from the digital orthophotography were digitized to provide landward limits of the two-dimensional model grid. A time-varying water surface elevation boundary condition (measured tide) was specified along the southern open boundary of the system based on the tide gage data collected in Vineyard Sound.

Once the mesh and boundary conditions were set, the model was recalibrated to ensure accurate predictions of tidal circulation. Various friction and eddy viscosity coefficients were adjusted, through several model calibration simulations for the system, to obtain agreement between measured and modeled water surface elevations. The recalibration procedures required the revised hydrodynamic model to meet the same calibration tolerances as the original MEP Model. The calibrated model provides the requisite information for future detailed examination of circulation, tidal attenuation, and inlet widening alternatives.
4.2.1 Grid Generation

The finite element grid generation process was completed through use of the SMS package. Digital aerial orthophotos from 2009 and bathymetry survey data were imported to SMS, and a finite element grid was generated to represent Bournes Pond. The entire system was represented by two-dimensional (depth-averaged) elements. The finite element grid for the system provided the detail necessary to evaluate accurately the variation in hydrodynamic properties within the estuary. Fine resolution was required to simulate channel constrictions through the flood shoal and along the length of the inlet which significantly impacts the system hydrodynamics. The completed grid consists of 4,176 nodes, which describe 1,773 two-dimensional (depth averaged) quadratic elements. The completed grid is shown in Figures 4-1 to 4-2. Reference water depths at each node of the model were interpreted from the bathymetry data. The final interpolated grid bathymetry is shown in Figure 4-3.

Grid resolution was governed by two factors: 1) expected flow patterns, and 2) the bathymetric variability of the region. Relatively fine grid resolution was employed where complex flow patterns were expected. For example, smaller node spacing was utilized across the flood and ebb shoal and through the inlet to provide a more detailed analysis in this region of rapidly varying flow. Widely spaced nodes were often employed in areas where flow patterns are not likely to change dramatically, such as in uniform sections along the pond where there were no significant constrictions or bathymetric variations.

Figure 4-1. Details of the finite element mesh around the inlet to Bournes Pond.
Figure 4-2. The hydrodynamic model finite element mesh developed for Bournes Pond system to examine widening the inlet.
Figure 4-3. Bathymetric data interpolated to the finite element mesh of hydrodynamic model. Depth contours are in feet referenced to NAVD88.

4.2.2 Boundary Condition Specification

Three types of boundary conditions were employed for the hydrodynamic model: 1) "slip" boundaries, 2) tidal elevation boundaries, and 3) flow boundaries representing freshwater flow into the system. All of the elements with land borders have "slip" boundary conditions, where the direction of flow was constrained shore-parallel. The model generated all internal boundary conditions from the governing conservation equations.
The hydrodynamic model was forced at the open boundary using water elevations measurements. The measured time series of water levels consists of all physical processes affecting variations of water level: tides, winds, atmospheric pressure, and other non-tidal oscillations of the water surface. The rise and fall of the tide in Vineyard Sound is the significant driving force for estuary circulation in Bournes Pond. Dynamic (time-varying) model simulations specified a new water surface elevation at the downstream boundary every 10 minutes. The model specifies the water elevation at the boundary, and uses this value to calculate water elevations at every nodal point within the system, adjusting each value according to solutions of the model equations. For example, changing water levels in Vineyard Sound produce variations in surface slopes within the system; these slopes drive water either into the system (if water is higher in the sound) or out of the system (if water is higher in the pond).

4.3 CALIBRATION

After developing the finite element grid and specifying boundary conditions, the model was recalibrated to match the calibration parameters that were set with the original MEP model. Calibration ensured the model predicts accurately what was observed during the field measurement program. Numerous model simulations were required to calibrate the model, with each run varying specific parameters such as friction coefficients, turbulent exchange coefficients, and minor modifications to the system bathymetry (aimed at improving the grid where data was sparse) to achieve a best fit to the data.

Calibration of the hydrodynamic model required a close match between the modeled and measured tides at each of the gage locations where tides were measured. Initially, the model was calibrated by the visual agreement between modeled and measured tides. To refine the calibration procedure, water elevations were output from the model at the same locations in the system where tide gages were installed, and the data were processed to calculate standard error as well harmonic constituents (of both measured and modeled data) over the 6-day calibration period. The amplitude and phase of four primary tidal constituents (K1, M2, M4, and M6) were compared and the corresponding errors for each were calculated. The intent of the calibration procedure is to minimize the error in amplitude and phase of the individual constituents. In general, minimization of the M2 amplitude and phase becomes the highest priority, since this is the dominant constituent based on amplitude and overall energy. The M2 constituent represents the largest semi-diurnal (twice daily) lunar component. Emphasis is also placed on the M4 and K1 constituents, as the M4 constituent represents a higher frequency harmonic of the M2 lunar tide (twice the frequency of the M2), results from frictional dissipation of the M2 tide in shallow water and hence is important to simulate tide propagation upriver. The K1 solar component represents the largest diurnal (once daily) tide constituents. M6 represents the shallow water overtide of the M2 semi-diurnal constituent.

The calibration was performed for an approximate five-day period, beginning 0000 hours EDT October 17, 2012 and ending 0400 EDT October 22, 2012. The calibration period was preceded by a 24-hour model spin-up period to ensure the model was stable and accurately performing the tidal hydrodynamic simulation. The selected time period spanned the transition from a new moon which represents the spring (bi-monthly maximum) tide ranges, also where tidal currents are greatest, to neap tides. Throughout the calibration period, the tide range was between 2 and 3 feet. The ability to model a range of flow conditions is a primary advantage of a numerical tidal circulation model. Modeled tides were evaluated for time (phase) lag and height damping of dominant tidal constituents. The calibrated model was used to analyze existing detailed flow patterns and compute residence times.
4.3.1 Friction Coefficients

Friction inhibits flow along the bottom of river channels or other flow regions where water depths can become shallow and velocities relatively high. Friction is a measure of the channel roughness, and can cause both significant amplitude attenuation and phase delay of the tidal signal and retard tidal flow. Friction was approximated in RMA-2 as a Manning coefficient. First, Manning's friction coefficient values of 0.028 were specified for all elements. This value corresponds to typical Manning's coefficients determined experimentally in smooth winding earth-lined channels with some cobble and gravel along the sides.

To improve model accuracy, friction coefficients were varied throughout the model domain, after initially matching the Manning's coefficients were matched to bottom type. Final model calibration runs incorporated various specific values for Manning's friction coefficients, depending upon flow damping characteristics of separate regions within the estuary. Manning's values for different bottom types were initially selected based ranges provided by the Open Channel Hydraulics (Chow, 1959), and values were incrementally changed when necessary to obtain a close match between measured and modeled tides. Small changes in these values did not change the accuracy of the calibration.

4.3.2 Turbulent Exchange Coefficients

Turbulent exchange coefficients approximate energy losses due to internal friction between fluid particles. The significance of turbulent energy losses increases where flow is swifter, such as inlets and bridge constrictions. According to King (1990), these values are proportional to element dimensions (numerical effects) and flow velocities (physics). In most cases, the modeled systems were relatively insensitive to turbulent exchange coefficients because there were no regions of strong turbulent flow. Typically, model turbulence coefficients were set between 20 and 65 lb-sec/ft².

4.3.3 Comparison of Modeled Tides and Measured Tide Data

Several calibration model runs were performed to determine how changes to various parameters (e.g. friction and eddy viscosity coefficients) affected the model results. These trial runs achieved good agreement between the model simulations and the field data. Comparison plots of modeled versus measured water levels at each gage locations is presented in Figures 4-4 and 4-5. At all gaging stations RMS errors were less than 0.08 ft (~1.0 inch) and computed R² correlation was better than 0.98. Errors between the model and observed tide constituents were less than 1.0 inches for all locations, and generally less than an half-inch, suggesting the model accurately predicts tidal hydrodynamics within the Bournes Pond.
Figure 4-4. Comparison of water surface variations simulated by the model (dashed red line) to those measured within the system (solid blue line) for the calibration time period, for the Vineyard Sound gage.

Figure 4-5. Comparison of water surface variations simulated by the model (dashed red line) to those measured within the system (solid blue line) for the calibration time period in Bournes Pond.
Although visual calibration achieved reasonable modeled tidal hydrodynamics, further tidal constituent calibration was required to quantify the accuracy of the models. Calibration of $M_2$ constituent was the highest priority since $M_2$ constituent accounted for a majority of the forcing tidal energy. Due to the duration of the model runs, four dominant tidal constituents were selected for constituent comparison: $K_1$, $M_2$, $M_4$, and $M_6$. Measured tidal constituent amplitudes and time lags ($\phi_{\text{lag}}$) shown in Table 4-1 for the calibration period differ from those in Table 3-2 because constituents were computed for only the 5-day section of the tide record represented in Table 3-2. Table 4-1 compares tidal constituent amplitude for modeled and measured tides within Bournes Pond. The constituent calibration resulted in excellent agreement between modeled and measured tides in Bournes Pond. The errors associated with tidal constituent amplitude were less than 0.1 ft, which is better than the order of the accuracy of the tide gage ($\pm 0.12$ ft). The comparisons of tidal constituent phase are shown in Table 4-1 for constituent $M_2$ and $M_4$; $K_1$ and $M_6$ were not compared since they do not represent the dominate tidal constituents. Time lag errors less than the time increment resolved by the model (1/6 hours or 10 minutes), indicating good agreement between the model and data calibration.

<table>
<thead>
<tr>
<th>Table 4-1.</th>
<th>Comparison of Tidal Constituents calibrated RMA2 model versus measured tidal data for the period October 17 to October 22, 2012.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Model Verification Run</strong></td>
<td><strong>Constituent Amplitude (ft)</strong></td>
</tr>
<tr>
<td>Location</td>
<td>$M_2$</td>
</tr>
<tr>
<td>Bournes Pond</td>
<td>0.82</td>
</tr>
<tr>
<td>Measured Tidal Data</td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td><strong>Constituent Amplitude (ft)</strong></td>
</tr>
<tr>
<td>$M_2$</td>
<td>$M_4$</td>
</tr>
<tr>
<td>Bournes Pond</td>
<td>0.87</td>
</tr>
<tr>
<td><strong>Error</strong></td>
<td></td>
</tr>
<tr>
<td>Location</td>
<td><strong>Constituent Amplitude (ft)</strong></td>
</tr>
<tr>
<td>$M_2$</td>
<td>$M_4$</td>
</tr>
<tr>
<td>Bournes Pond</td>
<td>0.04</td>
</tr>
</tbody>
</table>

4.4 EVALUATION OF THE INLET WIDENING ALTERNATIVE FOR BOURNES POND

The calibrated numerical model serves as a useful tool that can be used to accurately simulate design alternatives and analysis tidal hydrodynamics within Bournes Pond. Using model inputs of bathymetry and tide, current velocities and flow rates can be determined at any point in the model domain. This is a very useful feature of a hydrodynamic model, where a limited amount of collected data can be expanded to determine the physical attributes of the system in areas where no physical data record exists.

The analysis to determine the appropriate inlet width began by evaluating a series of increasing inlet widths. By increasing the inlet width, the volume of water entering the system over each tidal cycle, which is defined as tidal prism, is increased. The increase in tidal prism results in improved tidal flushing due to a large volume of water being exchanged over each cycle. The tidal prism will increase as the inlet width increases until the tidal attenuation through the inlet system approaches zero. However, once the inlet becomes too wide, the movement of tidal flows in and out of the system no longer can sustain the inlet cross-section as wave-
induced sediment transport along the coast will cause shoaling and currents through the inlet would not be strong enough to transport the sediment out of the inlet throat. By iterating the width of the inlet while accounting for changes in tidal velocity, the inlet cross section can be tuned to optimize the average maximum peak tidal velocities through the inlet channel to ensure cross-sectional stability.

Friction coefficients, turbulent exchange coefficients, forcing conditions, and numerical formulations were all carried forward from the calibrated hydrodynamic model for the evaluation of the widened the inlet. The purpose of the hydraulic modeling is to determine changes and trends that occur in Bournes Pond as a result of the implementation of the proposed inlet modifications. Therefore, no alterations were made to the model parameters (e.g. friction and turbulent exchange coefficients).

The initial inlet sizing utilized a uniform channel through Menauhant Beach, with no structural elements from a bridge structure changing the tidal flow characteristics. A graph of the average maximum peak velocities through the inlet channel at varying channel widths is shown in Figure 4-6. The model results illustrate that as the inlet width increases from the existing width (50-feet), the maximum tidal velocities decrease until the flow is no longer able to transport sediment. For sand-sized material, maintaining a maximum tidal current velocity of approximately 3 feet per second generally inhibits channel shoaling. At larger inlet widths, the cross section is no longer stable and would either shoal in or require frequent dredging to maintain the cross section. The goal for the inlet widening at Bournes Pond is to improve tidal flushing to mitigate for nutrient loading to the system, therefore it is important that the inlet remain stable so flushing is consistent and not require significant maintenance to sustain the inlet. The analysis determined that an inlet width of approximately 90 feet would increase tidal flushing and would maintain the inlet cross-sections under typical tidal conditions.

After determining the optimal cross-sectional area, four (4) bridge/culvert alternatives were developed for Menauhant Road crossing over the widened inlet. A full discussion of the bridge selection process and criteria are presented in the main body of the Technical Memorandum. The brief description of the four alternatives is below:

- Scenario 1: Single span bridge providing a 90-foot inlet.
- Scenario 2: Two span bridge with 45 foot opens with a single pier supporting the bridge within the channel.
- Scenario 3: Existing Menauhant Road Bridge crossing with two 19-foot wide box culverts placed east of the existing inlet.
- Scenario 4: Five 19-foot wide box culverts spanning the inlet.
Figure 4-6. Average maximum peak velocities for flood and ebb tides through the inlet to Bournes Pond. Inlet velocities that are greater than the sediment movement threshold indicate a stable inlet channel that can be generally maintained by tidal currents.

The selection process with the town-appointed oversight committee selected Scenario 2, the double span bridge over the widened inlet as the preferred alternative. The hydrodynamic model was updated to reflect the bridge pier and abutment configuration for the double span alternative. The proposed bridge will be a double span structure, with a total length of 93 feet. The bridge is supported on vertical walled pile supported bridge abutments, with a pile supported pier centered between the two spans. The inlet jetties will be approximately 93 feet apart with 1.5H:1V side slopes. The channel bottom will be at approximately -6.0 feet NAVD88. The revised hydrodynamic model grid is shown in Figure 4-7. The finite element grid areas extending beyond the immediate area around the bridge and inlet remain unchanged from the calibrated model. This allows the for the proposed bridge layout to be evaluated without concerns about grid variations altering flow characteristics elsewhere in the system that have been calibrated and validated.
Figure 4-7. Details of the refined finite element mesh around the proposed double span bridge.

The model was re-run over a series of inlet widths to examine the sensitivity of the tidal velocities through the inlet relative to minor changes in inlet width and the resulting influence on tidal prism volumes entering the pond. The sensitivity analysis confirmed that the 93 foot bridge with two 45-foot spans and a 3 foot wide pier centered in the channel would maximize flushing, while balancing inlet velocities to minimize maintenance dredging requirements.

Examining the differences in water surface elevation between the widened inlet and existing conditions at the tide station in Bournes Pond reveals that the modifications to the inlet will have appreciable effect upon the propagation of tide into Bournes Pond. This was anticipated, due to almost doubling the size of the inlet from existing conditions. Results from the numerical model are presented in Figure 4-8. In the top plot, tide attenuation is apparent by the higher elevation of the low tides, and also by the time delay of the tide signal inside the pond. In the bottom plot, tidal attenuation is dramatically reduced for the proposed double span 90 ft-wide inlet, to the point where there is little difference between the range and phase of the Bournes Pond tide signal relative to the offshore Vineyard Sound tides. The changes in tidal delays between the inlet configurations demonstrates that the widened inlet provides a significant improvement to the tidal attenuation, as shown in reduction of the $M_2$ tidal delay in Table 4-2. The delays in Table 4-2 differ from those in Table 3-3 because constituents were computed for a 5-day section of the entire recorded represented in Table 3-3. The reduction in tidal attenuation demonstrates that the proposed inlet width has maximized the potential flushing within Bournes Pond.
Figure 4-8. Plots showing a comparison of typical tides for modeled existing conditions (top plot) and proposed 90 ft-wide double span bridge inlet widen project (bottom plot) to Bournes Pond.

| Table 4-2. M₂ tidal attenuation computed within hydrodynamic model for Bournes Pond. |
|---|---|---|
| Location  | Existing M₂ Delay (minutes) | Proposed M₂ Delay (minutes) |
| Vineyard Sound | --- | --- |
| Bournes Pond | 52.5 | 26.5 |

Examining the predicted flow patterns in inlet reveals minor changes between the inlet configurations. The magnitude of the current speed through the inlet is reduced, but the flow patterns and pathways are very similar. The flood tide accelerates through the inlet and accelerates beneath bridge as the cross section decreases and splits due to the bridge pier. The depth-averaged velocities regularly peak around 2.5-3.3 ft/s on flooding tides, while on ebb tides the velocities are lower around 2.0-2.7 ft/s. This is due to an asymmetry in the length of the flood and ebb tides, where the duration of the flood tide is shorter than the ebb tide. The longer duration ebb tide decreases the flow velocities needed to transport equal volumes of water out of the estuary system. Examining flow immediately around the inlet shows an asymmetry of the flow entering the system due to the differing jetty lengths, but does not result in adverse conditions as a result of the asymmetry. A close-up of the model output is presented in Figure 4-9, which shows contours of flow velocity, along with velocity vectors which indicate the direction and magnitude of flow, for a single model time-step, at the portion of the tide where maximum ebb velocities occur at the bridges. On the ebb tide, the changes associated with the
widened inlet result in minor changes to the flow patterns. The influence of the new bridge pier is similar to the changes observed on the flood tide. The flow velocities around the abutments and pier are reduced due to the wider cross section.

Figure 4-9. Example of hydrodynamic model output with the new bridge for a single time step where maximum flood velocities occur for this tide cycle. Color contours indicate flow velocity, and vectors indicate the direction and magnitude of flow.

Tidal exchange and nutrient loading are the primary mechanisms controlling estuarine water quality within Bournes Pond. The goal of this analysis has been to determine if enlarging the inlet to increase tidal exchange offers a more effective approach for improving water quality in the pond over other nutrient management approaches. To quantify the changes to tidal exchange between existing conditions and the proposed widened inlet, the volume of water (based on the mean volumes computed for the simulation period) in the system and tidal prism (or volume entering the system through a single tidal cycle) were computed. These provide an indication of the water quality changes that could be realized by improving the inlet. Table 4-3 present the system volumes and tidal prisms for both inlet configurations in cubic feet. The increased inlet width causes a decrease in the mean volume of the system while increasing the volume of water exchanged; therefore, the wider inlet will improve water quality within Bournes Pond. The increased inlet channel enhances the efficiency of the system, allowing the water surface within the pond to respond more quickly to fluctuations in the offshore tides. The enhanced efficiency increases the volume of water entering and exiting the system.
Table 4-3. Average high, mid and low tide volumes, with mean tide prism for Bournes Pond, for existing inlet conditions, and for the proposed 90 ft-wide double span bridge inlet modification.

<table>
<thead>
<tr>
<th></th>
<th>existing inlet</th>
<th>90 ft-wide inlet</th>
<th>% change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mean High Tide Volume</td>
<td>26,495,012 ft³</td>
<td>26,760,038 ft³</td>
<td>1.0%</td>
</tr>
<tr>
<td>Mean Tide Volume</td>
<td>20,628,030 ft³</td>
<td>20,290,713 ft³</td>
<td>-1.6%</td>
</tr>
<tr>
<td>Mean Low Tide Volume</td>
<td>14,761,049 ft³</td>
<td>13,821,388 ft³</td>
<td>-6.4%</td>
</tr>
<tr>
<td>Mean Prism Volume</td>
<td>11,733,963 ft³</td>
<td>12,938,650 ft³</td>
<td>10.3%</td>
</tr>
</tbody>
</table>

Figure 4-10. Example of hydrodynamic model output throughout for a single time step where maximum flood velocities occur for this tide cycle. Color contours indicate flow velocity, and vectors indicate the direction and magnitude of flow.
5. WATER QUALITY MODELING

In general, it is anticipated that structural changes that reduce tidal attenuation in an estuary will decrease TN concentrations, resulting in improved water quality. Following the hydraulic study of Bournes Pond inlet, the optimized inlet size was evaluated relative to potential water quality improvements. This optimized inlet maintains typical tidal current conditions that will ensure a stable inlet system, as peak daily current velocities will exceed 3 feet per second. To assess water quality improvements, a two-dimensional finite element water quality model, RMA-4, was employed to study the effects of nitrogen loading in the Bournes Pond estuary systems. The RMA-4 model has the capability for the simulation of advection-diffusion processes in aquatic environments. It is the constituent transport model counterpart of the RMA-2 hydrodynamic model used to simulate the fluid dynamics of the Bournes Pond. Like RMA-2 numerical code, RMA-4 is a two-dimensional, depth averaged finite element model capable of simulating time-dependent constituent transport. The RMA-4 model was developed with support from the US Army Corps of Engineers (USACE) Waterways Experiment Station (WES), and is widely accepted and tested. Applied Coastal staff has utilized this model in water quality studies of other tidal embayments within southeastern Massachusetts as part of the Massachusetts Estuaries Project (MEP).

The overall approach involves modeling total nitrogen as a non-conservative constituent, where bottom sediments act as a source or sink of nitrogen, based on local biochemical characteristics. This modeling represents summertime conditions, when algal growth is at its maximum. Total nitrogen modeling is based upon various data collection efforts and analyses presented in previous sections of this report. Nitrogen loading information was derived from the Cape Cod Commission watershed loading analysis (based on the USGS watersheds), as well as the measured bottom sediment nitrogen fluxes. Water column nitrogen measurements were utilized as model boundaries and as calibration data. The updated hydrodynamic model output provided the remaining information (tides and bathymetry) needed to parameterize the water quality model of the Bournes Pond system.

Since the overall purpose of the work was to determine whether inlet widening represents a cost-effective method for nutrient management in the Bournes Pond system, the approach required comparison of the water quality improvements associated with the optimized inlet relative to a scenario that produced similar water quality in the estuary as a result of loading reductions. An outline of the approach included:

1. The updated RMA-2 hydrodynamic model developed to investigate inlet widening was utilized as the basis for the RMA-4 total nitrogen model. The updated hydrodynamic model included a more detailed model grid than the original MEP analysis and was designed to evaluate the near-field flow regime through the inlet, as well as incorporating updated bathymetric survey information.
2. The updated RMA-4 model for existing conditions incorporated the nitrogen loading that formed the basis for the MEP technical report. In addition, water quality data utilized to calibrate the model also was derived from the MEP analysis. The updated RMA-4 total nitrogen model was then calibrated to existing conditions.
3. The RMA-4 total nitrogen model also was run for the updated hydrodynamic conditions associated with the optimized tidal inlet. This water quality model run also utilized existing loading conditions.
4. The existing conditions RMA-4 model then formed the basis for determining the effective amount of total nitrogen that is removed from the system by widening the inlet and increasing the tidal flushing. The modeled reduction of upland nitrogen load was
performed by removing load for the southern portion of the estuary first and then sequentially removing load further to the north until the TN conditions at the sentinel station matched the modeled concentration of the widened inlet under existing loading. In this manner, a cost per pound removal of nitrogen could be computed or alternatively, the equivalent load reduction related to number of households also could be computed.

5.1 WATER QUALITY MODELING CALIBRATION

The existing conditions water quality model required recalibration after existing hydrodynamic model was updated. The water quality model was utilized the updated hydrodynamics model, but utilized the data from the original MEP Report for external nitrogen loads from the watersheds, internal nitrogen loads from the sediment (benthic flux), measurements of nitrogen in the water column and salinity measurements.

The three primary nitrogen loads to Bournes Pond have not changed from the MEP Report. Mass loading of nitrogen into water quality model included:

1. Sources developed from the results of the watershed analysis,
2. Estimates of direct atmospheric deposition,
3. Summer benthic regeneration; and
4. Point source input developed from measurement of the freshwater entering through Bournes Brook.

Nitrogen loads from each separate sub-embayment watershed were distributed across the sub-embayment. For example, the combined watershed direct atmospheric deposition loads for Isreals Cove were evenly distributed at grid cells that formed the perimeter of the embayment. Benthic regeneration loads were distributed among another sub-set of grid cells which are in the interior portion of each basin.

The loadings used to model present conditions in Bournes Pond systems are given in Table 5-1. Watershed and depositional loads were taken from the results of the analysis of Section IV of the MEP Report. Summertime benthic flux loads were computed based on the analysis of sediment cores in Section IV of the MEP Report. The area rate (g/sec/m²) of nitrogen flux from that analysis was applied to the surface area coverage computed for each sub-embayment, resulting in a total flux for each embayment is listed in Table 5-1.

| Table 5-1. Sub-embayment and surface water loads used for total nitrogen modeling of the Bournes pond systems, with total watershed N loads, atmospheric N loads, and benthic flux. These loads represent present loading conditions for the listed sub-embayments. |
|---|---|---|
| sub-embayment | watershed load (kg/day) | direct atmospheric deposition (kg/day) | benthic flux net (kg/day) |
| Bournes Pond | 9.61 | 1.61 | 28.45 |
| Isreals Cove | 2.05 | 0.26 | -0.32 |
| Surface Water Sources | | | |
| Bournes Brook (Bournes Pond) | 3.29 | | |

36
In addition to mass loading boundary conditions set within the model domain, concentrations along the model open boundaries were specified. The model uses concentrations at the open boundary to Nantucket Sound during the flooding tide periods of the model simulations. TN concentrations of the incoming water are set at the value designated for the open boundary. The boundary concentration in Nantucket Sound was set at 0.285 mg/L, based on SMAST data from the Sound. At the head of the system, TN concentrations for Bournes Brook were calculated based on the watershed loading and freshwater recharge rate.

The model was then recalibration to the standards set forth in the MEP Report. The recalibration involved varying the model dispersion coefficients so that model output of nitrogen concentrations matched measured data. Comparisons between model output and measured nitrogen concentrations were compared for the calibration, the root mean squared (rms) errors are less than 0.03 mg/L for the Bournes Pond Nitrogen model, and an $R^2$ correlation coefficient of 0.97 which indicates the model has an exceptional fit for Bournes Pond and matches the calibrated model used within the MEP Report.

A contour plot of calibrated model output is shown in Figure 5-1; the color contours indicate nitrogen concentrations throughout the model domain. The output in these figures show average total nitrogen concentrations, computed using the full 5-tidal-day model simulation output period.
Figure 5-1. Contour plot of average total nitrogen concentrations from results of the present conditions loading scenario, for Bournes Pond. The approximate location of the sentinel threshold station for Bournes Pond (B3) is shown.
5.2 WIDENED INLET EVALUATION

To gauge the influence of the widened inlet on the total nitrogen concentrations within Bournes Pond, the existing water quality was rerun using the hydrodynamic model for the widened inlet to examine the influence increase tidal exchange has upon the system. The external and internal loads to system were unchanged from the existing conditions model as shown in Table 5-2.

A second scenario was run based on the existing conditions RMA-4 model to determine the effective amount of total nitrogen that was removed from the system by widening the inlet and increasing the tidal flushing. The modeled reduction of upland nitrogen load was performed by removing load for the southern portion of the estuary first and then sequentially removing load further to the north until the TN conditions at the sentinel station matched the modeled concentration of the widened inlet under existing loading. The equivalent load reduction required in the existing conditions model to provide an equivalent load reduction to the widened inlet is shown in Table 5-2. The 56-percent reduction in nitrogen loading equates to a reduction of 1,995 kg per year within the watershed when compared to the 93-ft Double Span Bridge.

<table>
<thead>
<tr>
<th>sub-embayment</th>
<th>present load (kg/day)</th>
<th>Widened Inlet (kg/day)</th>
<th>Widened Inlet % change</th>
<th>Equivalent Load Reduction (kg/day)</th>
<th>Equivalent Load Reduction % change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bournes Pond</td>
<td>9.61</td>
<td>9.61</td>
<td>0.0%</td>
<td>4.14</td>
<td>-56.9%</td>
</tr>
<tr>
<td>Israels Cove</td>
<td>2.05</td>
<td>2.05</td>
<td>0.0%</td>
<td>2.05</td>
<td>0.0%</td>
</tr>
<tr>
<td>Surface Water Sources</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bournes Brook</td>
<td>3.29</td>
<td>3.29</td>
<td>0.0%</td>
<td>3.29</td>
<td>0.0%</td>
</tr>
</tbody>
</table>

The results from each of the water quality models, widened inlet and equivalent load reduction are shown in Tables 5-3 and 5-4. The nitrogen concentrations decrease system wide in both scenarios. The total N concentrations have the most significant reduction in the lower portion of Bournes Pond, and steady decrease up the system as expected. The widened inlet produces slightly higher reductions in nitrogen concentrations in the lower portion of the system do to the increased exchange with waters from Nantucket Sound. In the upper reaches, the reductions are slightly lower since the loading to the system remains unchanged, while in the equivalent modeling scenario the loading is reduced and marginally reduces total nitrogen at higher percentage. Color contours of model output for the widened inlet scenario is present in Figure 5-2 and for the equivalent load reduction in Figure 5-3. The range of nitrogen concentrations shown is the same as for the plot of present conditions in Figure 5-1, which allows direct comparison of nitrogen concentrations between inlet configuration and loading scenarios.
### Table 5-3. Comparison of model average total N concentrations from present loading and present loading with a widen inlet, with percent change against background nitrogen concentrations, for the Bournes Pond systems. Sentinel threshold station for Bournes Pond is B3 and is in bold print.

<table>
<thead>
<tr>
<th>Sub-Embayment</th>
<th>monitoring station</th>
<th>present (mg/L)</th>
<th>Widened Inlet (mg/L)</th>
<th>% change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bournes Pond - head</td>
<td>B1</td>
<td>0.927</td>
<td>0.915</td>
<td>-1.9%</td>
</tr>
<tr>
<td>Bournes Pond - upper</td>
<td>B2</td>
<td>0.837</td>
<td>0.815</td>
<td>-3.9%</td>
</tr>
<tr>
<td><strong>Bournes Pond - mid</strong></td>
<td><strong>B3</strong></td>
<td><strong>0.578</strong></td>
<td><strong>0.555</strong></td>
<td><strong>-7.8%</strong></td>
</tr>
<tr>
<td>Bournes Pond - lower</td>
<td>B4</td>
<td>0.412</td>
<td>0.395</td>
<td>-13.3%</td>
</tr>
<tr>
<td>Israels Cove</td>
<td>B5</td>
<td>0.581</td>
<td>0.564</td>
<td>-6.0%</td>
</tr>
<tr>
<td>Bournes Pond - lower</td>
<td>B6</td>
<td>0.338</td>
<td>0.330</td>
<td>-15.6%</td>
</tr>
</tbody>
</table>

### Table 5-4. Comparison of model average total N concentrations from present loading and present conditions loading scenario with reductions in septic loading to match TN concentrations at the sentinel threshold station with the widened inlet scenario, with percent change against background nitrogen concentrations, for the Bournes Pond systems. Sentinel threshold station for Bournes Pond is B3 and is in bold print.

<table>
<thead>
<tr>
<th>Sub-Embayment</th>
<th>monitoring station</th>
<th>present (mg/L)</th>
<th>Equivalent Load Reduction (mg/L)</th>
<th>% change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bournes Pond - head</td>
<td>B1</td>
<td>0.927</td>
<td>0.912</td>
<td>-2.3%</td>
</tr>
<tr>
<td>Bournes Pond - upper</td>
<td>B2</td>
<td>0.837</td>
<td>0.814</td>
<td>-4.1%</td>
</tr>
<tr>
<td><strong>Bournes Pond - mid</strong></td>
<td><strong>B3</strong></td>
<td><strong>0.578</strong></td>
<td><strong>0.555</strong></td>
<td><strong>-7.8%</strong></td>
</tr>
<tr>
<td>Bournes Pond - lower</td>
<td>B4</td>
<td>0.412</td>
<td>0.399</td>
<td>-10.0%</td>
</tr>
<tr>
<td>Israels Cove</td>
<td>B5</td>
<td>0.581</td>
<td>0.563</td>
<td>-6.3%</td>
</tr>
<tr>
<td>Bournes Pond - lower</td>
<td>B6</td>
<td>0.338</td>
<td>0.332</td>
<td>-11.4%</td>
</tr>
</tbody>
</table>
Figure 5-2. Contour plot of average total nitrogen concentrations from results of the widened inlet with two 45-foot bridge openings with present conditions loading scenario, for Bourne Pond. The approximate location of the sentinel threshold station for Bourne Pond (B3) is shown.
Figure 5-3. Contour plot of average total nitrogen concentrations from results of the present conditions loading scenario with reductions in septic loading to match TN concentrations at the sentinel threshold station with the widened inlet scenario, for Bournes Pond. The approximate location of the sentinel threshold station for Bournes Pond (B3) is shown.
Appendix BP-1-3
Backup Information for Wastewater Nitrogen Removal Cost Estimates
<table>
<thead>
<tr>
<th>Nitrogen Concentrations:</th>
<th>mg/l</th>
<th>Recharge Rates:</th>
<th>in/yr</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wastewater</td>
<td>35</td>
<td>Impervious Surfaces</td>
<td>40</td>
</tr>
<tr>
<td>Road Run-off</td>
<td>1.5</td>
<td>Natural and Lawn Areas</td>
<td>27.25</td>
</tr>
<tr>
<td>Roof Run-off</td>
<td>0.75</td>
<td>Water Use/Wastewater:</td>
<td></td>
</tr>
<tr>
<td>Direct Precipitation on Embayments and Ponds</td>
<td>1.09</td>
<td>For Parcels wo/water accounts:</td>
<td>gpd</td>
</tr>
<tr>
<td>Natural Area Recharge</td>
<td>0.072</td>
<td>Single Family Residence</td>
<td>151</td>
</tr>
<tr>
<td>Fertilizer:</td>
<td></td>
<td>Commercial Properties</td>
<td>122 per 1,000 ft² of building</td>
</tr>
<tr>
<td>Average Residential Lawn Size (ft²) *</td>
<td>5,000</td>
<td>Industrial Properties</td>
<td>112 per 1,000 ft² of building</td>
</tr>
<tr>
<td>Residential Watershed Nitrogen Rate (lbs/lawn) *</td>
<td>1.08</td>
<td>For Parcels w/water accounts:</td>
<td>Measured annual water use</td>
</tr>
<tr>
<td>Nitrogen Fertilizer Rate for golf courses, cemeteries, and public parks determined by site-specific information</td>
<td></td>
<td>Wastewater volume determined by multiplying water use by 0.9</td>
<td></td>
</tr>
</tbody>
</table>

Following the assignment of all parcels to individual watersheds, tables were generated for each of the 33 sub-watersheds summarizing water use, parcel area, frequency, sewer connections, private wells, and road area. As mentioned above, these tables were then condensed to 21 subwatersheds based upon the time of travel analysis (<10 yr vs. > 10 yr) discussed above.

The 21 individual sub-watershed assessments were then integrated to generate nitrogen loading tables relating to the each of the individual estuaries and their major components: Great Pond main stem Perch Pond, Upper (fresh) and Lower (fresh) Coonamessett River; Green Pond Estuary and Backus Brook; and Bournes Pond main stem, Israel Cove and Bournes Brook. The sub-embayments represent the functional embayment units for the Linked Watershed-Embayment Model’s water quality component.

For management purposes, the aggregated sub-embayment watershed nitrogen loads are partitioned by the major types of nitrogen sources in order to focus development of nitrogen management alternatives. Within the Great, Green and Bournes Pond systems the major types of nitrogen loads are: wastewater (septic systems and the Ashumet Plume), fertilizer, impervious surfaces, direct atmospheric deposition to water surfaces, and recharge within natural areas (Table IV-4). The output of the watershed nitrogen loading model is the annual mass (kilograms) of nitrogen added to the contributing area of component sub-embayments (and freshwater ponds and streams), by each land use category (Figures IV-4 a-c). This annual watershed nitrogen input is then adjusted for natural nitrogen attenuation during transport to the estuarine system before use in the embayment water quality sub-model. Natural attenuation within the upper watershed to each estuary is also directly measured (Section IV.2) and compared to the attenuated annual watershed nitrogen load from the land-use sub-model.
In order to estimate wastewater flows within the study area, MEP staff also obtained parcel by parcel water use information from the Town of Falmouth. This information included two years of water use information with the final reading in May 2003 for the majority of parcels, which are billed on a semiannual basis, and one complete year of data (mid-2002 to mid-2003) for approximately 300 accounts that are billed on a quarterly basis. Water use information was linked to the parcel and assessors data using GIS techniques. Water use for each parcel was converted to an annual volume for purposes of the nitrogen loading calculations. No wastewater treatment facilities (WWTFs) currently exist in the watersheds, but the nitrogen additions are included from the old MMR WWTF effluent discharge beds that are the source of the Ashumet Valley Plume of treated wastewater.

IV.1.2 Nitrogen Loading Input Factors

Wastewater/Water Use

All wastewater within the Great Pond, Green Pond, and Bournees Pond watersheds is returned to the aquifer through individual on-site septic systems. Wastewater based nitrogen loading from the parcels using on-site septic systems is based upon the measured water-use, nitrogen concentration in wastewater (35 mg N/L) and nitrogen loss estimates within the septic tank and soil adsorption system (25%). Loss in passage through the septic system used by MEP (Howes and Ramsey 2000, Weiskel and Howes 1991) is consistent with other regional studies (Brawley et al. 2000, Costa et al. 2001). The best quantitative information on Title 5 septic system nitrogen removals (21%-25%) was developed at the DEP’s Alternative Septic System Test Center at MMR. Multi-year monitoring of Title 5 septic system performance revealed that nitrogen removal within the septic tank was small (1%-3%), with most of the removal occurring within the soil adsorption system (Costa et al. 2001).

Wastewater engineering studies conventionally assume 90% of water used in a town is converted to wastewater (e.g., Massachusetts Water Resources Authority 1983, Stearns and Wheier, 1999). In order to check the reliability of parcel water use as a proxy for wastewater flow, average influent flow at three nearby WWTFs (Mashpee Commons, Willowbend, and the Town of Falmouth municipal facility) was compared to parcel water use within the respective service areas. These WWTFs, which are located to the east and west of the Great, Green and Bournees Pond watersheds have more diverse mixes of land uses within them, however, the analysis is useful as a local check of the 90% engineering assumption. The review of the WWTFs found that 79% of the Mashpee Commons (primarily commercial) water use is returned to the WWTF, 87% of the Willowbend water use is returned, and 87% of the water use in the Town of Falmouth sewer service area is returned to the WWTF. This analysis confirms that 90% return flow is an appropriate general adjustment when using water use in the nitrogen loading calculations within the Great Pond, Green Pond, and Bournees Pond watersheds.

While almost all of the developed parcels within the study watersheds have corresponding water use accounts 3% did not; all of these latter parcels are residential and are assumed to utilize private wells for drinking water. In order to complete the nitrogen loading, the average water use from parcels with water use accounts (Table IV-2) was applied to the parcels assumed to be on private wells. Of the 215 developed parcels in the study area without water use in the available database; 139 are in the Great Pond watershed, 9 are in the Green Pond watershed, and 67 are in the Bournees Pond watershed. Average water use was also used for determining nitrogen loads from new development determined in the buildout analyses.

In order to provide an independent validation of the residential water use average within the study area, MEP staff reviewed US Census population values. The state on-site wastewater
NEW SILVER BEACH WASTEWATER PROJECT
ACCOUNTING SUMMARY

Appropriated - TM
Art 72 11/97 $3,567,000
Art 24 04/02 $1,200,000
Art 29 11/06 $7,733,000
Total: $12,500,000

Borrowed
Bonded Total $1,650,000
SRF Loan Total $7,775,000
Total: $9,425,000

Cost Distribution Summary for Betterment
Total Project Cost for Betterment $9,103,055.82
Cost to Be Paid By Property Owners Through Betterments (70%) $6,372,139.07
Cost to Be Paid By Town (30%) $2,730,916.74

Number of Assessed Properties 231
Betterment Cost Per Property $27,585.02

→ $39,407 / property (2007 costs)
Scaled to 2013 at 1.03% / year
→ $47,050 / property

Use $47,000 / property in 2013.