04-1581-01.009

MARTIN SLOUGH ENHANCEMENT FEASIBILITY STUDY EUREKA, CALIFORNIA

Prepared for:
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Redwood Community Action Agency
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2006

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Winzler & Kelly Project No. 04-158101-009

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I. EXECUTIVE SUMMARY

The study area for the Martin Slough Enhancement Feasibility Study consists of the general flood plain between Swain Slough and the upper (second) Fairway Drive stream crossing in the lower Martin Slough watershed. The study area is located in and adjacent to the southeast portion of the City of Eureka, and is partially within the coastal zone. Existing problems that have been identified in the Martin Slough study area include obstructed fish access, poor fish habitat, poor sediment routing, lack of riparian habitat, and frequent prolonged flooding that has a negative economic impact on current land uses.

Winzler & Kelly Consulting Engineers teamed with Michael Love & Associates and Coastal Analysis LLC to develop an enhancement plan to improve fish access, enhance aquatic habitat, improve sediment transport, and reduce flooding impacts on land use activities within the study area.

To accomplish these goals, a Technical Advisory Committee (TAC) was established and TAC meetings were organized and scheduled by the Natural Resources Services (NRS) Division of Redwood Community Action Agency (RCAA) to foster discussion between interested stakeholders such as the property owners, regulators, and the design team. Throughout the process the TAC provided input and helped guide the project direction and content.

Martin Slough has a watershed area of approximately 5.4 square miles, and natural channel length of over 10 miles, with approximately 7.5 miles of potential salmonid fish habitat supporting coho salmon and cutthroat trout. However, the existing tidegates partially block upstream salmonid migration.

The lower portion of the watershed flows through low gradient bottomland containing the golf course and pastureland. Many of the stream channels flow from gulches that contain mature second-growth redwood forests. The upper portions of the watershed are either in urban settings, or are recently harvested timberlands slated for future residential or mixed use development.

Determining project hydrology was an important aspect of the research. Hydrographs were developed for existing and anticipated future land-use conditions to determine how changes in runoff characteristics influence effectiveness of the different project alternatives. Version 2.2.2 of the ACOE Hydrologic Engineering Center's Hydrologic Modeling System software (HEC-HMS), that simulates precipitation-runoff and flow routing processes, was utilized to compute hydrographs for selected rainfall events.

Summary of Alternatives

The following four alternatives were identified and refined as more information became available based on the results of the analysis conducted throughout the study.

Alternative 1: The No Action Alternative (Existing Conditions)

The No Action Alternative would leave the system as it exists today. This alternative is important for permitting considerations and also for comparing alternatives, allowing a familiar starting point for comparisons to be made.

Alternative 2: No Tidegates or Levee (Full Tidal Influence)

The No Tidegates or Levee (Full Tidal Influence) Alternative would result in removing the existing tidegates and the levee at Swain Slough. Based on land and tidal elevations, this alternative would open the majority of the project area to full tidal influence, allowing the system to transform back towards its pre-development state.

Alternative 3: New Tidegates and New Ponds (Muted Tide)
The New Tidegates and New Ponds (Muted Tide) Alternative would consist of removing the existing tidegates, installing new tidegates with a habitat door designed to create a muted tidal prism and facilitate fish passage, increasing the size of existing ponds and creating new ponds.

Alternative 4: New Tidegates, Storage Ponds, and Modified Channel (Muted Tide)
The New Tidegates, Storage Ponds, and Modified Channel (Muted Tide) Alternative is similar to Alternative 3, but includes improvements to the existing channel and a corresponding larger habitat door to accommodate the larger available tidal prism. This alternative consists of removing the existing tidegates, installing new tidegates with a habitat door designed to create a muted tide cycle and facilitate fish passage, increasing the size of existing ponds, creating new ponds, and making channel modifications throughout the project area.

Several different approaches were used to evaluate the alternatives. A simplified numerical model of tidegate hydraulics was created in a spreadsheet to allow for rapid analysis of the effectiveness of different tidegate designs in providing fish passage and flood routing within the project area. Fish passage analysis of the tidegates was conducted for each alternative. Passage conditions were evaluated using the stream crossing design criteria developed by NOAA Fisheries (2001) and CDFG (2002).

The geomorphic stability of enlarging the Martin Slough channel within the project area to increase conveyance area for both flood flows and a diurnal tidal exchange was analyzed using design guidelines developed for tidal channels. This was done because reintroducing a muted tide cycle into the project area would result in large volumes of water flowing up and down the channel with each tide cycle, changing the fluvial processes that maintain the channel.

To assist in determining potential impacts and evaluate potential permitting issues for the different alternatives, a wetland and biological reconnaissance investigation was conducted to determine the approximate size and location of wetlands, and sensitive plant and animal habitats within the potential footprint of the alternatives developed.

Hydraulic modeling of the alternatives was conducted with the two-dimensional finite-element model, ADCIRC (Luettich et al 1992). Objectives of the hydraulic modeling were to evaluate and compare alternatives in terms of inundation levels, inundation duration, and sediment transport for 2-year and 10-year storm events.

Graphical Comparison of Alternatives
The inundation effects of the different alternatives were evaluated based on the ADCIRC hydraulic model results over a seven day period using streamflows resulting from a 2 and 10 year rainfall event. Model results for each alternative were compared graphically. Comparisons show that Alternative 2 produces more inundation during high tide than during the respective rainfall events. Alternative 4 has the greatest potential to reduce inundation, but the model still shows

wide spread inundation during the peaks of the storm events. However, Alternative 4 indicates a faster recovery time resulting in less inundation time than the other alternatives. Although the duration that the fields and golf course are inundated is reduced, the ponds and channel retain water to provide aquatic habitat.

Quantitative Inundation Comparison of Alternatives

The TIDEGATE model, based on a simplified lump flow routing hydraulic model, was used to provide quantitative inundation comparisons for the different alternatives. Quantitative comparisons were evaluated based on the number of hours certain elevations are inundated. The quantitative information confirmed the basic conclusions drawn from the graphical comparisons.

Summary Comparison of Alternative Results

With four alternatives and twenty criteria to consider, the need arose for a method to help evaluate the alternatives. The following table provides a method for the overall comparison of results. With all the criteria listed alongside each alternative, comparisons are easier to make. In addition to qualitative written descriptions of how each alternative addresses the project criteria, the descriptions were given color codes to help convey the cumulative benefit of any one alternative compared to another alternative. The color green was chosen for the most benefit, yellow was chosen for some benefit, and red was chosen for the least benefit or potential negative influence such as the highest project cost. Project criteria with no improvement were left white. The criteria are not weighted or otherwise ranked, and therefore there is no implied best or worse alternative. In fact, each alternative has its benefits and potential problems and the determination of which alternative is “best” will depend on which criteria are most important to the individual(s) making the comparison.

This report concludes our preliminary planning-level analysis of four alternative conceptual plans for the enhancement of Martin Slough. This information is presented for use as a decision making tool to assist project stakeholders in the selection of a preferred alternative.

Table 1.1 Comparison of Alternatives

Criteria	Alternative 1 No Action Alternative (Existing Conditions)	Alternative 2 No Tidegates or Levee (Full Tidal Influence)	Alternative 3 New Tidegates and New Ponds (Muted Tide)	Alternative 4 New Tidegates, Ponds, and Modified Channel (Muted Tide)
Fish Passage and Fish Access for Juveniles and Adults				
1. Maximize Migration Access at Tidegates during Fish Migration Flows	No Improvement	Most Improvement	Some Improvement	Some Improvement
Fish Habitat				
2. Maximize Estuarine Habitat	No Improvement	Some Improvement	Some Improvement	Most Improvement
3. Increase Channel Complexity	No Improvement	No Improvement	No Improvement	Most Improvement
Riparian Corridor				
4. Increase Riparian Habitat	No Improvement	No Improvement	Some Improvement	Most Improvement
5. Increase Riparian Canopy	No Improvement	No Improvement	Some Improvement	Most Improvement
Water Quality				
6. Decrease Nutrient Impacts	No Improvement	Some Improvement	Some Improvement	Most Improvement
7. Decrease Sedimentation	No Improvement	Some Improvement	Some Improvement	Most Improvement
Wetlands				
8. Improve Wetland Habitat	No Improvement	Some Improvement	Most Improvement	Most Improvement
9. Increase Open Water Area of Wetlands	No Improvement	Some Improvement	Most Improvement	Most Improvement
10. Increase Diversity of Wetland Types	No Improvement	Some Improvement	Some Improvement	Most Improvement
Flood Impacts				
11. Reduce Flood Inundation Area	No Improvement	No Improvement/ Potentially Worse	Some Improvement	Most Improvement
12. Reduce Frequency of Flooding	No Improvement	No Improvement/ Potentially Worse	Some Improvement	Most Improvement
13. Minimize Duration of Flooding	No Improvement	No Improvement/ Potentially Worse	Some Improvement	Most Improvement
Existing Land Uses				
14. Maintain Agricultural Land Use	No Improvement	Likely Worse	Some Improvement	Most Improvement
15. Maintain Eureka Municipal Golf Course	No Improvement	Likely Worse	Some Improvement	Most Improvement
16. Allow for full Build-out Potential for City/County	No Improvement	No Improvement	Some Improvement	Most Improvement
17. Allow for Installation and Maintenance Access for City's Martin Slough Sewer Interceptor Project	No Improvement	No Improvement/ Potentially Worse	Some Improvement	Most Improvement
Project Permitting				
18. Consider ability to Obtain Permits	Permitting efforts for maintenance may increase with time	Potentially Very Difficult	Potentially Very Difficult	Moderate Effort Required
Cost of Improvements				
19. Consider Order of Magnitude Opinion of Probable Construction Costs	No Cost	Low Cost	Moderate Cost	Highest Cost
Project Maintenance				
20. Consider Need for Ongoing Maintenance	No Improvement	Potentially Worse in short term, lessoning over time	Potentially Worse	Most Improvement

II. INTRODUCTION, GOALS, AND SCOPE

1.0 INTRODUCTION

The Martin Slough Enhancement Project is located in and adjacent to the southeast portion of the City of Eureka, and terminates with its confluence with Swain Slough as shown in Figure 1.1. Martin Slough is the first tributary to Elk River via Swain Slough. The mouth of Martin Slough is separated from Swain's Slough by a levee and tidegates. The Martin Slough watershed includes both City and County jurisdictions, with the project area owned by the City of Eureka (approximately 120 acres) and a private landowner (approximately 40 acres). The project area is partially within the coastal zone.

The Martin Slough and Elk River estuary are part of the larger Humboldt Bay ecosystem that accommodates a variety of waterfowl, wading birds and shorebirds, several species of fish and other aquatic organisms, passerines, and raptors. Not much is known relative to the historic composition of the lower portions of Martin Slough. However, it is apparent from its elevation relative to tidewater and its geomorphic features that the lower portions of Martin Slough consisted of estuarine habitat, likely composed of some salt marsh and slough channels along with other more brackish water habitats. Although much of the historic estuary has been converted to other land use, some estuarine habitat still exists. That habitat has been severely degraded by the installation of tidegates at the confluence of Martin Slough with Swain Slough and other land management practices. These modifications also have had a pronounced effect on flood routing and sedimentation in the lower channel.

The Martin Slough watershed land use includes a mix of residential, agricultural, timberlands, and municipal infrastructure. Humboldt County's Eureka Community Plan includes future residential development of the southeastern portion of the Martin Slough watershed. This currently forested area will likely be eventually phased out of its current timber production zone (TPZ) status to allow for residential or mixed-use development. This conversion could modify the watershed hydrology and potentially result in increased storm water runoff. Its actual affect on peak flows within Martin Slough will be dependent on the measures taken by future development to address storm water runoff, currently set for no net increase by the County.

The project area is currently zoned Public Facility and Agriculture Exclusive. Municipal infrastructure directly within the project area includes the City maintained Fairway Drive, a natural gas line, an existing sewer line, a planned and partially designed sewage interceptor line, and the Eureka Municipal Golf Course. The Humboldt Community Services District also has existing sewer infrastructure near Fairway Drive.

Martin Slough has a watershed area of approximately 5.4 square miles, and natural channel length of over 10 miles with approximately 7.5 miles of potential salmonid fish habitat supporting coho salmon and cutthroat trout. However, the existing tidegates partially block upstream salmonid migration. The lower portion of the watershed flows through low gradient bottomland containing the golf course and pastureland. Many of the stream channels flow from gulches that contain mature second-growth redwood forests. The upper portions of the watershed are either in urban settings, or are recently harvested timber lands slated for future residential areas.

The Martin Slough Enhancement Feasibility Study area consists of the general flood plain between Swain Slough and the upper (second) Fairway Drive stream crossing in the lower Martin Slough watershed (Figure 1.1). Existing problems that have been identified in the Martin Slough study area include limited fish access, poor fish habitat, large sediment loads, poor sediment routing, lack of riparian habitat, and frequent prolonged flooding that has a negative economic impact on current land use.

Winzler & Kelly Consulting Engineers has teamed with Michael Love & Associates and Coastal Analysis LLC to develop a plan to improve fish access, enhance aquatic habitat, and reduce flooding impacts on land use activities within the study area. Winzler & Kelly is the prime consultant, providing project management for the development of the enhancement plan, coordination with the client and Winzler & Kelly's sub consultants, biological work, construction estimates, overall design assistance, and report preparation. Michael Love & Associates is primarily responsible for the hydrologic analysis, the tidegate model development, fish passage analysis, and channel design. Coastal Analysis LLC is primarily responsible for the two-dimensional finite-element hydraulic modeling of Martin Slough and for a simplified evaluation of sediment transport. The Natural Resources Services (NRS) Division of Redwood Community Action Agency (RCAA) administered the project and is responsible for the Technical Advisory Committee (TAC) and landowner coordination.

The TAC was comprised of agency representatives, land owners, and land managers plus the team of consultants and representatives of RCAA. The TAC had the following entities represented at one or more meetings:

City of Eureka;	Lisa Shikany (Planning), Gary Boughton (Engineering), Mike Zoppo (Property Management)
Course Co (golf course lessees);	Don Roller, Ray Davies, Bruce Perisho
Land Owners;	Gene Senestraro, Bob Barnum
State Coastal Conservancy;	Michael Bowen
U.S. Army Corps of Engineers;	David Ammerman (Permitting)
NOAA Fisheries;	Keytra Meyes, Margaret Tauzer, Chuck Glasgow
CA Department of Fish & Game;	Michelle Gilroy
County of Humboldt;	Rob Burnett and Chris Whitworth (Public Works), Alyson Hunter and Tom Hofweber (Community Development)
California Coastal Commission;	Jim Baskin
RCAA	Don Allan, Michele Copas
Michael Love & Associates	Michael Love
Winzler & Kelly	Steven Allen

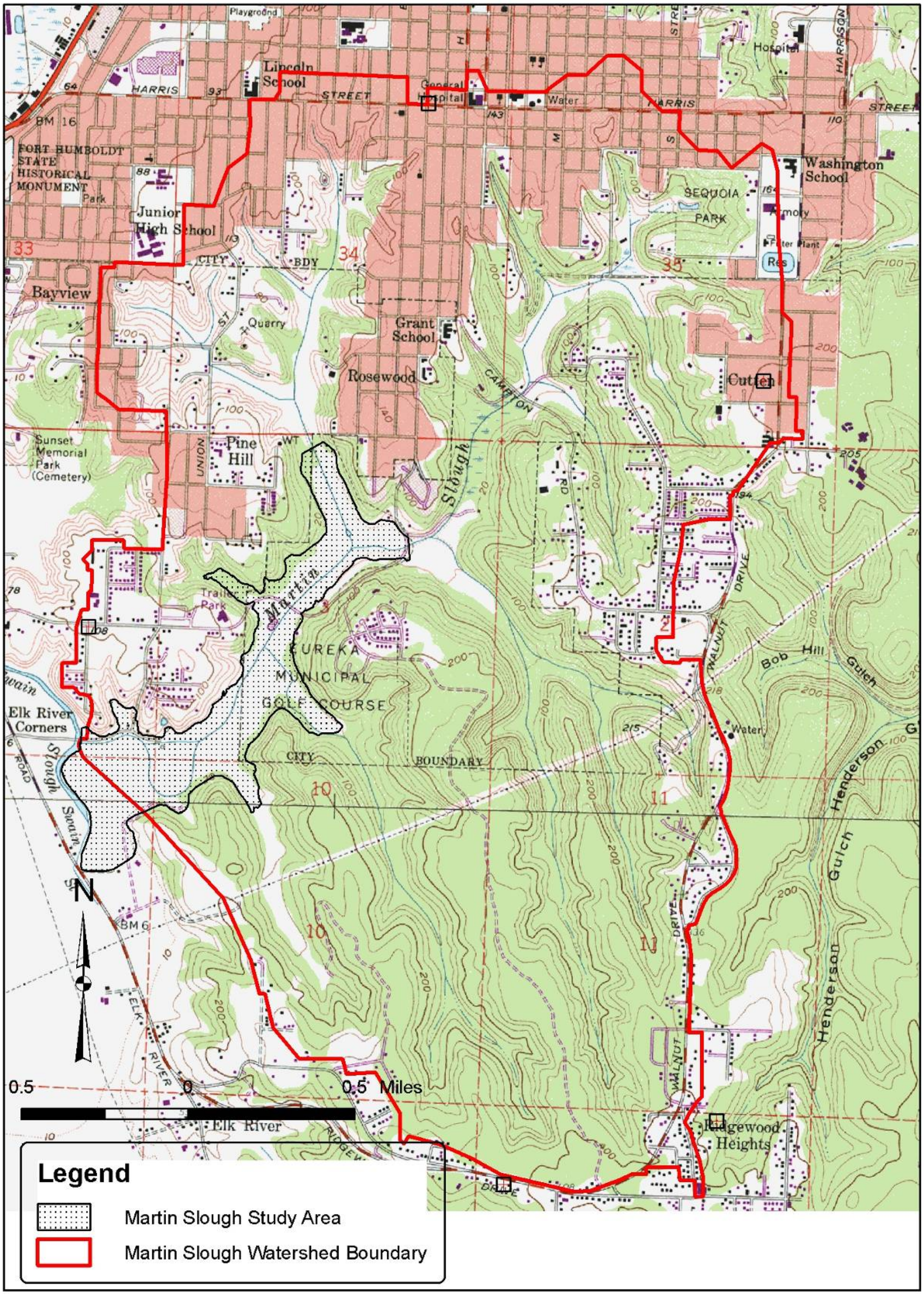

Figure 1.1 Study Area Site Map w/ Watershed Boundary

2.0 PROJECT GOALS AND OBJECTIVES

The goals of this project are to develop alternatives for lower Martin Slough, within the project boundaries, that will:

1. Enhance aquatic and riparian habitat,
2. Improve fish access from Swain Slough,
3. Reduce flood impacts to current land use,
4. Improve sediment transport.

The challenge is to develop a plan that can satisfy as many of the identified project goals and objectives as possible. However, often the objectives seem to be in direct conflict with one another. For example, in some cases salmonid habitat and riparian enhancement appear to be in direct conflict with other objectives such as routing of flood waters.

The broader goals of the project include working with all interested stakeholders to understand the issues and problems and then develop alternatives to achieve broad based support. This project provides an opportunity for stakeholders to work together to create a positive change to the lower Martin Slough watershed.

As part of the TAC meetings, lists of goals and objectives along with other issues and site constraints were compiled. From this, the project team developed and the TAC reviewed a list of specific project objectives (presented in Chapter 8.0). These specific objectives were later used to qualitatively evaluate the effectiveness of each alternative.

The project team endeavored to address all project goals with solutions that are complementary to each other. Analyzing flood impacts required more focus on hydrology and hydraulics than might normally be associated with an enhancement project. The magnitude of effort in this report associated with analyzing flood impacts reflects the complexity of those issues in this watershed. Solutions were developed with consideration of all the stated goals and objectives of the project.

3.0 SCOPE OF PROJECT

The Martin Slough Enhancement Feasibility Study scope that this report represents was developed in conjunction with Redwood Community Action Agency (RCAA). Initially all project related information was reviewed and discussions followed to determine key environmental issues and consider various approaches to accomplish the desired project goals. The following project scope provides a preliminary planning level analysis of four different alternatives for use as a decision making tool. The result of this effort is to provide useful information about the various alternatives so the benefits and tradeoffs of each alternative can be evaluated. The objective is to allow project stakeholders to select a preferred alternative to help guide an enhancement plan though environmental review, final design, permitting, construction, and post construction monitoring.

To best accomplish these goals, a Technical Advisory Committee (TAC) was established and TAC meetings were organized and scheduled by RCAA to foster discussion between interested stakeholders including property owners, regulators, and the design team. The TAC provided input along the way and helped guide the project direction and content.

To help identify possible alternatives, a base map of the project area was developed consisting of aerial photos and 2-foot contours obtained from existing aerial photogrammetry. The project area consists of the general flood plain between Swain Slough and the upper (second) Fairway Drive stream crossing in the lower Martin Slough watershed.

A hydrologic model of present and future conditions was prepared using the Army Corps of Engineers Hydrologic Engineering Center's Hydrologic Modeling System software (HEC-HMS) software for the entire watershed. The effort focused on developing hydrographs for the sub drainages that flow into Martin Slough within the project area. This information was used in the hydraulic models.

A brief evaluation of pre-development conditions was conducted utilizing historical maps and photographs. The goal was to try and determine what the project area may have been like prior to the levees being built along Swain Slough.

Utilizing the above information, four conceptual enhancement alternatives were developed in conjunction with the TAC. The first alternative is the "No Action" alternative. The No Action alternative simply evaluates the existing conditions. The second alternative restores full tidal influence by removing tidegates and the levee at Swain Slough. The third alternative involves installing new tidegates and expanding and creating new ponds along Martin Slough. The fourth alternative involves installing new tidegates, expanding and creating new ponds, and making channel modifications through the project area.

Next a hydraulic model for lower Martin Slough was developed utilizing additional limited survey data that was collected to better define the channel gradient and cross sections. The two-dimensional finite-element model ADCIRC (Luettich et al 1992) was used with observed tidal conditions as a downstream boundary control and inflow hydrographs representing the two year and ten year design storms as input. A separate tidegate model was used to evaluate and size the tidegates. A fish passage analysis was also completed at the tidegate interface to Martin Slough.

The biological and wetland reconnaissance fieldwork consisted of identifying likely areas of biological and wetland resources within the potentially impacted project area and marking the approximate locations over aerial photos. The areas were then digitized and added to the project base map. The goal was to provide planning level background information that would be helpful for the process of evaluating alternatives which minimize impacts to biological and wetland resources. See Chapter 14.0 for a detailed methodology of this effort.

The final scope item involved documenting the above information, comparing the results of the alternatives, and compiling the information into a report. This report represents the final product for this scope of work.

4.0 TECHNICAL ADVISORY COMMITTEE (TAC) MEETINGS

As briefly mentioned above, TAC meetings were organized and scheduled by RCAA to encourage dialogue between stakeholders and the design team. Stakeholders invited included but were not limited to representatives from the City of Eureka, County of Humboldt, RCAA, NOAA Fisheries, Army Corps of Engineers (ACOE), Fish & Game (CDFG), Coastal

Commission, Coastal Conservancy, Gene Senestraro (private land owner), and CourseCo, the management company which leases and operates the Eureka Municipal Golf Course. A total of seven TAC meetings were held at Eureka City Hall. One additional meeting was held as a field trip at the project study area.

Goals for the TAC included keeping communication open between project stakeholders, regulators, and the design team. With ongoing communication, the TAC was able to review and comment on the direction and content of the study. Items discussed and reviewed included the project scope, project criteria, project site map, hydrology results, development of alternatives, tidegate configurations, and hydraulic modeling results. Aspects of each alternative design were regularly discussed, such as permitting, wetlands, restoration of habitat, fisheries access and habitat, land use impacts, funding and maintenance. A draft copy of this report was provided to the TAC members for their review and comment. TAC meeting information, including sign-in sheets, meeting agendas, and meeting notes are included in Appendix C.

III. BACKGROUND INFORMATION

5.0 PROJECT BASE MAP

To help develop the Martin Slough Enhancement Feasibility Study aerial photogrammetry of the project area was used as the basis of the project base map. The aerial photogrammetry images with two-foot elevation contours were provided by the City of Eureka. The aerial photogrammetry used was flown in 2001 by Cartwright Aerial Surveys, Inc. of Sacramento.

5.1 Additional Surveying

As the photogrammetry provided only two-foot contours of the project area, additional survey information was needed to better define the drainage characteristics of the low gradient Martin Slough channel within the project area. Additional surveying of the site consisted of conducting a focused site survey to collect additional data for the hydraulic model. A total station was utilized to collect topographic select data from Swain Slough to upper Fairway Drive. Data collected included points for tops and toes of channel banks, channel thalweg, and ground shots for numerous channel cross sections. Additionally, tops and toes of Swain Slough levee near the existing Martin Slough tidegates were collected to better understand the downstream boundary. The results of the survey were utilized in conjunction with the photogrammetry to develop the surface mesh for the hydraulic model. The survey information collected as part of this project was not used to modify the original two-foot contour topographic surface provided by the City from the aerial photogrammetry.

Kayak being used to collect deeper water points

6.0 EVALUATION OF PRE-DEVELOPMENT CONDITIONS

For the purpose of this study, pre-development conditions refer to the Martin Slough study area as it existed prior to levees being built along Swain Slough. While our research of County files did turn up several old maps and photographs, nothing was found that pre-dated the levee along Swain Slough. While nothing conclusive can be said about the pre-development conditions in the Martin Slough study area, the information reviewed does provide some insights as to what the pre-development conditions may have been.

The pre-development vegetation of Martin Slough is presumed to have been a mixed Sitka Spruce (*Picea sitchensis*)/willow (*Salix* spp.) forest transitioning to tidal salt marsh. Extreme upper limits of the project area could possibly have been forested in portions by coast redwood (*Sequoia sempervirens*). Transition between forest and tidal salt marsh would likely have been comprised of brackish water and high groundwater tolerant willows, sedges (*Carex* spp.), bulrush (*Scirpus* ssp.) and rush (*Juncus* spp.). Salt marsh vegetation may well have dominated much of the study area prior to the dike construction. The tidal flats could well have been vegetated by pickleweed (*Salicornia virginiana*) and salt grass (*Distichlis spicata*). In the non-forested transitional areas brackish vegetation may have been soft rush (*Juncus effusus*), silverweed (*Potentilla anserina*), small-headed bulrush (*Scirpus microcarpus*), and tufted hairgrass (*Deschampsia caespitosa*). Some evidence of this interpretation of pre-development vegetation is provided below.

6.1 Historical Aerial Photographs

1941 Lower Martin Slough appears as today (Dikes/Senestraro Barn/Pine Hill Road/Myers Road). Golf Course property appears to be an operating ranch with a north central (above Holes 5 and 6) home, ranch facilities, and open fields. The properties adjacent to the east were primarily well developed timber stands. The properties located immediately to the north and west were occupied by residential structures. Adjacent properties in the vicinity of Martin Slough appear to have been used for agricultural/grazing land uses. A small triangle of land in the Hole 1 fairway region appears to be 50 percent vegetated with willows or short, broad-leaved trees. Reference used: Humboldt Co. Public Works aerial photo 11/23/1941, CVL-5B-149.

1954 Little change for the length of study area (as compared to 1941). A recent substantial barn is present on the ranch at Holes 5 and 6. A substantial road is parallel to upper Martin Slough along present day Hole 3 fairway. The property adjacent to the east has developed timber cover and appears to have recent road construction/logging. The properties located immediately to the north and west were occupied by residential structures. Adjacent properties in the vicinity of Martin Slough appear to have been used for agricultural/grazing land uses. A small triangle of land in the Hole 1 fairway region appears to be 50 percent vegetated with willows or short, broad-leaved trees. Reference used: Humboldt Co. Public Works aerial photo 7/21/1954, CVL-1N-86.

1962 The lower Martin Slough is unchanged (as compared to 1954). The golf course front nine is under construction. The pond between Holes 3-4 has been built. Fairway Drive to the Club House is under construction. Single family residences are developed on the north and west sides with Lundbar Hills access road now constructed to the east. Timberlands to the east are now laced with logging roads and most trees have been removed. Reference used: Humboldt Co. Public Works aerial photo HCN-2 10A-206.

6.2 Soil Survey Data

1921 The entire Martin Slough study area is classified as the Bayside Soil Series. Flood plain loam soil described as originally vegetated with oak, alder, willow, spruce and in farthest reaches redwood.

Reference: Watson, E. B. and S. W. Cosby. 1925. Humboldt County Soil Survey. U. S. Dept. of Agriculture, Bureau of Soils. Wash. D.C. Map from 1921. Field Operations. Humboldt Co. Public Works, Natural Resource Division.

1965 The entire Martin Slough study area is classified as Bayside 3 and Bayside 4 Soils Series. Flood plain and diked tidal lands soil described as originally vegetated in stream basin with willow, spruce and rush and in tidal reaches with pickleweed, silverweed and rush.

Reference: McLaughlin, J. and F. Harradine. 1965. Soils of Western Humboldt County, California. Dept. of Soils and Plant Nutrition. U. C. Davis and County of Humboldt.

6.3 Map Data

1854/1890 Early survey maps (Township/Range/Sections) cover the Martin Slough area and indicate vegetation along the Humboldt Bay margin (salt marsh/swamps/overflowed land) but no vegetation description is provided for Martin Slough. Reference: Humboldt Co. Public Works, Natural Resource Division

1870 Lower Martin Slough appears to be treated as plowed fields; apparently tidal lands have been diked as early as 1870. Salt marsh is restricted to areas closer to the Eureka waterfront and Fields Landing. The central portion of Martin Slough appears to be vegetated in non-conifer tree cover, willows possibly. The upper portions are illustrated in what one would interpret as conifer forest (Sitka spruce/coast redwood). Reference: Rodgers, A. F. and E. F. Dickens. 1870. U. S. Coast Survey Map, Part of Humboldt Bay.

1921 Property owners, tract names (early housing developments), timber lands (Excelsior Investment Co.) and agricultural pursuits (Russ Claim) are shown on this post-development map. The Russ Claim in the upper portion of Martin Slough is possibly a grazing claim on the flats of Martin Slough. Belcher Map. 1921.

1938 Early subdivision mapping of Eureka shows the upper portion of Martin Slough as “bottom land” and none of which is subdivided. The Russ Claim appears to be a housing tract. The bottom land is shown as Compton-Excelsior Annexation, with barn and house shown in the 5th and 6th Hole area. Lentell, J. N. 1938. Humboldt Land Title Co. Map of Eureka, California.

6.4 Pre-Development Summary

No clear understanding of the pre-development conditions are available from the above referenced materials. It is likely that tidal conditions occurred through much of the length of the Martin Slough drainage into the golf course. The existing low elevation terrain would have allowed high tides to extend well into the golf course. With this scenario, vegetation that has adapted to the tide water influence would have dominated the study area. Very little of the historical salt tolerant vegetation currently remains except for a narrow strip of vegetation along the slough itself (up to the lower golf course irrigation pond) and the lower Martin Slough pastures opposite the tidegate where the existing leaky tidegates provide brackish water influence.

Some settling of the land in the Martin Slough study area may have occurred after the Swain Slough levees were built. This has been documented in other tidal systems that have had the tidal influence removed by levees or other means. Under this scenario, the tidally influenced areas could have been less prevalent than we currently see. However, the frequent inundation of the project area by slow moving silt laden waters and resulting deposition has likely prevented substantial subsidence since the Swain Slough levees were built.

7.0 PROJECT HYDROLOGY

Determining project hydrology was an important aspect of the Martin Slough Enhancement Feasibility Study. Hydrologic conditions were characterized through:

- Collection of hydrologic data within the project area
- Development of numerical model for estimating hydrographs at various locations throughout the watershed resulting from design rainfall events (i.e. 2-year 24-hour rainfall event).
- Hydraulic modeling of the project area for characterizing existing conditions and examining hydrologic conditions associated with different project alternatives.

Hydrographs were also developed for potential future land-use conditions to determine if changes in runoff characteristics may influence effectiveness of the different project alternatives. The methodology is described in Chapter 7.2.2.1.

7.1 Hydrologic Monitoring

Through a separate contract with RCAA, Graham Mathews and Associates collected hydrologic data within the project area to help characterize existing conditions and to provide data for calibration of hydrologic and hydraulic models for use in the restoration planning and design phases. The data collection consisted of discharge measurements, continuous stage recording within the stream and on both sides of the tidegates, rainfall recording, and monitoring peak water levels using crest gages. Refer to Appendix A for location information of each monitoring station.

Hydrologic data collection efforts were performed from February through June 2003, and the continuous stage recording at Fairway Drive was reinitiated from November 2003 to January 2004. Unfortunately, the continuous stage recorders at the tidegates were vandalized shortly after installation, providing less than two weeks of data. Although limited, this data did provide important information to help calibrate the model. A summary of data collection methods and monitoring results are presented in Appendix A.

7.2 Hydrologic Modeling

Hydrologic modeling is an essential tool for predicting flow regimes when long term continuous flow gauge data is not available. Hydrologic modeling can also be used to study the effects of changing land uses on the runoff characteristics of a watershed. The development and use of a hydrologic model of the entire Martin Slough Watershed was the most appropriate approach for the following reasons:

(1) Martin Slough lacks historical streamflow data to perform a probabilistic prediction for recurrence flow regimes,

(2) Development and evaluation of alternatives requires design hydrographs and estimates of total volume of water entering the project area, and

(3) A component of this study includes examining the effects of future land use development on drainage characteristics within the project area

Version 2.2.2 of the ACOE Hydrologic Engineering Center's Hydrologic Modeling System software (HEC-HMS) was utilized to compute flow regimes based on desired rainfall events. Similar to its predecessor, HEC-1, HEC-HMS simulates precipitation-runoff and flow routing processes.

Following is a general description of hydrologic modeling activities and results. For a detailed explanation of the hydrology study's methods, data acquisition and inputs, and model results, refer to the study's detailed hydrology report located in Appendix A.

7.2.1 Model Development

Procedures for developing, simulating and interpreting results from the HEC-HMS model were followed using both the ACOE HEC-HMS Technical Reference and User Manuals. In general, initial model development followed the USDA Soil Conservation Service (SCS) methods for predicting outflow hydrographs (NRCS 1986; NRCS 2002).

Using stream flow data collected at the upper Fairway Drive stream crossing and precipitation data at the golf course during the initial phase of this project, the model was calibrated and its reliability of modeling flows was validated. During the calibration process the standard SCS unit hydrograph and lag time with constant baseflow method failed to adequately describe runoff characteristics within the watershed. As a result, alternative methods were used that more accurately model flows, as described below.

The first step of model development was division of the watershed into 44-subdrainages, referred to in HEC-HMS as *subbasins*. A subbasin element represents a complete watershed that is separated into three separate processes: *loss rate, transform, and baseflow.* The quantity of rainfall that falls and infiltrates is represented by a loss rate method. The excess rainfall which does not infiltrate and becomes runoff is represented by a transform method. Groundwater contributions to channel flow are represented with a baseflow method.

Contributing runoff from a subbasin into a defined stream channel is modeled in HEC-HMS using open channel flow principles, and referred to as a *reach* element. The attenuation characteristics and travel time of water flowing through a reach is dependent on length, slope, friction, flow depth and channel storage. The confluence location where two or more reaches combine is referred to as a *junction* element. Unlike a subbasin or reach element, physical properties are not assigned to a junction element. A junction element is strictly for computation purposes within the model and a location for which the user may view flow results (ACOE, 2001).

7.2.2 Watershed Delineation and Hydrologic Characteristics

Field reconnaissance was conducted to determine flow path direction for each street and city block within the Martin Slough Watershed for delineating subbasin divides, channel reaches, and flow paths (Figure 7.1). Watershed characteristics such as locations of day-lighted underground storm drains, land cover, topography and the need to examine runoff hydrographs at specific locations, all factored into the division process. In total, 44 subbasins and 15 individual channel reaches were identified.

A topographic map of the watershed containing 2-foot contours was provided by the City of Eureka and used to determine basin and channel slopes. Channel dimensions, which were needed for routing flows, where measured in the field at representative locations within each reach. This information is included in Appendix A.

The diverse land coverage within the watershed was classified and divided into six discrete land covers: commercial, dense urban, sparse urban, grasslands, timber and reservoir (Table 7.1). Within each of the 44-subbasins, land cover was visually delineated using georectified aerial photos in Arcview 3.3. An example of commercial land cover can best be defined as the business district in the Henderson Center area. The definitions used for delineating dense urban and sparse urban land covers are residential lot sizes of roughly 1/6-acre or less and greater than 1/6-acre, respectively. Land coverage associated with grasslands includes pasture, grazing rangeland, and golf course fairways. Land coverage associated with timber includes forested areas regardless of tree density, age or species. The reservoir land coverage was solely used to account for the City of Eureka's roofed reservoir adjacent to Sequoia Park.

Table 7.1 Land cover area within the Martin Slough Watershed for current conditions and anticipated future full build-out conditions.

	Current Conditions		Full Build-Out Scenario	
Land Cover	**Area (acres)**	**Portion of Watershed (%)**	**Area (acres)**	**Portion of Watershed (%)**
Grassland	397	11.27	234	6.63
Dense Urban	1,028	29.17	2,293	65.06
Commercial	16	0.46	16	0.46
Timber	1,944	55.16	948	26.89
Sparse Urban	138	3.91	33	0.93
Reservoir	1	0.03	1	0.03
Total Area	**3,524**	**100**	**3,524**	**100**

7.2.2.1 Full Build-Out Scenario

As part of this project we examined possible affects on future peak flows and runoff volumes associated with potential future land use changes within the watershed. The goal was to consider the hydrologic implications of a future full build-out scenario. To accomplish this, we worked with City and County staff to identify potential future land-use changes considered allowable. For example, a large portion of the southern watershed is expected to eventually transition from current timber production to residential and mixed land-use. Additionally, within the currently developed residential areas of the watershed located mostly within the City limits, further infill is expected to occur.

As part of the Martin Slough Sewer Interceptor Project, City and County staff helped develop a map that showed currently undeveloped areas within the watershed that had slopes greater than 30% or are considered wetlands (Figure 7.2). These areas, which consist of mostly gulches, were considered non-developable as part of the sewer interceptor project. For the full build-out scenario we assumed that areas receiving the non-developable designation would continue having the same land cover as currently designated. Sequoia Park was also included in our model as an additional area whose current land use would remain unchanged.

The remaining areas within the Martin Slough watershed that fell outside of the non-developable areas described above and are not currently designated as commercial or industrial were assumed to become dense urban (a HEC-HMS land cover designation) based on discussions with City and County staff. The HEC-HMS model utilizes these land cover designations to determine runoff and assumes no detention basins. Using GIS, we identified the proportion of different land uses within each subbasin for this full build-out scenario. To account for the residential infilling City and County staff suggested we use in the hydrologic model runoff characteristics (curve numbers) associated with residential lot sizes equivalent to 1/8-acre or less for designated dense urban land cover. Table 7.1 shows the distribution of the six different land covers for current conditions and the full build-out scenario.

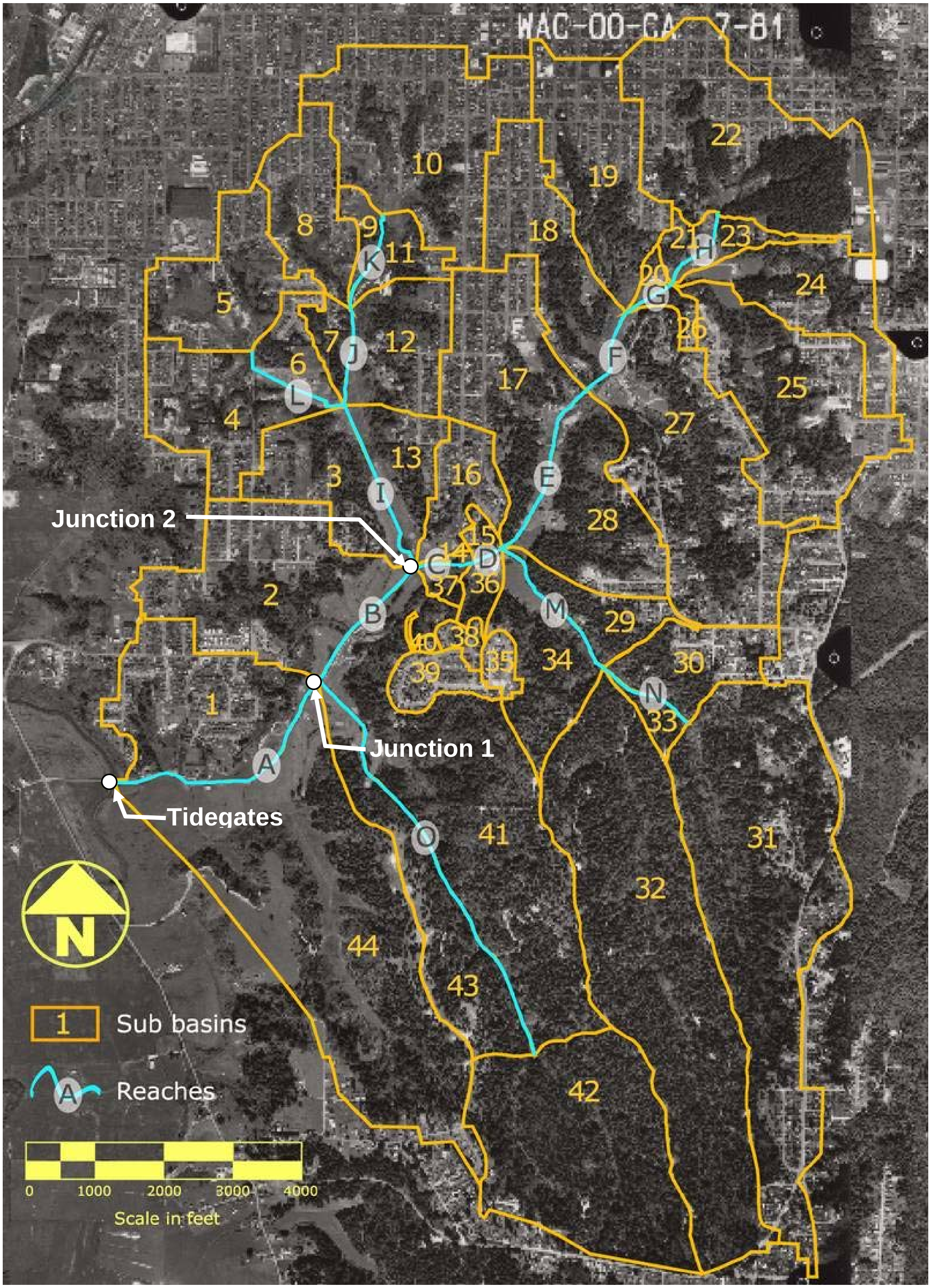

Figure 7.1 A 2000 aerial map of Martin Slough Watershed, with the 44-subbasins, and 15 channel reaches delineated. Map created by Natural Resources Services, RCAA.

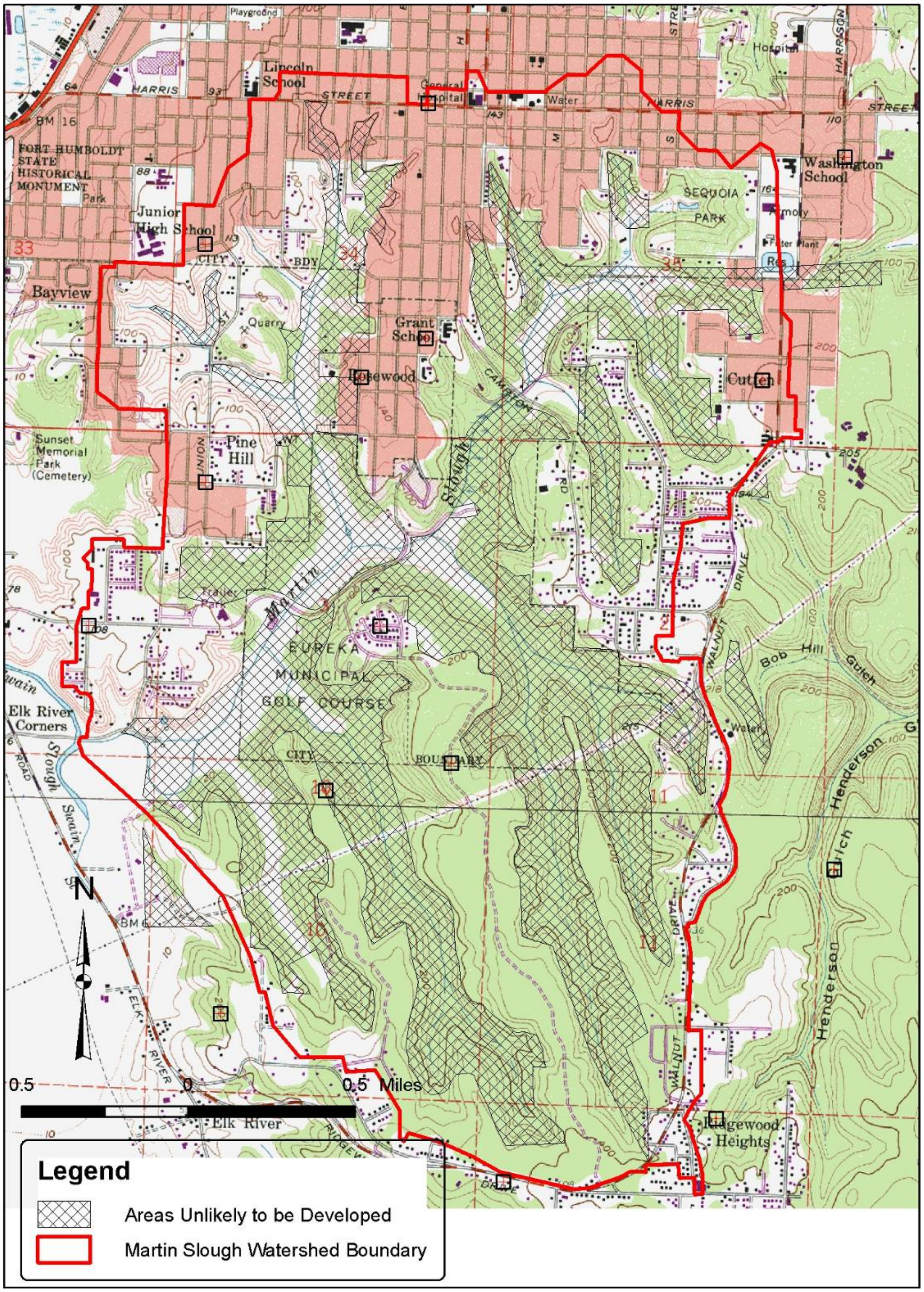

Figure 7.2 Areas within the Martin Slough Watershed designated for no-further development. Land cover within these areas were assumed to remain unchanged within the full build-out scenario.

7.2.3 Model Approach and Calibration

During model calibration, initial runs were performed following the standard SCS approach. For calibration of the model, discrete rainfall events recorded at the rain gauge located on the golf course were input into HEC-HMS and the modeled output flows at upper Fairway Drive were compared to actual flows measured at that location. Initially, the standard SCS method greatly over-estimated peak flows and modeled flows returning to baseflow far quicker than observed. As a result, alternative methods were explored that could more accurately model flows.

After trying various accepted approaches for modeling losses, transformation and baseflow, the final calibrated model used:

- **Loss** - Constant initial loss and SCS curve numbers based on land use and soil type to determine proportion of rainfall that becomes runoff.
- **Transformation** - The Snyder Unit Hydrograph (UH) method for transforming excessive rainfall into runoff,
- **Baseflow** - Recession curve returning to a constant baseflow.

The initial loss, Snyder UH coefficients and recession curve constants were successfully calibrated to produce predicted hydrographs that followed relatively accurately observed hydrographs at Upper Fairway Drive near Junction 2 shown in Figure 7.2. The calibration effort balanced accurately predicting both peak flows and total volume of runoff associated with individual storm event (Figure 7.3).

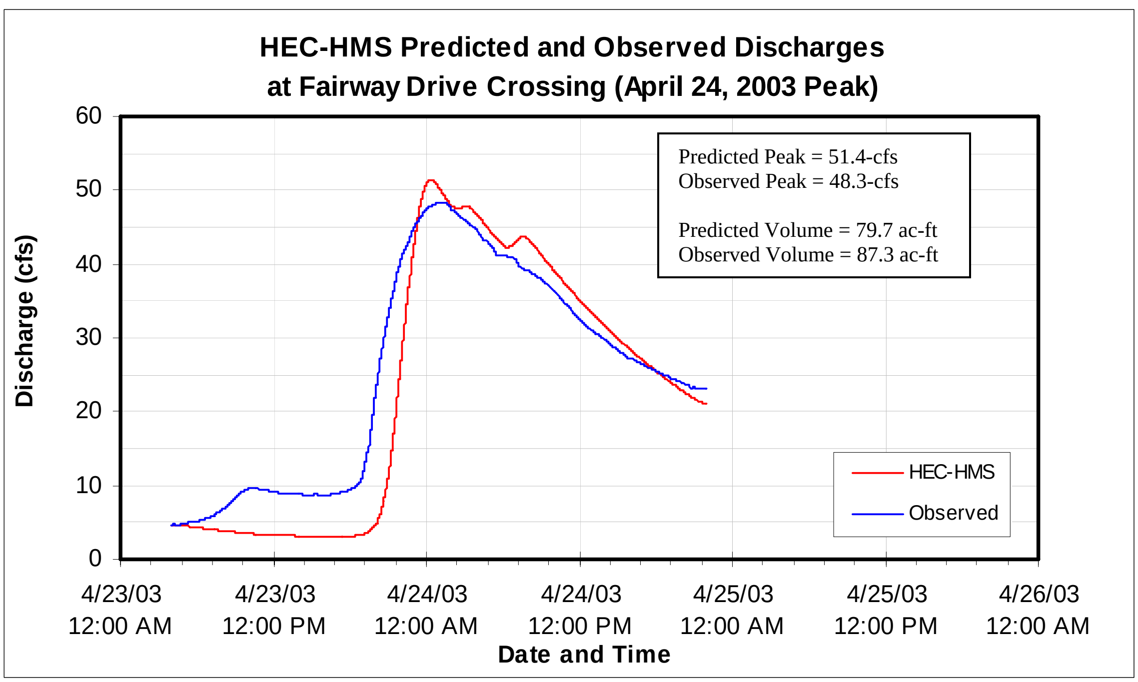

Figure 7.3 Observed and HEC-HMS predicted flows at Upper Fairway Drive crossing during precipitation event beginning on April 24, 2003.

7.2.4 Final Results from Design Rainfall Events

Once the model was calibrated, 24 hour rainfall events with recurrence intervals of 2 years, 10 years, and 100 years were input into the model and predicted outflow hydrographs were generated for various locations throughout the watershed. The 24 hour rainfall events used for the Martin Slough Watershed were based on records from Eureka (Table 7.2). The resulting predicted hydrographs were then incorporated into the hydraulic modeling phases of the overall project.

Table 7.2 24 hour rainfall events of varying return periods for Eureka (1904-1999), developed by the California Department of Water Resources.

Return Period	24-hour Rainfall Event (inches)
2-year	2.67
10-year	4.17
100-year	5.86

7.2.5 Results for Current Land Use

The peak flow and total volume of runoff predicted at Junctions 1 and 2 (see Figure 7.1) and at the tidegates are listed in Table 7.3. It is important to note that HEC-HMS does not take into account backwater effects when routing flows through the watershed. Therefore, the peak flow estimates fail to account for substantial backwater effects created by the tidegates. For this reason, flows into the project area above potential backwater effects were used as inputs into the hydraulic model, which does account for backwater effects.

Table 7.3 HEC-HMS predicted peak discharge and total flow volume at three locations.

Return Period of 24-hr Rainfall Event	Junction 1		Junction 2		Tidegate	
	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)
2-year	141	322	131	281	149	397
10-year	315	667	286	579	330	833
100-year	554	1,130	499	983	583	1,400

7.2.6 Comparison with Previous Study

A similar hydrologic study of the Martin Slough Watershed was conducted by Oscar Larson and Associates (OLA) in 1990 for the City of Eureka and Humboldt County Departments of Public Works. The study used the ACOE HEC-1 model, the precursor to HEC-HMS, to predict rainfall-runoff processes within the watershed (OLA, 1990). The earlier HEC-1 model, which followed the standard SCS method, produced peak flow estimates substantially greater than the HEC-HMS results for this project. For example, for the 10-year 24-hour design storm the predicted peak flow at the tidegates was 1,320 cfs while the HEC-HMS model predicted a peak of 330 cfs. However, the HEC-HMS model estimated outflow volumes that were nearly a third more than predicted with the earlier HEC-1 results. The discrepancy between results can be explained by the differences in methods employed. Most notable is that the previous study assumed Antecedent Moisture Conditions (AMC) III conditions, which results in considerably higher peak flows. We considered it prudent to use AMC II conditions given that NRCS now recommends avoiding usage of AMC I and III (Ponce and Richard Hawkins, 1996.).

Absent any streamflow data for Martin Slough, the earlier model was uncalibrated. They used the standard SCS UH and Lag Time method, while we determined during calibration that the Snyder method was more applicable. Also, the previous study used a Type 1 SCS Hypothetical Storm instead of the appropriate Type 1A. The Type 1 distribution assumes a higher intensity storm relative to a Type 1A and is applicable for regions further south.

Even though the earlier study had a larger peak flow, the HEC-HMS model predicted substantially more outflow volume. Since a goal of this project is to reduce the duration of flooding within the lower portions of the watershed through improved drainage at the tidegates, it is most essential to have accurate estimates of the volume of water that must be drained during one or more tidal cycles. Given that this HEC-HMS model was calibrated to existing streamflow data, it is reasonable to assume that it produces realistic estimates of flow hydrographs for the watershed, and is suitable for use as input into the hydraulic model of the project area.

7.2.7 Final Results for Full Build-Out Scenario

As part of this project we examined possible affects on future peak flows and runoff volumes associated with potential future land use changes within the watershed. For modeling purposes the only parameter changed in HMS was the subbasin area-weighted curve number, which is a function of the land use within the subbasin. Anticipated future land use was determined using the criteria outlined in Chapter 7.2.2.1. All other model input remained the same as used for existing conditions. Table 7.4 summarizes the peak flow and total runoff volume predicted for the full build-out scenario. Comparing results from simulations using current conditions (Table 7.3), the full build-out scenario predicts at the tidegate a 62% increase in the peak flow and 54% increase in runoff volume associated with a 2-year 24-hour rainfall event. For the 10-year 24-hour rainfall event peak flows and volumes at the tidegate are predicted to increase 50% and 42%, respectively.

Table 7.4 Comparison of HEC-HMS predicted peak flows and runoff volumes at the tidegate for existing land use conditions and the full build out scenario.

Return Period of 24-hr Rainfall Event	**Existing Land Use**		**Full Build-Out Scenario**	
	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)
2-year	149	397	242	612
10-year	330	833	495	1,184
100-year	583	1,400	823	1,896

The peak flow and volume estimates associated with the full build-out scenario are conservative, and assumes future development will not have any storm water detention facilities. If detention basins or other methods are employed in future developments, we would expect full build-out peak flows and related volumes would be less than those shown in Table 7.4, especially for the 2-year and 10-year rainfall events.

IV. PROJECT ALTERNATIVES

8.0 DEVELOPMENT OF PROJECT ALTERNATIVES

Project alternatives were developed based on numerous factors. Their development was partially an iterative process. As the alternatives were refined more information became available based on the results of various tasks conducted throughout the study. The following sections describe how the project alternatives were developed.

8.1 Current Land Use

Project alternatives were developed based on current land use combined with the ability to make modifications based on the current and projected future land use as well. The no tidegate alternative was requested as part of the project scope. Two main land owners are included in the project area. The upper project area is predominately owned by the City of Eureka and the land is used as the Eureka Municipal Golf Course. The lower project area is predominately owned by a single private landowner and the land is used for agricultural grazing. Both land owners intend to maintain their current land use and all alternatives developed take this desired land use into consideration.

Both land owners also expressed willingness to participate in this study process to develop potential alternatives and are willing to consider allowing some of their land to change uses to make improvements to Martin Slough. Neither land owner is interested in giving up use of the majority of their land for any project. Rather than designing alternatives based on a particular design storm event or other hydraulic criteria, this study developed conceptual alternatives based on the criteria of the project discussed below within the constraints of the amount of land available to accommodate current land uses.

8.2 Criteria Used for Alternative Development

Based on the myriad goals for the project, criteria were developed to help guide alternative development and evaluate the potential impacts or benefits of the various alternatives. The multiple criteria reflect the project goals and fit into general categories. The draft criteria developed by the project team were then discussed at a TAC meeting for their review, input, and modification. Neither the categories nor the criteria were weighted or otherwise ranked. Thus the scope of this project is not to determine which categories, criteria, or alternatives are preferred. Rather, the scope of this project is to provide useful information so that decisions may be made by the TAC regarding how various alternatives address the myriad goals of the project. The following table summarizes the categories and criteria that were determined to be useful to help develop and evaluate the potential impacts and benefits of various alternatives:

Table 8.1 Criteria Used For Alternative Development

Categories	Criteria
Fish Passage and Fish Access for Juveniles and Adults	1. Maximize Migration Access at Tidegates during Fish Migration Flows
Fish Habitat	2. Maximize Estuarine Habitat 3. Increase Channel Complexity for fish habitat
Riparian Corridor	4. Increase Riparian Habitat 5. Increase Riparian Canopy
Water Quality	6. Decrease Nutrient Impacts 7. Decrease Sedimentation
Wetlands	8. Improve Wetland Habitat 9. Increase Open Water Wetlands 10. Increase Diversity of Wetland Types
Flood Impacts	11. Reduce Flood Inundation Area 12. Reduce Frequency of Flooding 13. Minimize Duration of Flooding
Existing Land Uses	14. Maintain Agricultural Land Use 15. Maintain Eureka Municipal Golf Course 16. Allow for full Build-out Potential for City/County 17. Allow for Installation and Maintenance Access for City's Martin Slough Sewer Interceptor Project
Project Permitting	18. Consider Ability to Obtain Permits
Cost of Improvements	19. Consider Order of Magnitude Opinion of Probable Construction Costs
Project Maintenance	20. Consider Need for Ongoing Maintenance

To develop potential alternatives for the Martin Slough Enhancement Feasibility Study, all of the above criteria were taken into account by the project team. Clearly it would not be a benefit to develop an alternative that does not address any of the project goals or is simply not feasible. In addition, the project team was asked to include certain alternatives, specifically the first two alternatives. All alternatives were brought to a TAC meeting for their review and approval prior to proceeding with the analysis. Following are brief descriptions of the alternatives, with more detailed descriptions given in Section V, Chapter 15.0.

9.0 DESCRIPTION OF ALTERNATIVES

9.1 Alternative 1: The No Action Alternative (Existing Conditions)

The No-Action Alternative would leave the system as it exists today. With this alternative existing conditions are quantified based on data and results from calibrated hydraulic and hydrologic models. Including the No Action alternative is essential for comparing alternatives, allowing a familiar starting point for comparisons to be made.

9.2 Alternative 2: No Tidegates or Levee (Full Tidal Influence)

Part of the scope of this project included considering removal of the control structures to return lower Martin Slough to a full tidally influenced system. This concept also made sense from an ongoing maintenance perspective, which was an interest of the landowners. The No Tidegates or Levee (Full Tidal Influence) Alternative would consist of removing the existing tidegates and the levee at Swain Slough. Essentially this would most closely approximate the system in its pre-development state. Based on land and tidal elevations, this alternative would open the majority of the project area to full tidal influence.

9.3 Alternative 3: New Tidegates and New Ponds (Muted Tide)

The New Tidegates and New Ponds Alternative would consist of removing the existing tidegates, installing new tidegates with a habitat door which is sized to create a muted tidal prism and facilitate fish passage, increasing the size of existing ponds, and creating new ponds. The Swain Slough levee would also be repaired to alleviate the existing low points that allow tidal waters to pour over the levee and enter the adjacent pastures during high tides. The features of Alternative 3 are described in Figure 9.1.

The new tidegates would replace the existing undersized tidegates, which would improve discharge capacity and provide a habitat door allowing fish passage when the other gates are closed and allow some seawater to flow upstream to create muted tide cycle. The main tidegates would also include side-hinged doors which fully open with less flow than top hinged doors. These will increase fish migration access compared to existing conditions. The habitat door would allow adult and juvenile fish passage and is sized based on available volume of the tidal prism upstream of the tidegates. The habitat door would remain open until the water level within Martin Slough rises to elevation 5 feet based on the North American Vertical Datum of 1988 (NAVD88). Once the desired level in Martin Slough is reached, the door would close to prevent additional seawater from entering Martin Slough. Although this alternative does not include any changes to the stream channel, the increased muted tide cycle would increase the daily flushing of the channel. This increase in tidal prism (diurnal volume of water that ebb and flows) within Martin Slough would increase channel bank and bed scour, likely causing the channel to widen and deepen. To provide some control over the muted tide cycle the water level in which the door would close will be adjustable, allowing for adaptive management of the muted tide cycle.

The new and expanded ponds would create additional habitat for rearing salmonids, waterfowl, and other aquatic and semi-aquatic species. The ponds would also provide additional storage capacity for storm flows, reducing the amount of time higher ground is inundated. This alternative would increase the size of three existing ponds on the golf course. Two new additional ponds would be added, one on the golf course and one on the downstream private property adjacent to the new tidegates. It is anticipated that this alternative would provide a range of estuarine habitat with varying salinity values. The highest salinity values would be adjacent to the tidegates, and the lowest salinity would be found farther upstream. Salinity values would likely fluctuate from summer to winter months, being higher in the summer when less fresh water is entering the drainage. The golf course would likely need to use the upper irrigation pond as their primary irrigation source. The additional ponds with varying salinity values would be beneficial to juvenile salmonids and other fish and wildlife species. The ponds would be planted with a variety of wetland and riparian vegetation. In the pasture, the new vegetation would be protected by riparian fence to help protect it from grazing.

Notes:

New tidegates to increase conveyance and restore limited tidal influence, construction and enlargement of ponds to increase floodwater storage and provide enhanced fisheries and waterfowl habitat .

Pond 1 - Construct new pond, predominantly saltwater with saltmarsh. (5.5 acres)

Pond 2 & 3 - Enlarge existing ponds. Seasonal brackish, turning predominantly saltwater during low flow. (1.25 acres & 1.9 acres)

Pond 4 - New seasonal freshwater pond turning brackish during low flow. (1.9 acres)

Pond 5 - Enlarged year round freshwater pond, serving as primary source for irrigation for golf course. (1.7 acres)

Martin Slough Enhancement Plan

Alternative - 3

New Tidegates & New Ponds (Muted Tide)

Figure 9.1 Features of Alternative 3

9.4 Alternative 4: New Tidegates, Storage Ponds, and Modified Channel (Muted Tide)

This alternative is similar to Alternative 3, but includes improvements to the existing channel. The New Tidegates, Storage Ponds and Modified Channel Alternative consists of removing the existing tidegates, installing new tidegates with a habitat door designed to create a muted tide cycle and facilitate fish passage, increasing the size of existing ponds, creating new ponds, and making channel modifications throughout the project area. The Swain Slough levee would also be repaired to alleviate the existing low points that allow tidal waters to pour over the levee and enter the adjacent pastures during high tides. Alternative 4 are described in Figure 9.2.

With this alternative, the existing Martin Slough channel would be enlarged within the project area to increase conveyance for both flood flows and diurnal tidal exchange. In addition, the remnant meander channel would be used as the new main channel, alleviating flow in the ditch past the barn. The adjacent subdrainage channel would be modified to flow into the meander channel. Reintroducing a muted tide cycle into the project area would result in large volumes of water flowing up and down the channel with each tide cycle, changing the fluvial processes that help maintain channel geometry. We relied on design guidelines developed for tidal channels. It is anticipated the width of the new channel would remain unchanged at the very upper reaches of the project area and would increase in width in the downstream direction to accommodate the additional volume of water entering the channel from the new ponds and subdrainages. In addition to the width of the actual channel, it is anticipated that an additional 15 to 20 feet on each side (and additional 30 to 40 feet total) of the channel within the project area would be planted with some wetland and mostly riparian vegetation.

The new tidegates would replace the existing undersized tidegates, which would improve discharge capacity and provide a habitat door allowing fish passage and a muted tidal prism. This habitat door is larger than in Alternative 3 in order to achieve the same tidal inundation with the larger volume tidal prism available with this alternative. The main tidegates would also include side-hinged doors that would increase fish migration access compared to existing conditions. The habitat door would allow adult and juvenile fish passage. The habitat door would remain open until the water level within Martin Slough rises to 5 feet (NAVD88). The water level in Martin Slough that closes the door will be adjustable. Once the desired level is reached, the tide is held back from entering Martin Slough.

The new and expanded ponds would create additional habitat for rearing salmonids, waterfowl, and other aquatic and semi-aquatic species. The ponds would also provide additional storage capacity for storm flows, reducing the amount of time higher ground is inundated. This alternative would increase the size of three existing ponds on the golf course. Two new ponds would be added, one on the golf course and one on the downstream private property adjacent to the new tidegates. It is anticipated that this alternative would provide a range of estuarine habitat with varying salinity values. The highest salinity values would be adjacent to the tidegates, and the lowest salinity would be found farther upstream. Salinity values would likely fluctuate from summer to winter months, being higher in the summer when less fresh water is entering the drainage. The golf course would likely need to use the upper irrigation pond as their primary irrigation source. The additional ponds with varying salinity values would be a large benefit for juvenile salmonids and other species. The ponds would be planted with a variety of wetland and riparian vegetation. The new vegetation in the pasture would be protected by riparian fence.

Notes:

New tidegates to increase conveyance and restore limited tidal influence, construction and enlargement of ponds to increase floodwater storage and provide enhanced fisheries and waterfowl habitat, and enlarged channel to increase floodwater and tide water conveyance through project area.

Pond 1 - Construct new pond, predominantly saltwater with saltmarsh (5.5 acres)

Pond 2 & 3 - Enlarge existing ponds. Seasonal brackish, turning predominantly saltwater during low flow. (1.25 acres & 1.9 acres)

Pond 4 - New seasonal freshwater pond turning brackish during low flow (1.9 acres)

Pond 5 - Enlarged year round freshwater pond, serving as primary source for irrigation for golf course (1.7 acres)

New channel dimensions - Trapezoidal in shape with 1.5:1 (H:V) side slopes and bottom elevation set at 0 ft. Stable tidal channel geometry based on published relationships between diurnal tidal prism and slough channel dimensions.

Reach	Top Width (ft)	Length (ft)
①	65	2,520
②	55	2,415
③	40	782
④	30	514
Ⓐ	15	750
Ⓑ	25	270

Martin Slough Enhancement Plan

Alternative - 4

New Tidegates, Ponds & Channel Improvements (Muted Tide)

Figure 9-2 Features of Alternative 4

10.0 TIDEGATE MODEL DEVELOPMENT

I must go down to the seas again,
for the call of the running tide
Is a wild call and a clear call
that may not be denied.
--John Masefield

A simplified numerical model (referred to as TIDEGATE) of tidegate hydraulics was created in an Excel spreadsheet to allow for rapid analysis of the effectiveness of different tidegate designs in providing fish passage and flood routing within the project area. The prediction of water levels upstream of the tidegate and velocity through the tidegate were conducted using a control volume analysis that is based on the principles of lumped flow routing (hydrologic routing) and predicts the water surface elevation upstream of the tidegates and the flow through the tidegates.

For this analysis, the project area was characterized by a storage-inundation curve, which relates the volume of water in the project area to the corresponding water surface elevation. This relationship is a function of the provided topography within the project area. Streamflow entering the project area was described by either observed or simulated hydrographs. Flows can also exit or enter the project area through the tidegates. Flows through the tidegates are computed as a function of the water surface elevation within Martin Slough and the tidewater elevation within Swain Slough. As the volume of water within the project area changes, the stage-inundation curve is used to determine the new water level. A conceptual diagram of the TIDEGATE model and its components are illustrated in Figure 10.1.

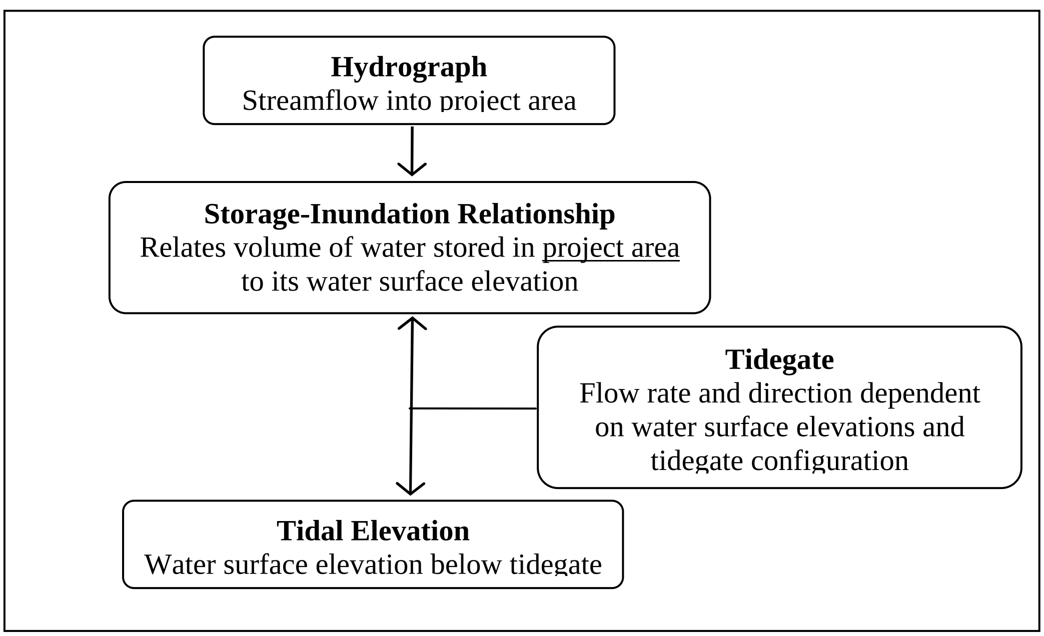

Figure 10.1 Diagram of TIDEGATE model used for Martin Slough

10.1 TIDEGATE Model Limitations

The TIDEGATE model relies on the storage-inundation relationship to determine the amount of water that enters or exists through the tidegates. The model assumes that the channel can instantaneously convey water to the tidegates. The model does not directly account for hydraulic routing times associated with waters flowing from one side of the project area to the other. To improve stability and accuracy of the model, it was run at one minute time steps. For this project we compared the model output with observed conditions to verify its ability to accurately predict water surface elevations upstream of the existing tidegates.

The TIDEGATE model served as a means of quickly examining numerous tidegate configurations with relative ease. Suitable alternatives identified with the TIDEGATE model were then modeled in detail using the 2-dimensional finite element hydraulic model, which does account for hydraulic routing of water throughout the project area.

10.2 Design 7-day Swain Slough Tide Cycle

Modeling flows through the tidegates at the Swain Slough boundary required information about the actual tide cycles within Swain Slough. From February 12-22, 2003, tidal data was recorded in Swain Slough near the Martin Slough tidegates, as well as water surface elevation in Martin Slough just upstream of the tidegates. The first seven days of the Swain Slough tidal data was used as the “design tide cycle” for modeling each of the alternatives. This seven day period consisted of relatively extreme high and low tides, with the North Spit Humboldt Bay tidal station (Station 9418767) recording a low tide below -1.0 feet (using NAVD 88 datum) and a high tide above 8.0 feet (Figure 10.2). High tides within Swain Slough closely match those observed at the North Spit station. However, low tides within Swain Slough were prevented from dropping below 1.5 feet. Swain Slough is a relatively high elevation tidal slough, with portions having a bottom elevation between 0 and 1 foot elevation. Additionally, there is possibly a grade control point in Swain Slough near the Martin Slough tidegates that prevents Swain Slough from draining below the 1.5 foot elevation.

10.3 Storage-Inundation Relationships

Creating the TIDEGATE model required developing a storage-inundation relationship for the project area based on provided topography. This relates the volume of water stored in the project area to the corresponding water surface elevation. This relationship was created for current conditions using a combination of the topographic base map provided by the City of Eureka containing 2-foot contour intervals combined with the limited detailed topographic survey data of the channels collected as part of this study (Figure 10.3). Some error in the volume calculations is expected due to the level of accuracy associated with the 2 foot contour topographic data. For Alternatives 3 and 4, the volume associated with the additional ponds and enlargement of the channel were calculated from the City provided topography and used to create new storage-inundation relationships for those two alternatives.

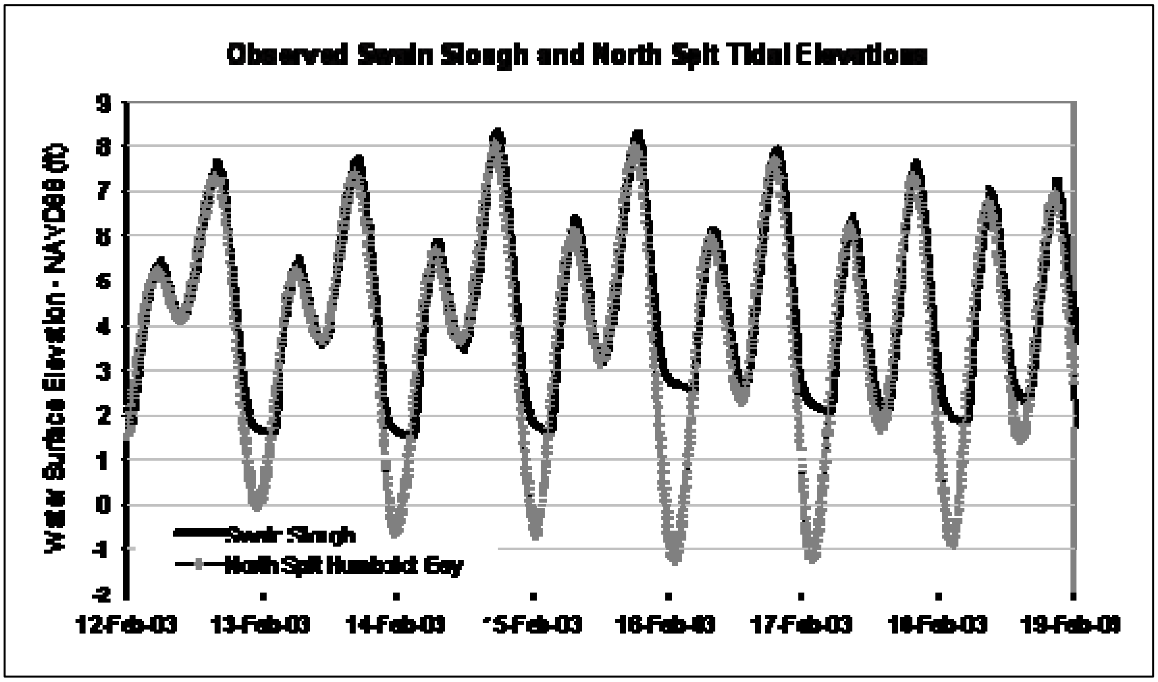

Figure 10.2 Comparisons of observed tides in Swain Slough and at the North Spit of Humboldt Bay North Spit (tidal station 9418767). Swain is an high elevation slough channel, with tides not dropping below 1.5 feet during the observation period.

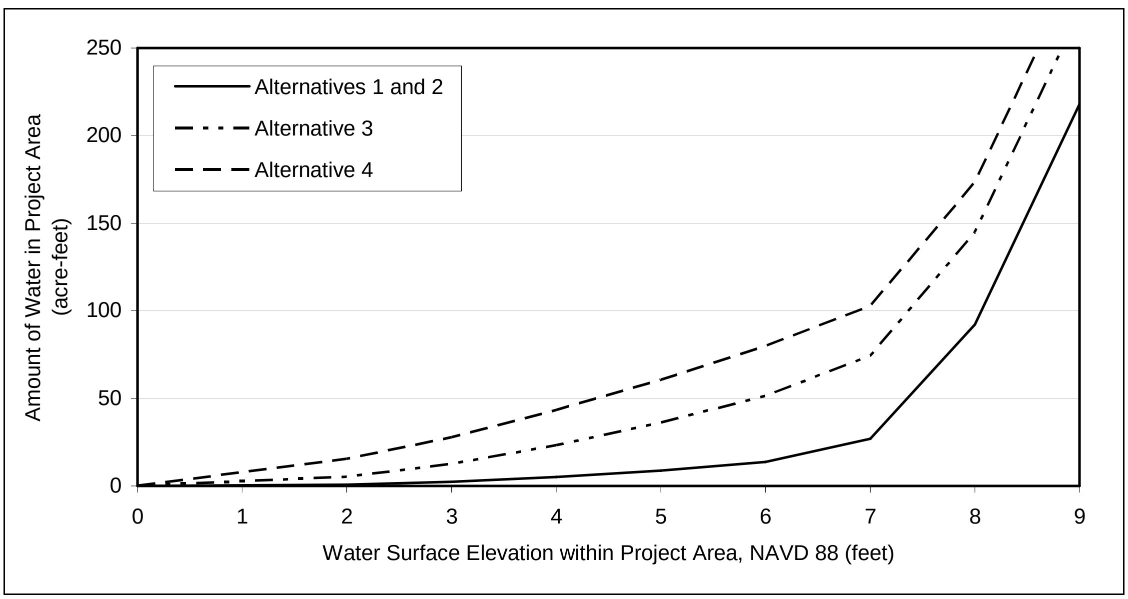

Figure 10.3 Rating curves for each of the four alternatives relating the volume of water stored in the project area and the elevation of inundation. Alternative 1 and 2 use existing topography only, while Alternative 3 includes additional storage ponds and Alternative 4 includes the storage ponds and enlargement of the main channel.

10.4 Tidegate Hydraulics

Flows through the tidegates were calculated using procedures outlined in the Army Corps of Engineers (ACOE) Hydraulic Engineering Center River Analysis System (HEC-RAS) Hydraulic Reference Manual, Version 3.1. The calculated flows were a function of the shape and vertical placement of each gate and the water levels in both Swain Slough and Martin Slough. Flows were calculated using either modified weir or orifice equations, depending on the corresponding water levels.

10.5 Model Calibration

Observed water levels and stream flows from the February 2003 data were used to calibrate the TIDEGATE model. Water levels were recorded immediately above and below the existing tidegates for one complete week in February 2003 (Figure 10.4). This data was utilized as our design tide cycle for Swain Slough and provided an understanding of current tidegate performance. Additionally, during this same time period streamflows were recorded at the upper Fairway Drive crossing. Low flow conditions persisted through February 15th, with a small rise in flows on the16th and 17th.

We also measured the dimensions and surveyed the elevations of the existing tidegates. While surveying, substantial leakage through the tidegates during high tide was observed and noted. Shortly after these observations, two of the three existing tidegate and culvert assemblies were replaced with new culvert pipes and the old tidegates were re-attached. All observations used for characterizing existing conditions were made prior to this recent tidegate maintenance, which occurred in fall 2004 and reduced the amount of water leaking through the gates. Therefore this change did not influence our analysis as the data used for calibration were collected previously.

For model input we used the dimensions and elevations of the existing tidegates, the recorded tidal levels in Swain Slough, and flow measured at Upper Fairway Drive and scaled-up by the drainage area of the entire watershed. We also simulated leakage through the tidegates by including in the model a small opening that allowed water to flow from Swain Slough into Martin Slough. Through trial and error, we found that an opening with an area of 2.2 square feet best described the sum of all the tidegate leakages that existed in February 2003.

The TIDEGATE model was able to simulate existing conditions relatively well. The largest discrepancy between observed and predicted values occurred at low tide. The observed water level in Martin Slough likely failed to drop as low as Swain Slough tidal elevations due to the heavy, top-hinged, cast iron gates. At low tides there appears to be insufficient stream flow pressure to keep the existing gates open, reducing outflow. However, the tidegate hydraulic equations used in the model do not account for the weight of the gates, likely explaining the discrepancy. This problem does not arise when modeling other types of gates, such as the lighter aluminum side hinged and top-hinged gates.

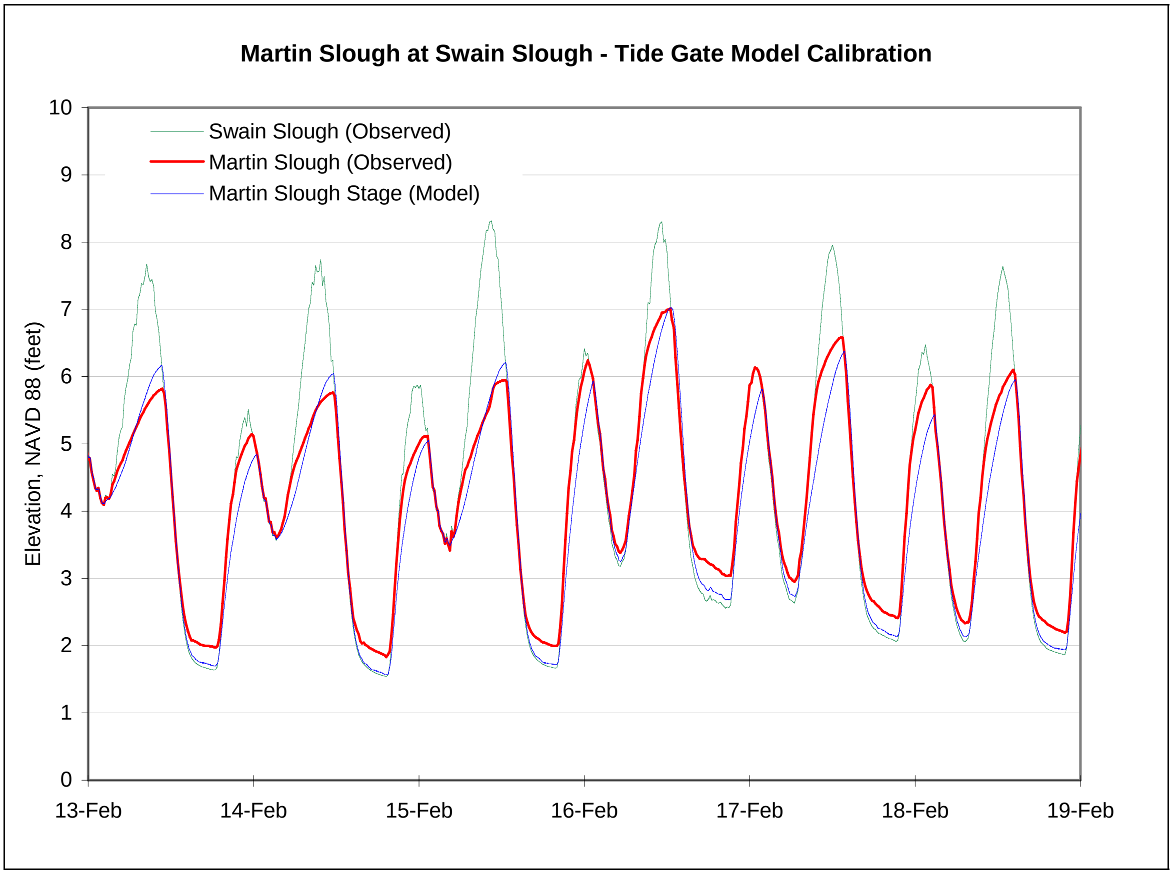

Figure 10.4 Observed and predicted water levels at the upstream side of the existing Martin Slough tidegates. Observations made in 2003, prior to recent tidegate maintenance. The tidegate model output closely followed observed water levels.

10.6 Selection of Preferred Tidegate Design

The TIDEGATE model was used to evaluate the performance of different tidegate designs. The objectives of a new tidegate structure are to improve fish passage, increase the amount and diversity of aquatic habitats (salt, brackish, and freshwater marshes as well as in-channel and side-channel slough habitat), reduce sedimentation within the channel, and improve drainage of floodwaters during storm events. These objectives can be satisfied through increasing the outflow capacity (conveyance area) of the tidegates and reintroducing tidal influence within Martin Slough. Unfortunately, most of the lands within the project area are below elevation 8-feet and would be regularly inundated by the tide if the tidegates were removed. One means of reestablishing tidal influence is to create a muted tide cycle by designing the drainage structures to limit the ebb and flow of bay water into and out of the project site. A muted tide cycle can allow for the establishment of salt and brackish marsh habitats, greatly improve the amount of time fish passage is provided, and improve sediment transport within the channel through increased tidal flushing.

Numerous different tidegate sizes, types, and configurations were analyzed using the TIDEGATE model. Each tidegate alternative was modeled using the 2-year and 10-year design storms and results were evaluated against the following criteria:

- Provide habitat door opening sufficiently large enough to allow filling of Martin Slough channel with each tide to at least elevation 5-feet to provide adequate volume of seawater for sediment and nutrient flushing and enlarge estuarine habitat.

- Minimize amount of time habitat door is closed to provide maximum migrational access to Martin Slough.

- Minimize flood inundation of land above elevation 6-feet, which is the approximate elevation of the top of the existing Martin Slough channel in the project area.

- Minimize amount of time velocities through the gate openings exceed fish passage criteria (2 fps for juvenile salmonids and 6 fps for adult salmon and steelhead).

- Avoid over-designing size of tidegate structure to minimize construction costs.

The "habitat door" is a gate that allows tidal waters to flow from Swain Slough into Martin Slough. Several of the alternatives use a habitat door that closes when the water surface in Martin Slough reaches elevation 5.0 feet. Table 10.1 summarizes results for several of the final tidegate alternatives.

Table 10.1 Comparisons of tidegate performance over a 7-day period using the 2-year design storm. All gate inverts were placed at elevation 0.0-ft (NAVD 88). Water surface elevations above 6.0-ft begin to cause minor flooding of the golf course and pasture. The tidegates were all evaluated using the storage-inundation curve for existing conditions.

		Conveyance Area of Gate Opening (ft^2)		2-yr Design Storm			
No.	**Gate Configuration**	**Outflow (tidegate)**	**Inflow (habitat door)**	**Water Surface Inundation Above El. 6' (hrs)**	**All Gates Closed (hrs)**	**Velocity >2 fps (hrs)**	**Velocity > 6 fps (hrs)**
(E)	3 - 4' dia. CMP's w/leaks (Existing Conditions)	37.6	2.2	16.7	75.9	42.5	3.7
1	3 - 6'x6' boxes & 2'x2' habitat door closes @ El. = 5'	108.0	4.0	10.4	33.9	10.9	0.0
2	3 - 6'x6' boxes (No habitat door)	108.0	0.0	9.8	108.4	10.9	0.0
3	3 - 6'x6' boxes & 1'x1' habitat door closes @ El. = 5'	108.0	1.0	9.8	17.7	12.9	0.0
4	3 - 6'x6' boxes & 1.5'x1.5' habitat door closes @ El. = 5'	108.0	2.3	10.1	33.9	10.9	0.0
5	2 - 6'x6' boxes & 1.5'x1.5' habitat door closes @ El. = 5'	72.0	2.3	10.6	33.5	20.5	0.6
6	2 - 8'x8' boxes & 1.5'x1.5' habitat door closes @ El. = 5"	128.0	2.3	10.3	33.8	11.3	0.0
7	4 - 6'x6' boxes & 1.5'x1.5' habitat door closes @ El. = 5'	162.0	2.3	10.0	34.2	8.0	0.0
8	3 - 6'x6' boxes & 1.5'x1.5' permanent opening	108.0	2.3	14.6	0.0	11.0	0.0

We found through our analysis that increasing the size of the outflow gates failed to decrease the peak flood elevation. However, larger outflow gates did reduce the duration of inundation. For example, under existing conditions the 2-year design storm inundates lands above elevation 6.0 feet for approximately 16.7 hours. By nearly tripling the outflow gate size, inundation above elevation 6.0 feet is reduced by almost 7 hours, a 41% decrease. Including a habitat door into the tidegate structure only had a minor affect on flood inundation times.

The size of the habitat door determines the amount of tide water that enters the project area. An opening that is too small will not create the desired muted tide upstream of the gates. Also, addition of upstream ponds or enlargement of the channel by design or through scour will increase the available tidal prism. If the habitat door is not sized large enough to fill the newly available volume, the height of the muted high tide will fail to reach its design level. An opening that is too large will either cause upstream flooding or, if the habitat door is designed to close at a given water level, will close too quickly resulting in blockage of upstream fish movement. It is best to design a habitat door that has an adjustable opening to allow for flexibility in the operation to better meet design objectives.

The main 6-ft x 6-ft tidegates could be side hinged, top hinged, or a combination of both. Side hinged tidegates open fully without much force, while top hinged gates require more force to open. Top hinged gates provide a smaller opening at lower flows, opening fully during larger flows when there is adequate water to push the gate wide open. Side hinged gates open wide at lower flows which helps to minimize water velocities to provide optimal conditions for fish passage. Multiple side hinged gates in a single structure could cause adverse hydraulic effects, since during low and moderate flow conditions there may not be enough water draining through each gate to keep them fully open throughout the ebb tide. Splitting the low flows through multiple side hinged gates could drain Martin Slough faster than desired causing the gates to close prematurely and restrict fish passage. Combining one top hinged gate and one side hinged gate allows the majority of water during lower flow conditions to drain through the side hinged gate, keeping it fully open. During higher flows there would be sufficient force to open all the gates, providing the desired hydraulic capacity.

10.7 Preferred Tidegate Design

Comparing the performance of numerous tidegate configurations to the objectives and criteria previously listed, we found Configuration 1 to be the preferred structure. It consists of three 6-ft x 6-ft openings for outflow and a habitat door with an adjustable opening that closes mechanically when the water surface upstream of the gate is greater than elevation 5.0 feet. These gates would be placed in two locations, one set of two 6-ft x 6-ft openings at the existing tidegate location and one set of a 6-ft x 6-ft opening and the habitat door approximately 100 feet to the south along the Swain Slough levee. The two locations help with exit flow dynamics from the channel and lower pond. The habitat door would be at least 2-ft x 2-ft in size, depending upon other hydraulic considerations. All of the openings were modeled with their inverts set at elevation 0.0 feet. The following two figures (Figure 10.5 and 10.6) show conceptual drawings of the tidegate configurations.

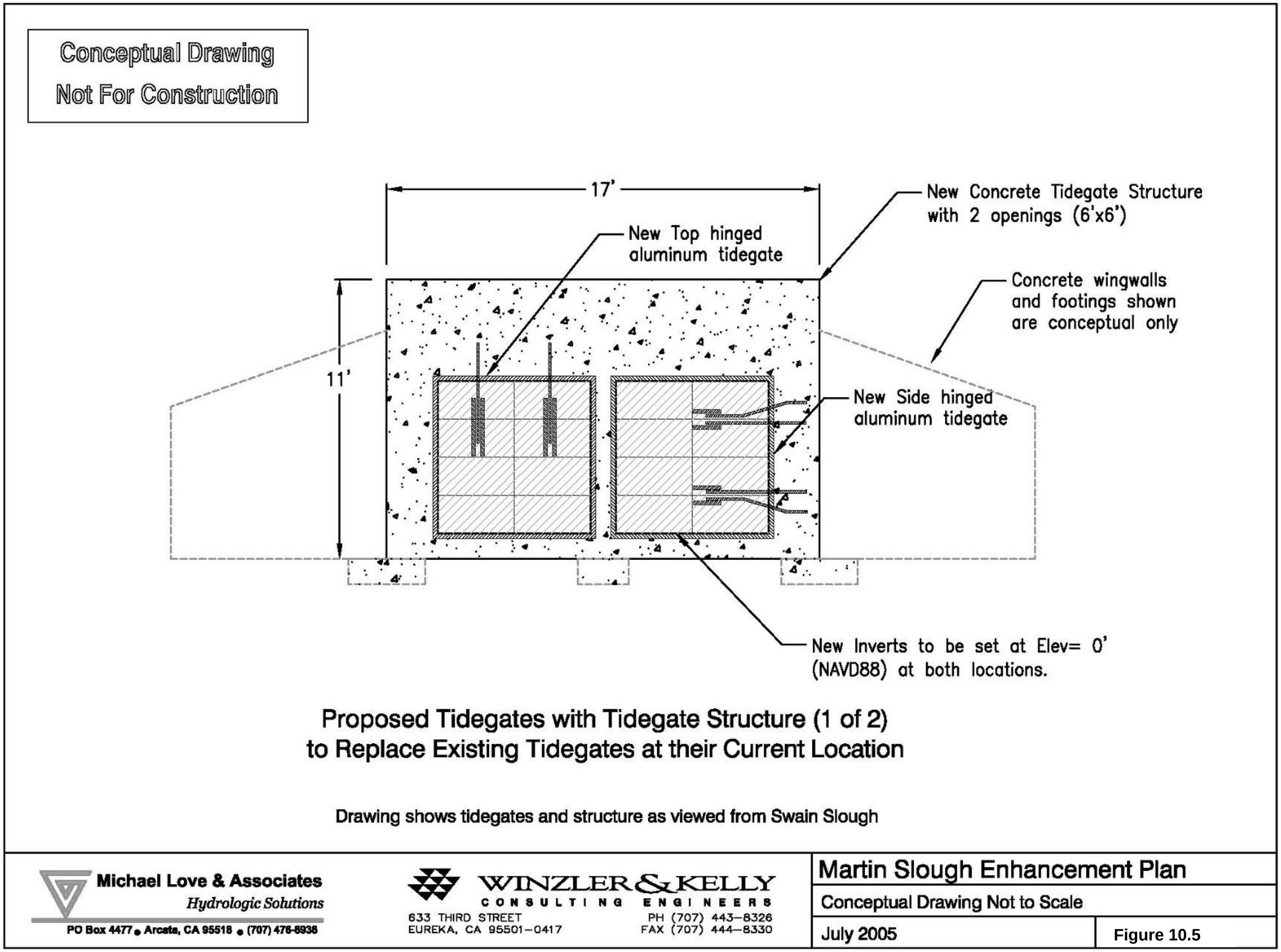

Figure 10.5 Martin Slough Tidegate Conceptual Drawing 1

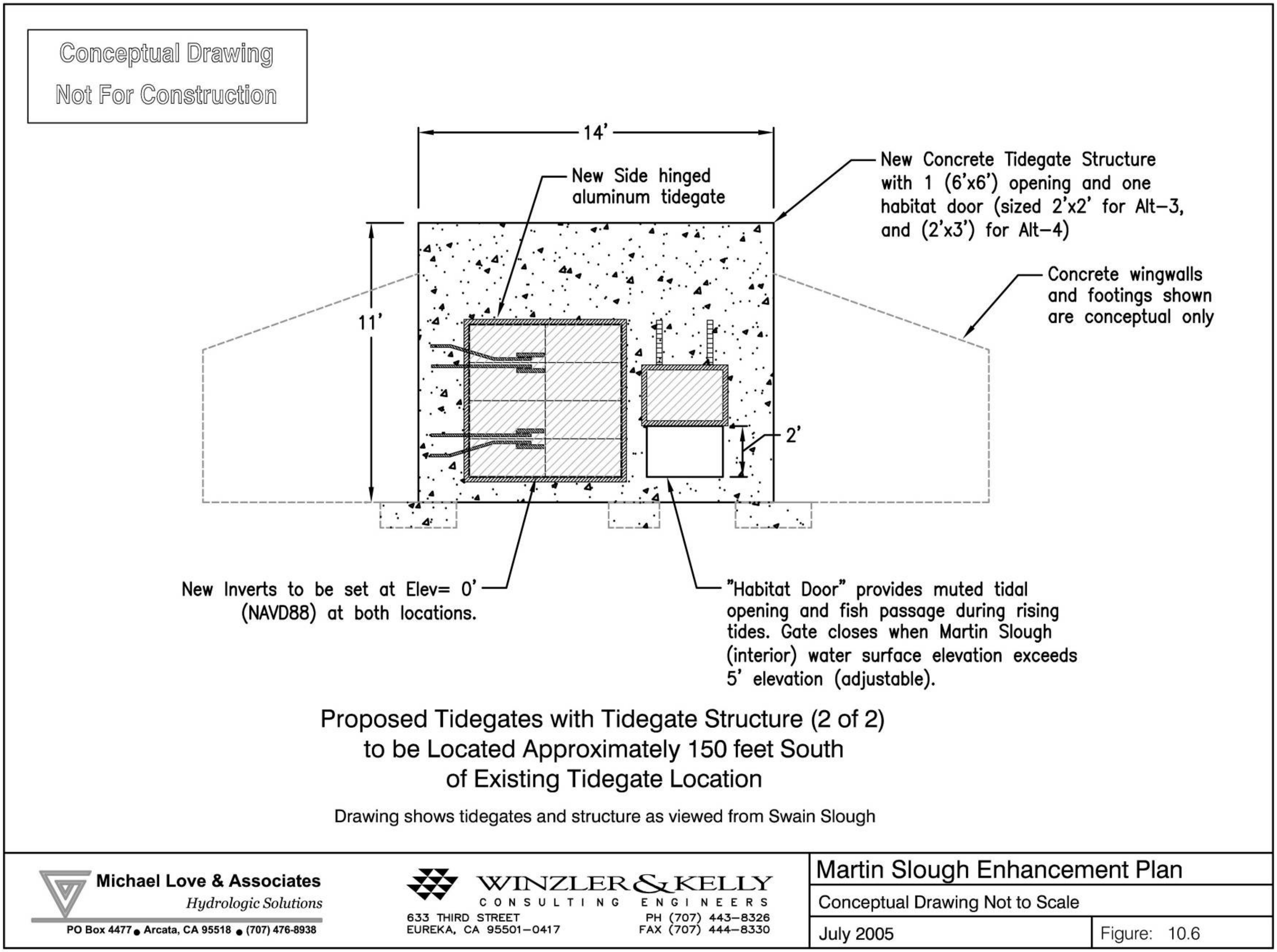

Figure 10.6 Martin Slough Tidegate Conceptual Drawing 2

11.0 FISH PASSAGE ANALYSIS

Fish passage analysis of the tidegates was conducted for the different alternatives using the lumped flow routing model described in the previous section, referred to as the TIDEGATE model. Passage conditions were evaluated using the stream crossing design criteria developed by NOAA Fisheries (2001) and CDFG (2002).

Generally, upstream fish passage can be blocked at a tidegate for several different reasons:

1. Water depth within the tidegate structure is insufficient
2. Outflow water velocities are excessive
3. All gates are closed
4. Gate not opened wide enough for the fish to swim through

Flows through the tidegate are cyclical due to the daily tidal variations within Swain Slough. As a result, hydraulic conditions (water velocities and depths) through the tidegates are a function of the tidal elevation within Swain Slough and the water surface elevation within Martin Slough. Therefore, water velocities and depths are constantly changing with the tides and streamflow.

11.1 Fish Passage Design Flows

It is widely understood that fish passage cannot reasonably be provided at all flows. Additionally, it is unlikely that fish are migrating upstream during large flood events, so providing passage during such events is unnecessary. Additionally, at small falls or shallow riffles within natural channels, fish are often delayed in migrating upstream until flow conditions change. As such, delaying upstream migration at stream crossing structures for short periods is considered acceptable in many situations. For example, it is typically not necessary to make the structure passable when flow conditions within the adjacent channel are considered impassable (e.g. shallow depths).

The lower and upper fish passage design flows are intended to define the range of streamflows in which adequate passage conditions should be provided for a specified fish species and lifestage. The goal is to design a structure that provides suitable upstream passage conditions at streamflows between these flows. When streamflows are outside the design flow range, fish passage does not need to be provided.

Fish passage design flows have been in use for many years on projects dealing with non-tidally influenced stream crossings. However, they have not generally been applied to tidal systems such as Martin and Swain Sough. For this reason, we have examined fish passage conditions for each alternative using a variety of approaches aimed at providing sufficient insight into how each alternative affects fish passage.

11.2 Fish Passage Design Flow Criteria

Fish passage design flows are defined by NOAA Fisheries (2001) and CDFG (2002). Lower and upper fish passage design flows are prescribed for adult salmon and steelhead, adult rainbow and cutthroat trout, and juvenile salmonids.

Fish passage design flow guidelines are prescribed in terms of exceedance flows, which are obtained from flow duration curves (FDC's) (Table 11.1). Flow duration is a cumulative frequency showing the average percentage of time that specific flows are equaled or exceeded. For example, the adult salmon and steelhead upper fish passage design flow is defined as the 1% exceedance flow. This is the flow at the location of interest (i.e. the tidegates) that is equaled or exceeded on average 1% of the time during a year. The remaining 99% of the time flows at that location would be less than the 1% exceedance flow.

Table 11.1 Fish passage design flow criteria, as defined by NOAA Fisheries (2001) and CDFG (2002). These criteria were applied to assessment of passage conditions at existing and proposed tidegates.

Species and Lifestage	Lower Design Flow	Upper Design Flow
Adult Salmon and Steelhead	50% exceedance flow or 3 cfs (whichever is greater)	1% exceedance flow
Adult Rainbow and Cutthroat Trout	90% exceedance flow or 2 cfs (whichever is greater)	5% exceedance flow
Juvenile Salmonids	95% exceedance flow or 1 cfs (whichever is greater)	10% exceedance flow

11.3 Martin Slough Flow Duration Curve

Streamflow within Martin Slough was gauged from 2/13/2003 to 1/8/2004 at the upper Fairway Drive crossing by Graham Mathews and Associates. Since this record is far too short to construct a representative long-term flow duration curve for Martin Slough, it was necessary to create a synthetic FDC to estimate fish passage design flows.

The FDC for Martin Slough was derived indirectly using flow records from the Little River in Humboldt County. The USGS operated Little River streamflow gauging station (Table 11.2) is relatively close to Eureka, has been in operation for a substantial period, and exhibits flow characteristics similar to other smaller low elevation coastal streams and rivers (Lang, Love, and Trush, 2004). During the period when both Martin Slough and Little River flow gages were operational, the two streams appeared to behave similarly (Figure 11.1). However, on a per unit drainage area basis, Martin Slough consistently had lower flows than Little River. The drainage area for Martin Slough at the tidegates is 5.51 square miles, and Little River 40.5 square miles.

To develop a relationship between flows in Martin Slough and Little River, a flow duration curve was created for both using only the period when both gauging stations were in operation (2/13/2003 to 1/8/2004). Then the following relationship was developed relating flows in the two streams:

$$Exceedance\_Flow_{Martin} = 0.7485 \bullet Drainage\_Area_{Martin}\left(\frac{Exceedance\_Flow_{LittleR}}{Drainage\_Area_{LittleR}}\right)^{0.8558}$$

Figure 11.1 Flow Duration Curve Equation

We constructed a synthetic flow duration curve for Martin Slough using this regression equation and the Little River flow duration curve created using all 49 years of daily average flows (Figure 11.3). The regression equation appears to provide an accurate correlation up to roughly

30 cfs/mi^2 for the Little River. Using the regression equation, this is equivalent to a flow of 75 cfs in Martin Slough.

Table 11.2 USGS streamflow gauging station utilized for developing the Martin Slough Flow Duration Curve.

Station Number	Stream Name	Drainage Area (sq. miles)	Record Length (years)	Coverage
11481200	Little River Nr Trinidad	40.50	49	1956-2004

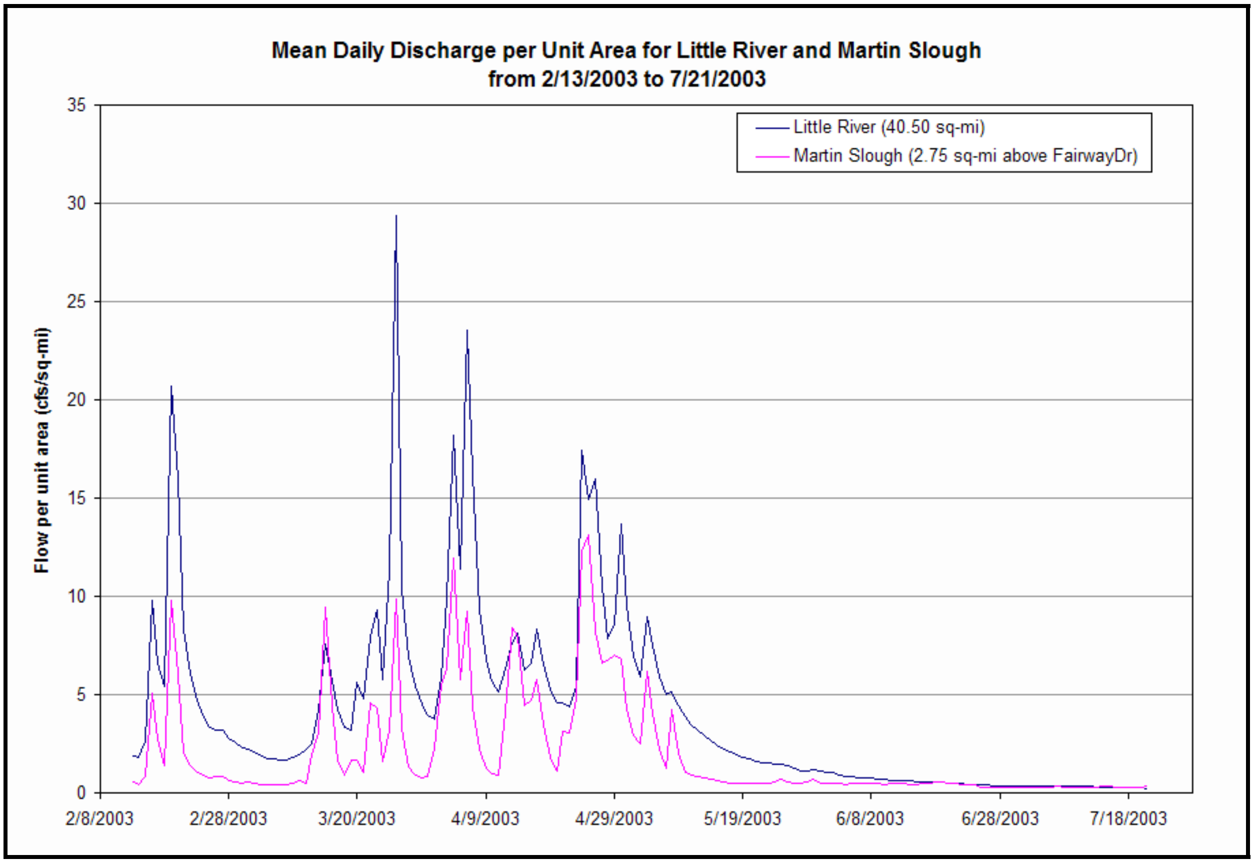

Figure 11.2 Daily average flows per unit drainage area for Martin Slough at Fairway Drive and Little River during the period from 2/13/03 to 7/21/03. Note the shape of the Martin Slough hydrograph closely resembles the Little River hydrograph. However, the flow per unit area in Martin Slough was consistently less than observed in Little River.

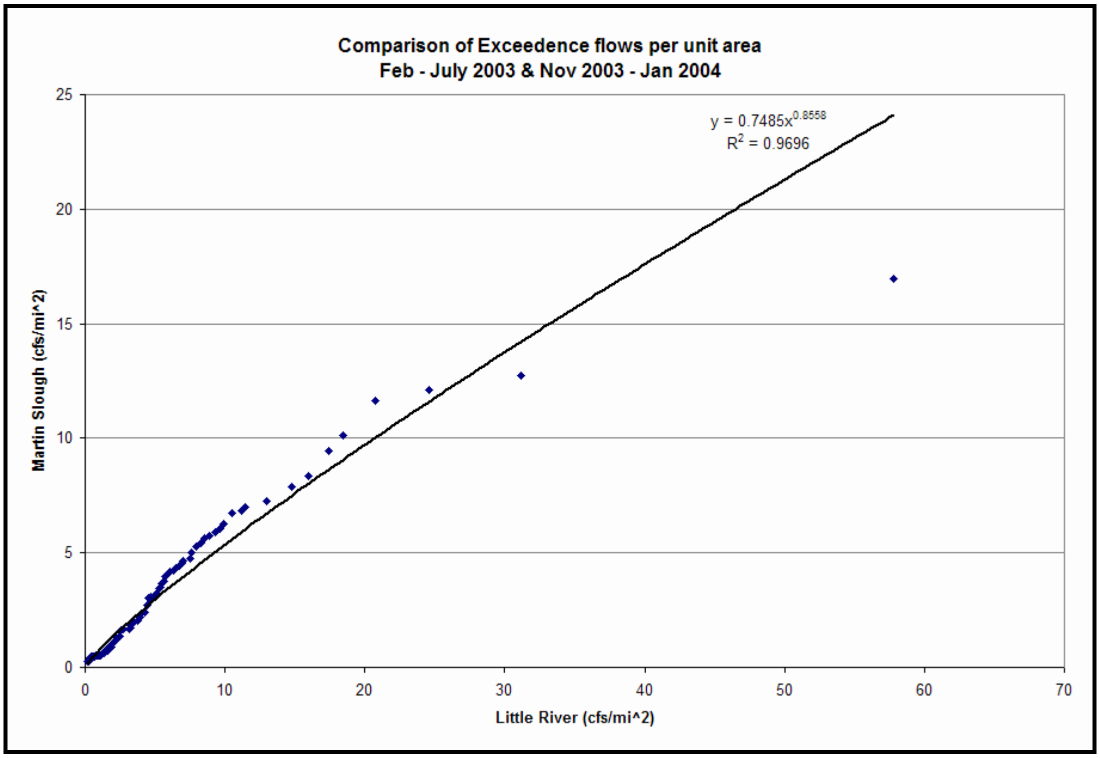

Figure 11.3 Relationship between exceedance flows for the Little River and Martin Slough during the period when both stream gauging stations were operating simultaneously. This relationship was used to create a synthetic long term flow duration curve for Martin Slough.

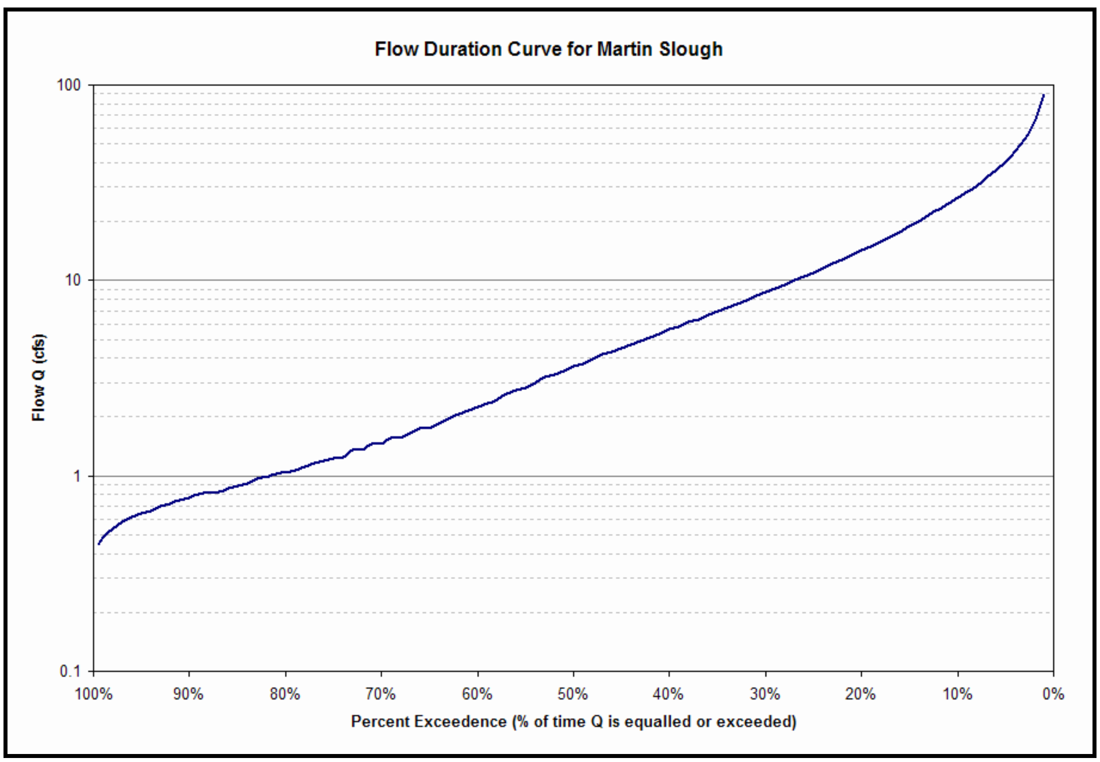

Figure 11.4 Synthetic flow duration curve for Martin Slough at the tidegates (drainage area 5.51 mi2). Curve was created based on the Little River flow duration curve and a relationship between flows in Little River and Martin Slough.

11.4 Fish Passage Design Flows for Martin Slough

Using the fish passage design flow criteria shown in Table 11.1 and the synthetic flow duration curve in Figure 11.3, fish passage design flows were calculated for the Martin Slough tidegates at the confluence with Swain Slough (Table 11.3).

Table 11.3 Fish passage design flows for Martin Slough at the confluence with Swain Slough.

Species and Lifestage	Lower Fish Passage Flow	Upper Fish Passage Flow
Adult Salmon and Steelhead	3.6 cfs	89 cfs
Adult Rainbow and Cutthroat Trout	2.0 cfs	41 cfs
Juvenile Salmonids	1.0 cfs	27 cfs

11.5 Fish Passage Guidelines

The CDFG (2002) and NOAA Fisheries (2001) fish passage guidelines prescribe minimum water depths and maximum average water velocities for fish passage at stream crossings (Table 11.4). To provide unimpeded adult and juvenile passage, depth and velocity criteria listed below should be satisfied between the lower and upper fish passage design flows. However, CDFG and NOAA Fisheries guidelines recognize the criteria cannot always be satisfied, and suggest the criteria be applied as a project goal instead of a strict requirement. Also, criteria were originally developed for stream crossings, and may not be directly applicable to tidegates.

11.6 Fish Passage Conditions

Water depth at the tidegate is controlled by the invert (bottom) elevation of the tidegate and the water surface elevations within Swain and Martin Sloughs. The lowest observed water surface elevation in Swain Slough was 1.54 feet and Swain Slough has a controlling bed elevation of approximately 1.0 feet just downstream of the tidegates. Of the three existing tidegate culverts, the lowest invert elevation is 0.9 feet. The lowest observed tide in Swain Slough was 1.54 feet. At this tide Swain Slough would backwater the culvert no more than 0.64 feet, which violates the minimum depth requirement for adult salmon and steelhead (Table 11.4). The invert of the proposed gates for Alternatives 3 and 4 are set at elevation 0.0 feet, creating a minimum water depth within the tidegate structures of 1.54 feet at the lowest observed tide, surpassing all fish passage depth requirements. Additionally, setting the invert elevation to 0.0 feet would match the channel elevation of Swain Slough at the tidegate outlet.

Table 11.4 CDFG and NOAA Fisheries fish passage depth and velocity criteria applied to the passage analysis of existing and proposed Martin Slough tidegates.

Species and Lifestage	Minimum Water Depth	Max. Water Velocity (distance < 60 ft)
Adult Salmon and Steelhead	1.00 ft	6 fps
Adult Rainbow and Cutthroat Trout	0.67 ft	4 fps
Juvenile Salmonids	0.50 ft	2 fps[1]

[1] Given the short length of the tidegate structures, a water velocity corresponding to juvenile salmonid burst swim speeds was used for analyzing juvenile passage instead of the 1 fps recommended by CDFG and NOAA Fisheries.

11.7 Water Velocities and Blockage to Passage

Water velocities at the tidegate were estimated using the TIDEGATE model. Fish passage was considered blocked when (1) the out-flowing water velocities (flowing from Martin Slough to Swain Slough) exceeded the criteria listed above or (2) when all the gates were closed, including the "habitat door" in Alternatives 3 and 4.

The first type of analysis examined conditions at the upper fish passage design flow for each species and lifestage. This was done by running the TIDEGATE model using the seven-day design tide cycle for Swain Slough combined with a constant streamflow for Martin Slough equal to the fish passage design flow. For Alternative 2, the levee breach was modeled as a 20-feet wide opening. Table 11.5 summarizes the amount of time passage conditions were satisfied for each alternative. It also reports the amount of time fish passage would be blocked due to excessive velocities or closed gates.

At the upper design flow the existing tidegates (Alternative 1) provide suitable upstream passage conditions for adult salmon and steelhead 74% of the time. As would be expected, the Alternative 2, the levee and tidegate removal option, would provide unimpeded adult salmon and steelhead passage at the upper design flow.

For the two tidegate replacement alternatives, excessive velocity for adult salmon and steelhead does not occur. However, fish are blocked approximately 22% of the time due to the gates being closed more frequently. Since the proposed gates are approximately three times larger than the existing, they drain the stored waters more quickly, resulting in them being open for slightly less time. By incorporating the habitat door into Alternatives 3 and 4, which closes when Martin Slough reaches water surface elevation 5-feet, we minimize the amount of time the doors close. Without the habitat door, the proposed tidegates would block fish passage far more often. It is also important to consider that blockage only occurs for short periods at and around high tide, imposing minimal delay on a migrating fish.

Table 11.5 The amount of time upstream fish passage conditions are satisfied at the upper fish passage design flow for Martin Slough when using the 7-day design tide cycle in Swain Slough. Summing the percent of time passable with the percent of time blocked due to excessive velocity and closed gates equals 100%.

	Conditions at Upper Fish Passage Design Flows[1]								
	Adult Salmon & Steelhead			Adult Rainbow & Cutthroat Trout			Juvenile Salmonids		
		% of Time Blocked			% of Time Blocked			% of Time Blocked	
Design Alternatives	% of Time Passable	Excessive Velocity[2]	Tidegates Closed	% of Time Passable	Excessive Velocity[2]	Tidegates Closed	% of Time Passable	Excessive Velocity[2]	Tidegates Closed
1 – Existing conditions	**74**	8	18	**57**	8	35	**15**	43	42
2 – No tidegates or levee	**100**	0	0	**98**	2	0	**95**	5	0
3 – New tidegates and new ponds	**78**	0	22	**83**	0	17	**73**	13	13
4 – New tidegates and ponds, modified channel	**79**	0	21	**83**	0	17	**64**	21	15

Notes:

[1]Upper Fish Passage Design Flows for Martin Slough at Swain Slough:
Adult Salmon & Steelhead, 1% Exceedance (89 cfs)
Adult Rainbow & Cutthroat Trout, 5% Exceedance (41 cfs)
Juvenile Salmonids, 10% Exceedance (27cfs)

[2] Velocity criteria (velocities in downstream direction):
6 fps for Adult Salmon and Steelhead,
4 fps for Adult Rainbow and Cutthroat Trout, and
2 fps for Juvenile Salmonids.

Passage conditions for juvenile salmonids at the upper design flow varied the most between alternatives. Under existing conditions passage was only provided 15% of the time, while the new tidegate alternatives provided suitable juvenile passage conditions at the upper design flow as much as 73% of the time. It is interesting to note that even for the levee breach alternative, Alternative 2, juvenile fish were blocked 5% of the time due to excessive velocities. This suggests that even under natural conditions average water velocities within the slough channels can, at times, likely exceed the 2 ft/s threshold.

11.7.1 Passage Conditions during the 2-year Design Storm

We also examine passage conditions over the entire period of design storm resulting from the 2-year 24-hour rainfall event. This was done by running the TIDEGATE model using the seven-day design tide cycle for Swain Slough combined with the 7-day design flow hydrograph for Martin Slough. Results were summarized by the number of hours velocities exceeded the 2 ft/s and 6 ft/s thresholds, as well as the total hours the gates were closed to fish passage over the 7-day period (Table 11.6).

During the 2-year design storm the TIDEGATE model estimated the existing gates being closed nearly 76 hours (45% of the time during the 7-day period). In contrast, the tidegates in Alternatives 3 and 4 were only closed 13.8 hours and 19.0 hours, respectively.

11.7.2 Other Considerations

In addition to depth and velocity barrier conditions that were modeled, the tidegates themselves can create difficulties for fish passage. Although difficult to quantify, observations at this and other installations have shown that the heavy cast-iron top-hinged gates that are currently installed at Martin Slough appear to not open wide enough for fish to pass through (Figure 11.5). New proposed gates for Alternatives 3 and 4 are made of lighter aluminum that open with little effort. Additionally, the side-hinged gates swing fully open to provide a very large area for fish to swim through.

Table 11.6 Amount of time fish passage conditions were not met during the 2-year design storm. The model was run for a period of seven days (168 hours).

2-yr Design Storm, 7-day Period			
	Hours[1] Velocity Greater than		
Alternative	2 fps	6 fps	Hours[1] Tidegates are Closed
1 - Existing Conditions	42.5	3.7	75.9
2 - Remove tidegate / Levee Breach *	18.3	0.9	0.0
3 - New tidegate, storage ponds	23.3	0.0	13.8
4 - New tidegate, storage ponds, modified channel	35.9	0.0	19.0

Notes:

** Simulated with a 20 ft wide opening*

[1] Cumulative hours during 7 days of continuous flow modeled through tidegate

Figure 11.5 Existing Martin Slough tidegates during ebb tide. At lower flows the top-hinged tidegates may not open wide enough for adult salmon and steelhead to swim through.

12.0 MARTIN SLOUGH CHANNEL SIZING

Alternative 4 involves enlarging the Martin Slough channel within the project area to increase conveyance area for both flood flows and diurnal tidal exchange. This requires defining a design channel shape, including top widths, bottom widths, bottom elevation, and side slopes of banks.

Reintroducing a muted tide cycle into the project area will result in large volumes of water flowing up and down the channel with each tide cycle, changing the fluvial processes that maintain the channel. Unfortunately, widely used sizing techniques typically do not address channels that are tidally influenced (Rosgen, 1996; Montgomery and Buffington, 1993). Instead, we relied on using design guidelines developed for tidal channels.

12.1 Methods for Sizing Slough Channels

In estimating stable channel dimensions for the tidally influenced sections of Martin Slough, we assumed the channel would function hydraulically and geomorphically as a tidal slough channel. Hydraulics of muted daily tidal fluctuations are anticipated to control stable channel dimension for Alternative 4, rather than hydraulics caused by less frequently occurring storm flows draining from the watershed. Designing a channel that is larger than appropriate may result in the channel being unable to adequately flush sediments, filling-in until it reaches a stable size. If the designed channel is undersized, the constant tidal ebb and flow will scour the channel banks and bed, causing the channel to widen and deepen until it reaches equilibrium. This condition is discussed in more detail on pages 68 and 73.

Our approach was to size the channel using design guidelines developed for tidal channels and then model the new proposed channel using the 2-dimensional hydraulic model to determine its ability to also convey storm flows. We utilized hydraulic geometry relationships developed by Philip Williams & Associates (1995; 2004). They were developed primarily from slough channels found within the San Francisco Bay region. However, they also included data from slough channels from the Monterey Bay and San Diego Bay areas. Since slough channel geometry is mostly a function of the tidal prism (volume of water exchanged during a tidal cycle) and physical properties of the soils, these relationships are considered widely applicable and not limited to the locations in which they were developed. One note of caution is that most of the slough channels used to create the relationships drain salt marshes, and are not directly connected to large freshwater inputs.

A series of three different regression equations were used. They relate the diurnal tidal prism (volume of water exchanged) between mean lower low water (MLLW) and mean higher high water (MHHW) within the slough channel to the:

(1) Cross sectional area of channel below MHHW
(2) Channel top width at MHHW
(3) Maximum channel depth below MHHW

The MHHW level for the muted tide cycle in Martin Slough was estimated to be 5.0-feet, equal to the elevation at which the habitat door would be designed to close. Based on the observed tide cycles in Swain Slough, the MLLW for Martin Slough was estimated to be 2.0-feet. The new bottom elevation of the channel was placed at elevation 0.0-feet, equal to the elevation in which the inverts of the tidegates would be placed. The design cross section is trapezoidal, as recommended by Philip Williams & Associates (1995). Based on numerous channel cross sections we surveyed throughout Martin Slough, the native bank material appeared to be stable at approximately 1.5(H):1.0(V) side slopes (Figure 12.1).

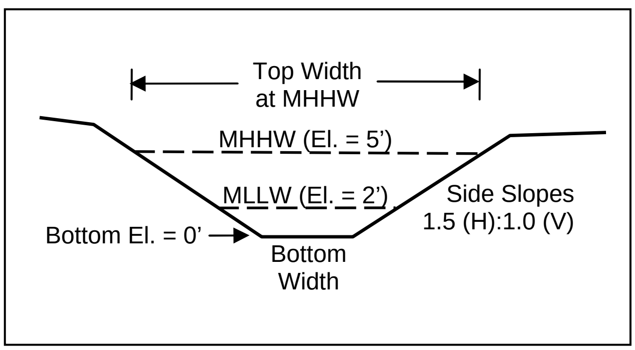

Figure 12.1 Typical channel cross sections for Martin Slough.
Top and bottom width varies with location.

The tidal prism is composed of the volume within the contributing portions of the slough channel plus the volumes of any contributing ponds or marshes. The further up the channel you travel the smaller the tidal prism becomes, causing stable channel dimensions to decrease. To account for the changing tidal prism we divided the channel into reaches (Figure 12.2).

With the length, bottom elevation (0 feet), shape (trapezoidal), and side slopes (1.5H:1V) for each channel reach selected, we solved for the channel top width, depth, and area. Finding a satisfactory solution was iterative since the tidal prism is partially a function of the channel shape. Table 12.1 summarizes the estimated stable slough channel dimensions for each reach.

Figure 12.2 Martin Slough was divided up into reaches for determining new channel dimensions. The main channel was divided into five individual reaches. The two tidally influenced tributaries are denoted as A and B.

Table 12.1 Estimated stable channel dimensions for modified stream reaches within Martin Slough as part of Alternative 4. Dimensions were derived from published relationships.

Reach	Length (ft)	Contributing Tidal Prism (acre-ft)	Top Width at MHHW (ft)	Bottom Width (ft)
1	2,520	29.3	60	45
A	750	0.4	15	5
2	2,415	20.3	50	35
3	780	9.2	35	20
4	514	2.9	25	10
B	270	0.5	20	5

12.2 References

Montgomery, D.R. and Buffington J.M. 1993. Channel classification, prediction of channel response, and assessment of channel condition. Report TFW-SH10-93-002. WA State Timber/Fish/Wildlife Agreement.

Philip Williams & Associates, Ltd. and P.M. Faber. 2004 *Design guidelines for tidal wetland restoration in San Francisco Bay.* Prepared for The Bay Institute and CA State Coastal Conservancy, Oakland CA. 83 pp.

Philip Williams & Associates, Ltd. 1995. *Design guidelines for tidal channels in coastal wetlands.* Prepared for the USACOE Waterways Experiment Station. 40 pp.

Rosgen, D. 1996. Applied river morphology. Wildland Hydrology, Pagosa Springs, CO.

13.0 HYDRAULIC ANALYSIS

A computational model that calculates water level and current velocity was developed to evaluate and compare project alternatives. This section provides the objectives of the modeling effort together with brief descriptions of the model and development for the Martin Slough study area. For the full hydraulic analysis report, see Appendix B.

13.1 Goals and Description of Model

Goals of the hydraulic modeling are to evaluate and compare project alternatives in terms of inundation levels, inundation duration, and sediment transport for 2-year and 10-year storm events.

Hydraulic modeling was conducted with the two-dimensional finite-element model, ADCIRC (Luettich et al 1992). This model was selected for the analysis because of its great flexibility in representation of bathymetric and topographic features, robust wetting and drying capabilities, representation of discharge and stage inputs, and proven performance for overland flooding calculations. ADCIRC calculates water-surface elevation and two horizontal components of current velocity on a finite-element mesh. This type of mesh allows for great detail to be specified where needed, such as the stream channels in the Martin Slough study area, and for coarser resolution in regions where detailed calculations are not needed, such as the higher-elevation areas of the subject study area.

13.2 Model Development and Approach

Development of the ADCIRC model for Martin Slough required topographic information to represent the stream, drainage ditches, ponds, and upland areas. Two sources of topographic information were utilized in the model. A digital terrain model containing 2-ft contours, from the City of Eureka, provided wide-area topographic information. A focused supplemental stream and bank survey conducted for the present study provided detailed cross-sectional elevations at selected locations in the study area. Topographic information from both of these sources was combined and applied to generate the computational mesh for the existing condition. All action alternatives involved modification of the existing condition mesh.

Topographic data were provided with the vertical datum being NAVD 88 and horizontal coordinates of California State Plane Zone 1 with all units in feet. For application within ADCIRC, these data were converted to the vertical datum of Mean Sea Level at North Spit Coast Guard Station in units of meters and geographic horizontal coordinates (longitude and latitude) in units of decimal degrees. All calculations and grids shown herein have been converted back to NAVD 88 (feet).

The modeling approach taken in this study was to conduct simulations for the various alternatives in which the model results could be directly compared in terms of inundation area and inundation duration. Inundation events were specified as tributary input for 2-yr and 10-yr design storms. The watershed model was applied to calculate tributary discharges from the four primary drainages entering the Martin Slough study area. These discharges were then provided to the hydraulic model as upstream input.

Downstream input varied among the alternatives according to the specific configuration. Input was specified as water-surface elevation, and if tidegates were present, discharges combined with water-surface elevation. For simulations representing tidegates, the discharges and water-surface elevations were calculated by the separate TIDEGATE model.

The computational mesh for the No-Action Alternative is shown in Figure 13.1. The mesh was developed from the topographic surface data. Each node represents a computational point in the hydraulic model. This mesh is comprised of 6,586 nodes and 12,624 elements. The downstream boundary, located at the confluence of Martin Slough and Swain Slough represents the tidegate configuration presently in place.

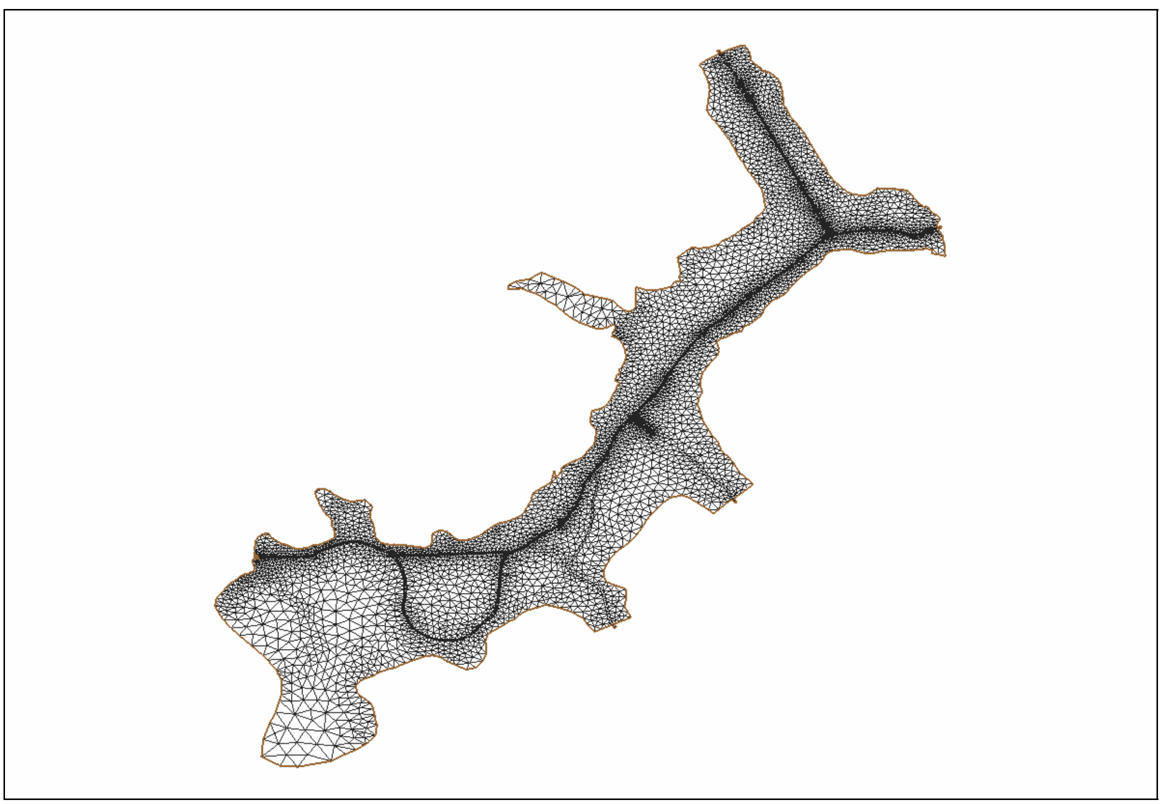

Figure 13.1 Computation mesh for the No-Action Alternative (Existing Condition).

The computational mesh for Alternative 4 (modified tidegates, additional storage ponds, and modified channel) is shown in Figure 13.2. This mesh is comprised of 7,706 nodes, 14,861 elements. Downstream tidegate boundaries are located at the confluence of Martin Slough and Swain Slough, and at the proposed tidegate location.

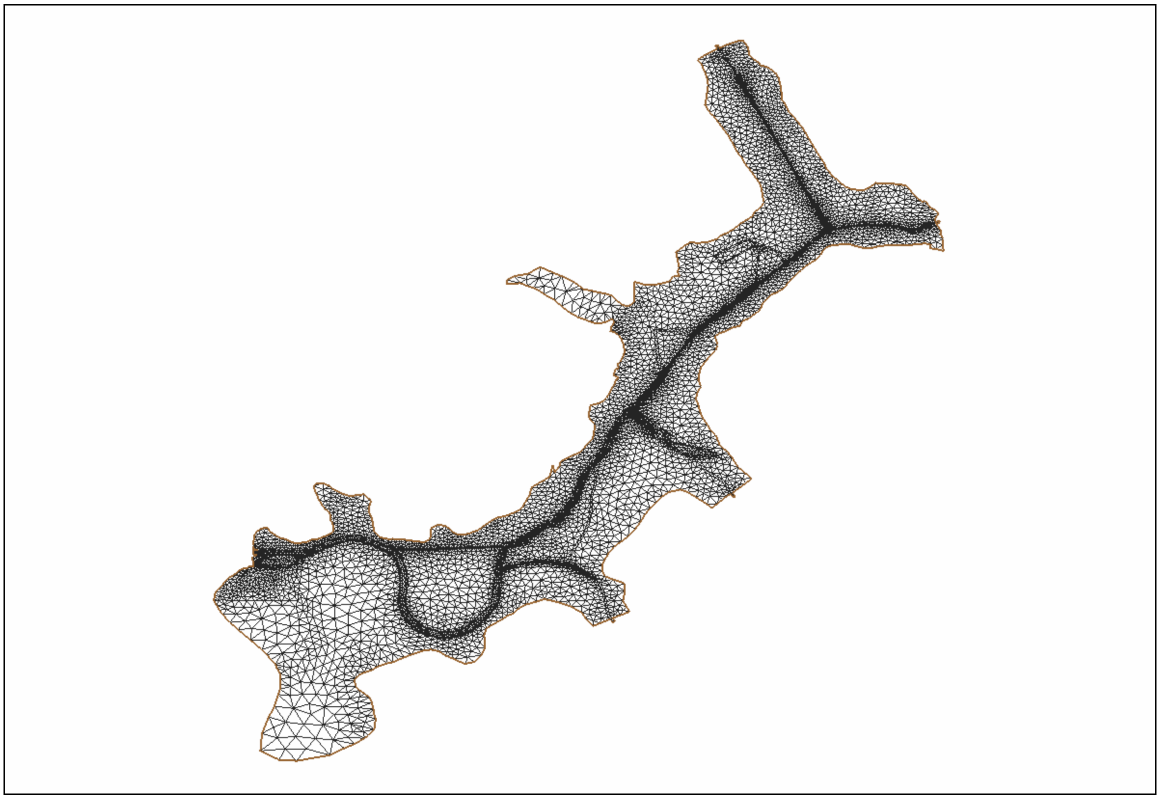

Figure 13.2 Computation mesh for the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative.

Details of each mesh at the downstream end are provided to show individual configurations. Figure 13.3 shows the downstream mesh region for the No-Action Alternative 1. Figure 13.4 shows the downstream mesh region for Alternative 4.

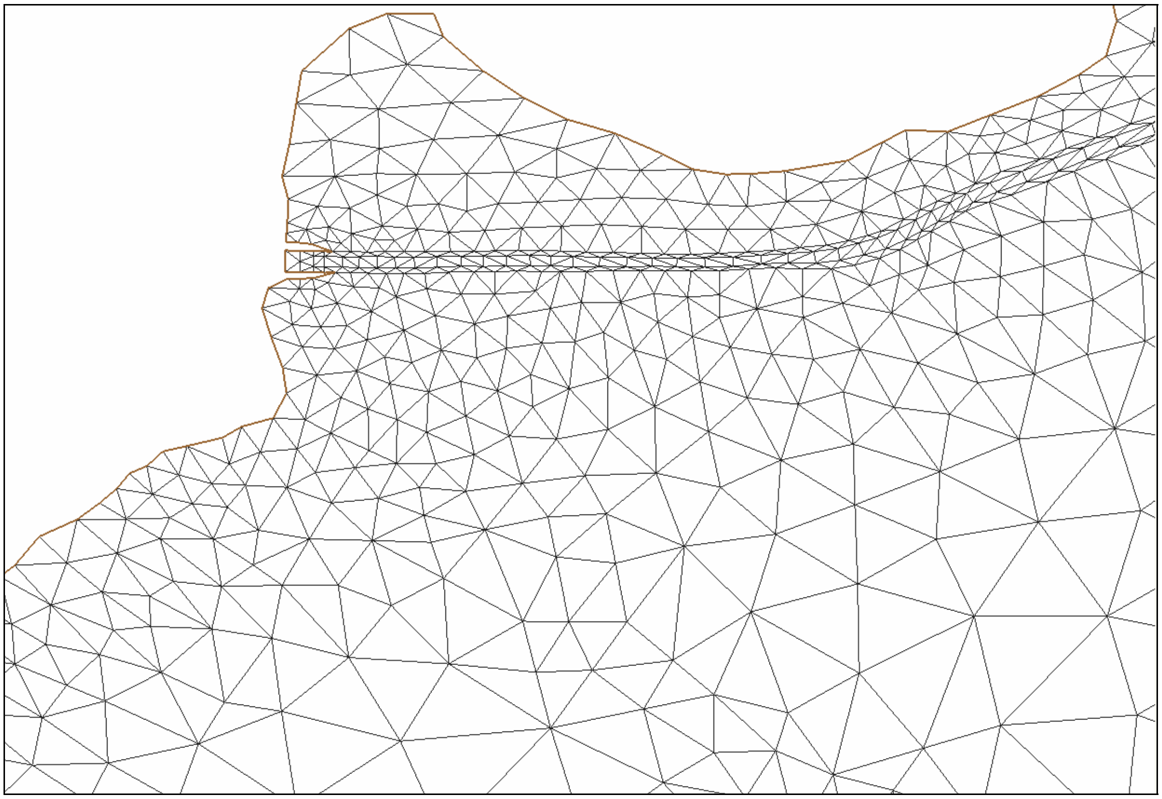

Figure 13.3 Downstream mesh detail for the No-Action Alternative (Existing Condition).

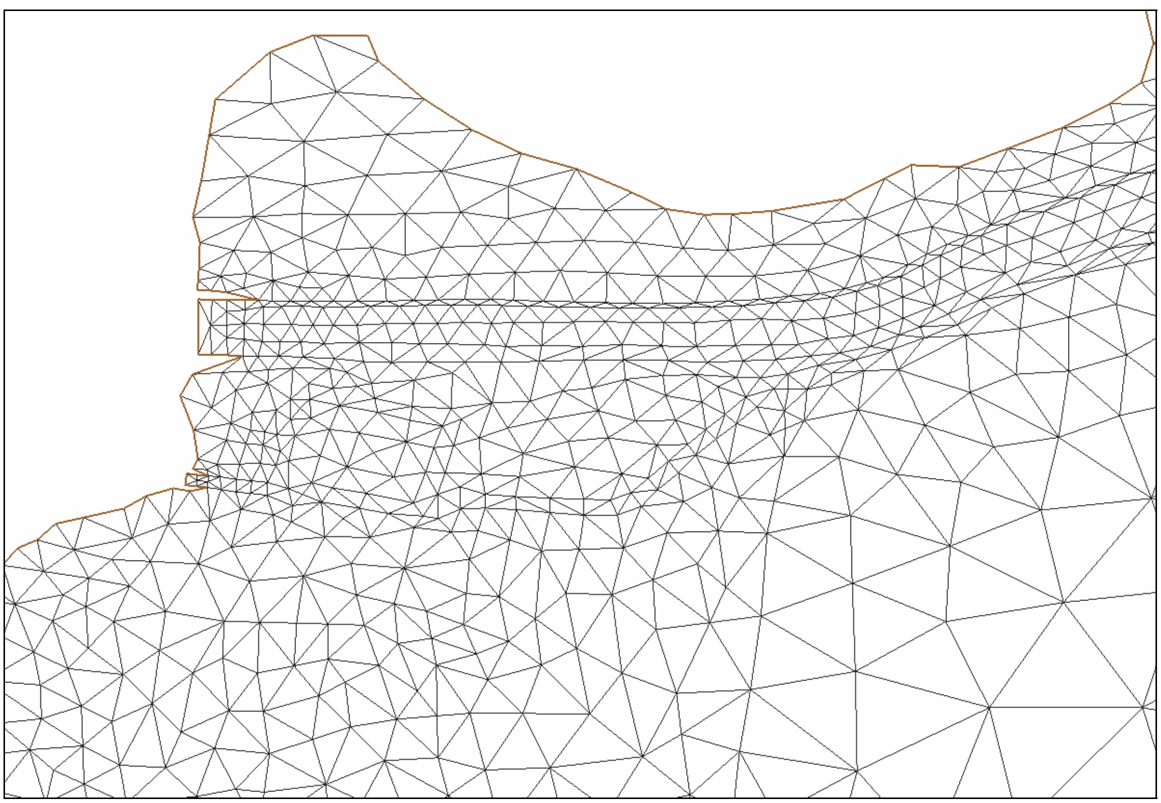

Figure 13.4 Downstream mesh detail for the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative.

Contour plots of mesh topography are shown in Figures 13.5 and 13.6 for the No-Action Alternative 1 and Alternative 4, respectively. Figure 13.6 shows the additional storage provided by the new ponds, increased area of existing ponds, and the widened and deepened Martin Slough channel together with a new channel extending west from the southernmost tributary and entering Martin Slough in the large channel bend.

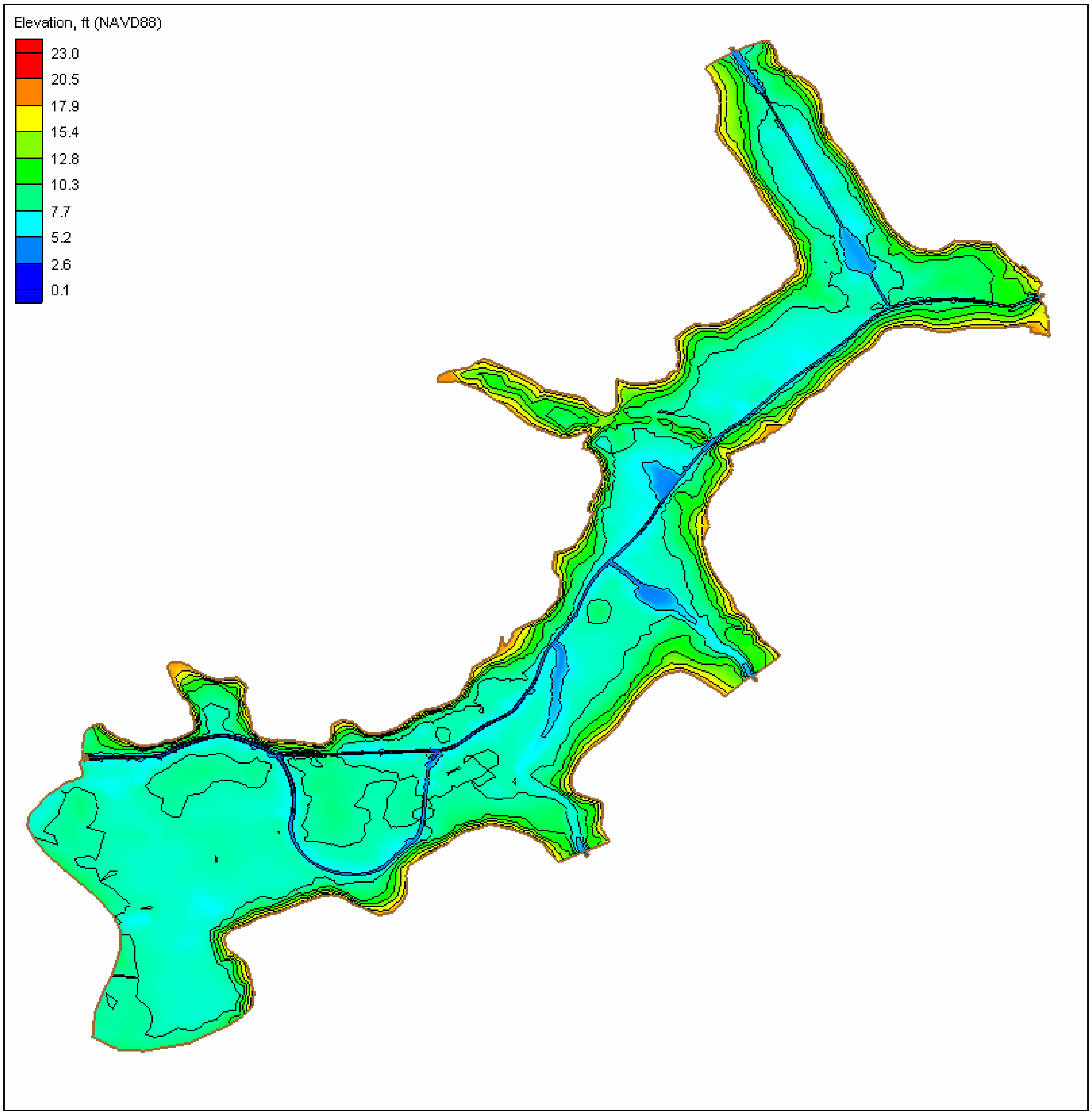

Figure 13.5 Mesh topographic surface for Alternative 1, the No-Action Alternative (Existing Conditions).

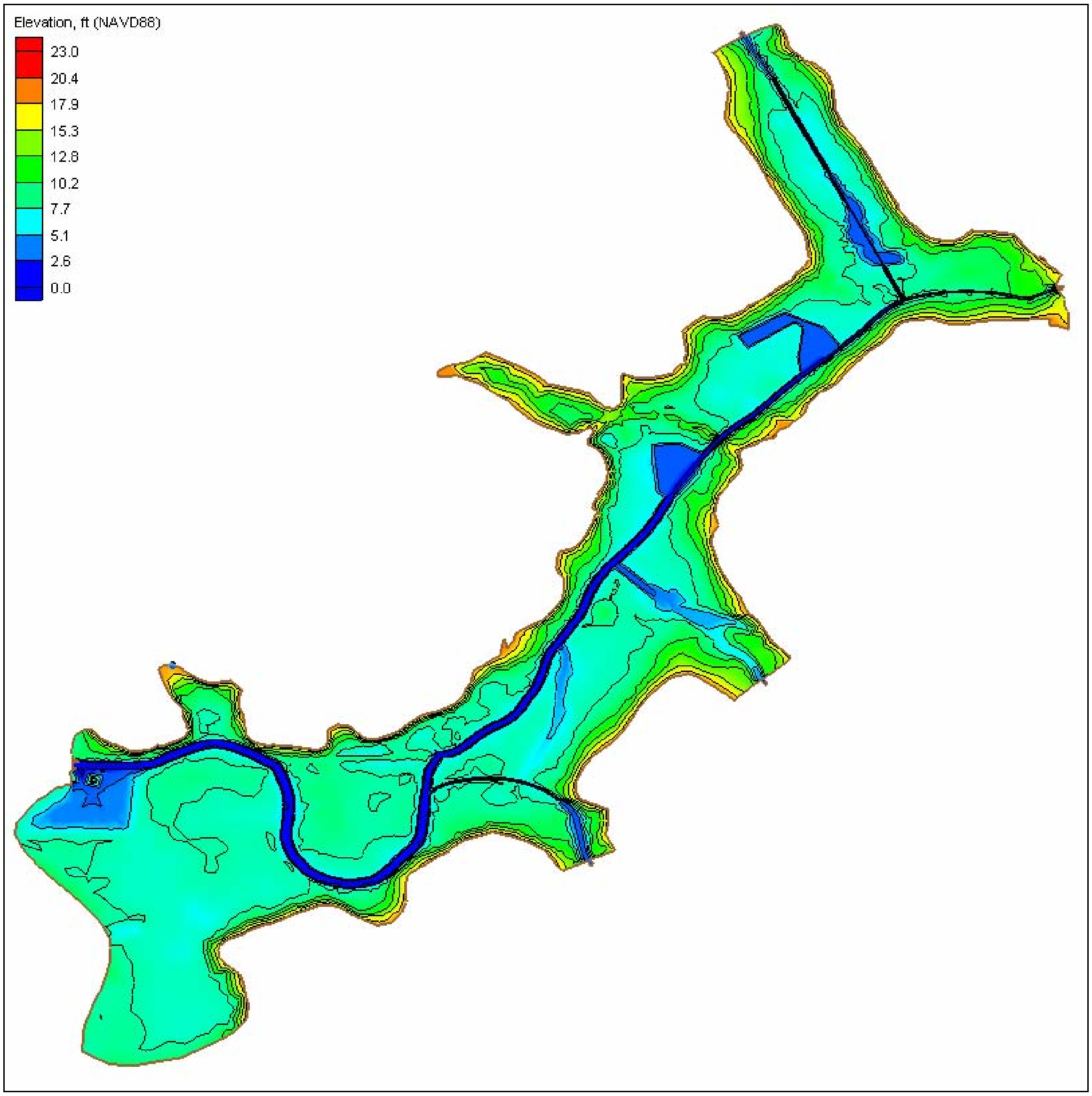

Figure 13.6 Mesh topographic surface for Alternative 4the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative.

Forcing of the boundary conditions in the hydraulic model for the Martin Slough study area was specified as tributary discharges, tidegate discharges, and downstream water-surface elevation values. Discharges were calculated by the watershed model and tidegate model for alternatives in which tidegates are present. Downstream boundaries were forced with water-surface elevation values. For Alternatives 3 and 4 with tidegates, boundary forcing included utilizing water-surface elevations predicted by the TIDEGATE model at nodes just upstream of the tidegate location, and water discharge volumes calculated by the TIDEGATE model at the tidegates.

Simulations of flood inundation were computed for 2-year and 10-year events for each alternative. All simulations were conducted with the following computational specifications: time step of 0.02 seconds, friction coefficient of 0.007 (dimensionless), and eddy viscosity coefficient of 5 m^2/s. Resolution along the Martin Slough channel has typical along-channel spacing of about 18 ft, with spacing ranging from approximately 14 to 40 ft. Maximum node spacing in the mesh is about 160 ft.

The finite-element mesh was developed, calibrated, and verified for water level over a 7.6-day time frame starting on February 12, 2003. Data availability for model verification was limited. Water-level gages deployed during the February 2003 measurement interval were located near the existing tidegates, one on the downstream side and one just upstream. A comparison of measured and calculated water level at the location of the upstream gauge is shown in Figure 13.7. During the first approximately 0.7 days of the simulation, the calculated water level is lower than the measured due to the required time for the model to adjust to the boundary forcing, which is an initial interval in which the model input is slowly brought up to full forcing. After this initial adjustment period, calculated and measured water levels are nearly identical, indicating good agreement. Because water-level data are not available for locations away from the existing tidegate, verification of water level upstream was not possible for this study.

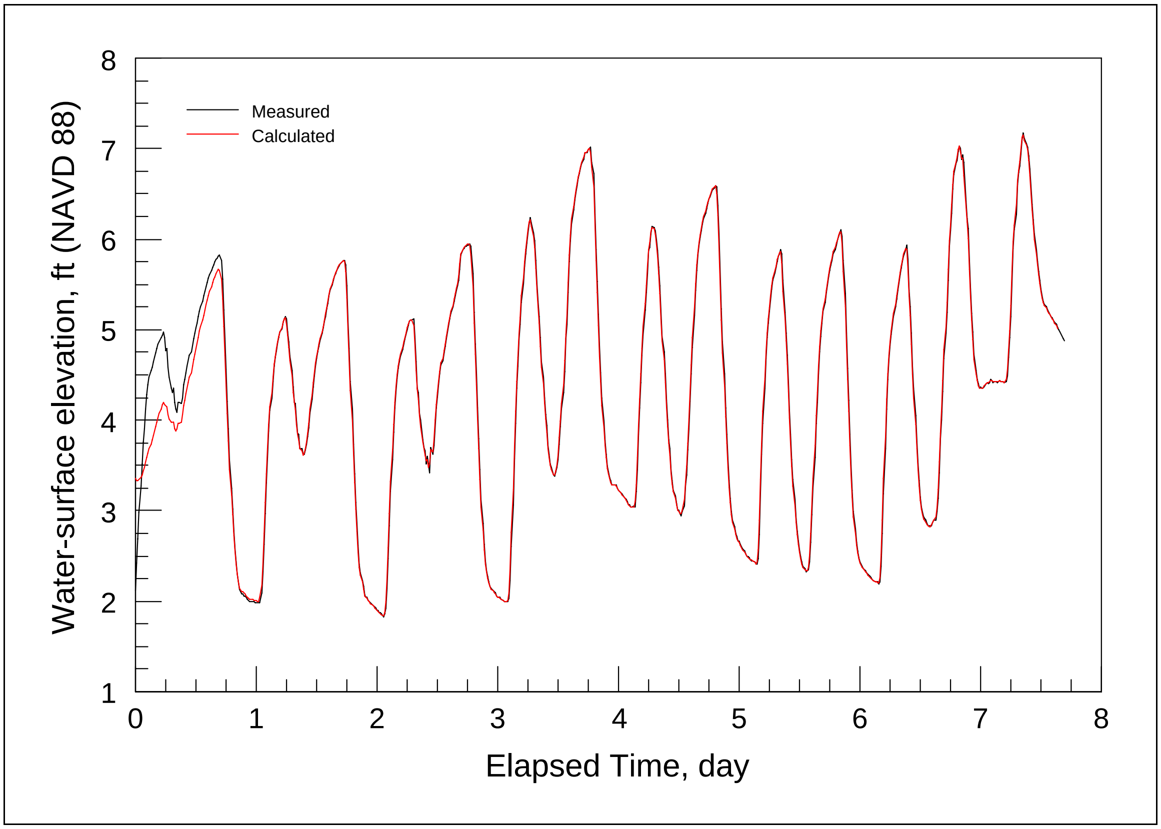

Figure 13.7 Comparison of measured and calculated water level for the February 2003 calibration period at the location of the water-level gauge located just upstream of the existing tidegate.

13.3 Sediment Transport

Sediment transport considerations were based on a simplified analysis. Sediment transport modeling was not part of the project scope. Results presented are based on calculated velocities, grain size range at the site, and water depth. The circulation model ADCIRC does not calculate sediment transport so locations of erosion and deposition must be inferred from the model results. Sediment samples from the Martin Slough channel consisted of sands, clays, and silts. Sand size ranged from medium sand (0.50 to 0.25 mm) to very fine sand (0.10 to 0.05 mm). Medium sand having a diameter of 0.50 mm in water depth of 1 m (3.28 ft) requires a current speed of approximately 2 ft/s (0.6 m/s) for initiation of suspension (Militello and Kraus 2001). Sand of this diameter in shallower water, or sand having smaller diameter and in 3.28 ft or less of water requires weaker velocity for initiation of suspension. For example, 0.50 mm sand in 0.66 ft (0.2 m) of water requires a current speed of about 1.44 ft/s (0.44 m/s) for initiation of suspension and 0.10 mm sand in 0.66 ft (0.2 m) of water depth requires a current speed of about 1.08 ft/s (0.33 m/s) for initiation of suspension. Thus, for the range of sand sizes found in the Martin Slough channel and the channel depths under normal and flood conditions, velocities in the range of 1 to 2 ft/s are sufficient to initiate the sand into suspension, after which it can be carried downstream. Because the transport of cohesive material is significantly more complex, we assume here that clays and silts will be initiated into suspension over the same range of current speeds as sand. The sediment transport analysis also assumes that material is lying on the channel bed, and not caught in vegetation or armored by debris.

Deposition of material will begin to occur if the current speed is less than that required for initiation of suspension. Because the Martin Slough channel contains mixed grain sizes, deposition will begin to occur over a range of velocities with dependence on grain properties and water depth. Here, we take a simplified approach and assume that deposition will begin to occur below the lowest current speed required for initiation of suspension for fine sand. Thus, it will be assumed that deposition can take place in the Martin Slough channel at velocities less than 1 ft/s.

The Martin Slough watershed consists of soils that are prone to erosion. Although not included in the scope of this project, locating and designing sediment basins would be helpful to minimize the impact of sediment transfer into the channel and adjacent storage ponds. The golf course currently maintains several small sediment basins in known trouble areas. With the difficulty in transporting sediments through a low gradient system, efforts to reduce the sediment load in the up slope areas and removing sediments prior to them reaching the main channel are recommended.

13.4 Hydraulic Analysis References

Luettich, R. A., Westerink, J. J., and Scheffner, N. W. (1992). *ADCIRC: An advanced three-dimensional circulation model for shelves, coasts, and estuaries; Report 1: Theory and methodology of ADCIRC-2DDI and ADCIRC-3DDI*. Technical Report DRP-92-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Militello, A., and Kraus, N. C. 2001. *Shinnecock Inlet, New York, Site Investigation, Report 4, Evaluation of Flood and Ebb Shoal Sediment Source Alternatives for the West of Shinnecock Interim Project, New York*. Coastal Inlets Research Program Technical Report ERDC-CHL-TR-98-32. U. S. Army Engineer Research and Development Center, Vicksburg, MS.

14.0 WETLAND AND BIOLOGICAL RECONNAISSANCE

The properties known as APN 301-031-08 and 18 are the Eureka Municipal Golf Course and APN 301-161-03 and 301-221-01 are owned by Gene Senestraro. The subject properties are located in south central Eureka off of Fairway Drive and just south of the City of Eureka off of Pine Hill Road (See Figure 14-1 for project boundaries, wetlands locations, soil pit locations, osprey nest locations and Lyngbye's sedge locations.) The wetlands investigation and sensitive plant survey covered approximately 40 acres total, which is bordered by Swain's Slough to the south and Eureka Municipal Golf Course, holes 1-9 to the north. Parcels APN 301-161-03, 301-221-01 and portions of 301-031-08 are in the coastal zone.

14.1 Investigation Purpose

The purpose of this investigation was to determine the approximate size and location of wetlands, sensitive plants, and sensitive animal locations within the potential footprint of alternative projects developed as part of the Martin Slough Enhancement Feasibility Study. This effort is a planning level effort intended to help guide the selection of a preferred alternative by providing information regarding biological resources that may impact or be impacted by alternative projects.

14.2 Wetlands Investigation & Sensitive Plant Survey Methodology

The wetlands investigation was conducted by Winzler & Kelly on June 23, 2005, following the Army Corps of Engineers (COE) criteria from the Corps of Engineers Wetlands Delineation Manual (1987). Vegetation and soils data were collected throughout the site. Hydrologic conditions were observed. Primary determination of the wetlands boundary was made based on vegetation, hydrology and soil characteristics. Coastal Commission requires only one wetland characteristic to be present to determine a wetland boundary, therefore the uplands found were determined to lack hydric soils, wetland hydrology and wetland plants.

The methodology for the sensitive plant survey for the Martin Slough project area included the review of topographic maps, aerial photography maps, and the Eureka Quad California Department of Fish and Game Natural Diversity Data Base prior to and during the survey to determine potential sensitive species occurrence.

The surveys were conducted following protocol developed by James Nelson for the California Department of Fish and Game (DFG 2000). An intuitively controlled, seasonally appropriate survey was conducted that sampled the identified potential habitat. The survey was high in coverage (95-100%). Plants were identified to the lowest taxonomic level (genus or species) necessary for rare plant identification. The scientific nomenclature follows the Jepson Manual (Hickman 1993).

14.3 Sensitive Plant Species Historically Reported Near Site

The California Natural Diversity Database (CNDDB) includes historical records for eight species within the Eureka 7.5 minute USGS quadrangle:

1) The pink sand-verbena (*Abronia umbellata* ssp. *breviflora*) is attributed to numerous collections on North Spit.

2) The marsh milkvetch (*Astragalus pycnostachyus* var. *pycnostachyus)* was reported historically in salt marshes near Samoa.

3) Lyngbye's sedge (*Carex lyngbyei*) was reported near North Spit and Eureka Slough.

4) Oregon coast Indian paintbrush (*Castilleja affinis* ssp. *litoralis*) had been reported in 1918 from the coastal dunes of the Eureka vicinity.

5) Humboldt Bay owl's clover (*Castilleja ambigua* ssp. *humboldtiensis*) occurs from nearby Elk River Slough in 1986, and other salt marsh habitats throughout Humboldt Bay.

6) Pt. Reyes bird's beak (*Cordylanthus maritimus* ssp. *palustris*) known from a nearby 1987 collection site on Elk River Spit, and widespread salt marsh habitats in Humboldt Bay.

7) Humboldt Bay wallflower (*Erysimum menziesii* ssp. *eurekense*) is known from widespread North and South Spit dune habitats.

8) Pacific gilia (*Gilia capitata* ssp. *pacifica*) from an old collection noted as the sandy field behind Bucksport.

9) Dark-eyed gilia (*Gilia millefoliata*) occurs from nearby Elk River sand spit in 1998, and other dune habitats throughout Humboldt Bay.

10) Sand pea (*Lathyrus japonicus*) known from a nearby 1915 collection site on Elk River Spit.

11) Beach layia (*Layia carnosa*) is known nearby Elk River Spit and from widespread North and South Spit dune habitats.

12) Western sand spurrey (*Spergularia canadensis* var. *occidentalis*) is known only in California from Humboldt Bay. The collection is from a vague Samoa salt marsh at an unknown location.

13) Northern clustered sedge (*Carex arcta*) is known from coastal bogs, fens and moist coastal forests. Locally it was reported in 1912 at an unknown location.

14) Meadow sedge (*Carex praticola*) is associated with wet meadows but the exact 1915 Eureka location is unknown.

15) Coast fawn lily (*Erythronium revolutum*) is found in bogs, fens, coastal upland forests although the Eureka site reported in 1918 in unknown.

16) Marsh pea (*Lathyrus palustris*) was recently located near the Elk River estuary (City of Eureka, et. al., 2004).

17) Indian pipe (*Monotropa uniflora*) is attributed to a 1971 Redwood Acres coast redwood forest site but has never been since relocated.

18) Howell's montia (*Montia howellii*) is known only from a 1916 collection but has recently been discovered in and along compacted unpaved roads east of Redwood Acres.

19) Maple-leaved checkerbloom (*Sidalcea malchorides*) is recently known from the Martin Slough area (City of Eureka, et. al., 2004).

20) Siskiyou checkerbloom (*Sidalcea malvaflora* ssp. *patula*) was reported from 1905 and 1944 collections but the Bucksport site was in decline and has not been seen since.

21) Coast checkerbloom (*Sidalcea oregana* ssp. *eximia*) found in the Elk River Valley in 1907 but has not been rediscovered.

22) Marsh violet (*Viola palustris*) reported from an unknown Eureka location in 1922.

All species included on List 1 and 2 (herein referred to as sensitive species) of the California Native Plant Society's (CNPS) Inventory of Rare and Endangered Vascular Plants of California (Tibor 2001) were reviewed to determine potential presence in the vicinity of the Martin Slough project area. The CNPS inventory includes all species listed as rare or endangered by the Federal and State governments.

The following documents were reviewed prior to field work to become familiar with the Martin Slough project area.

McLaughlin, J. and F. Harradine. 1965. *Soils of Western Humboldt County, California*. University of California, Davis, County of Humboldt. Eureka, CA.

The soils map of the Martin Slough project area based on the above reference indicates that Bayside 3 (Ba 3) and Bayside 4 (Ba 4) wetland soils (silty clay loam, imperfectly drained, 0-3% slopes) exist throughout the lower 1/3 of the project area (Senestraro). The Eureka Municipal Golf Course is considered residential, business industrial areas (UI), and the soils present in the section within the golf course are unclassified.

California Coastal Commission. 1986. Eureka Quad Map, Coastal Zone. Post LCP Certification Permit and Appeal Jurisdiction, County of Humboldt, City of Eureka.

The coastal zone map reviewed above indicates that the coastal zone occurs from the southern end of the Eureka Municipal Golf Course to the southern end of the project throughout the Senestraro property. The California Coastal Commission area of primary permitting jurisdiction includes 100 feet on either side of Martin Slough for the entire length of the slough within the Coastal Zone. Humboldt County jurisdiction occurs beyond the 100 foot area adjacent to Martin Slough within the Coastal Zone outside of City Limits.

U. S. Fish & Wildlife Service. 1987. National Wetlands Inventory, Eureka Quad. Portland, OR.

The following wetland types are mapped in the project area in the above inventory:

Riverine, lower perennial, unconsolidated bottom, permanently flooded (and portions with excavated channel), riverine, upper perennial, unconsolidated bottom, permanently flooded, palustrine, emergent, persistent, seasonally flooded (partially drained/ditched), temporary flooded (partially drained/ditched), palustrine, unconsolidated bottom, permanently flooded.

City of Eureka, Roberts, Kemp & Associates, SHN Engineers & Geologists, Inc., Brown & Caldwell, Mad River Biologists, Thomas Payne & Associates and Jamie Roscoe & Associates. 2004. Draft Environmental Impact Report, Martin Slough Interceptor Project, SCH No. 2002082043. Vol. II, Biological Studies. Eureka, CA.

The following sensitive species were reported in the Martin Slough project area:

Lyngbye's sedge (*Carex lyngbyei*) is found from the southern extent of the project (along the Marin Slough channel) to the Eureka Municipal Golf Course lower irrigation pond.

Osprey (*Pandion halieatus*) nests were located within 500 feet of the project area. One nest is in a mature coast redwood (*Sequoia sempervirens*) above the 2-3 holes of the Eureka Municipal Golf Course, north of Fairway Drive. The second nest, in a redwood snag, is above the golf course, at the 10th hole south of Fairway Drive.

Northern Red-legged Frog (*Rana aurora aurora*) was found in the Martin Slough channel immediately south of the golf course.

David Ammerman. 2005. Personal Communication. Mr. Ammerman of the U. S. Army Corps of Engineers was contacted regarding previous wetland delineation projects in the vicinity of the southern terminus (Senestraro). He believed that there had been tidegate projects which were submitted under nationwide permits but did not locate any delineation reports for that area in his project files (Younger/Senestraro/Reardon).

14.4 Data Collection Methodology

14.4.1 Wetlands Botanical Methodology

Vegetation data collected consisted of determining the dominant species for tree, shrub and herbaceous layers at each site location sampled. The species were then classified as to whether or not they are wetlands indicators, using the standard reference for plant wetlands indicators: National List of Plant Species that Occur in Wetlands: California (Region O) (1988). The document classifies plants based on the probability that they would be found in wetlands, ranging from Obligate (OBL, almost always in wetlands), Facultative/wet (FACW, 67% to 99% in wetlands), Facultative (FAC, 34% to 66% in wetlands), Facultative/up (FACU, 1% to 33% in wetlands) to Uplands (UPL, <1% in wetlands). Plants not listed are included in the uplands category. If 50% or greater of the dominant plant species at each sample location are classified OBL, FACW or FAC, the vegetative mix is determined to be hydrophytic (wetlands plants).

14.4.2 Wetlands Soils Methodology

Soil test pits were dug to a depth of approximately 20 inches. The 1987 Manual's procedures were combined with the Natural Resources Conservation Service's (NRCS) definition of hydric soils (Changes in Hydric Soils of the United States, Federal Register, Volume 60, No. 37, February 24, 1995) and Field Indicators of Hydric Soils in the United States, 1998. Care was taken to observe mottling (iron concentrations) and to distinguish between chromas of 1 and 2. Color indicators of hydric soils used in this delineation were as follows:

1. Matrix chroma of 2 or less in mottled soils (1987 Manual)
2. Matrix chroma of 1 or less in unmottled soils (1987 Manual)
3. Colors (evidence of saturation) determined at 12 inches depth in poorly drained or very poorly drained soil (NRCS)

Colors were determined at a depth of 10 inches. Colors were determined on moist ped surfaces, which had not been crushed.

14.4.3 Wetlands Hydrology Methodology

The delineation was performed during mid summer. Direct evidence of soil saturation (soil saturation, standing water, etc.) was present when the investigation was performed due to extensive late spring rains and higher than average accumulated rainfall. Wetlands hydrology conditions were based on drainage patterns and in some cases, the presence of algal mats. Topographic position, FAC-Neutral Test and oxidized root channels were also used as a secondary indicator of wetlands/upland boundaries.

The wetlands determination was made with an emphasis on redoximorphic soils features, hydrology and hydrophytic vegetation. An area was determined to be wetlands when soil, vegetation or assumed hydrology met the wetlands criteria explained above (one parameter approach). An area was determined to be uplands when the area lacked hydrophytic vegetation, hydric soil and the evidence of wetlands hydrology. There was a strong correlation between the presence of hydrophytic vegetation, hydric soils and wetlands plots.

14.5 Results of Wetlands Investigation and Sensitive Plant Survey

The parameters used to identify a wetland are the characteristics of the soils, hydrology and vegetation. To define a wetland, the ACOE (1987) requires that all three parameters show attributes of persistent soil saturation. The California Coastal Commission (CCC) requires that one of the three parameters show wetlands characteristics to be a wetland regulated by the Coastal Act. Three on-site parameters analyzed, vegetation, soils and hydrology, are described below.

Hydrophytic vegetation was dominant within the site wetlands. Typical wetlands vegetation at the site included:

- Small-headed bulrush (Scirpus microcarpus)
- Meadow foxtail (Alopecurus genticulatus)
- Creeping buttercup (Ranunculus repens)
- Fowl manna grass (Glyceria elata)
- Shining willow (Salix lucida ssp lasiandra)
- Iris-leafed rush (Juncus ensifolius)
- Lyngbye's sedge (Carex lyngbyei)
- Pacific water parsley (Oenanthe sarmentosa)
- Pacific silver-weed (Potentilla anserina)

Upland vegetation such as white clover (*Trifolium repens*), Kentucky bluegrass (*Poa pratensis*), English daisy (*Bellis perennis)*, hairy cat's ear (*Hypochaeris radicata*) and sweet vernal grass (*Anthoxanthum ododratum*) were dominant in most upland locations.

Although not strongly hydrophytic, vegetation such as annual bluegrass (*Poa annua*), bird's foot trefoil (*Lotus corniculatus*), California blackberry (*Rubus ursinus)* and perennial ryegrass (*Lolium perenne*) were commonly found in both uplands and wetlands locations. All the above aforementioned species are FACW or FAC designated indicator species (U.S. Fish and Wildlife Services, 1988). All upland plots were determined by the lack of all three parameters.

Wetlands soil exhibited redoximorphic features typically found in hydric soils. These features included distinct or prominent mottles (iron concentrations) at or above 12 inches from the surface. Uplands soils typically had colors of 2.5Y 3/1, 3/2, 4/2, 5/2, 3/3, 4/4, 4/6, 6/8 with no redoximorphic features. Upland soils with a color of 10YR3/1 were due to high organic matter, not reduction. Wetlands (hydric) soils typically had a matrix chroma at 2.5Y 3/2, 4/2 with iron concentrations (7.5YR 4/4, 7.5YR 4/6 or 10YR 5/1).

Hydrologic conditions were present to confirm the wetlands/upland boundary. Although the investigation was performed in the early summer season, hydrologic conditions were observed in many of the mapped locations.

Lyngbye's sedge was found along the immediate edge of Martin's Slough. The location of Lyngbye's sedge is closely associated with brackish waters that reach up to the irrigation pond just south of Fairway Drive (Figure 14-1). Lyngbye's sedge is a CNPS List 2 species. The list 2 species are those species considered rare, threatened or endangered in California but are not listed. Any proposed impacts to Lyngbye's sedge populations need to be considered in preparation of permitting or environmental documentation.

Within approximately 400-500 feet of the project area, two active Osprey nests were found (Figure 14-1). One nest is in a mature coast redwood (*Sequoia sempervirens*) above the 2-3 holes of the Eureka Municipal Golf Course, north of Fairway Drive. The second nest, in a redwood snag, is above the golf course, at the 10th hole south of Fairway Drive. Osprey is a California Species of Special Concern and the California Department of Fish & Game recommends a 500 foot buffer from construction activity during any time of occupied nest behavior.

14.6 Reconnaissance Summary

A reconnaissance level wetlands investigation and sensitive plant survey at the site was conducted on June 23, 2005 within the limits of the Martin Slough study area. The wetlands investigation determined that Estuarine and Palustrine Emergent, Palustrine Shrub-Scrub and Palustrine Forested Wetlands occur on the subject properties. The primary wetlands are found associated with or adjacent to the Martin Slough drainage. The sensitive plant species survey determined the presence of a California Native Plant Society List 2 species, Lyngbye's sedge (*Carex lyngbyei*), which grows from near the 18th fairway at the irrigation pond and down Martin Slough for the length of the project area along the waterway. In addition to wetlands and sensitive plant species on site, there is one active Osprey (*Pandion haliaetus*) nest above the golf course 2nd hole (and an additional active osprey nest off site above the 10th hole). Figure 14.1 at the end of this section shows the results of this work on the Project Base Map.

14.7 Special Terms and Conditions

To achieve the investigation objectives stated in this report, we based our opinions on the information available during the period of the investigation, June 23, 2005. This report does not authorize any individuals to develop, fill or alter the wetlands investigated. Verification of a formal delineation by jurisdictional agencies is necessary prior to the use of this site for development purposes (this effort was only a reconnaissance). Permits to affect wetlands must be obtained from the involved government agencies. If filling occurs under permitted authority, care should be given to maintenance and placement of a sufficient quantity of fill to prevent reestablishment of wetlands. Land use practices and regulations can change thereby affecting current conditions and delineation results.

This report was prepared for Redwood Community Action Agency. Winzler & Kelly is not liable for any action arising out of the reliance of any third party on the information contained within this report.

14.8 Wetland and Biological References

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NORTH
ACTIVE OSPREY NEST
COUNTY
EUREKA CITY LIMITS
COASTAL ZONE BOUNDARY
LIMITS OF BIOLOGICAL AND WETLANDS RECONNAISSANCE STUDY AREA
COUNTY
EUREKA CITY LIMITS
COUNTY
ACTIVE OSPREY NEST
LEGEND:
UPLANDS
BOUNDARY WETLAND/UPLAND
WETLANDS
-XXXXXXXX- LYNGBYE'S SEDGE (CAREX LYNGBYEI)
SOIL PIT LOCATIONS
0' 400' 800'
SCALE IN FEET
MARTIN SLOUGH ENHANCEMENT PLAN BIOLOGICAL BASE MAP
FIGURE 14.1
WINZLER & KELLY
FILE: J:\CAD\JOBS\2004\04158101\dwg\Martin Slough Base Map.dwg DATE: Aug 12 05 @ 1:28pm

V. ALTERNATIVE RESULTS

15.0 DISCUSSION OF ALTERNATIVE RESULTS

This section discusses each alternative's results. The results of each alternative were evaluated and then compared to a list of criteria set by the TAC that are combined into the following categories: Fish Passage and Fish Access for Juvenile and Adult Salmonids, Fish Habitat, Riparian Corridor, Water Quality, Wetlands, Flood Impacts, Existing Land Uses, Project Permitting, Cost of Improvements, and Project Maintenance.

15.1 Alternative 1: The No Action Alternative (Existing Conditions)

The No-Action Alternative would leave the system as it exists today. This alternative is important for permitting considerations and for comparing alternatives, providing a familiar starting point for comparisons to be made. See the description of alternatives section for the full description of this alternative.

15.1.1 Discussion of Results

Since the No-Action Alternative would leave the system as it exists today, there are not many results to discuss. Existing problems would remain essentially as they are today. It is important to note that leaving the system as it exists today will require ongoing maintenance, as the existing system does not sufficiently route sediments through the drainage. This is exacerbated by the lack of riparian canopy, which allows grasses and other vegetation to dominate the channel. The fast growing vegetation slows water velocities and captures sediment, thus compounding the problem. Maintaining the current conveyance area of the channel requires regular dredging, which necessitates permits from State and Federal agencies. The permitting portion of this alternative could change in the future, potentially requiring more time and effort to obtain permits. There are no construction costs related to this alternative.

The table below summarizes how Alternative 1 would address the project criteria. Following the table is a discussion with graphical output of the ADCIRC hydraulic model results.

Table 15.1 Criteria Matrix for Alternative 1 – No Action (Existing Conditions)

Criteria	Effects
Fish Passage and Fish Access for Juveniles and Adults	
1. Maximize Migration Access at Tidegates during Fish Migration Flows	No Improvement
Fish Habitat	
2. Maximize Estuarine Habitat	No Improvement
3. Increase Channel Complexity	No Improvement
Riparian Corridor	
4. Increase Riparian Habitat	No Improvement
5. Increase Riparian Canopy	No Improvement
Water Quality	
6. Decrease Nutrient Impacts	No Improvement
7. Decrease Sedimentation	No Improvement

Criteria	Effects
Wetlands	
8. Improve Wetland Habitat	No Improvement
9. Increase Open Water Area of Wetlands	No Improvement
10. Increase Diversity of Wetland Types	No Improvement
Flood Impacts	
11. Reduce Flood Inundation Area	No Improvement
12. Reduce Frequency of Flooding	No Improvement
13. Minimize Duration of Flooding	No Improvement
Existing Land Uses	
14. Maintain Agricultural Land Use	No Improvement
15. Maintain Eureka Municipal Golf Course	No Improvement
16. Allow for full Build-out Potential for City/County	No Improvement
17. Allow for Installation and Maintenance Access for City's Martin Slough Sewer Interceptor Project	No Improvement
Project Permitting	
18. Consider ability to Obtain Permits	Permitting efforts for maintenance may increase with time
Cost of Improvements	
19. Consider Order of Magnitude Opinion of Probable Construction Costs	Not Estimated
Project Maintenance	
20. Consider Need for Ongoing Maintenance	No Improvement

15.1.2 ADCIRC hydraulic model results

Most flooding of the existing system for the 2-year event takes place on the golf course property upstream of the large remnant channel meander. Inundated areas downstream of the meander tend to drain before areas upstream. The site remains strongly inundated through 2 days of simulation, but inundation is substantially reduced by day 3. Reduction in inundation is gradual from day 3 to day 7 with the remaining water being located primarily near existing ponds and in low lying areas.

It is important to note that the existing pasture on the southern portion of the modeled project area is a low lying area where the model indicated varying degrees of inundation through day 7. This modeling output is likely due to the limitations of the topographic 2-foot contours used to build the model surface. The model does not consider groundwater flow or infiltration, so low lying areas that do not have a well-defined drainage outlet result in isolated ponds that show up as inundated areas in the model output. Detailed topographic information in that area would alleviate this result. While not included in the scope of this project, some low lying areas prone to frequent flooding could benefit from small drainage improvements. While this project focused on the larger scale drainage features of lower Martin Slough, it became apparent that localized drainage problems were the cause of extended ponding as the water simply had no place to drain.

Flooding for the 10-year event occurs over most of the study area, inundating a substantially larger area than the 2-year event, particularly in the southwestern portion (pasture). Inundated areas downstream tend to drain before those upstream. The upstream portions of the site remain widely inundated throughout the simulation, and the largest reduction in inundation occurs between the peak and day three, then slowing over the remaining four days.

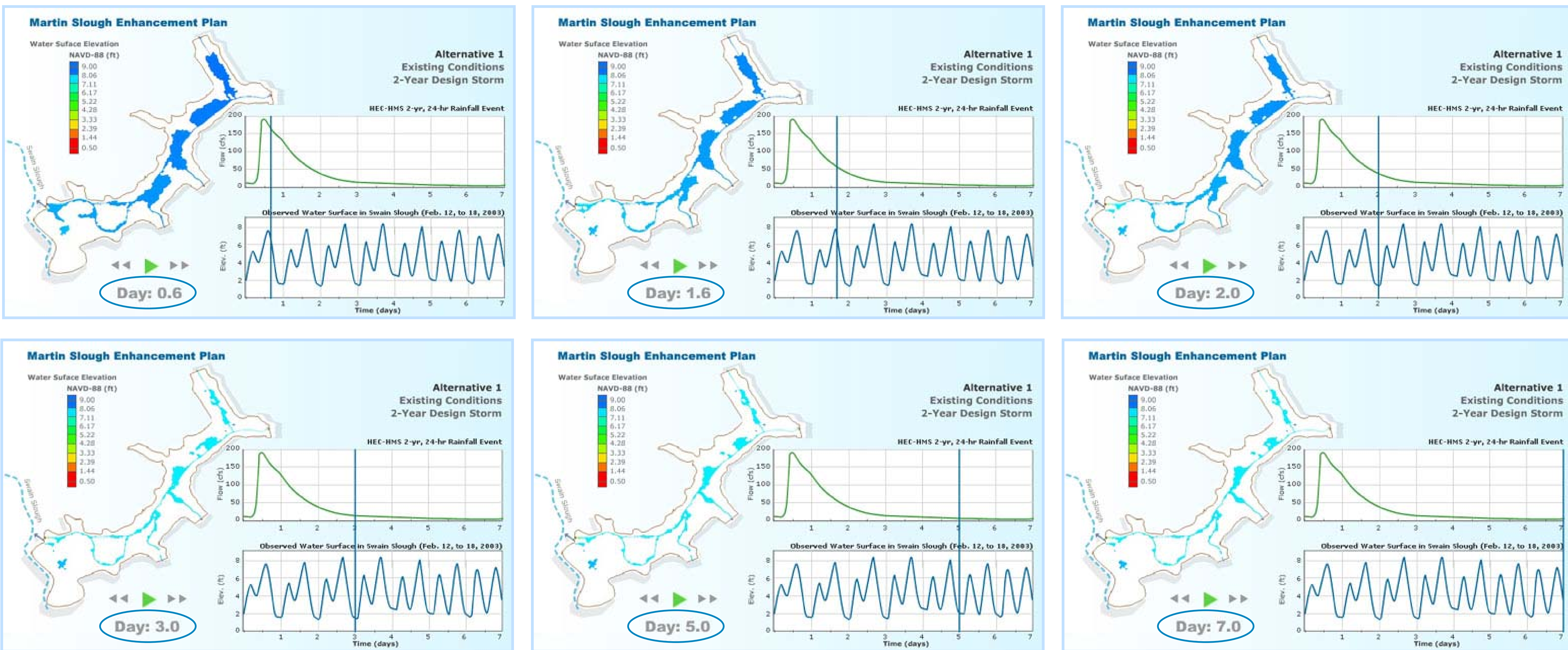
The figures on the following two pages summarize the qualitative inundation output of the ADCIRC hydraulic model for Alternative 1 for the 2-year and 10-year events, respectively.

Martin Slough Enhancement Plan
ADCIRC Hydraulic Model Graphic Results

Alternative 1 (Existing Conditions):
2 Year Rainfall Event **Figure 15.1**

Hydraulic modeling output shown below was generated with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). Each graphic displays the project area with surface water inundation corresponding to the time step indicated. The two charts to the right of the project area represent the design storm input hydrograph and the downstream tidal boundary condition used in the model. The vertical bar through both charts represents the model time step which correlates to the inflow hydrograph, downstream tidal conditions, and the model results graphically shown.

Time steps of the graphic output shown were chosen to represent near maximum inundation conditions followed by several days of draining after the design storm had passed.

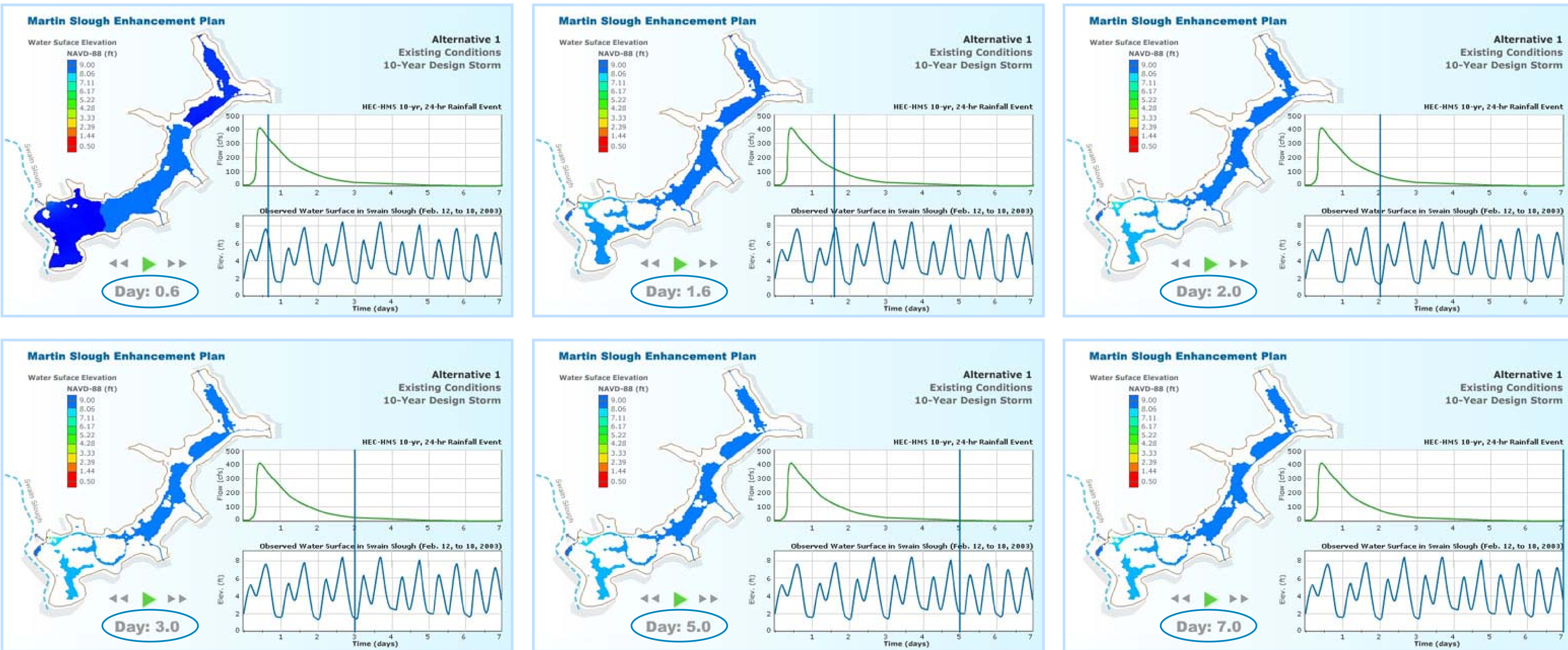


Martin Slough Enhancement Plan
ADCIRC Hydraulic Model Graphic Results

Alternative 1 (Existing Conditions): 10 Year Rainfall Event — Figure 15.2

Hydraulic modeling output shown below was generated with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). Each graphic displays the project area with surface water inundation corresponding to the time step indicated. The two charts to the right of the project area represent the design storm input hydrograph and the downstream tidal boundary condition used in the model. The vertical bar through both charts represents the model time step which correlates to the inflow hydrograph, downstream tidal conditions, and the model results graphically shown.

Time steps of the graphic output shown were chosen to represent near maximum inundation conditions followed by several days of draining after the design storm had passed.



15.2 Alternative 2: No Tidegates or Levee (Full Tidal Influence)

The No Tidegates Full Tidal Influence Alternative would consist of removing the existing tidegates and the levee at Swain Slough. Essentially this would cause changes in the system that would eventually return the system towards its pre-development state. This alternative would open the majority of the project area to regularly occurring tidal inundation, based on land and tidal elevations.

This alternative was modeled by removing the tidegates and a twenty foot wide portion of Swain Slough from the hydraulic model's downstream boundary. Everything else remained the same as the existing conditions modeled in Alternative 1. See the description of alternatives for the full description of this alternative (Section V).

15.2.1 Discussion of Results

Conversion of the lower Martin Slough watershed towards pre-development conditions by re-introducing a full tidal cycle would preclude the current agricultural and golf course uses. This could occur as the high tide level, approximately 8 foot elevation (NAVD88), would convert much of the existing grasslands in the pasture and golf course into salt tolerant plant communities. Also, conversion to salt marsh would likely result in colonization by invasive cord grass (*spartina densiflora*).

While this alternative would provide the most fish migration access, the alternative as presented would likely not increase the channel complexity and would only marginally improveme estuarine habitat. While this alternative could be modified to improve these criteria, other criteria such as more frequent tidal inundation of the project area may preclude this from being a viable option.

It is important to note that while this alternative does not include any changes to the stream channel, the re-introduced full tide cycle would increase the daily hydraulic flushing of the channel. This increase in tidal prism (diurnal volume of water that ebb and flows) within Martin Slough would increase channel bank and bed scour, likely causing the existing channel to widen and deepen substantially. These channel changes would likely extend upstream throughout the project area. Left to occur on their own, these changes could have negative impacts on existing roadways, access and utility crossings, the bridge footings on Fairway Drive, and other related items. The scoured sediment would be transported with the outgoing tides into Swain Slough and Humboldt Bay. We do not recommend allowing this scour to occur as described above due to the uncertainties involved and the related potential for damage to property, infrastructure, and sedimentation of downstream habitat including the Bay.

As noted above, this alternative could require considerable repair and maintenance relating to channel scour. While costs were not estimated for maintenance, they could be substantial, especially if roadways, bridges, and utilities were affected. Once the system stabilized, the required maintenance would likely be greatly reduced and could be less than the existing conditions. The uncertainties of project impacts could make this project difficult to permit.

The order of magnitude estimate of probable construction costs for this alternative is $150,000. A breakdown of costs is included in Appendix E.

The table below summarizes how Alternative 2 would address the project criteria. Following the table is a discussion with graphical output of the ADCIRC hydraulic model results.

Table 15.2 Criteria Matrix for Alternative 2 – No Tidegates or Levee (Full Tidal Influence)

Criteria	Effects
Fish Passage and Fish Access for Juveniles and Adults	
1. Maximize Migration Access at Tidegates during Fish Migration Flows	Most Improvement
Fish Habitat	
2. Maximize Estuarine Habitat	Some Improvement
3. Increase Channel Complexity	No Improvement
Riparian Corridor	
4. Increase Riparian Habitat	No Improvement
5. Increase Riparian Canopy	No Improvement
Water Quality	
6. Decrease Nutrient Impacts	Some Improvement
7. Decrease Sedimentation	Some Improvement
Wetlands	
8. Improve Wetland Habitat	Some Improvement
9. Increase Open Water Area of Wetlands	Some Improvement
10. Increase Diversity of Wetland Types	Some Improvement
Flood Impacts	
11. Reduce Flood Inundation Area	No Improvement/ Potentially Worse
12. Reduce Frequency of Flooding	No Improvement/ Potentially Worse
13. Minimize Duration of Flooding	No Improvement/ Potentially Worse
Existing Land Uses	
14. Maintain Agricultural Land Use	Likely Worse
15. Maintain Eureka Municipal Golf Course	Likely Worse
16. Allow for full Build-out Potential for City/County	No Improvement
17. Allow for Installation and Maintenance Access for City's Martin Slough Sewer Interceptor Project	No Improvement/ Potentially Worse
Project Permitting	
18. Consider ability to Obtain Permits	Potentially Very Difficult
Cost of Improvements	
19. Consider Order of Magnitude Opinion of Probable Construction Costs	Lowest Cost
Project Maintenance	
20. Consider Need for Ongoing Maintenance	Potentially Worse in short term, lessoning over time

15.2.2 ADCIRC hydraulic model results

Most flooding of the existing system for the 2-year event takes place primarily upstream of the large remnant channel meander and in the pasture downstream of the meander. Over the 7-day simulation interval, overall water level is minimally reduced. Free propagation of the tidal flux into the study area, combined with higher tidal elevations resulted in prolonged inundation over all areas that experienced flooding. Inundation expanse and duration are greater than that for the No Action Alternative for the 2-year event.

It is important to note that the existing pasture on the southern portion of the modeled project area is a low lying area where the model indicated varying degrees of inundation through day 7. This modeling output is likely due to the limitations of the topographic 2-foot contours used to build the model surface. The model does not consider groundwater flow or infiltration, so low lying areas that do not have a well-defined drainage outlet result in isolated ponds that show up as inundated areas in the model output. Detailed topographic information in that area would alleviate this result. While not included in the scope of this project, some low lying areas prone to frequent flooding could benefit from small drainage improvements. While this project focused on the larger scale drainage features of lower Martin Slough, it became apparent that localized drainage problems were the cause of extended ponding as the water simply had no place to drain.

Flooding for the 10-year event takes place over most of the study area. Inundation levels are notably reduced after 2 days, but standing water remains in the upstream areas and downstream pasture at 7 days. Free propagation of the tidal flux into the study area, combined with higher tidal elevations resulted in prolonged inundation over all areas that experienced flooding. Inundation expanse and duration are greater than that for the No Action Alternative for the 10-year event.

The tables on the following two pages summarize the qualitative inundation output of the ADCIRC hydraulic model for Alternative 2 for the 2-year and 10-year events, respectively.

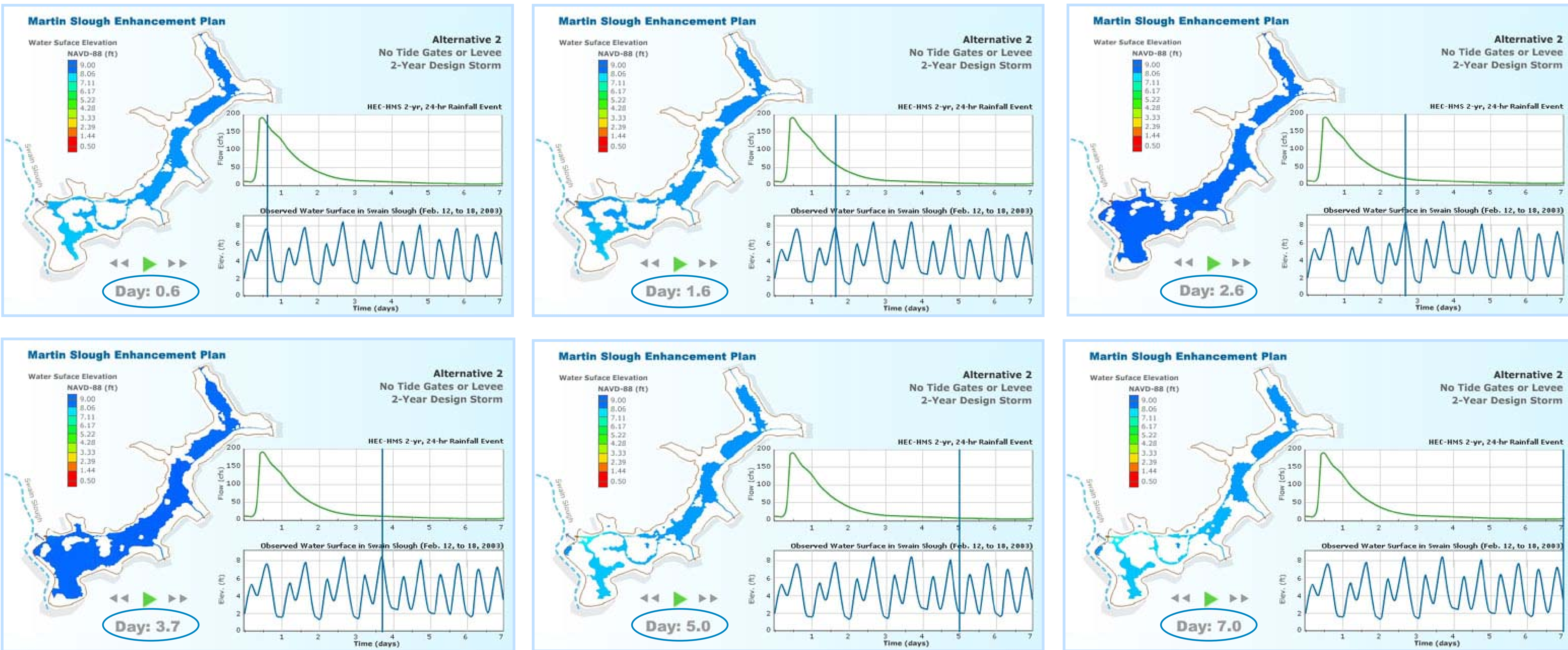
Martin Slough Enhancement Plan
ADCIRC Hydraulic Model Graphic Results

Alternative 2 (No Tidegates or Levee): 2 Year Rainfall Event

Figure 15.3

Hydraulic modeling output shown below was generated with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). Each graphic displays the project area with surface water inundation corresponding to the time step indicated. The two charts to the right of the project area represent the design storm input hydrograph and the downstream tidal boundary condition used in the model. The vertical bar through both charts represents the model time step which correlates to the inflow hydrograph, downstream tidal conditions, and the model results graphically shown.

Time steps of days 1.6, 2.6, and 3.7 shown as they correlate to elevated high tides which show substantial inundation with no tidegates or levees separating Martin Slough from Swain Slough.

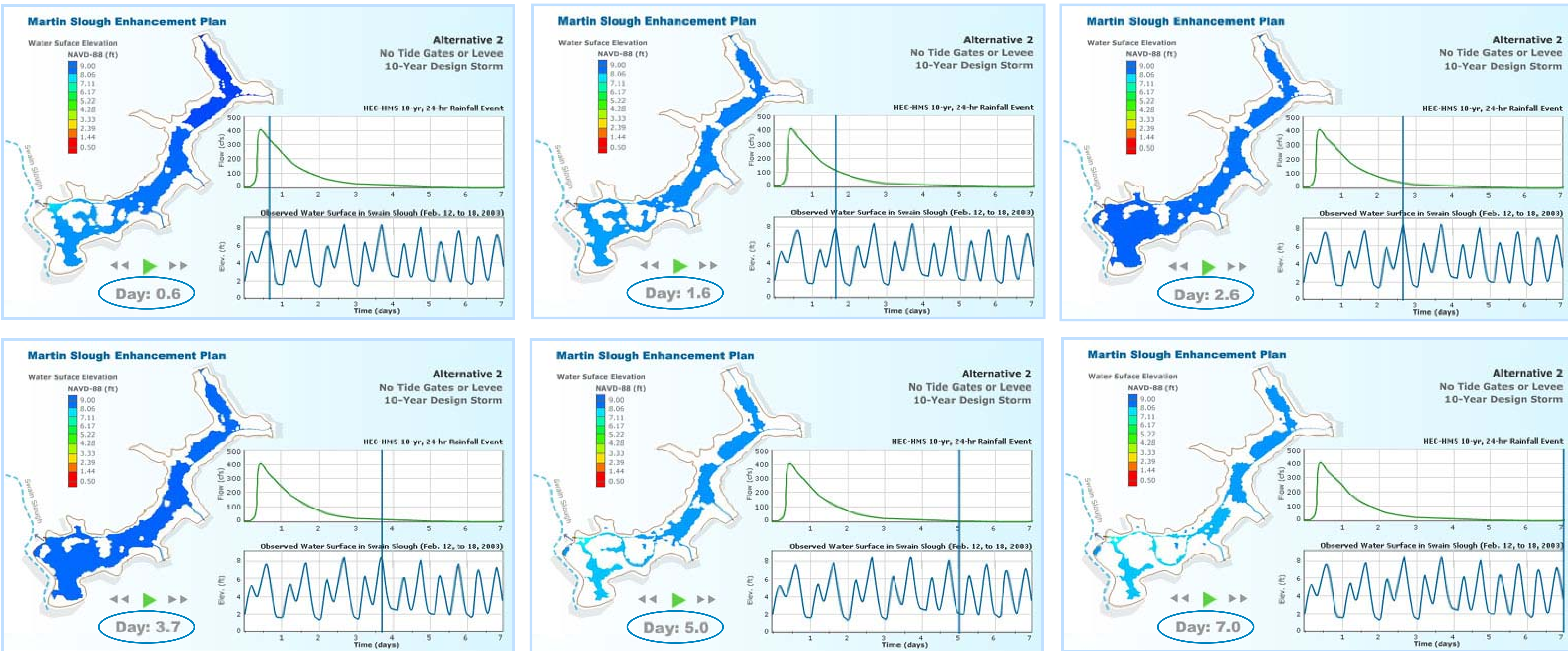


Martin Slough Enhancement Plan
ADCIRC Hydraulic Model Graphic Results

Alternative 2 (No Tidegates or Levee): 10 Year Rainfall Event — Figure 15.4

Hydraulic modeling output shown below was generated with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). Each graphic displays the project area with surface water inundation corresponding to the time step indicated. The two charts to the right of the project area represent the design storm input hydrograph and the downstream tidal boundary condition used in the model. The vertical bar through both charts represents the model time step which correlates to the inflow hydrograph, downstream tidal conditions, and the model results graphically shown.

Time steps of days 1.6, 2.6, and 3.7 shown as they correlate to elevated high tides which show substantial inundation with no tidegates or levees separating Martin Slough from Swain Slough.



Winzler & Kelly Consulting Engineers
Michael Love & Associates
Coastal Analysis LLC

15.3 Alternative 3: New Tidegates and New Ponds (Muted Tide)

The New Tidegates and New Ponds Alternative will consist of removing the existing tidegates, installing new tidegates with a 2 foot by 2 foot habitat door designed to create a muted tidal prism and facilitate fish passage, increasing the size of existing ponds, and creating new ponds. See the description of alternatives for the full description of this alternative (Chapter 9.0).

15.3.1 Discussion of Results

This alternative results in some improvement for most of the project criteria. The new tidegates provide improved fish passage and the new and expanded ponds would provide more estuarine habitat. The riparian corridor would see some improvement with the new and expanded ponds being re-vegetated along their edge. Water quality would see some improvement as the new tidegates and muted tide would result in increased channel flushing, especially in the summer months when the fresh water inflow is lowest and its ability to dilute the tidal water is reduced.

Wetland habitat would improve with the new and expanded ponds by providing more open water wetland area and diversity of wetland types. While modeling salinity levels was not included as part of this project, the new and expanded ponds are located at varying distances from the tidegates and would likely experience varying salinity levels. We would expect to see higher salinity values near the tidegates and lower values further upstream where there is more fresh water influence. Additionally, we would expect to see seasonal fluctuations with lower salinity levels in the winter months when there is more storm flow input and higher salinity levels in the summer when the flows within the channel would be predominately tidal.

It is important to note that while this alternative does not include any constructed changes to the stream channel, the increased muted tide cycle would increase the daily flushing of the channel. This increase in tidal prism (diurnal volume of water that ebb and flows) within Martin Slough would increase channel bank and bed scour, likely causing the existing channel to widen and deepen. These channel changes would likely extend upstream throughout the project area. Left to occur on their own, these changes could have negative impacts on existing roadways, access and utility crossings, the bridge footings on Fairway Drive, and other related structures. The scoured sediment would be transported with the outgoing tides into Swain Slough and Humboldt Bay. We do not recommend allowing this scour to occur as described above due to the uncertainties involved and the related potential for damage to property, infrastructure, and sedimentation of downstream habitat.

This alternative did show some improvement in reducing flood impacts. The new and expanded ponds provide some storage volume for storm flows, and the new tidegates would provide three times the surface area, allowing more water to drain, resulting in a reduced duration of flooding. The main constraint preventing better drainage appeared to be the existing channel. Even with the new tidegates and additional storage capacity of the ponds, the existing channel as modeled does not provide sufficient conveyance to move the water through the drainage to the tidegates.

This alternative results in some improvement for most of the project criteria. The new tidegates provide improved fish passage and the new and expanded ponds would provide more estuarine habitat. The riparian corridor would see some improvement with the new and expanded ponds being re-vegetated along their edges. The amount of riparian benefit would depend upon how much riparian habitat is ultimately designed and planted. Water quality would see some improvement as the new tidegates and the muted tide would result in increased channel flushing, especially in the summer months when fresh water inflow is lowest.

As noted above, this alternative could require a lot of repair and maintenance relating to channel scour. While costs were not estimated for maintenance, they could be substantial, especially if roadways, bridges, and utilities are affected. Once the system stabilized, the required maintenance would likely be greatly reduced and could be less than the current conditions.

The order of magnitude estimate of probable construction costs for this alternative is $2,400,000. A breakdown of costs is included in Appendix E. A large portion of the costs is related to the hauling and disposal of excavated soil. With approximately 90,000 cubic yards of excavated material estimated to need to be hauled away, we assumed a disposal site could be found within two miles of the site. There has not been any site identified, and this one item could have a large impact on the actual construction cost. We also assumed that temporary haul roads would need to be provided in order to move the excavated material from the ponds to a public road. Traffic control was included as a cost for the movement of trucks on and off the public roadway to haul off the excavated material. There is a possibility that some material could be placed on the land within the project area. While most of the land within the project area is expected to be jurisdictional wetlands, some material may be allowed to be spread out, as long as the jurisdictional status of the wetlands does not change. This approach would require some work to delineate the existing wetlands and collect adequate data to permit the project. If allowed, this approach could help to reduce the quantity of material that needs to be hauled off site, helping to reduce construction costs. Existing upland areas (areas that are not jurisdictional wetlands) within or near the project site could also potentially be used to place excavated material and could help to reduce the cost of hauling excavated material.

The uncertainties of project impacts could make this project difficult to permit.

The table below summarizes how Alternative 3 would address the project criteria. Following the table is a discussion with graphical output of the ADCIRC hydraulic model results.

Table 15.3 Criteria Matrix for Alternative 3 – New Tidegates and New Ponds (Muted Tide)

Criteria	Effects
Fish Passage and Fish Access for Juveniles and Adults	
1. Maximize Migration Access at Tidegates during Fish Migration Flows	Some Improvement
Fish Habitat	
2. Maximize Estuarine Habitat	Some Improvement

Criteria	Effects
3. Increase Channel Complexity	No Improvement
Riparian Corridor	
4. Increase Riparian Habitat	Some Improvement
5. Increase Riparian Canopy	Some Improvement
Water Quality	
6. Decrease Nutrient Impacts	Some Improvement
7. Decrease Sedimentation	Some Improvement
Wetlands	
8. Improve Wetland Habitat	Most Improvement
9. Increase Open Water Area of Wetlands	Most Improvement
10. Increase Diversity of Wetland Types	Some Improvement
Flood Impacts	
11. Reduce Flood Inundation Area	Some Improvement
12. Reduce Frequency of Flooding	Some Improvement
13. Minimize Duration of Flooding	Some Improvement
Existing Land Uses	
14. Maintain Agricultural Land Use	Some Improvement
15. Maintain Eureka Municipal Golf Course	Some Improvement
16. Allow for full Build-out Potential for City/County	Some Improvement
17. Allow for Installation and Maintenance Access for City's Martin Slough Sewer Interceptor Project	Some Improvement
Project Permitting	
18. Consider ability to Obtain Permits	Potentially Very Difficult
Cost of Improvements	
19. Consider Order of Magnitude Opinion of Probable Construction Costs	Moderate Cost
Project Maintenance	
20. Consider Need for Ongoing Maintenance	Potentially Worse

15.3.2 ADCIRC hydraulic model results

Most flooding of the existing system for the 2-year event takes place primarily upstream of the large remnant channel meander and in the pasture downstream of the meander. After 1 day, there is a notable reduction in inundation just upstream of the meander. After 2 days, inundation has been reduced significantly in all areas with the exception of the downstream pasture. Strong inundation reduction continues up to Day 3, and then tapers off such that there is minimal change through Day 7. Inundation expanse and duration are comparable to the No-Action Alternative for the 2-year event, except that the downstream pasture experiences greater flooding for this alternative. This increase in flooding in the downstream pasture could probably be alleviated by creating a channel connecting the inundated area to the downstream pond to improve drainage.

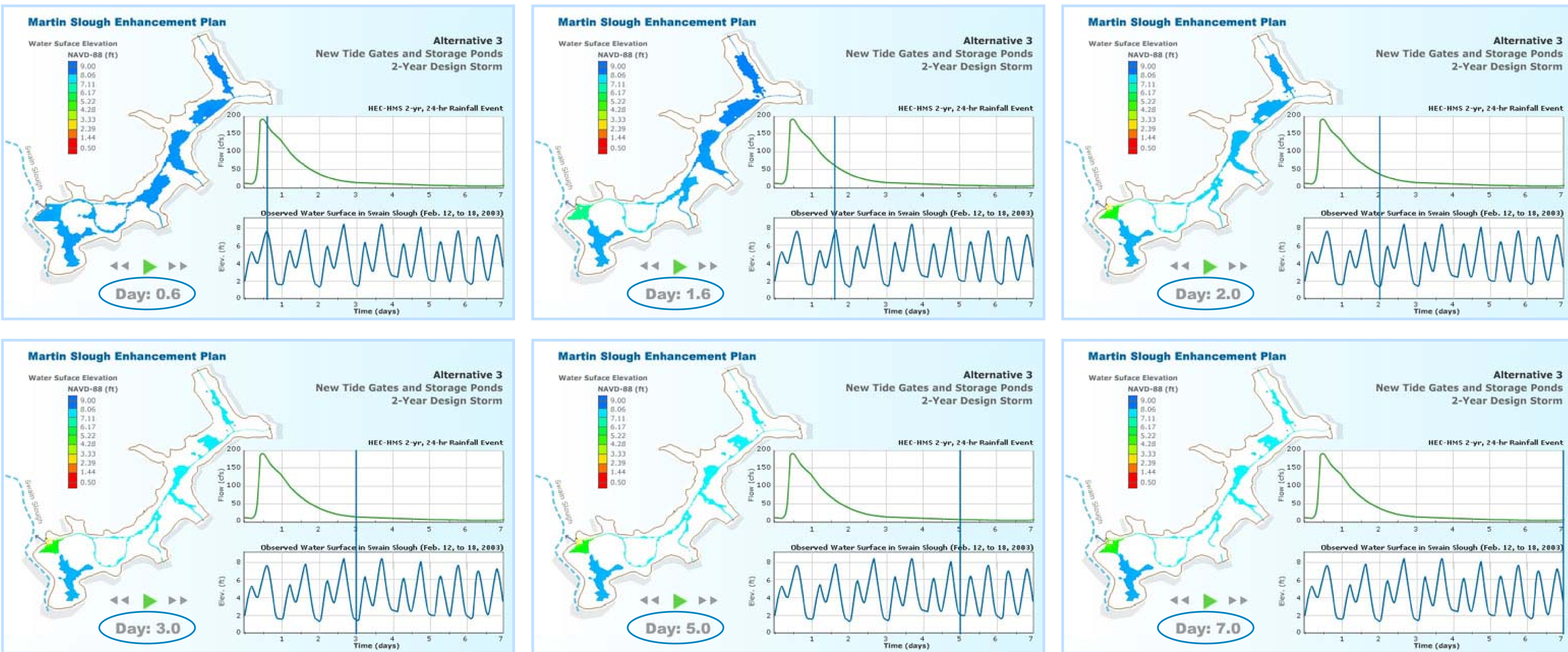
It is important to note that the existing pasture on the southern portion of the modeled project area is a low lying area where the model indicated varying degrees of inundation through day 7. This modeling output is likely due to the limitations of the topographic 2-foot contours used to build the model surface. The model does not consider groundwater flow or infiltration, so low lying areas that do not have a well-defined drainage outlet result in isolated ponds that show up as inundated areas in the model output. Detailed topographic information in that area would alleviate this result. While not included in the scope of this project, some low lying areas prone to frequent flooding could benefit from small drainage improvements. While this project focused on the larger scale drainage features of lower Martin Slough, it became apparent that localized drainage problems were the cause of extended ponding as the water simply had no place to drain.

Flooding for the 10-year event takes place upstream of the remnant channel meander and in the pasture downstream of the meander. After 1 day, there is a notable reduction in inundation near to and downstream of the meander. After 2 days, inundation has been reduced in all areas. Between Days 3 and 7, inundation is slowly reduced. Inundation expanse and duration are generally comparable to the No-Action Alternative for the 10-year event, except that the downstream pasture and vicinity of the channel meander experience significantly less flooding for this alternative. Upstream of the channel meander, this alternative produces a slightly smaller inundation area, as compared to the No-Action Alternative, but the relative difference is not significant.

The tables on the following two pages summarize the qualitative inundation output of the ADCIRC hydraulic model for Alternative 3 for the 2-year and 10-year events, respectively.

Hydraulic modeling output shown below was generated with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). Each graphic displays the project area with surface water inundation corresponding to the time step indicated. The two charts to the right of the project area represent the design storm input hydrograph and the downstream tidal boundary condition used in the model. The vertical bar through both charts represents the model time step which correlates to the inflow hydrograph, downstream tidal conditions, and the model results graphically shown.

Time steps of the graphic output shown were chosen to represent near maximum inundation conditions followed by several days of draining after the design storm had passed.

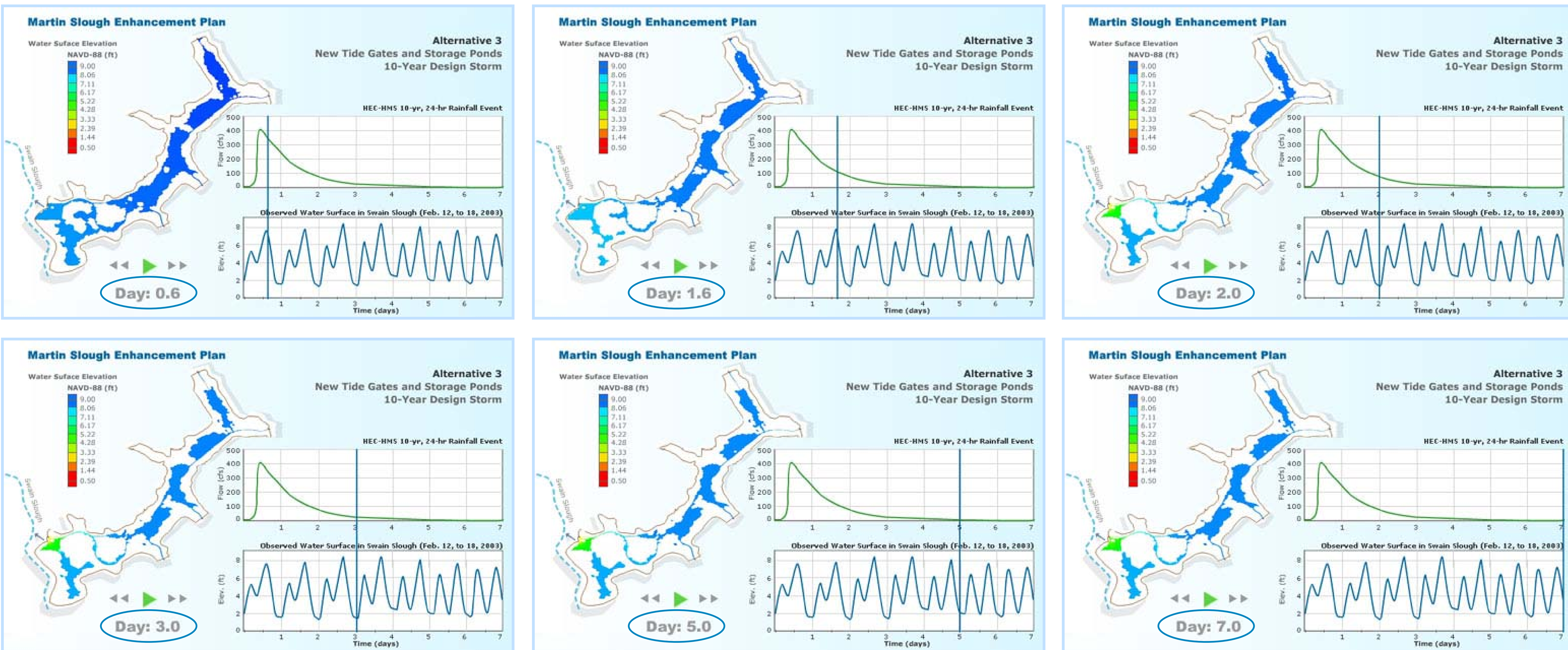


Martin Slough Enhancement Plan
ADCIRC Hydraulic Model Graphic Results

Alternative 3 (New Tidegates and New Ponds): 10 Year Rainfall Event — Figure 15.6

Hydraulic modeling output shown below was generated with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). Each graphic displays the project area with surface water inundation corresponding to the time step indicated. The two charts to the right of the project area represent the design storm input hydrograph and the downstream tidal boundary condition used in the model. The vertical bar through both charts represents the model time step which correlates to the inflow hydrograph, downstream tidal conditions, and the model results graphically shown.

Time steps of the graphic output shown were chosen to represent near maximum inundation conditions followed by several days of draining after the design storm had passed.



Winzler & Kelly Consulting Engineers
Michael Love & Associates
Coastal Analysis LLC

15.4 Alternative 4: New Tidegates, Storage Ponds, and Modified Channel (Muted Tide)

This alternative is similar to Alternative 3, but includes improvements to the existing channel. The New Tidegates, Storage Ponds and Modified Channel Alternative consists of removing the existing tidegates, installing new tidegates with a habitat door designed to create a muted tide cycle and facilitate fish passage, increasing the size of existing ponds, creating new ponds, and making channel modifications throughout the project area. See Chapter 9.0 for more discussion.

15.4.1 Discussion of Results

This alternative results in the most improvement for all of the project criteria. The new tidegates provide improved fish passage and the new and expanded ponds would provide more estuarine habitat. The riparian corridor would see the most improvement with the new and expanded ponds and channel being re-vegetated along their edge. Water quality would see the most improvement as the new tidegates, muted tide, and increased channel volume would result in increased channel flushing, especially in the summer months when fresh water inflow is lowest.

Wetland habitat would improve with the new and expanded ponds by providing more open water wetland area and diversity of wetland types. While modeling salinity levels was not included as part of this project, the new and expanded ponds are located at varying distances from the tidegates and would likely experience varying salinity levels. We would expect to see higher salinity values near the tidegates and lower values further upstream where there is more fresh water influence. Additionally, we would expect to see seasonal fluctuations with lower salinity levels in the winter months when there is more storm flow input and higher salinity levels in the summer when the flows within the channel would be predominately tidal.

This alternative showed the most improvement to help reduce flood impacts. The new and expanded ponds with the larger channel provide additional storage volume for storm flows. The larger channel and tidegates conveys water through the drainage faster, allowing more water to leave the drainage, resulting in less flooding and greatly reduced duration of flooding. Except for small storm events (less than a 2-year event), the project area would experience nearly the same initial flood inundation as the current conditions. This is due to the alternative improvements being limited to channel, pond, and tidegate improvements. The big difference between this and other alternatives is that this alternative reduces the amount of time the project area is flooded.

This alternative would require the least amount of maintenance. While costs were not estimated for maintenance, they would likely be minimal. Anticipated maintenance would likely include maintaining newly vegetated areas and potentially repairing isolated eroded areas. Once the new system stabilized and the vegetation became well established, only minimal maintenance would be expected.

The order of magnitude estimate of probable construction costs for this alternative is $3,900,000. A breakdown of costs is included in Appendix E. A large portion of the costs is related to the hauling and disposal of excavated soil. With approximately 140,000 cubic yards of excavated material estimated to need to be hauled away, we assumed a disposal site could be found within two miles of the site. There has not been any site identified, and this one item could have a large impact on the actual construction cost. We also assumed that temporary haul roads would need to be provided in order to move the excavated material from along the channel to a public road. Traffic control was included as a cost for the movement of trucks on and off the public roadway to haul off the excavated material. There is a possibility that some material could be placed on the land adjacent to the channel. While most of the land within the project area is expected to be jurisdictional wetlands, some material may be allowed to be spread out as long as the jurisdictional status of the wetlands function does not change. This approach would require some work to delineate the existing wetlands and collect adequate data to permit the project. If allowed, this approach could help to reduce the quantity of material that needs to be hauled off, helping to reduce construction costs. Existing upland areas (areas that are not jurisdictional wetlands) within or near the project site could also potentially be used to place excavated material and could help to reduce the cost of hauling off excavated material.

Because this project incorporates the anticipated channel modifications, the project impacts would be easier to predict. Without the likelihood of channel scour and downstream sedimentation, project permitting should be easier than earlier alternatives. However, due to the magnitude of the project and number of permits that would be required, the anticipated permitting effort would still be moderate.

The table below summarizes how Alternative 4 would address the project criteria. Following the table is a discussion with graphical output of the ADCIRC hydraulic model results.

Table 15.4 Criteria Matrix for Alternative 4 – New Tidegates, Storage Ponds, and Modified Channel Improvements (Muted Tide)

Criteria	Effects
Fish Passage and Fish Access for Juveniles and Adults	
1. Maximize Migration Access at Tidegates during Fish Migration Flows	Some Improvement
Fish Habitat	
2. Maximize Estuarine Habitat	Most Improvement
3. Increase Channel Complexity	Most Improvement
Riparian Corridor	
4. Increase Riparian Habitat	Most Improvement
5. Increase Riparian Canopy	Most Improvement
Water Quality	
6. Decrease Nutrient Impacts	Most Improvement
7. Decrease Sedimentation	Most Improvement
Wetlands	
8. Improve Wetland Habitat	Most Improvement
9. Increase Open Water Area of Wetlands	Most Improvement
10. Increase Diversity of Wetland Types	Most Improvement
Flood Impacts	
11. Reduce Flood Inundation Area	Most Improvement
12. Reduce Frequency of Flooding	Most Improvement

Criteria	Effects
13. Minimize Duration of Flooding	Most Improvement
Existing Land Uses	
14. Maintain Agricultural Land Use	Most Improvement
15. Maintain Eureka Municipal Golf Course	Most Improvement
16. Allow for full Build-out Potential for City/County	Most Improvement
17. Allow for Installation and Maintenance Access for City's Martin Slough Sewer Interceptor Project	Most Improvement
Project Permitting	
18. Consider ability to Obtain Permits	Moderate Effort Required
Cost of Improvements	
19. Consider Order of Magnitude Opinion of Probable Construction Costs	Highest Cost
Project Maintenance	
20. Consider Need for Ongoing Maintenance	Most Improvement

15.4.2 ADCIRC hydraulic model results

Most flooding of the existing system for the 2-year event takes place primarily upstream of the large remnant channel meander with some flooding in the pasture downstream of the meander. After 1 day, all areas experiencing flooding have been drained with the exception of parts of the downstream pasture. The downstream pasture retains standing water throughout the 7 days. Draining of this pasture could be improved by constructing a channel to route water to the pond located adjacent to the tidegates. Inundation expanse at peak flooding is less than that for the No-Action Alternative. Inundation duration is greatly improved with this alternative, with water levels over most of the study site returning a non-inundation level within 1 day.

It is important to note that the existing pasture on the southern portion of the modeled project area is a low lying area where the model indicated varying degrees of inundation through day 7. This modeling output is likely due to the limitations of the topographic 2-foot contours used to build the model surface. The model does not consider groundwater flow or infiltration, so low lying areas that do not have a well-defined drainage outlet result in isolated ponds that show up as inundated areas in the model output. Detailed topographic information in that area would alleviate this result. While not included in the scope of this project, some low lying areas prone to frequent flooding could benefit from small drainage improvements. While this project focused on the larger scale drainage features of lower Martin Slough, it became apparent that localized drainage problems were the cause of extended ponding as the water simply had no place to drain.

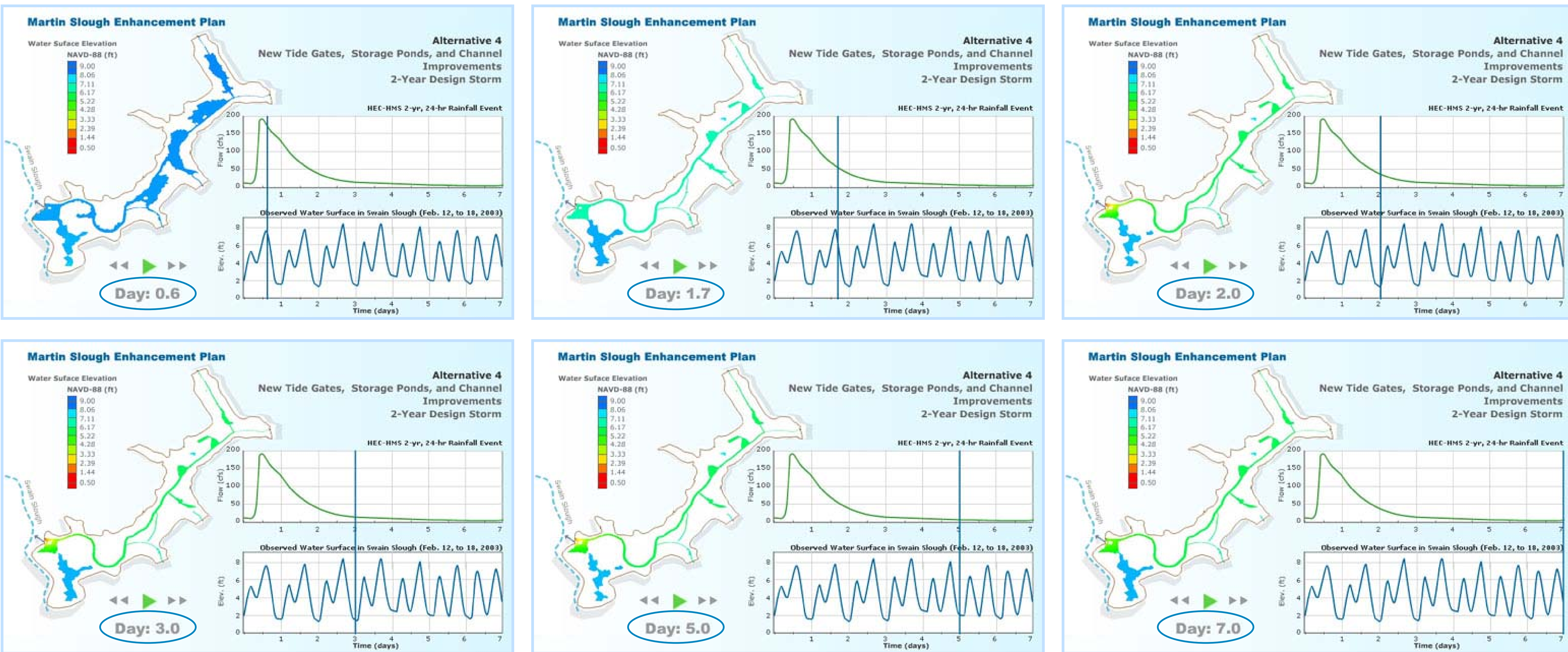
Flooding for the 10-year event takes place upstream of the remnant channel meander and in the pasture downstream of the meander. After 1 day, the inundation is substantially reduced, particularly upstream of the meander. By day 2, all areas experiencing flooding have been drained with the exception of parts of the downstream pasture. The downstream pasture retains standing water throughout the 7 days. Draining of this pasture could be improved by constructing a channel to route water to the pond located adjacent to the tidegates. Inundation expanse at peak flooding is significantly less than that for the No-Action Alternative over the entire study site. Inundation duration is greatly improved with this alternative, as compared to the No-Action

Alternative, with most water draining off of the flooded areas within 1 day, and a return to a no-inundation state by day 2, with the exception of the downstream pasture.

The tables on the following two pages summarize the qualitative inundation output of the ADCIRC hydraulic model for Alternative 4 for the 2-year and 10-year events, respectively.

Hydraulic modeling output shown below was generated with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). Each graphic displays the project area with surface water inundation corresponding to the time step indicated. The two charts to the right of the project area represent the design storm input hydrograph and the downstream tidal boundary condition used in the model. The vertical bar through both charts represents the model time step which correlates to the inflow hydrograph, downstream tidal conditions, and the model results graphically shown.

Time steps of the graphic output shown were chosen to represent near maximum inundation conditions followed by several days of draining after the design storm had passed.



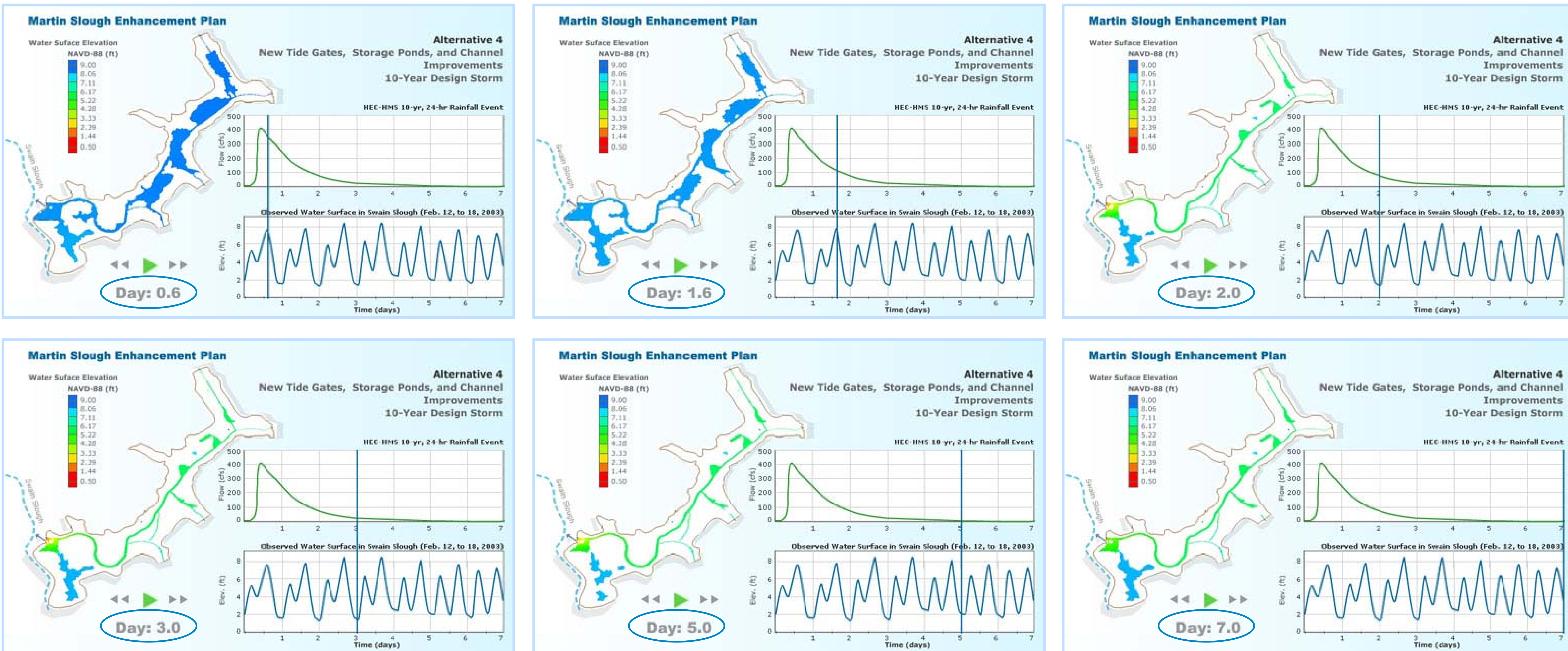
Martin Slough Enhancement Plan
ADCIRC Hydraulic Model Graphic Results

Alternative 4 (New Tidegates, New Ponds, and Modified Channel): 10 Year Rainfall Event

Figure 15.8

Hydraulic modeling output shown below was generated with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). Each graphic displays the project area with surface water inundation corresponding to the time step indicated. The two charts to the right of the project area represent the design storm input hydrograph and the downstream tidal boundary condition used in the model. The vertical bar through both charts represents the model time step which correlates to the inflow hydrograph, downstream tidal conditions, and the model results graphically shown.

Time steps of the graphic output shown were chosen to represent near maximum inundation conditions followed by several days of draining after the design storm had passed.



Winzler & Kelly Consulting Engineers
Michael Love & Associates
Coastal Analysis LLC

16.0 SUMMARY

The study area for the Martin Slough Enhancement Feasibility Study consists of the general flood plain between Swain Slough and the upper (second) Fairway Drive stream crossing in the lower Martin Slough watershed (Figure 1.1). The study area is located in and adjacent to the southeast portion of the City of Eureka, and is partially within the coastal zone. Existing problems that have been identified in the Martin Slough study area include obstructed fish access, poor fish habitat, poor sediment routing, lack of riparian habitat, and frequent prolonged flooding that has a negative economic impact on current land uses.

Winzler & Kelly Consulting Engineers teamed with Michael Love & Associates and Coastal Analysis LLC to develop an enhancement plan to improve fish access, enhance aquatic habitat, and reduce flooding impacts on land use activities within the study area.

The broader goals of the project included working with interested stakeholders to understand their issues and objectives and to develop alternatives that could receive broad support. It is recognized that there are different real or perceived problems and solutions, and a long history of landowners and the public living with the current situation. This project provides an opportunity for stakeholders to work together and create a positive change to the lower Martin Slough watershed.

To accomplish these goals, a Technical Advisory Committee (TAC) was established and TAC meetings were organized and scheduled by the Natural Resources Services (NRS) Division of Redwood Community Action Agency (RCAA) to foster discussion between interested stakeholders such as the property owners, regulators, and the design team. Throughout the process the TAC provided input and helped guide the project direction and content.

A comprehensive understanding of the pre-development conditions is not possible given the few available historical maps and photographs. For the purpose of this study, pre-development conditions refer to the Martin Slough study area as it existed prior to levees being built along Swain Slough. While our research did uncover several old maps and photographs, nothing was found that pre-dated the levee along Swain Slough. It is likely that prior to the levee, tidal conditions occurred through much of lower Martin Slough, from Swain Slough extending upstream through the golf course. Under these conditions, vegetation that has adapted to the salt water influence would have dominated the study area. Very little of the historical salt tolerant vegetation currently remains except for a narrow strip of vegetation along the slough itself (up to the lower golf course irrigation pond) and the lower Martin Slough pastures opposite the tidegate where the existing leaky tidegates provide brackish water influence.

Martin Slough has a watershed area of approximately 5.4 square miles, and natural channel length of over 10 miles, with approximately 7.5 miles of potential salmonid fish habitat supporting coho salmon, steelhead trout, and cutthroat trout. However, the existing tidegates partially block upstream salmonid migration.

The lower portion of the watershed flows through low gradient bottomland containing the golf course and pastureland. Many of the stream channels flow from gulches that contain mature second-growth redwood forests. The upper portions of the watershed are either in urban settings, or are recently harvested timberlands slated for future residential or mixed use development.

To help develop the Martin Slough Enhancement Feasibility Study, aerial photogrammetry of the project area was used as the basis of the project base map. The aerial photogrammetry images with two-foot elevation contours were provided by the City of Eureka. The aerial photogrammetry used was flown in 2001 by Cartwright Aerial Surveys, Inc. Since the photogrammetry provided only two-foot contours of the project area, additional survey information was collected to better define the drainage characteristics of the low gradient Martin Slough channel within the project area, which was then used for the hydraulic model.

Determining project hydrology was an important aspect of the research. Hydrologic conditions were characterized through the following means:

- Collection of hydrologic data within the project area (stream flow and precipitation)
- Development of a numerical model for predicting hydrographs at various locations throughout the watershed resulting from design rainfall events (i.e. 2-year 24-hour rainfall event).
- Hydraulic modeling of the project area for characterizing existing conditions and examining hydrologic conditions associated with different project alternatives.

Hydrographs were also developed for anticipated future land-use conditions to determine how changes in runoff characteristics influence effectiveness of the different project alternatives. Version 2.2.2 of the ACOE Hydrologic Engineering Center's Hydrologic Modeling System software (HEC-HMS), that simulates precipitation-runoff and flow routing processes, was utilized to compute hydrographs for selected rainfall events.

Project alternatives were developed based on current land use combined with the ability to make modifications based on the current and projected future land use as well. The no tidegate alternative was requested as part of the project scope. The upper project area is predominately owned by the City of Eureka and the land is used as the Eureka Municipal Golf Course. The lower project area is predominately owned by a single private landowner and the land is used for agricultural grazing. Both land owners intend to maintain their current land use and all alternatives developed considered this desired land use. Both land owners also expressed willingness to consider allowing some of their land to change uses to make improvements to Martin Slough.

The following four alternatives were identified and refined as more information became available based on the results of the analysis conducted throughout the study.

16.1 Alternative 1: The No Action Alternative (Existing Conditions)

The No-Action Alternative would leave the system as it exists today. This alternative is important for permitting considerations and also for comparing alternatives, allowing a familiar starting point for comparisons to be made.

16.2 Alternative 2: No Tidegates or Levee (Full Tidal Influence)

The No Tidegates Tidally Influenced Alternative would result in removing the existing tidegates and a 20 foot section of levee at Swain Slough. This alternative would open the majority of the project area to full tidal influence, based on land and tidal elevations, allowing the system to transform back towards its pre-development state.

16.3 Alternative 3: New Tidegates and New Ponds (Muted Tide)

The New Tidegates and New Ponds (Muted Tide) Alternative would consist of removing the existing tidegates, installing new tidegates with a habitat door designed to create a muted tide cycle and facilitate fish passage, increasing the size of existing ponds and creating new ponds.

16.4 Alternative 4: New Tidegates, Storage Ponds, and Modified Channel (Muted Tide)

The New Tidegates, Storage Ponds, and Modified Channel (Muted Tide) Alternative is similar to Alternative 3, but includes improvements to the existing channel and a corresponding larger habitat door. This alternative consists of removing the existing tidegates, installing new tidegates with a habitat door designed to create a muted tide cycle and facilitate fish passage, increasing the size of existing ponds, creating new ponds, and making channel modifications throughout the project area.

Several different approaches were used to evaluate the alternatives. A simplified numerical model of tidegate hydraulics was created in a spreadsheet to allow for rapid analysis of the effectiveness of different tidegate designs in providing fish passage and flood routing within the project area. Fish passage analysis of the tidegates was conducted for each alternative. Passage conditions were evaluated using the stream crossing design criteria developed by NOAA Fisheries (2001) and CDFG (2002).

The geomorphic stability of enlarging the Martin Slough channel within the project area to increase conveyance area for both flood flows and a diurnal tidal exchange was analyzed using design guidelines developed for tidal channels. This was done because reintroducing a muted tide cycle into the project area would result in large volumes of water flowing up and down the channel with each tide cycle, changing the fluvial processes that maintain the channel.

To assist in determining potential impacts and evaluate potential permitting issues for the different alternatives, a wetland and biological reconnaissance investigation was conducted to determine the approximate size and location of wetlands, and sensitive plant and animal habitats within the potential footprint of the alternatives developed.

Hydraulic modeling of the alternatives was conducted with the two-dimensional finite-element model, ADCIRC (Luettich et al 1992). Objectives of the hydraulic modeling were to evaluate and compare alternatives in terms of inundation levels, inundation duration, and sediment transport for 2-year and 10-year storm events.

16.5 Graphic Inundation Comparison of Alternatives

The inundation effects of the different alternatives were evaluated based on the ADCIRC hydraulic model results over a seven day period using streamflows resulting from a 2 and 10 year rainfall event. Model results for each alternative were compared graphically, as shown on Figure 16.1 and Figure16.2. Comparisons show that Alternative 2 produces more inundation during high tide than during the respective rainfall events. The high tides modeled near day 3 and day 5 graphically show the increased inundation area compared to the other alternatives. As described in the alternative results section, the ponding shown on the existing pasture on the southern portion of the modeled project remains through day 7 due to the limitations of using 2-foot contour data to model low lying areas that have less than 2 feet of elevation difference and are naturally slow draining. Detailed topographic information in that area would improve the accuracy of the modeling and likely reduce the resulting ponding in some of the low lying areas.

Alternative 4 has the greatest potential to reduce inundation, but the model still shows wide spread inundation during the peaks of the storm events. However, Alternative 4 indicates a faster recovery time resulting in less inundation time than the other alternatives. Although the duration that the fields and golf course are inundated is reduced, the ponds and channel retain water to provide aquatic habitat.

16.6 Quantitative Inundation Comparison of Alternatives

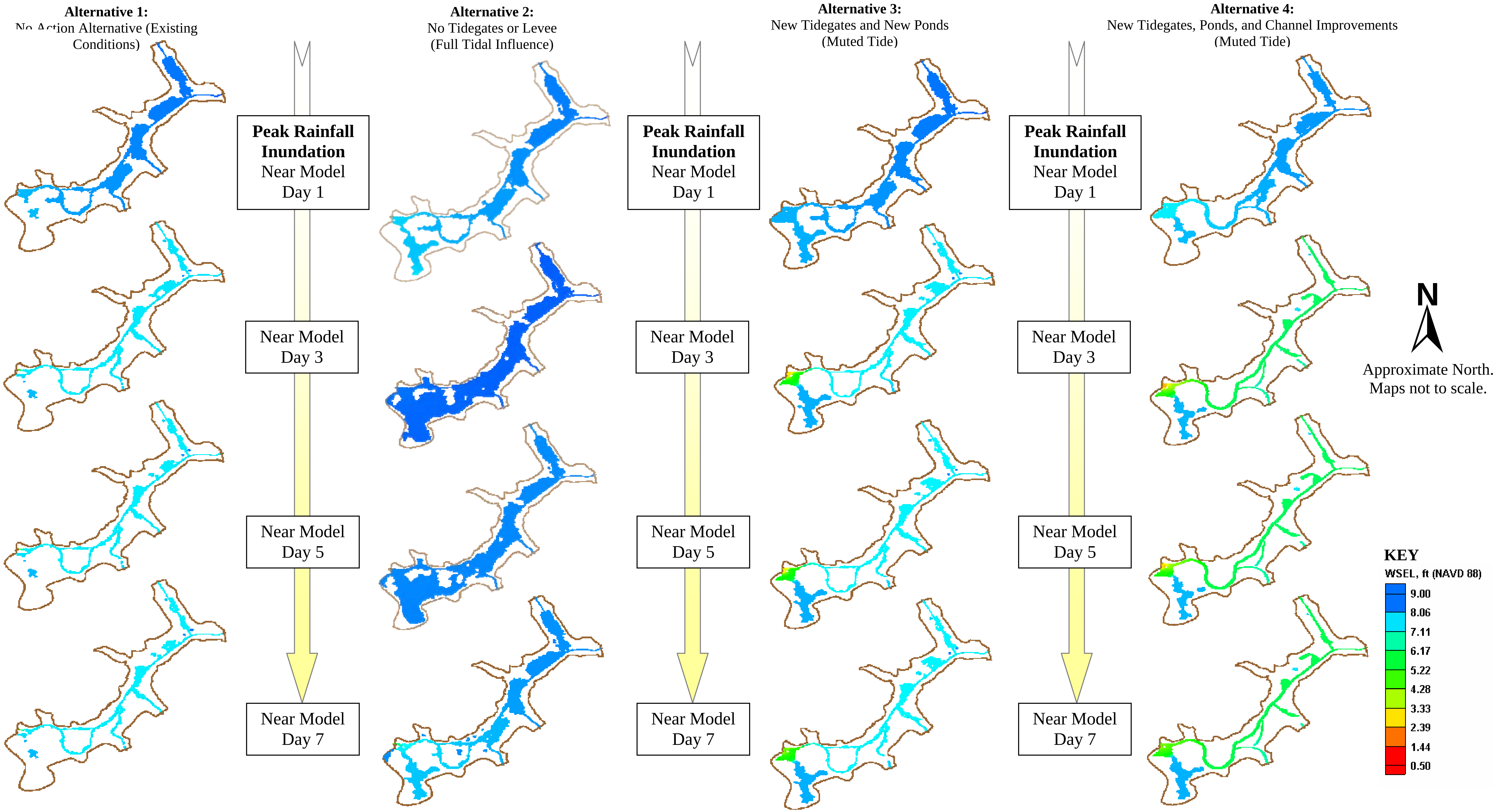
The TIDEGATE model, based on a simplified lump flow routing hydraulic model, was used to provide quantitative inundation comparisons for the different alternatives. Tables 16.1, 16.2, and Figure 16.3 provide quantitative comparisons of the number of hours certain elevations are inundated. The table provides data for the 2 year and 10 year design storms for both existing hydrologic (current) conditions and for (anticipated) future hydrologic conditions.

Martin Slough Enhancement Plan

Graphic Inundation Comparison of Alternatives from ADCIRC Hydraulic Model

2 Year Rainfall Event

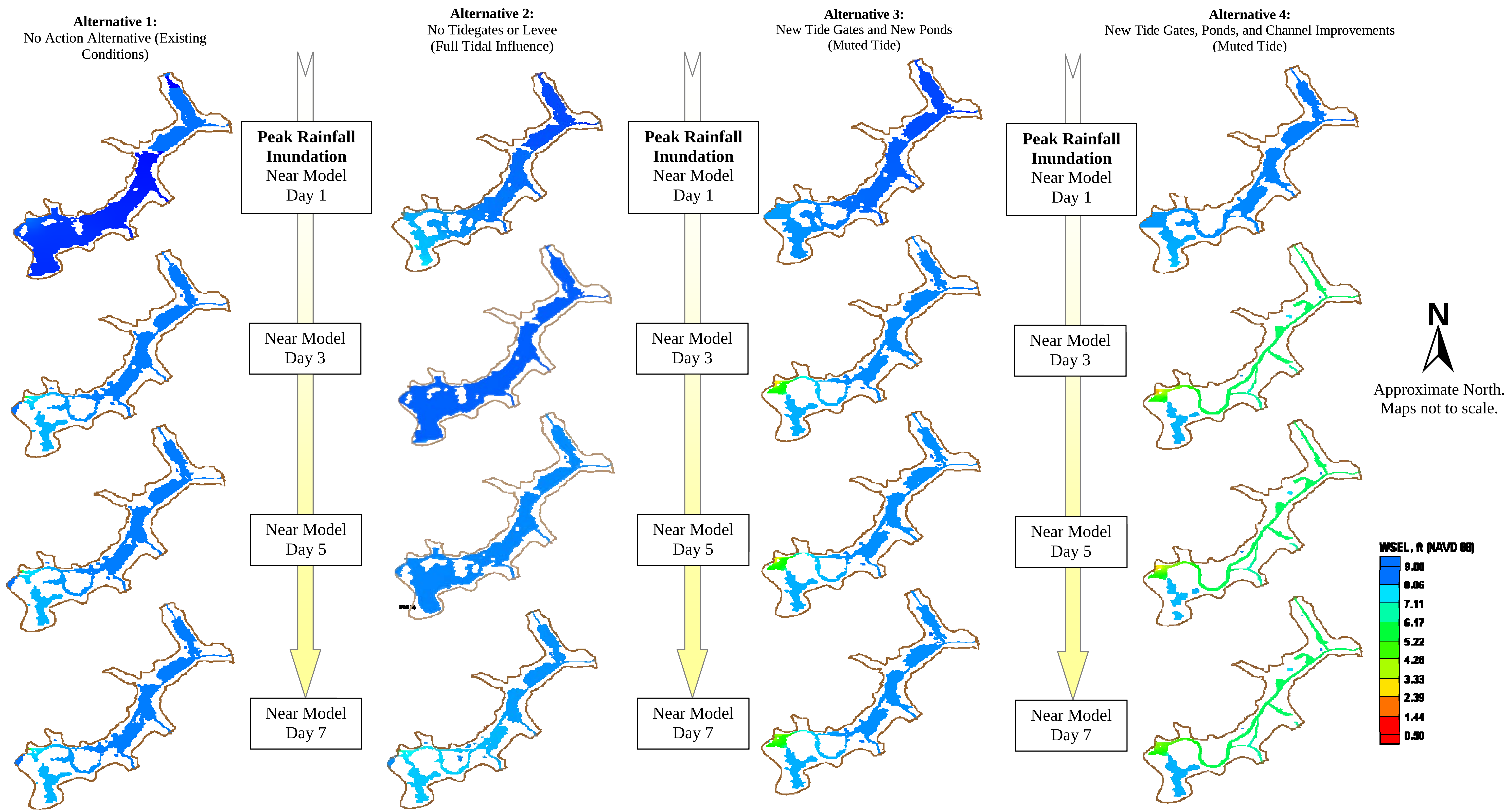
Figure 16.1



Martin Slough Enhancement Plan

Graphic Inundation Comparison of Alternatives from ADCIRC Hydraulic Model

10 Year Rainfall Event **Figure 16.2**



Martin Slough Enhancement Plan

Quantitative Inundation Comparison of Alternatives from the TIDEGATE Model

Table 16.1 – Estimated Cumulative time the 5, 6, 7, and 8 foot elevations are inundated within the project area for the 2-year and 10-year design storms with existing land use.

2-year Design Storm – Existing Land Use	**(Current Hydrologic Conditions)**			
	Hours* above Water Surface Elevation			
Alternative	**5 ft**	**6 ft**	**7 ft**	**8 ft**
1 - Existing Conditions	47.8	16.4	7.1	0.0
2 - Remove tidegate / Levee Breach‡	66.4	39.1	21.3	2.2
3 - New tidegate, storage ponds	22.2	7.3	3.0	0.0
4 - New tidegate, storage ponds, modified channel	25.8	6.3	1.9	0.0
10-year Design Storm – Existing Land Use	**(Current Hydrologic Conditions)**			
	Hours* above Water Surface Elevation			
Alternative	**5 ft**	**6 ft**	**7 ft**	**8 ft**
1 - Existing Conditions	63.8	24.7	15.0	0.0
2 - Remove tidegate / Levee Breach‡	67.4	40.0	22.0	2.3
3 - New tidegate, storage ponds	27.3	10.3	5.8	0.0
4 - New tidegate, storage ponds, modified channel	30.7	9.8	4.8	0.0

Table 16.2 – Cumulative time the 5, 6, 7, and 8 foot elevations are inundated within the project area for the 2-year and 10-year design storms with anticipated future land use.

2-year Design Storm – Full Build-Out Scenario	**(Future Hydrologic Conditions)**			
	Hours* above Water Surface Elevation			
Alternative	**5 ft**	**6 ft**	**7 ft**	**8 ft**
1 - Existing Conditions	53.2	19.8	10.1	0.0
2 - Remove tidegate / Levee Breach‡	66.8	39.5	21.6	2.3
3 - New tidegate, storage ponds	25.3	9.5	4.7	0.0
4 - New tidegate, storage ponds, modified channel	28.8	8.9	3.4	0.0
10-year Design Storm – Full Build-Out Scenario	**(Future Hydrologic Conditions)**			
	Hours* above Water Surface Elevation			
Alternative	**5 ft**	**6 ft**	**7 ft**	**8 ft**
1 - Existing Conditions	67.3	35.2	25.9	9.3
2 - Remove tidegate / Levee Breach‡	69.3	40.9	23.1	2.3
3 - New tidegate, storage ponds	32.0	12.9	7.6	0.0
4 - New tidegate, storage ponds, modified channel	34.9	11.7	6.2	0.0

* *Cumulative hours during 7 days of continuous flow modeled through tidegate.*

‡ *Simulated with a 20 ft wide opening set at bottom elevation of 0.0 feet.*

Note:

Inundation times shown above are based on a simplified Lump Flow Routing Model at the tidegate locations. The TIDEGATE model incorporates the design storm hydrograph and observed Swain Slough tidal elevations to predict water surface elevations at the tidegate locations. This model does not consider channel routing through the project area. Results from the simplified TIDEGATE model are for comparative purposes only.

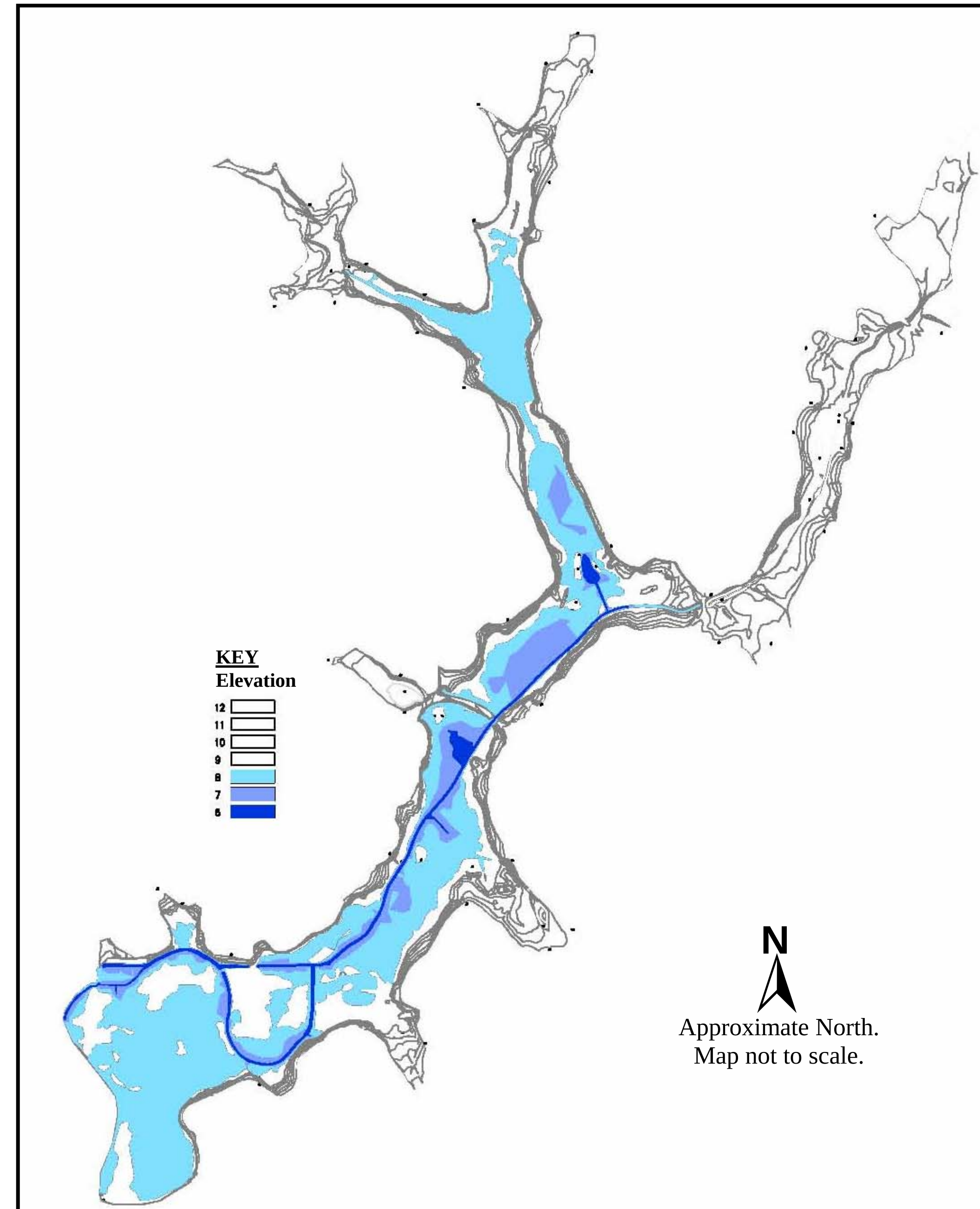


Figure 16.3 – Map of inundation for existing conditions for elevations between 5 and 8 feet (NAVD88) Inundation areas shown are based on existing topography only. Inundation areas shown utilized along with topographic surface to generate stage-volume relationships for use in the lump flow routing TIDEGATE model.

16.7 Summary Comparison of Alternative Results

With four alternatives and twenty criteria to consider, the need arose for a method to help evaluate the alternatives. The following table provides a method for the overall comparison of results. With all the criteria listed alongside each alternative, comparisons are easier to make. In addition to qualitative written descriptions of how each alternative addresses the project criteria, the descriptions were given color codes to help convey the cumulative benefit of any one alternative compared to another alternative. The color green was chosen for the most benefit, yellow was chosen for some benefit, and red was chosen for the least benefit or potential negative influence such as the highest project cost. Project criteria with no improvement were left white. The criteria are not weighted or otherwise ranked, and therefore there is no implied best or worse alternative. In fact, each alternative has its benefits and potential problems and the determination of which alternative is “best” will depend on which criteria are most important to the individual(s) making the comparison.

This report concludes our preliminary planning-level analysis of four alternative conceptual plans for the enhancement of Martin Slough. This information is presented for use as a decision making tool to assist project stakeholders in the selection of a preferred alternative.

Table 16.3 Comparison of Alternatives

Criteria	Alternative 1 No Action Alternative (Existing Conditions)	Alternative 2 No Tidegates or Levee (Full Tidal Influence)	Alternative 3 New Tidegates and New Ponds (Muted Tide)	Alternative 4 New Tidegates, Ponds, and Modified Channel (Muted Tide)
Fish Passage and Fish Access for Juveniles and Adults				
1. Maximize Migration Access at Tidegates during Fish Migration Flows	No Improvement	Most Improvement	Some Improvement	Some Improvement
Fish Habitat				
2. Maximize Estuarine Habitat	No Improvement	Some Improvement	Some Improvement	Most Improvement
3. Increase Channel Complexity	No Improvement	No Improvement	No Improvement	Most Improvement
Riparian Corridor				
4. Increase Riparian Habitat	No Improvement	No Improvement	Some Improvement	Most Improvement
5. Increase Riparian Canopy	No Improvement	No Improvement	Some Improvement	Most Improvement
Water Quality				
6. Decrease Nutrient Impacts	No Improvement	Some Improvement	Some Improvement	Most Improvement
7. Decrease Sedimentation	No Improvement	Some Improvement	Some Improvement	Most Improvement
Wetlands				
8. Improve Wetland Habitat	No Improvement	Some Improvement	Most Improvement	Most Improvement
9. Increase Open Water Area of Wetlands	No Improvement	Some Improvement	Most Improvement	Most Improvement
10. Increase Diversity of Wetland Types	No Improvement	Some Improvement	Some Improvement	Most Improvement
Flood Impacts				
11. Reduce Flood Inundation Area	No Improvement	No Improvement/ Potentially Worse	Some Improvement	Most Improvement
12. Reduce Frequency of Flooding	No Improvement	No Improvement/ Potentially Worse	Some Improvement	Most Improvement
13. Minimize Duration of Flooding	No Improvement	No Improvement/ Potentially Worse	Some Improvement	Most Improvement
Existing Land Uses				
14. Maintain Agricultural Land Use	No Improvement	Likely Worse	Some Improvement	Most Improvement
15. Maintain Eureka Municipal Golf Course	No Improvement	Likely Worse	Some Improvement	Most Improvement
16. Allow for full Build-out Potential for City/County	No Improvement	No Improvement	Some Improvement	Most Improvement
17. Allow for Installation and Maintenance Access for City's Martin Slough Sewer Interceptor Project	No Improvement	No Improvement/ Potentially Worse	Some Improvement	Most Improvement
Project Permitting				
18. Consider ability to Obtain Permits	Permitting efforts for maintenance may increase with time	Potentially Very Difficult	Potentially Very Difficult	Moderate Effort Required
Cost of Improvements				
19. Consider Order of Magnitude Opinion of Probable Construction Costs	No Cost	Low Cost	Moderate Cost	Highest Cost
Project Maintenance				
20. Consider Need for Ongoing Maintenance	No Improvement	Potentially Worse	Potentially Worse	Most Improvement

Appendix A

Hydrology Report

Hydrologic Analysis for Martin Slough

Prepared by:

Michael Love, Michael Love & Associates
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With assistance from:

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July 18, 2005

Table of Contents

1 Hydrologic Monitoring

Graham Mathews and Associates were contracted by Redwood Community Action Agency to collect hydrologic data at several locations within lower Martin Slough for use in developing project designs for marsh restoration, flood routing and detention, and calibration of hydrologic and hydraulic models.

Hydrologic monitoring included collection of precipitation data, continuous stage recorders, discharge measurements, and crest stage gages. Monitoring efforts began in February 2003 and continued through June 2003. The continuous stage recorder continued to operate into January 2004.

For details on methods and results of the hydrologic monitoring, refer to Appendix E.

2 Hydrologic Analysis and Modeling

Hydrologic modeling is an essential tool for predicting flow regimes when long term continuous flow gauge data is not available. Hydrologic modeling can also be used to study the effects of changing land uses on the runoff characteristics of a watershed. We chose to develop and use a hydrologic model of the entire Martin Slough Watershed for the following reasons:

(1) Martin Slough lacks historical streamflow data to perform a probabilistic prediction for recurrence flow regimes, and

(2) A component of this study includes examining the effects of future land use development on drainage characteristics within the project area

Version 2.2.2 of the ACOE Hydrologic Engineering Center's Hydrologic Modeling System software (HEC-HMS) was utilized to compute flow regimes based on desired rainfall events. Similar to its predecessor, HEC-1, HEC-HMS simulates precipitation-runoff and routing processes. The difference in the two models is subtle; aside from the graphical user interface introduced in HEC-HMS, the governing simulation routines found in HEC-1 were also used in developing HEC-HMS. Because of the inherent properties of the methodologies and routines within HEC-HMS, ACOE recommends modeling discrete storms only. Depending on the selected methods, a duration of time that includes several storms can be modeled, but the results should be critically analyzed (ACOE 2000).

2.1 Model Development

Procedures for developing, simulating and interpreting results from the HEC-HMS model were followed using both the ACOE HEC-HMS Technical Reference and User Manuals. The following flowchart (Figure 2.1) depicts the steps and methods employed in the hydrologic modeling, which are explained in detail within the sections to follow. In general, initial model development followed the USDA Soil Conservation Service (SCS) methods (NRCS 1986; NRCS 2002).

Using stream flow and precipitation data collected during the initial phase of this project, the model was calibrated and its reliability to predict flows was validated. During the calibration process the standard SCS unit hydrograph and lag time with constant baseflow method failed to adequately describe runoff characteristics within the watershed. As a result, alternative methods were used that more accurately predicted flows.

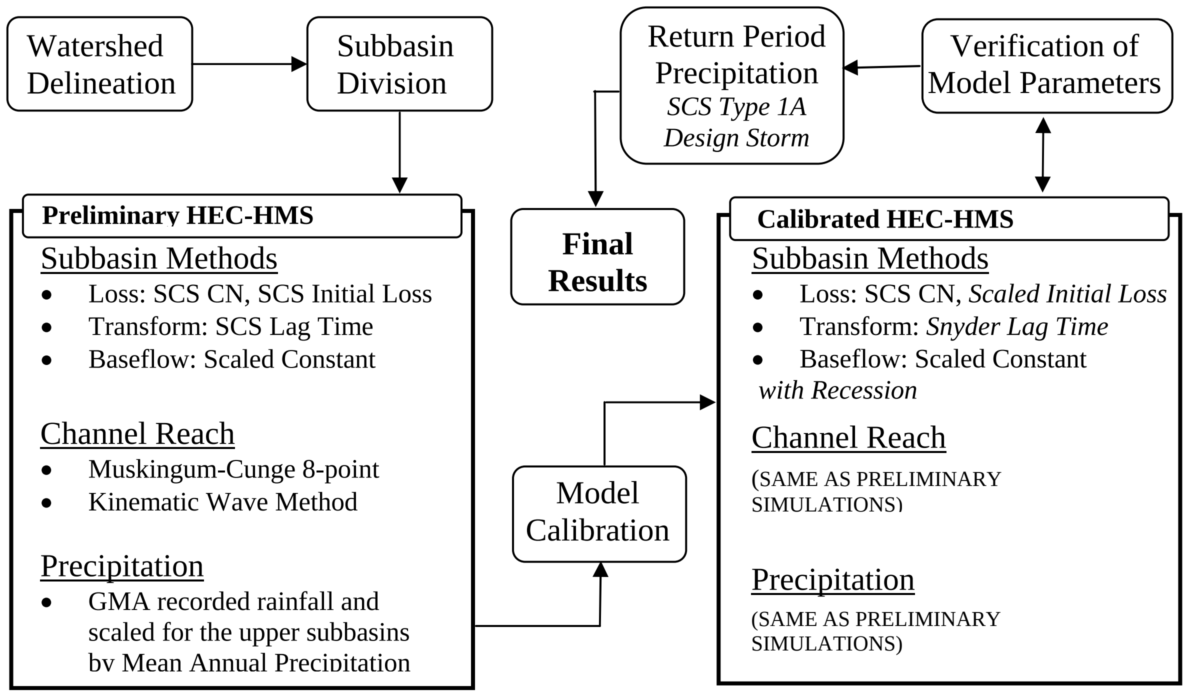

Figure 2.1 - Flowchart showing HEC-HMS model structure and methods used in development of the hydrologic model for Martin Slough.

2.1.1 Watershed Delineation

The first step of model development was division of the watershed into 44-subdrainages, referred to in HEC-HMS as *subbasins*. A subbasin element represents a complete watershed that is separated into three separate processes: *loss rate, transform, and baseflow.* The quantity of rainfall that falls and infiltrates is represented by a loss rate method. The excess rainfall which does not infiltrate and becomes runoff is represented by a transform method. Groundwater contributions to channel flow are represented with a baseflow method.

Contributing runoff from a subbasin into a defined stream channel is modeled in HEC-HMS using open channel flow principles, and referred to as a *reach* element. The attenuation characteristics and travel time of water flowing through a reach is dependent on length, slope, friction, flow depth and channel storage. The confluence location at which two or more reaches combine is referred to as a *junction* element. Unlike a subbasin or reach element, physical properties are not assigned to a junction element. A junction element is strictly for computation purposes within the model and a location for which the user may view flow results (ACOE, 2001).

Field reconnaissance to determine flow path direction for each street and city block within the Martin Slough Watershed was conducted to delineate subbasin divides, channel reaches, and flow paths (Figures 2.2, 2.3). Watershed characteristics such as locations of day lighted underground storm drains, land cover, topography and the need to examine runoff hydrographs at specific locations, all factored into the division process. Once delineated, all 44 subbasins and 15 reaches were entered into the HEC-HMS model (Figures 2.4).

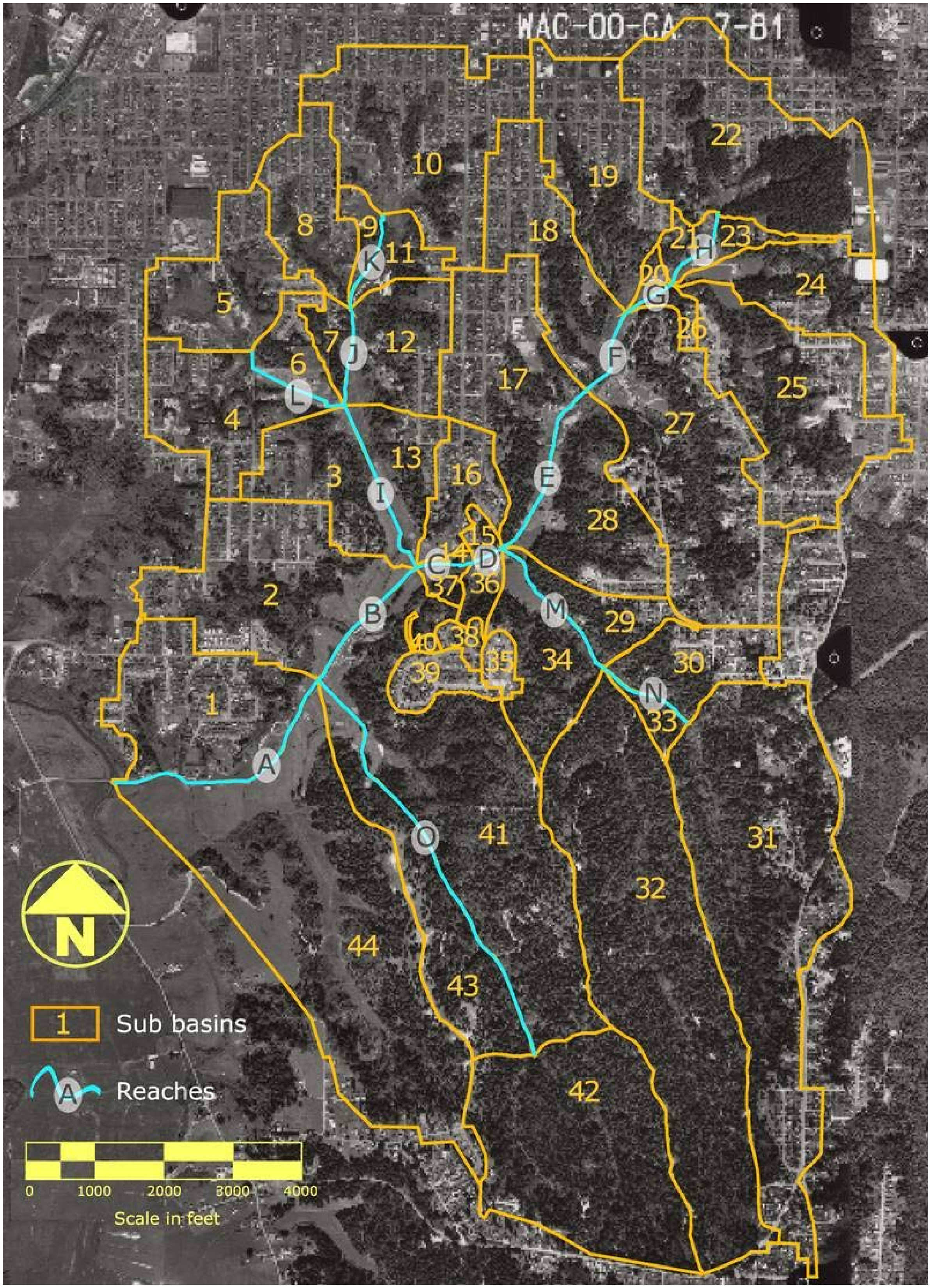

Figure 2.2 - – 2000 aerial map of Martin Slough Watershed, with the 44-subbasins, and 15 channel reaches delineated. Map created by Natural Resources Services, RCAA.

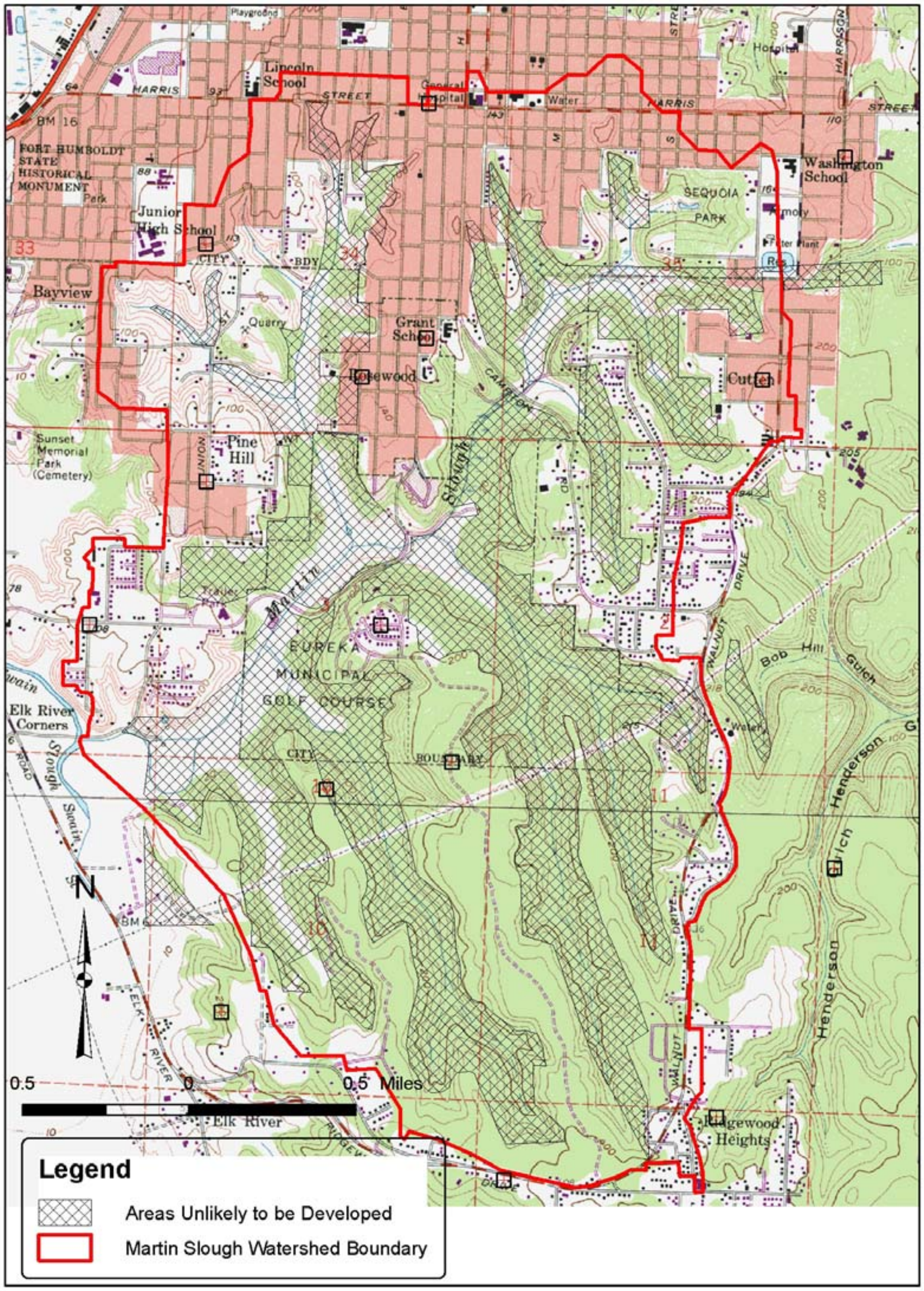

Figure 2.3 - – Areas within the Martin Slough Watershed designated for no-further development. Land covers within these areas were assumed to remain unchanged within the full build-out scenario.

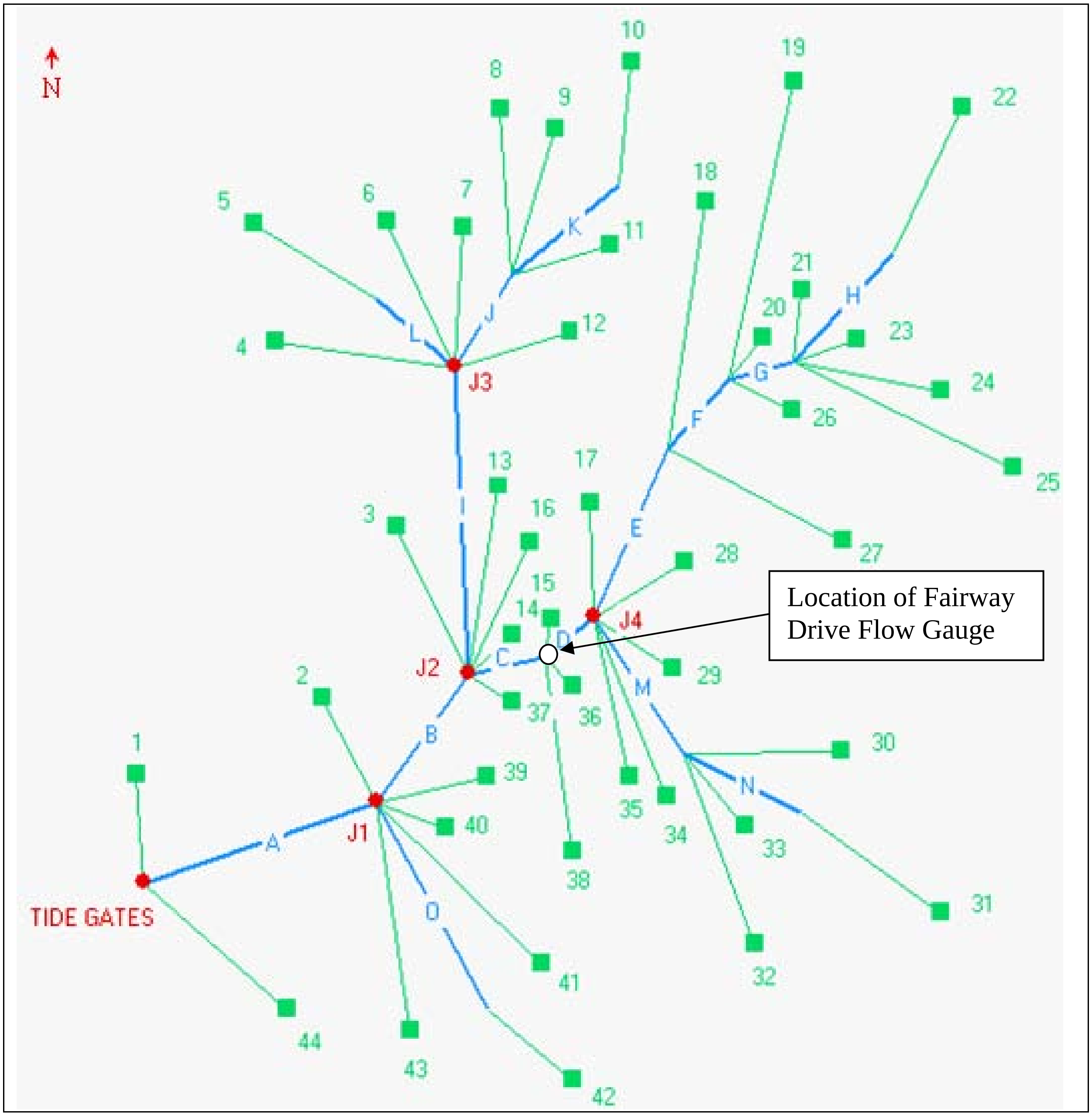

Figure 2.4 - HEC-HMS element network of Martin Slough Watershed including locations of 44 contributing subbasins (depicted by a green box), 15 channel reaches (shown as a blue line), and five channel reach junctions(indicated by a red dot)

2.1.2 Subbasin Characteristics and Processes

The diverse land coverage within the watershed was classified and divided into six discrete land covers: commercial, dense urban, sparse urban, grasslands, timber and reservoir (Table 2.1).

Table 2.1 - Land cover area within the Martin Slough Watershed.

	Current Conditions		Full Build-Out Scenario	
Land Cover	**Area (acres)**	**Portion of Watershed (%)**	**Area (acres)**	**Portion of Watershed (%)**
Grassland	397	11.27	234	6.63
Dense Urban	1,028	29.17	2,293	65.06
Commercial	16	0.46	16	0.46
Timber	1,944	55.16	948	26.89
Sparse Urban	138	3.91	33	0.93
Reservoir	1	0.03	1	0.03
Total Area	**3,524**	**100**	**3,524**	**100**

Within each of the 44-subbasins, land cover was visually delineated using georectified aerial photos in Arcview 3.3 (Appendix A, Table A-1). An example of commercial land cover can best be defined as the business district in the Henderson Center area. The definitions used for delineating dense urban and sparse urban land covers are residential lot sizes of roughly 1/6-acre or less and greater than 1/6-acre, respectively. Land coverage associated with grasslands includes pasture, grazing rangeland, and golf course fairways. Land coverage associated with timber includes forested areas regardless of tree density, age or species. The reservoir land coverage was solely used to account for the City of Eureka's roofed reservoir adjacent to Sequoia Park.

2.1.2.1 Full Build-Out Scenario

As part of this project we examined possible affects on future peak flows and runoff volumes associated with potential future land use changes within the watershed. The goal was to consider the hydrologic implications of a future full build-out scenario. To accomplish this, we worked with City and County staff to identify potential future land-use changes considered allowable. For example, a large portion of the southern watershed is expected to eventually transition from current timber production to residential and mixed land-use. Additionally, within the currently developed residential areas of the watershed located mostly within the City limits, further infill is expected to occur.

As part of the Martin Slough Sewer Interceptor Project, City and County staff helped develop a map that showed currently undeveloped areas within the watershed that had slopes greater than 30% or are considered wetlands. These areas, which consist of mostly gulches, were considered non-developable as part of the sewer interceptor project. For the full build-out scenario we assumed that areas receiving the non-developable designation would continue having the same land cover as currently designated. Sequoia Park was also included in our model as an additional area whose current land use would remain unchanged.

The remaining areas within the Martin Slough watershed that fell outside of the non-developable areas described above and are not currently designated as commercial or industrial were assumed to become dense urban based on discussions with City and County staff. Using GIS, we identified the proportion of different land uses within each subbasin for this full build-out scenario. To account for the residential infilling City and County staff suggested we use in the hydrologic model runoff characteristics (curve numbers) associated with residential lot sizes equivalent to 1/8-acre or less for designated dense urban land cover.

2.1.3 Loss Rate Method

The commonly used Soil Conservation Service (SCS) empirical curve number method was utilized in HEC-HMS to estimate total excess precipitation. The curve number (CN) represents the soil cover, land cover and antecedent moisture conditions of a watershed. A curve number was assigned to each of the land covers defined above, following the Technical Release 55 methodology (NRCS, 1986). Surface soil texture maps were obtained from the NRCS for the entire watershed and are shown in Appendix A, Figure A-1). For the purpose of this project, it was assumed that Antecedent Moisture Condition Type II (AMC II) curve numbers be applied. AMC II assumes that 0.5-inches to 1.1-inches of rain had fallen in the watershed of interest over the course of 5-days prior to the initiation of the design storm. Curve numbers developed for AMC II are the most widely used in hydrologic analysis when utilizing the SCS method. Additionally, the NRCS recommends avoiding the use of AMC I and III curve numbers (Ponce and Hawkins, 1996).

Because most subbasins within the Martin Slough watershed consist of several soil types and land covers, an area weighted composite curve number was calculated for each subbasin.

The initial loss, also referred to as initial abstraction, is the depth of rainfall that either infiltrates into the soil horizon or is captured in depressional storage sinks, and does not contribute to runoff. The initial abstraction was first computed for each subbasin using the empirical relationship developed by the SCS (NRCS, 1986).

$$I_a = 0.2\left[\frac{1000}{CN_{composite}} - 10\right] \qquad \text{(Equation 2.1)}$$

where:

I_a (inches)	=	SCS Initial Abstraction.
$CN_{composite}$	=	SCS Composite Curve Number for the subbasin

During calibration of the model, this method was found to over predict the initial abstraction. ACOE recommends an initial abstraction for forested areas to range between 10-20% of the total rainfall, and 0.1-0.2 inches for urban areas (ACOE, 2000). Equation 2.1 yielded initial abstractions far greater than the ACOE recommended values and resulted in the model-predicted streamflow not responding to precipitation until a substantial amount had fallen. For example, for several of the forested subbasins Equation 2.1 yielded initial abstractions surpassing two inches of precipitation (Appendix A, Table A-4). Due to this discrepancy, an alternative method for estimating initial abstraction was used that was based on the percentage of impervious area within each subbasin (Appendix A, Table 3).

Based on rainfall-runoff results during model calibration of AMC Type II storms, an average initial abstraction of 0.15-inches was assumed for the entire Martin Slough watershed. Next, for each subbasin the overall initial abstraction of 0.15 inches was scaled based on the percent area impervious within the subbasin. Equation 2.2 was developed to estimate the scaled initial abstraction in each subbasin.

$$I_{a\,i} = I_a * n \left[\frac{100 - \%\,Impervious_i}{\sum_{i=1}^{n} (100 - \%\,Impervious_i)} \right] \qquad \text{(Equation 2.2)}$$

where:

I_{ai} (inches)	=	Computed initial abstraction for each subbasin *i*.
I_a (inches)	=	Average initial abstraction within the entire watershed.
n	=	Total number of subbasins within watershed.
$\%\ Impervious_i$	=	Percent area of subbasin *i* comprised of impervious surfaces.

Scaling the overall initial abstraction of 0.15 inches by each subbasin's percent area impervious produced initial abstractions ranging from 0.19 inches for completely forested subbasins (no impervious area) to 0.07 inches for the most urbanized subbasins (65% of subbasin impervious). Initial abstraction values for each of the subbasins are provided in Appendix A, Table A-4.

2.1.4 Transform Method

Both the SCS Unit Hydrograph and Snyder Unit Hydrograph (UH) methods were explored in HEC-HMS for transforming excess rainfall into streamflow. Even though both methods incorporate similar empirical unit hydrograph principles, the results were very different relative to one another. The SCS UH methodology calculates the lag time based on hydraulic flow path length, average subbasin slope and the SCS curve number. The flow path for each subbasin was determined from field reconnaissance, mapped, and digitized for length in Arcview 3.3. A topographic map with 2-foot intervals provided by the City of Eureka was used in determining the average subbasin slopes. These values and the corresponding computed SCS Lag Times are tabulated in Appendix A, Table 5.

After several simulations and comparisons to observed flows from the Upper Fairway Drive streamflow gauge, we found the hydrographs produced using the SCS transform method did not adequately describe the shape of the observed hydrographs. It consistently over predicted the peak flows and under predicted the outflow volume associated with each rainfall event. Alternatively, the Snyder UH method was decided upon for final transform modeling.

Snyder's UH method requires specifying the standard lag time and two non-physically based coefficients that adjust the shape of the unit hydrograph. Assigning the appropriate value to the non-physical based coefficients is the primary reason the Snyder method is not widely used in hydrologic modeling. However, the method is applicable when observed flow data is available for calibration (ACOE, 2000). The Snyder UH methodology calculates the lag time based on two hydraulic flow lengths: (1) distance from the basin divide to the outlet, and (2) distance from the basin's centroid to the outlet following the same flow path. Using these flow lengths, we calibrated the model to observed flows by adjusting the two non-physical based parameters (Appendix A, Figure XX). The calibration focused on a balance between accurately predicting peak flow and total outflow volume. Throughout the calibration process the same parameters were applied uniformly to all subbasins. These values and the corresponding computed lag time for each subbasin are tabulated in Appendix A, Table 6.

2.1.5 Baseflow Method

Baseflow accounts for the quantity of flow contributed from groundwater, and not direct precipitation-runoff. For modeling design storms, each subbasin required an initial baseflow. This flow rate is easily recognized on the Martin Slough hydrograph as the lower flows occurring between peaks (Appendix D). The average baseflow occurring during the period in which flows were measured (February through June, 2003) was approximately 2 cfs at the upper Fairway Drive crossing. Initial baseflow for each subbasin was computed by scaling the 2 cfs by drainage area (Appendix A, Table 7).

The simplest and most commonly used method for estimating baseflow is to assume it's constant throughout the entire event. In addition to the constant baseflow method HEC-HMS offers an exponential recession method. The recession method models the lower portion of the hygrograph's receding limb assuming exponential decay. Incorporating the recession model generally requires having observed flow data for calibrating the recession constant.

Many factors associated with both watershed characteristics and channel hydraulics can greatly affect the slope of the receding hydrograph. The receding limb on the observed hydrographs at Fairway Drive confirmed that both the SCS and Snyder transform methods were predicting a receding limb far too steep. This indicated the observed flow was draining out of the watershed much slower and producing more overall volume than the model was predicting. Also, examining the hydrographs from two nearby gauged streams revealed recession limbs similar in shape to those observed in Martin Slough

(Appendix D). To more accurately match the characteristics of the observed Martin Slough hydrographs, the recession model in HEC-HMS was utilized.

The recession model parameters required by HEC-HMS include an exponential decay constant and a threshold flow. The decay constant is defined as the ratio of the current computed baseflow to the computed baseflow one day earlier. This parameter governs how rapidly the receding limb falls back towards the initial baseflow. The threshold flow defines the point at which the recession model begins to compute the receding limb flow rates. The threshold can be specified as a ratio to the computed peak flow or simply a flow rate (ACOE, 2000). Through calibration processes, the exponential decay constant value of 0.275, and a threshold ratio-to-peak value of 1.0 was assigned to all subbasins (Appendix A, Table 7).

2.1.6 Channel Routing Reaches

Even though HEC-HMS is considered to be strictly a hydrologic model, it still offers the capabilities of routing hydrographs from one subbasin through another using various hydraulic methods. Several modeling methods for routing water through channel reaches are offered within the program. Because of its inability to perform standard-step calculations, HEC-HMS only analyzes channel reach hydraulics using one representative cross-section flowing at normal depth. Factors associated with channel slope and floodplain interaction govern which method is appropriate for a particular reach.

Following the ACOE recommendations, the Muskingum-Cunge 8-point cross-section methodology was used to model the lower gradient channels with large floodplains (Reaches A through L). Reaches M through O consist of steeper gradient, incised channels, for which the Kinematic Wave method was utilized. Both methods require one cross-section that represents the entire reach. For the purpose of this model, representative cross-sectional geometry was obtained from field measurements within each reach. Channel reach lengths were determined using Arcview 3.3. Channel slope and floodplain shape and size were estimated using the 2-foot interval topographic map provided by the City of Eureka (Appendix B). Roughness values for each cross-section were developed through field inspection and followed tabulated recommendations (Chow, 1959).

Because of the simplistic nature of HEC-HMS hydraulic routing methods, backwater effects created from flow constrictions are not accounted for. Aside from the tidegates at the confluence of Martin Slough with Swain Slough, three other flow constriction locations were identified (Table 2.2). These constrictions may increase lag times during larger flow events, which may influence the magnitude and timing of observed peak flows compared to those predicted by the model.

Table 2.2 - Location and type of flow constrictions not modeled in HEC-HMS.

Location of Flow Constriction	Location of Flow Constriction in the HEC-HMS Model	Type of Flow Constriction
Martin Slough Tidegates	Reach A	(3) 48” CMP with flapgate
Campton Road Crossing	Junction of Reach E and F	(5) 48” RCP
Private Road Crossing Below Brogan Road	Reach G	48” CMP
O Street Crossing	Reach H	(3) 24” CMP

The tidegates located at the outlet of the Martin Slough Watershed cause substantial backwater effects due to several factors:

1. During higher tides the gates are closed, preventing the streamflow from exiting. During larger flow events this results in the filling of the channel and inundation of overbank areas extending from the tidegate through most of the golf course.

2. During periods of lower flows and high tides substantial saltwater leakage occurs through the closed gates. Additionally, during spring high tides the tidal waters overtop the Swain-Martin Slough levee adjacent to the tidegates. This creates tidal backwater effects within the channel that extend upstream past the lower Fairway Drive bridge crossing.

3. The tidegates, when open, have limited conveyance capacity. As a result, during larger flow events the backwater effects may persist for numerous tidal cycles.

The backwater effects caused by the existing tidegates are mostly limited to the detailed study area, which is comprised of the Senestrero Property and the Eureka Golf Course. Hydraulic conditions within the study area were analyzed using a 2-dimensional hydraulic model. HEC-HMS predicted flows entering the edge of the project area were used as flow inputs into the hydraulic model, thus avoiding errors associated with routing flows in reaches with backwater influences from the tidegates (Reaches A, B, C, and I).

2.2 Model Calibration and Validation

The HEC-HMS model was calibrated and results were validated using precipitation and stream flow data collected by Graham Mathews & Associates (GMA) between February and April, 2003. As is commonly the practice, we chose to assume Type II antecedent moisture conditions (AMC II) for the project design storms (such as the 2-year 24-hour rainfall event). Therefore, model parameters were calibrated to observed AMC II precipitation events.

2.2.1 Scaling Observed Precipitation

HEC-HMS offers various methods for assigning and modeling precipitation events. Methods for using either theoretical storms or actual incremental rainfall data are offered. The calibration and validation process relied on incremental precipitation data collected at the Eureka Golf Course during 2003. However, the rainfall was collected at an elevation of approximately 10-feet above mean sea level while the elevations of the upper subbasins are at nearly 400-feet elevation. Isopluvial maps indicate that subbasins 30, 31, 32, 33 and 42 are located within the region that receives a mean annual precipitation of 44 inches. The remaining subbasins, including the location of the Eureka Golf Course rain gauge are located within a region that receives a mean annual precipitation of 40-inches (Appendix C). In an attempt to account for orographic effects within the watershed, the observed precipitation data was scaled-up for five of the upper subbasins based on differences in mean annual precipitation. The scaled incremental rainfall was then used in the calibration and validation model simulations.

2.2.2 Selection of Storms for Model Calibration

The HEC-HMS model was developed as part of this project for predicting flows resulting from large rainfall events. Therefore, we selected some of the largest observed peak flow events that occurred during the period of record for calibrating the model. For selecting specific flow events for use in calibration and validation of model results, three criteria were established:

1. A discrete storm with a single-defined peak flow
2. A high peak discharge for the period of record
3. AMC II (0.5-inches to 1.1-inches of rainfall within the 5 days prior to storm event)

The three most discrete storms resulting in relatively high peak flows for the period of record and determined to be AMC II are listed in Table 2.3.

Flow events resulting in greater peak flows would have been utilized for model calibration, but they were either not discrete enough or determined to be AMC III storms.

Table 2.3 - AMC II Peak flow events at Fairway Drive crossing that were used for model calibration and validation.

Rank of Peak Flow Magnitude for Period of Record	Date and Time of Peak at Fairway Drive crossing	Peak Flow (cfs)	24-hour Rainfall (inches)
2	April 24, 2003 @1:00am	48.3	1.17”
5	March 26, 2003 @ 3:30am	42.9	1.25”
7	March 22, 2003 @ 5:30pm	33.4	0.79”

2.2.3 Model Calibration and Validation

Referring back to the model development flowchart (Figure 2.1), several methods used in the preliminary model simulations were altered prior to the final model simulations. Table 2.4 shows the sequence of applied HEC-HMS methods used during the calibration process. Calibration efforts focused on obtaining good estimations of both peak flow and total outflow volume with respect to observed flows.

Table 2.4 - Sequence of preliminary simulations used in calibrating the HEC-HMS model for all three selected storm events. The bold and italic lettering indicates the method that was changed from the preceding simulation.

HEC-HMS Process	Preliminary Simulation 1	Preliminary Simulation 2	Preliminary Simulation 3	Preliminary Simulation 4
Loss	SCS CN	SCS CN	SCS CN	SCS CN
	SCS Initial Abstraction	***Scaled Initial Abstraction***	Scaled Initial Abstraction	Scaled Initial Abstraction
Transform	SCS Lag Time	SCS Lag Time	***Snyder Lag Time***	Snyder Lag Time
Baseflow	Scaled Constant Baseflow	Scaled Constant Baseflow	Scaled Constant Baseflow	***Scaled Constant Baseflow w/ Recession***

Figure 2.5 shows the sequence of hydrograph results generated by changing the methods used in HEC-HMS. When calibrated, methods used in simulation 4 yielded the best-fit computed hydrograph for estimating both the peak flow and total runoff volume. The parameters for the baseflow recession and Snyder methodologies were calibrated to best match the modeled storms using both trial and error and optimization measures. The final parameters were decided upon based on balancing the model's ability to predict both peak flow and total outflow volume for each of the three observed storm events (Figures 2.6, 2.7 and 2.8).

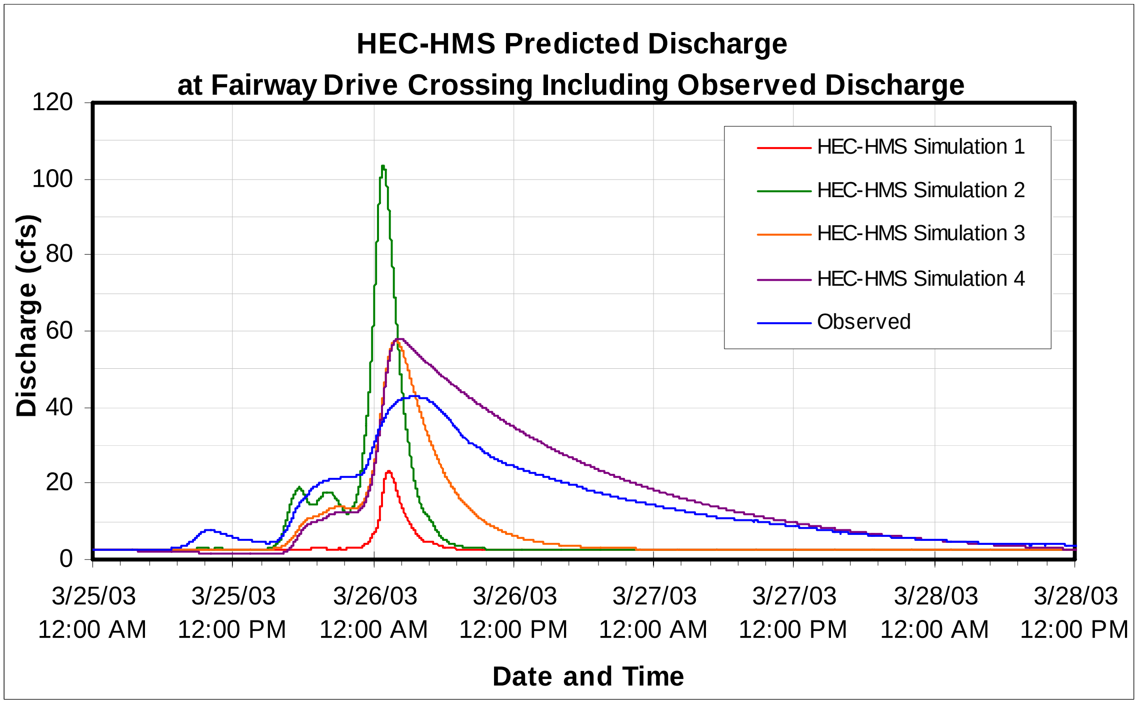

Figure 2.5 - Computed hydrographs at Fairway Drive crossing (Reach C) for each preliminary HEC-HMS simulation defined in Table 2.4.

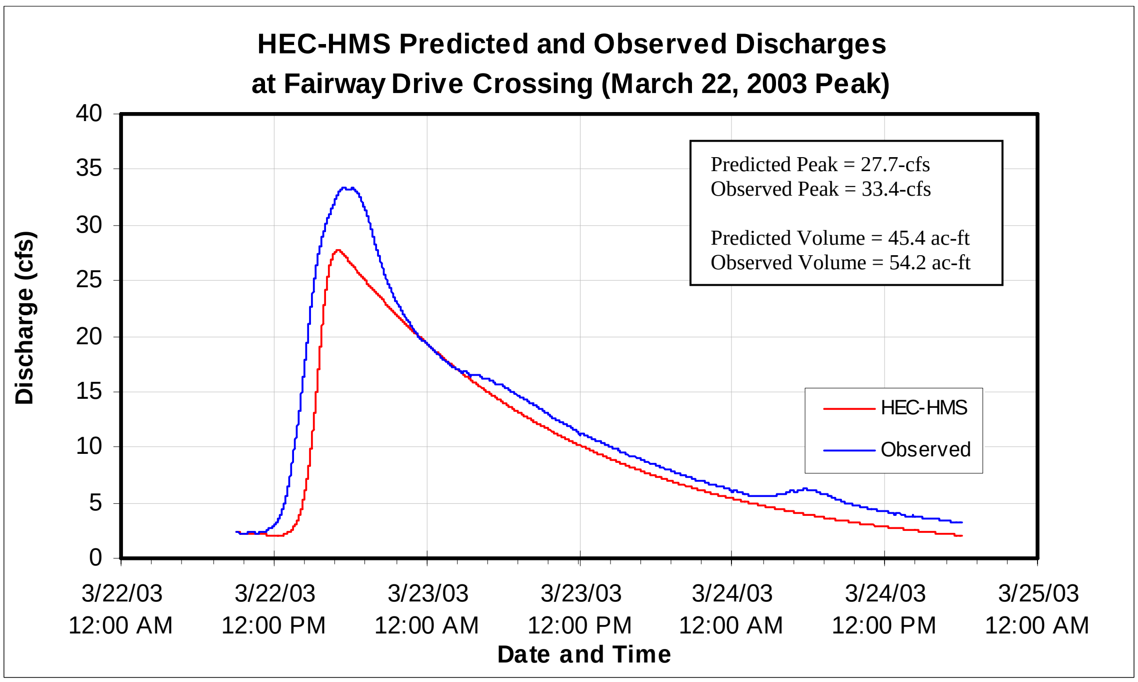

Figure 2.6 - Observed and HEC-HMS predicted flows at Fairway Drive crossing during precipitation event beginning on March 22, 2003.

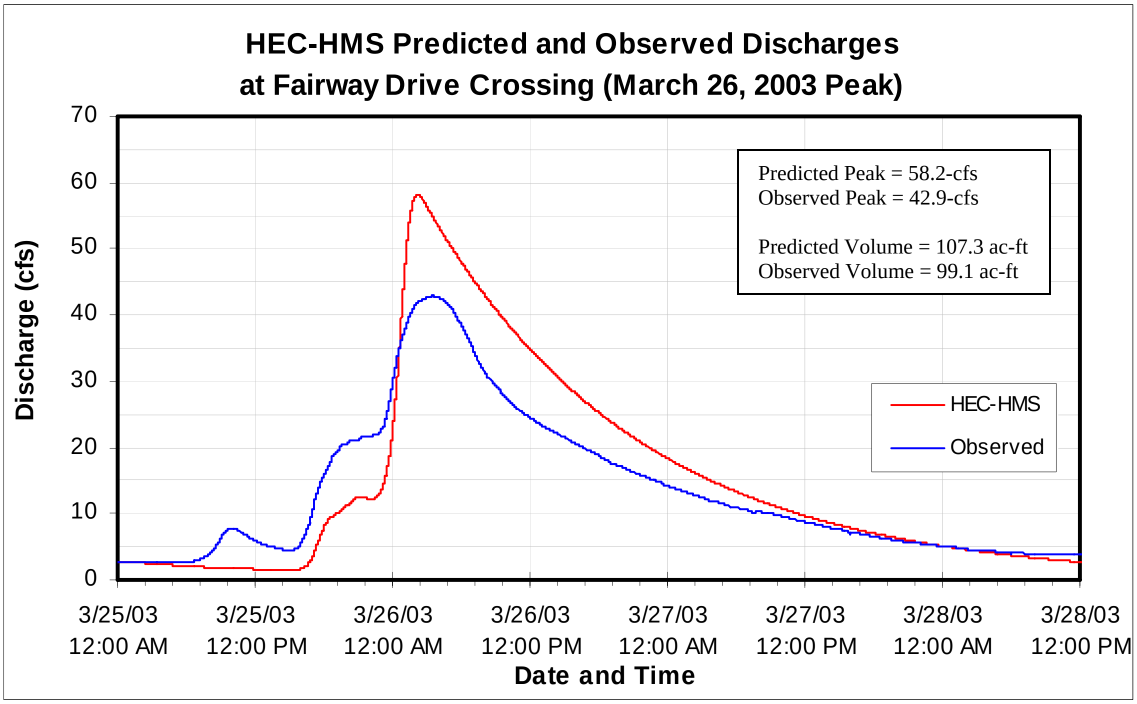

Figure 2.7 - Figure 2-1 – Observed and HEC-HMS predicted flows at Fairway Drive crossing during precipitation event beginning on March 26, 2003.

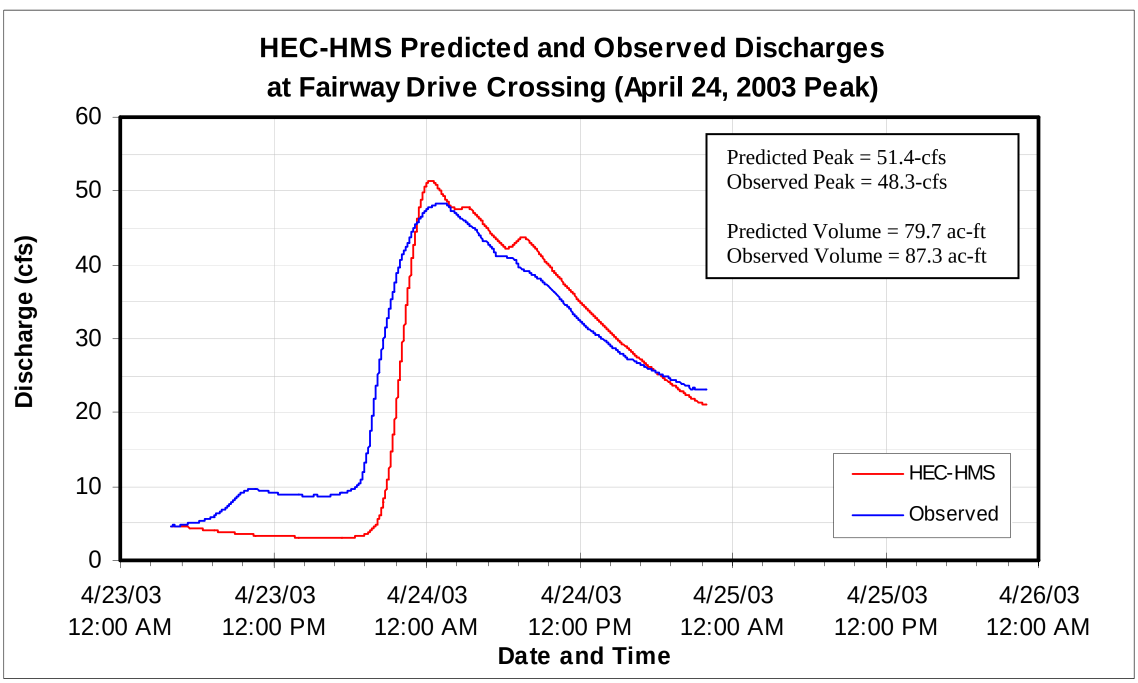

Figure 2.8 - Figure 2-2 - Observed and HEC-HMS predicted flows at Fairway Drive crossing during precipitation event beginning on April 24, 2003.

2.3 Predicted Flows for Design Storms

2.3.1 Design Precipitation Events

For the purpose of simulating design precipitation events within the final calibrated model, the 24 hour SCS Hypothetical Storm method was utilized in HEC-HMS. This method requires the 24 hour rainfall amount associated with a specific frequency. The method also requires the determination of a rainfall distribution. The SCS has defined four distributions within the United States based on storm intensity. The North Coast of California is considered to have a Type IA distribution (NRCS, 1986).

For the purpose of this project, recurrence rainfall events were obtained from data provided by the California Geological Survey (2003) for the recording station located at the Eureka National Weather office (Table 2.5).

Table 2.5 - 24 hour rainfall events of varying return periods for Eureka (1904-1999), developed by the California Department of Water Resources.

Return Period	24-hour Rainfall Event (inches)
2-year	2.67
5-year	3.59
10-year	4.17
25-year	4.88
50-year	5.38
100-year	5.86

The rainfall amounts were entered into HEC-HMS using the Type IA SCS Hypothetical Storm Distribution method. The resulting hydrographs from a 24-hour, 2-, 10- and 100-year recurrence rainfall events at Junctions 1, 2, 3 and the tidegates are presented below (Figures 2.9 through 2.12). Refer to Figure 2.3 for Junction locations. It is important to note that the predicted hydrograph at the tidegate assumes no backwater conditions and was not directly used as part of this study.

The peak flow and total outflow volume generated from each of the design storms are summarized in Table 2.6.

2.3.2 Use of Streamflow Hydrographs in Hydraulic Modeling

The hydrologic results predicted by HEC-HMS were directly incorporated into the hydraulic model for the project area. The hydraulic model encompassed the lowlands from the tidegate to the upper fairway crossing. Streamflow hydrographs at four different locations were provided as flow inputs to the hydraulic model (Figure 2.13). For hydraulic modeling purposes, flows from individual subbasins that drained directly into the project area were added to the closest of the four inflow locations (Martin 1 – 4). All four input locations were above the backwater effects of the tidegates. For a more accurate outflow hydrograph at the tidegates, refer to results from the hydraulic modeling effort.

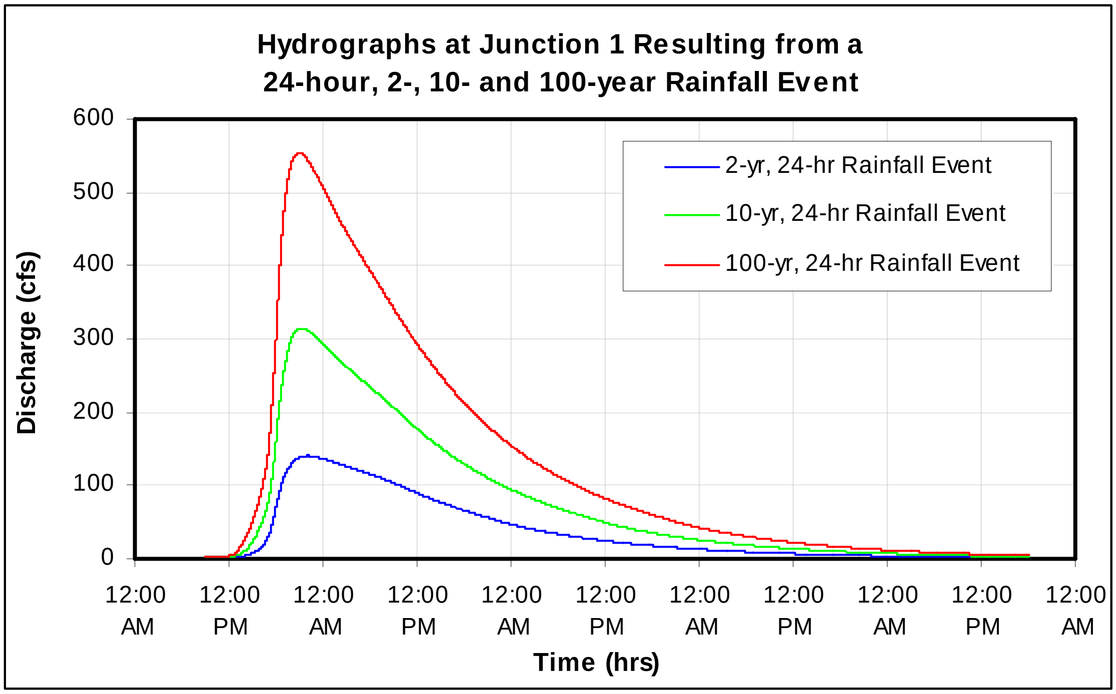

Figure 2.9 - HEC-HMS predicted hydrographs for Junction 1.

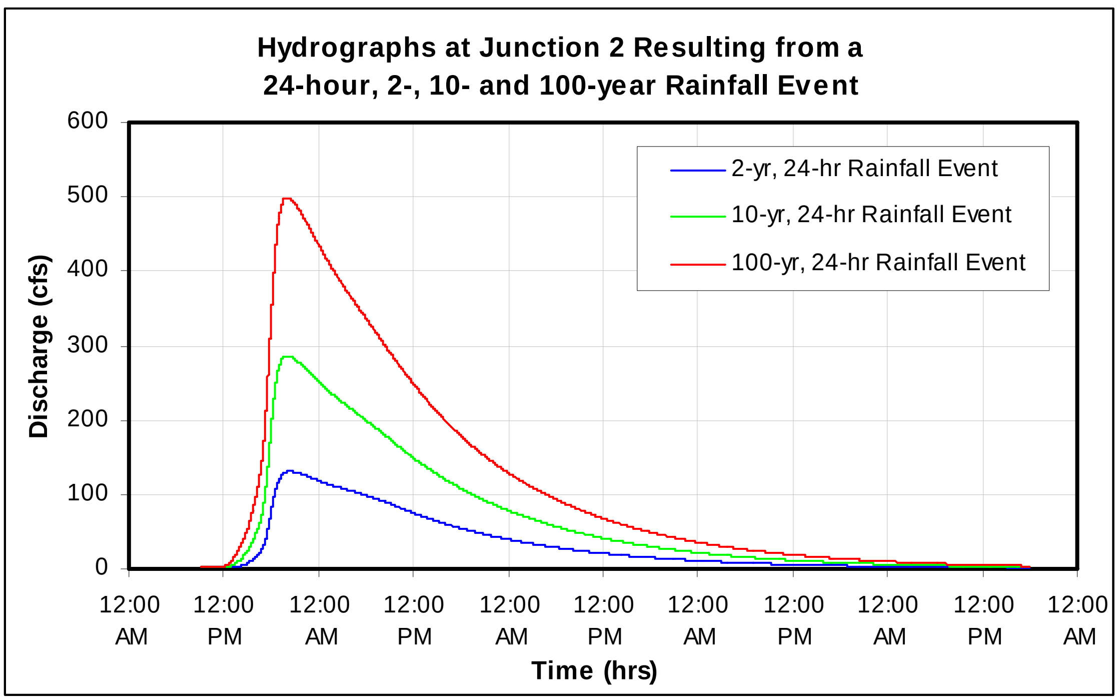

Figure 2.10 - Figure 2-3 - HEC-HMS predicted hydrographs for Junction 2.

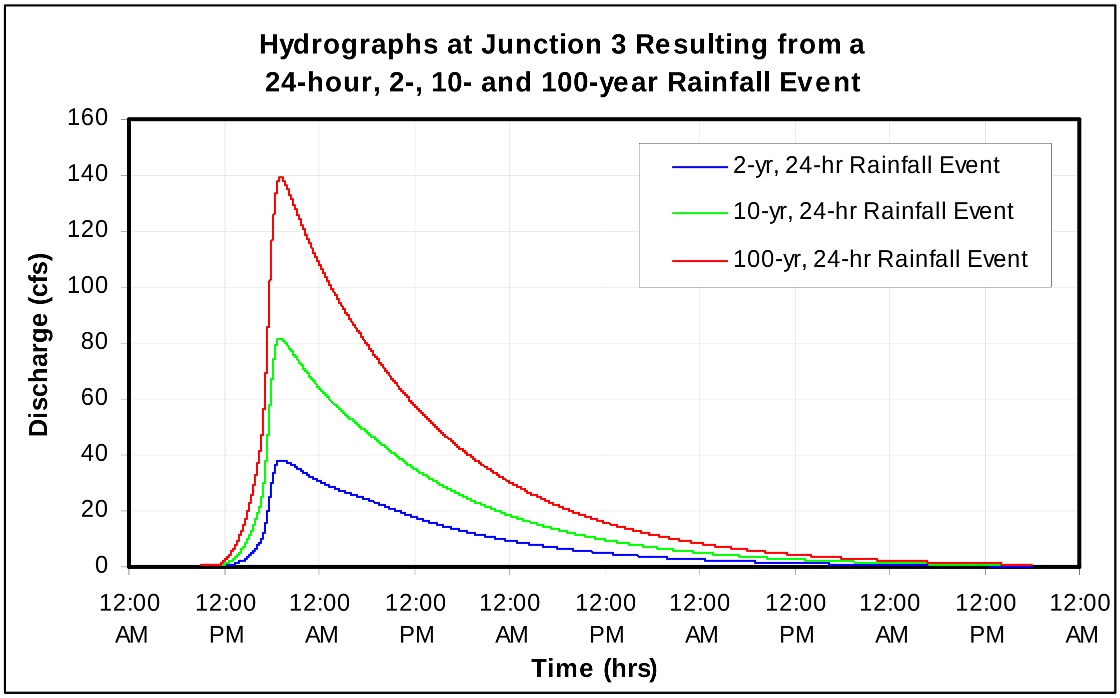

Figure 2.11 - HEC-HMS predicted hydrographs for Junction 3.

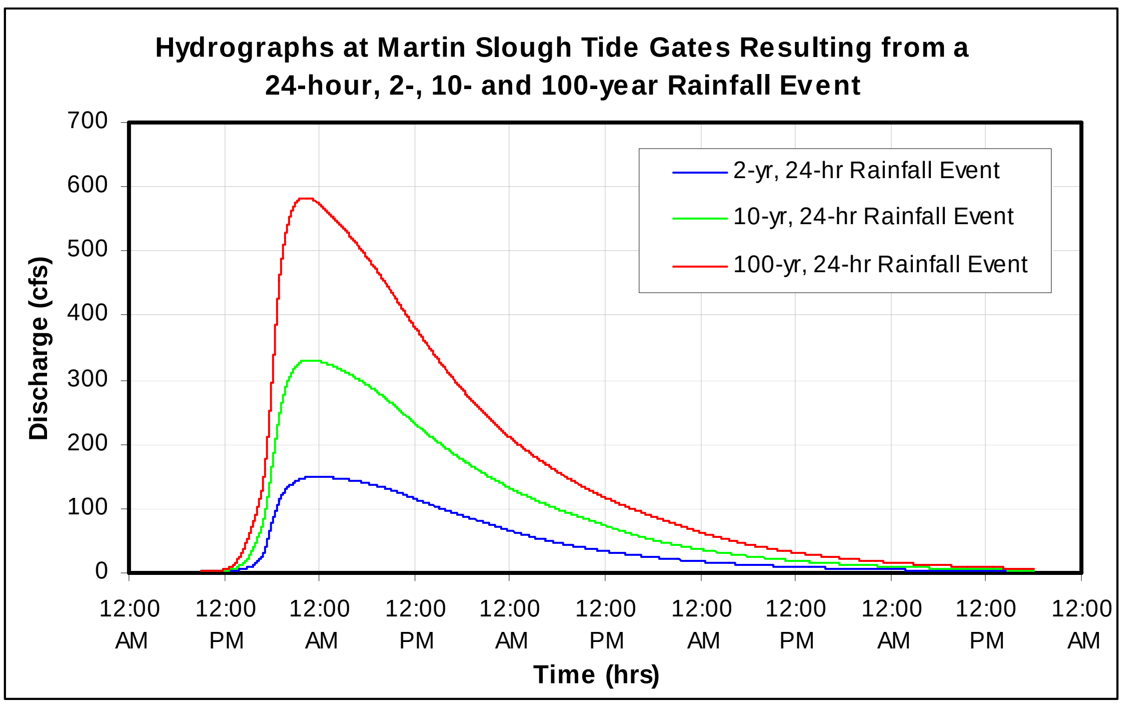

Figure 2.12 - HEC-HMS predicted hydrographs for Martin Slough tidegates, assuming no backwater conditions.

Figure 2.13 - Four locations of inflow hydrographs for hydraulic model.

Table 2.6 - HEC-HMS predicted peak discharge and total flow volume for each of the four locations.

Return Period of 24-hr Rainfall Event	Junction 1		Junction 2		Junction 3		Tidegate	
	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)
2-year	141	322	131	281	38	72	149	397
10-year	315	667	286	579	82	146	330	833
100-year	554	1,130	499	983	139	245	583	1,400

2.3.3 Comparison of Results to Earlier Hydrologic Study

A similar hydrologic study of the Martin Slough Watershed was conducted by Oscar Larson and Associates (OLA) in 1990 for the City of Eureka and Humboldt County Departments of Public Works. The study used the ACOE HEC-1 model, the precursor to HEC-HMS, to predict rainfall-runoff processes within the watershed (OLA, 1990). The earlier HEC-1 model produced peak flow estimates substantially greater than the HEC-HMS results for this project. For example, for the 10-year 24-hour design storm the predicted peak flow at the tidegates was 1,320 cfs while the HEC-HMS model predicted a peak of 330 cfs. However, the HEC-HMS model estimated outflow volumes that were nearly a third more than predicted with the earlier HEC-1 results. The discrepancy between results can be explained by the differences in methods used (Table 2.7 and 2.9). Most notable is that the previous study assumed AMC III conditions, which results in considerably higher peak flows. We considered it prudent to use AMC II curve numbers given that NRCS now recommends avoiding usage of AMC I and III (Ponce and Richard Hawkins, 1996.).

Table 2.7 - Attributes used in developing both the HEC-HMS model and OLA's HEC-1 model.

Model Attributes	HEC-1 Model (1990)	HEC-HMS Model (2005)
Number of Land Covers	3	6
Number of subbasins	7	44

Table 2.8 - Methods used in developing both the HEC-HMS model and OLA's HEC-1 model.

HEC-1 and HEC-HMS Processes	HEC-1 Model (1990)	HEC-HMS Model (2005)
Loss	AMC III SCS CN and Scaled Initial Abstraction of 0.02 to 0.08 inches	AMC II SCS CN and Scaled Initial Abstraction of 0.07 to 0.19 inches
Transform	SCS UH and Lag Times	Snyder UH and Lag Times
Baseflow	None	Scaled Constant w/ Recession
Reach Routing	Modified Puls	Muskingum and Kinematic
Precipitation	Type I Distribution 24-hour, 10-yr rainfall = 4.22 inches	Type IA Distribution 24-hour, 10-yr rainfall = 4.17 inches

Absent any streamflow data for Martin Slough, the model used in the previous study was uncalibrated. They used the standard SCS UH and Lag Time method, while we discovered during calibration that the Snyder method was more applicable. Also, the previous study used a Type I SCS Hypothetical Storm instead of the appropriate Type 1A. The Type I distribution assumes a higher intensity storm relative to a Type IA and is applicable for regions further south.

Even though the earlier study had a larger peak flow, the HEC-HMS model predicted substantially more outflow volume. Since this project is focused largely on reducing the duration of flooding within the lower portions of the watershed through improved drainage at the tidegates, it is most important to have accurate estimates of the volume of water that must be drained during one or more tidal cycles. Given that this HEC-HMS model was calibrated to existing streamflow data, it is reasonable to assume that it produces realistic estimates of flow hydrographs for the watershed, and is suitable for use as input into the hydraulic model of the project area.

2.3.4 Final Results for Full Build-Out Scenario

As part of this project we examined possible affects on future peak flows and runoff volumes associated with potential future land use changes within the watershed. The full build-out scenario and associated land use changes are described in section 2. For modeling purposes the only parameter changed in HMS was the subbasin area-weighted curve number, which is a function of the land use within the subbasin. All other model input remained the same as used for existing conditions. Table 2.9 summarizes the peak flow and total runoff volume predicted for the full build-out scenario. Comparing results from simulations using current conditions (Table 2.6), the full build-out scenario predicts at the tidegate a 62% increase in the peak flow and 54% increase in runoff volume associated with a 2-year 24-hour rainfall event. For the 10-year 24-hour rainfall event peak flows and volumes at the tidegate are predicted to increase 50% and 42%, respectively.

Table 2.9 - Full build out scenario - HEC-HMS predicted peak discharge and total flow volume for each of the four locations.

Return Period of 24-hr Rainfall Event	Junction 1		Junction 2		Junction 3		Tidegate	
	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)	Peak Flow (cfs)	Total Volume (ac-ft)
2-year	225	482	209	420	59	105	242	612
10-year	465	941	419	816	116	203	495	1,184
100-year	766	1,518	684	1,313	185	324	823	1,896

The peak flow and volume estimates associated with the full build-out scenario are conservative, and assumes new development will not have any storm water detention facilities.

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Hydrology Appendix A

HEC-HMS Model Input

Subbasin Parameters

Table A1 - Land cover type with associated areas for each subbasin, including totals for the entire Martin Slough Watershed.

Subbasin	Grassland (ft^2)	Dense Urban (ft^2)	Commercial (ft^2)	Timber (ft^2)	Sparse Urban (ft^2)	Reservoir (ft^2)	Total Area (ft^2)
1	900,505	2,467,445			1,650,397		5,018,347
2	1,420,302	2,389,567		1,961,824	343,068		6,114,761
3	789,245	1,130,306		913,459			2,833,010
4	432,871	1,725,274		813,721	414,895		3,386,761
5	1,366,279	1,682,182		374,577			3,423,038
6	171,374	112,596		361,953			645,923
7	171,374	112,596		361,953			645,923
8	311,117	1,798,430		465,851			2,575,398
9	124,882			207,580	172,111		504,573
10	218,209	4,826,038		1,590,477			6,634,724
11	109,125	163,313		621,189	42,546		936,173
12	320,569	844,710		1,110,346	311,873		2,587,498
13	545,104	465,600		763,666			1,774,370
14	65,893			34,496			100,389
15	28,215			122,551	122,481		273,247
16	186,222	1,227,088		212,023			1,625,333
17	294,386	2,266,408	291,069	1,490,842	408,846		4,751,551
18	300,874	2,132,545		1,176,916	194,553		3,804,888
19	210,019	2,421,916	288,051	1,499,031	50,940		4,469,957
20	122,536			78,253	40,557		241,346
21	108,582			254,300	130,905		493,787
22	316,988	3,633,856	121,434	3,263,289	68,688	46,570	7,450,825
23	192,799	43,509		345,962	58,006		640,276
24	190,225	1,366,445		1,330,439	57,107		2,944,216
25	346,969	2,976,655		2,871,352			6,194,976
26	15,246	134,604		170,523			320,373
27	467,025	2,758,601		3,755,087	799,533		7,780,246
28	104,856	760,676		2,666,427	371,887		3,903,846
29	22,013	310,979		1,089,956	43,643		1,466,591
30		887,854		1,265,406			2,153,260
31		3,915,821		8,141,888	277,260		12,334,969
32		66,812		12,946,930			13,013,742
33				438,395			438,395
34	424,902			2,211,660			2,636,562
35	90,998	325,165					416,163
36	118,053			409,557			527,610
37	64,482			246,598			311,080
38		339,124					339,124
39		959,138					959,138
40		81,534					81,534
41	848,458			9,785,116			10,633,574
42				8,086,082	240,207		8,326,289
43	323,400			3,593,116			3,916,516
44	5,582,304	447,426		7,632,823	208,988		13,871,541
Total (ft^2)	17,306,401	44,774,213	700,554	84,665,614	6,008,491	46,570	153,501,843
Total (acres)	397.3	1,027.9	16.1	1,943.7	137.9	1.1	3,523.9
Total (mi^2)	0.6208	1.6061	0.0251	3.0370	0.2155	0.0017	5.5061

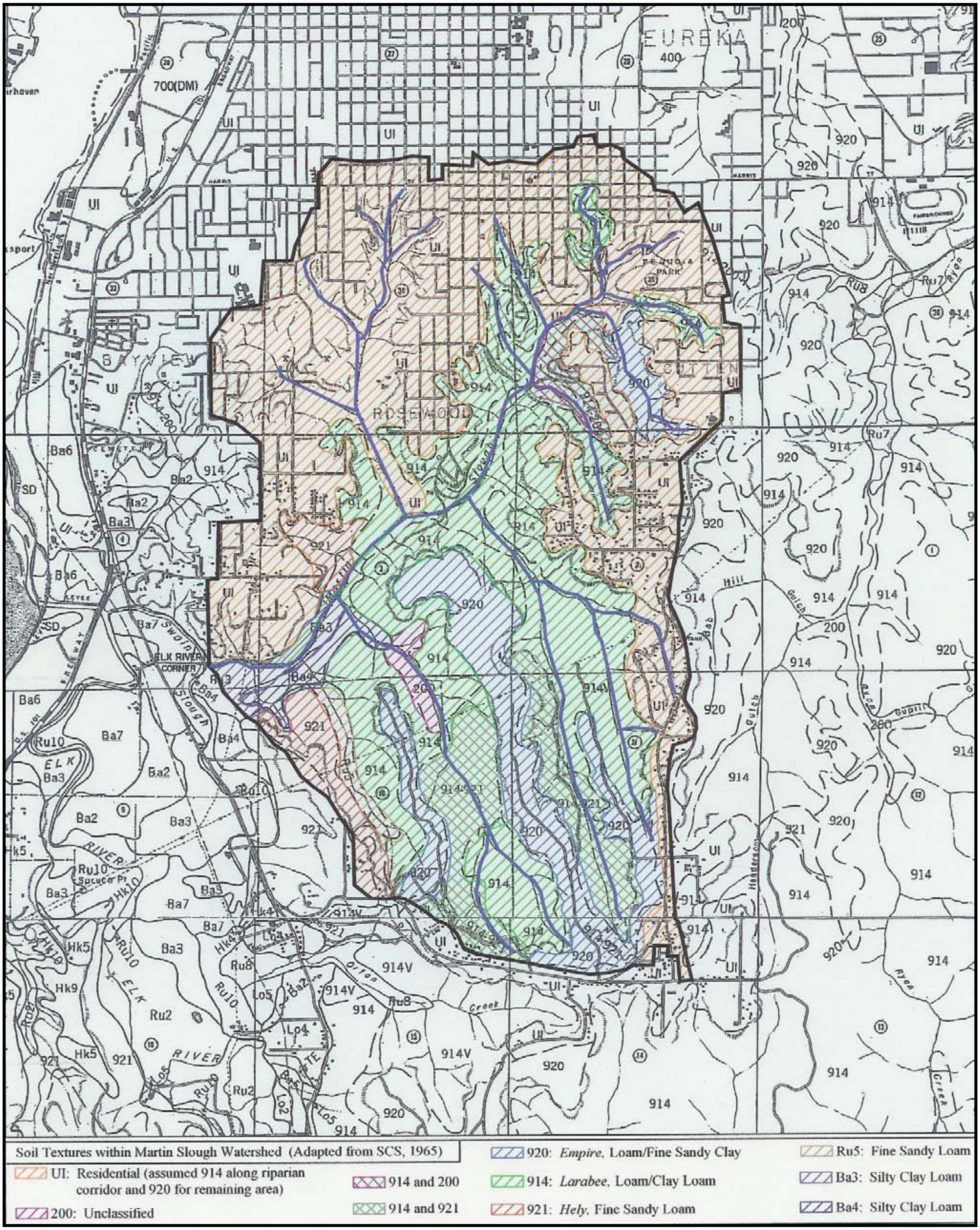


Figure A1 – Surface soil classifications within the Martin Slough Watershed (adapted from SCS, 1965 and 1975).

Table A2 – Aerial distribution land covers and corresponding curve numbers (CN) for each subbasin.

		Grassland		Dense Urban		Commercial		Timber		Sparse Urban		Reservoir			
Sub-basin	**Soils**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Total Area (ft^2)**	**Composite CN (AMC II)**
1	UI, Ba3	900,505	84	2,467,445	85					1,650,397	70			5,018,347	**80**
2	UI, 921, 914	1,420,302	69	2,389,567	85			1,961,824	30	343,068	70			6,114,761	**63**
3	UI, 914	789,245	69	1,130,306	85			913,459	55					2,833,010	**71**
4	UI	432,871	69	1,725,274	85			813,721	55	414,895	70			3,386,761	**74**
5	UI	1,366,279	69	1,682,182	85			374,577	55					3,423,038	**75**
6	UI	171,374	69	112,596	85			361,953	55					645,923	**64**
7	UI	171,374	69	112,596	85			361,953	55					645,923	**64**
8	UI	311,117	69	1,798,430	85			465,851	55					2,575,398	**78**
9	UI	124,882	69					207,580	55	172,111	70			504,573	**64**
10	UI	218,209	69	4,826,038	85			1,590,477	55					6,634,724	**77**
11	UI	109,125	69	163,313	85			621,189	55	42,546	70			936,173	**63**
12	UI	320,569	69	844,710	85			1,110,346	55	311,873	70			2,587,498	**68**
13	UI, 914	545,104	69	465,600	85			763,666	55					1,774,370	**67**
14	UI, 914	65,893	69					34,496	55					100,389	**64**
15	914	28,215	69					122,551	55	122,481	70			273,247	**63**
16	UI, 914	186,222	69	1,227,088	85			212,023	55					1,625,333	**79**
17	UI, 914	294,386	69	2,266,408	85	291,069	92	1,490,842	55	408,846	70			4,751,551	**74**
18	UI, 914	300,874	69	2,132,545	85			1,176,916	55	194,553	70			3,804,888	**74**
19	UI, 914	210,019	69	2,421,916	85	288,051	92	1,499,031	55	50,940	70			4,469,957	**74**
20	UI	122,536	69					78,253	55	40,557	70			241,346	**65**
21	UI	108,582	69					254,300	55	130,905	70			493,787	**62**
22	UI, 914	316,988	69	3,633,856	85	121,434	92	3,263,289	55	68,688	70	46,570	98	7,450,825	**71**
23	UI	192,799	69	43,509	85			345,962	55	58,006	70			640,276	**63**
24	UI, 920, 914	190,225	69	1,366,445	85			1,330,439	55	57,107	70			2,944,216	**70**
25	UI, 920, 914	346,969	69	2,976,655	85			2,871,352	55					6,194,976	**70**

Table A2 (continued) – Aerial distribution land covers and corresponding curve numbers (CN) for each subbasin.

		Grassland		Dense Urban		Commercial		Timber		Sparse Urban		Reservoir			
Sub-basin	**Soils**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Area (ft^2)**	**CN**	**Total Area (ft^2)**	**Composite CN (AMC II)**
26	914	15,246	69	134,604	85			170,523	55					320,373	**68**
27	UI, 914	467,025	69	2,758,601	85			3,755,087	55	799,533	70			7,780,246	**68**
28	UI, 914	104,856	69	760,676	85			2,666,427	55	371,887	70			3,903,846	**63**
29	UI, 914	22,013	69	310,979	85			1,089,956	55	43,643	70			1,466,591	**62**
30	UI, 914			887,854	85			1,265,406	55					2,153,260	**67**
31	UI, 920, 914			3,915,821	85			8,141,888	55	277,260	70			12,334,969	**65**
32	914-921, 920			66,812	85			12,946,930	50					13,013,742	**50**
33	914							438,395	55					438,395	**55**
34	920, 914	424,902	69					2,211,660	55					2,636,562	**57**
35	920 now (UI)	90,998	69	325,165	85									416,163	**82**
36	914	118,053	69					409,557	55					527,610	**58**
37	914	64,482	69					246,598	55					311,080	**58**
38	920 now (UI)			339,124	85									339,124	**85**
39	920 now (UI)			959,138	85									959,138	**85**
40	920 now (UI)			81,534	85									81,534	**85**
41	914-921, 920	848,458	69					9,785,116	47					10,633,574	**49**
42	914-921, 920							8,086,082	47	240,207	70			8,326,289	**48**
43	Ba, 920, 914-920	323,400	77					3,593,116	43					3,916,516	**45**
44	Ba, Ru5, 921, 914, 920	5,582,304	49	447,426	77			7,632,823	55	208,988	70			13,871,541	**54**

Table A3 – Composite percent impervious for each subbasin, used for scaling initial abstraction values between subbasins.

	Grassland		Dense Urban		Commercial		Timber		Sparse Urban		Reservoir			
Sub-basin	**Area (ft^2)**	**% Imp.**	**Area (ft^2)**	**% Imp.**	**Area (ft^2)**	**% Imp.**	**Area (ft^2)**	**% Imp.**	**Area (ft^2)**	**% Imp.**	**Area (ft^2)**	**% Imp.**	**Total Area (ft^2)**	**Composite % Imper.**
1	900,505	0	2,467,445	65		85		0	1,650,397	25		100	5,018,347	**40**
2	1,420,302	0	2,389,567	65		85	1,961,824	0	343,068	25		100	6,114,761	**27**
3	789,245	0	1,130,306	65		85	913,459	0		25		100	2,833,010	**26**
4	432,871	0	1,725,274	65		85	813,721	0	414,895	25		100	3,386,761	**36**
5	1,366,279	0	1,682,182	65		85	374,577	0		25		100	3,423,038	**32**
6	171,374	0	112,596	65		85	361,953	0		25		100	645,923	**11**
7	171,374	0	112,596	65		85	361,953	0		25		100	645,923	**11**
8	311,117	0	1,798,430	65		85	465,851	0		25		100	2,575,398	**45**
9	124,882	0		65		85	207,580	0	172,111	25		100	504,573	**9**
10	218,209	0	4,826,038	65		85	1,590,477	0		25		100	6,634,724	**47**
11	109,125	0	163,313	65		85	621,189	0	42,546	25		100	936,173	**12**
12	320,569	0	844,710	65		85	1,110,346	0	311,873	25		100	2,587,498	**24**
13	545,104	0	465,600	65		85	763,666	0		25		100	1,774,370	**17**
14	65,893	0		65		85	34,496	0		25		100	100,389	**0**
15	28,215	0		65		85	122,551	0	122,481	25		100	273,247	**11**
16	186,222	0	1,227,088	65		85	212,023	0		25		100	1,625,333	**49**
17	294,386	0	2,266,408	65	291,069	85	1,490,842	0	408,846	25		100	4,751,551	**38**
18	300,874	0	2,132,545	65		85	1,176,916	0	194,553	25		100	3,804,888	**38**
19	210,019	0	2,421,916	65	288,051	85	1,499,031	0	50,940	25		100	4,469,957	**41**
20	122,536	0		65		85	78,253	0	40,557	25		100	241,346	**4**
21	108,582	0		65		85	254,300	0	130,905	25		100	493,787	**7**
22	316,988	0	3,633,856	65	121,434	85	3,263,289	0	68,688	25	46,570	100	7,450,825	**34**
23	192,799	0	43,509	65		85	345,962	0	58,006	25		100	640,276	**7**
24	190,225	0	1,366,445	65		85	1,330,439	0	57,107	25		100	2,944,216	**31**
25	346,969	0	2,976,655	65		85	2,871,352	0		25		100	6,194,976	**31**

Table A3 (continued) – Composite percent impervious for each subbasin, used for scaling initial abstraction values between subbasins.

	Grassland		Dense Urban		Commercial		Timber		Sparse Urban		Reservoir			
Sub-basin	Area (ft^2)	% Imp.	Area (ft^2)	% Imp.	Area (ft^2)	% Imp.	Area (ft^2)	% Imp.	Area (ft^2)	% Imp.	Area (ft^2)	% Imp.	Total Area (ft^2)	Subbasin % Imper.
26	15,246	0	134,604	65		85	170,523	0		25		100	320,373	**27**
27	467,025	0	2,758,601	65		85	3,755,087	0	799,533	25		100	7,780,246	**26**
28	104,856	0	760,676	65		85	2,666,427	0	371,887	25		100	3,903,846	**15**
29	22,013	0	310,979	65		85	1,089,956	0	43,643	25		100	1,466,591	**15**
30		0	887,854	65		85	1,265,406	0		25		100	2,153,260	**27**
31		0	3,915,821	65		85	8,141,888	0	277,260	25		100	12,334,969	**21**
32		0	66,812	65		85	12,946,930	0		25		100	13,013,742	**0**
33		0		65		85	438,395	0		25		100	438,395	**0**
34	424,902	0		65		85	2,211,660	0		25		100	2,636,562	**0**
35	90,998	0	325,165	65		85		0		25		100	416,163	**51**
36	118,053	0		65		85	409,557	0		25		100	527,610	**0**
37	64,482	0		65		85	246,598	0		25		100	311,080	**0**
38		0	339,124	65		85		0		25		100	339,124	**65**
39		0	959,138	65		85		0		25		100	959,138	**65**
40		0	81,534	65		85		0		25		100	81,534	**65**
41	848,458	0		65		85	9,785,116	0		25		100	10,633,574	**0**
42		0		65		85	8,086,082	0	240,207	25		100	8,326,289	**1**
43	323,400	0		65		85	3,593,116	0		25		100	3,916,516	**0**
44	5,582,304	0	447,426	65		85	7,632,823	0	208,988	25		100	13,871,541	**2**

Table A4– Initial abstraction for each subbasin based on watershed average initial abstraction scaled by percent area impervious within the subbasin. Initial abstractions computed for each subbasin using the SCS method are also included for comparison.

Ave Initial Loss for entire watershed (in) =	0.15	(developed from calibration runs)		
Subbasin	**Composite Percent Impervious**	**Difference from 100**	**Scaled Initial Abstraction (in.)**	**Comparison SCS Initial Abstraction (in.)**
1	40	60	**0.1160**	0.50
2	27	73	**0.1420**	1.18
3	26	74	**0.1437**	0.82
4	36	64	**0.1238**	0.71
5	32	68	**0.1320**	0.65
6	11	89	**0.1720**	1.13
7	11	89	**0.1720**	1.13
8	45	55	**0.1059**	0.58
9	9	91	**0.1774**	1.15
10	47	53	**0.1023**	0.59
11	12	88	**0.1698**	1.20
12	24	76	**0.1470**	0.93
13	17	83	**0.1609**	0.98
14	0	100	**0.1940**	1.12
15	11	89	**0.1722**	1.17
16	49	51	**0.0988**	0.52
17	33	67	**0.1295**	0.71
18	38	62	**0.1208**	0.71
19	36	64	**0.1250**	0.69
20	4	96	**0.1858**	1.09
21	7	93	**0.1811**	1.22
22	33	67	**0.1308**	0.81
23	7	93	**0.1810**	1.19
24	31	69	**0.1345**	0.85
25	31	69	**0.1334**	0.85
26	27	73	**0.1410**	0.93
27	26	74	**0.1443**	0.94
28	15	85	**0.1648**	1.19
29	15	85	**0.1658**	1.22
30	27	73	**0.1420**	0.97
31	21	79	**0.1528**	1.08
32	0	100	**0.1933**	1.99
33	0	100	**0.1940**	1.64
34	0	100	**0.1940**	1.49
35	51	49	**0.0955**	0.45
36	0	100	**0.1940**	1.44
37	0	100	**0.1940**	1.45
38	65	35	**0.0679**	0.35
39	65	35	**0.0679**	0.35
40	65	35	**0.0679**	0.35
41	0	100	**0.1940**	2.10
42	1	99	**0.1926**	2.20
43	0	100	**0.1940**	2.41
44	2	98	**0.1892**	1.74
	Scaling Factor =	**3403**	**6.6000**	

Table A5– Parameters required in computing the SCS Lag Time for each subbasin.

Sub-basin	Composite CN (AMC II)	Length of Flow Path (ft)	Top of Subbasin Elev. (ft)	Bottom of Subbasin Elev. (ft)	Elevation Difference (ft)	Ave. Subbasin Slope (%)	SCS Lag Time for AMC II (hr)
1	80	5253	123	6	117	2.23	**0.81**
2	63	5428	122	6.2	115.8	2.13	**1.36**
3	71	4266	132	6.5	125.5	2.94	**0.77**
4	74	4197	129	6.8	122.2	2.91	**0.70**
5	75	2638	120	10	110	4.17	**0.39**
6	64	3237	81	6.8	74.2	2.29	**0.84**
7	64	1829	80	6.8	73.2	4.00	**0.40**
8	78	3678	116	9	107	2.91	**0.57**
9	64	2288	107	9	98	4.28	**0.47**
10	77	3817	146	22	124	3.25	**0.56**
11	63	2038	122	9	113	5.54	**0.39**
12	68	2871	128	6.5	121.5	4.23	**0.50**
13	67	3319	142	6.5	135.5	4.08	**0.59**
14	64	947	22	6.5	15.5	1.64	**0.37**
15	63	1079	108	7.8	100.2	9.29	**0.18**
16	79	3351	142	6.5	135.5	4.04	**0.43**
17	74	5196	123	8.2	114.8	2.21	**0.96**
18	74	4475	143	12	131	2.93	**0.74**
19	74	4642	148	18	130	2.80	**0.76**
20	65	1330	126	18	108	8.12	**0.22**
21	62	1250	68	27	41	3.28	**0.34**
22	71	3369	146	42	104	3.09	**0.62**
23	63	2628	166	27	139	5.29	**0.48**
24	70	4787	207	27	180	3.76	**0.76**
25	70	4864	209	27	182	3.74	**0.77**
26	68	1703	150	18	132	7.75	**0.24**
27	68	5552	217	12	205	3.69	**0.92**
28	63	3166	201	8.2	192.8	6.09	**0.52**
29	62	3846	209	8.2	200.8	5.22	**0.67**
30	67	2729	216	40	176	6.45	**0.40**
31	65	9153	400	64	336	3.67	**1.49**
32	50	9500	400	40	360	3.79	**2.19**
33	55	1839	154	40	114	6.20	**0.41**
34	57	4209	68	8.2	59.8	1.42	**1.56**
35	82	936	206	120	86	9.19	**0.09**
36	58	1430	176	7.8	168.2	11.76	**0.22**
37	58	885	110	6.5	103.5	11.69	**0.15**
38	85	1270	182	150	32	2.52	**0.21**
39	85	2253	206	70	136	6.04	**0.21**
40	85	386	182	164	18	4.66	**0.06**
41	49	7391	260	6.2	253.8	3.43	**1.95**
42	48	4092	400	150	250	6.11	**0.94**
43	45	6797	240	6.2	233.8	3.44	**2.00**
44	54	9287	245	6	239	2.57	**2.40**

Table A6– Parameters used for the Snyder method.

Subbasin	Length of Flow Path (ft)	Length Along Flow Path from Subbasin Centroid to Outlet (ft)	Non-physical Based Coefficient (C_t, unitless)	Snyder Lag Time (t_p, hrs)	Non-physical Based Coefficient (C_p, unitless)
1	5,253	4,060	2.5	2.31	0.45
2	5,428	2,714	2.5	2.06	0.45
3	4,266	2,279	2.5	1.82	0.45
4	4,197	2,720	2.5	1.91	0.45
5	2,638	1,319	2.5	1.34	0.45
6	3,237	1,619	2.5	1.51	0.45
7	1,829	1,044	2.5	1.12	0.45
8	3,678	2,212	2.5	1.73	0.45
9	2,288	1,511	2.5	1.34	0.45
10	3,817	1,909	2.5	1.67	0.45
11	2,038	1,200	2.5	1.20	0.45
12	2,871	1,436	2.5	1.41	0.45
13	3,319	2,069	2.5	1.64	0.45
14	947	640	2.5	0.79	0.45
15	1,079	664	2.5	0.83	0.45
16	3,351	2,010	2.5	1.63	0.45
17	5,196	3,143	2.5	2.13	0.45
18	4,475	2,210	2.5	1.83	0.45
19	4,642	2,359	2.5	1.89	0.45
20	1,330	665	2.5	0.89	0.45
21	1,250	625	2.5	0.86	0.45
22	3,369	826	2.5	1.25	0.45
23	2,628	1,314	2.5	1.34	0.45
24	4,787	2,369	2.5	1.91	0.45
25	4,864	3,220	2.5	2.10	0.45
26	1,703	852	2.5	1.03	0.45
27	5,552	2,800	2.5	2.10	0.45
28	3,166	1,583	2.5	1.49	0.45
29	3,846	2,217	2.5	1.75	0.45
30	2,729	1,365	2.5	1.37	0.45
31	9,153	3,117	2.5	2.52	0.45
32	9,500	3,650	2.5	2.67	0.45
33	1,839	920	2.5	1.08	0.45
34	4,209	2,105	2.5	1.77	0.45
35	936	500	2.5	0.73	0.45
36	1,430	680	2.5	0.91	0.45
37	885	442	2.5	0.70	0.45
38	1,270	635	2.5	0.86	0.45
39	2,253	900	2.5	1.14	0.45
40	386	175	2.5	0.41	0.45
41	7,391	3,528	2.5	2.45	0.45
42	4,092	2,046	2.5	1.74	0.45
43	6,797	3,398	2.5	2.36	0.45
44	9,287	4,900	2.5	2.90	0.45

Table A7– Extrapolated baseflow for each subbasin, assuming 2-cfs at the upper Fairway Drive crossing.

Assumed Baseflow at Fairway Drive Crossing (gauge) =	2.0	cfs
Area above Fairway Drive Crossing (gauge) =	76,595,950	ft^2
Including subbasins 15,17-36 and 38 (top of REACH C)	2.748	mi^2

Martin Slough Total Watershed Area (above tidegates) =	153,501,843	ft^2
(Including All Subbasins)	5.506	mi^2

Extrapolated Baseflow At Tidegates =	4.008	cfs

Subbasin	Subbasin Area (ft^2)	Percent of Watershed Above Fairway Drive Crossing	Scaled Baseflow (cfs)	Recession Constant	Threshold Flow (ratio to peak)
1	5,018,347	3.27	0.1310	0.275	1.0
2	6,114,761	3.98	0.1597	0.275	1.0
3	2,833,010	1.85	0.0740	0.275	1.0
4	3,386,761	2.21	0.0884	0.275	1.0
5	3,423,038	2.23	0.0894	0.275	1.0
6	645,923	0.42	0.0169	0.275	1.0
7	645,923	0.42	0.0169	0.275	1.0
8	2,575,398	1.68	0.0672	0.275	1.0
9	504,573	0.33	0.0132	0.275	1.0
10	6,634,724	4.32	0.1732	0.275	1.0
11	936,173	0.61	0.0244	0.275	1.0
12	2,587,498	1.69	0.0676	0.275	1.0
13	1,774,370	1.16	0.0463	0.275	1.0
14	100,389	0.07	0.0026	0.275	1.0
15	273,247	0.18	0.0071	0.275	1.0
16	1,625,333	1.06	0.0424	0.275	1.0
17	4,751,551	3.10	0.1241	0.275	1.0
18	3,804,888	2.48	0.0993	0.275	1.0
19	4,469,957	2.91	0.1167	0.275	1.0
20	241,346	0.16	0.0063	0.275	1.0
21	493,787	0.32	0.0129	0.275	1.0
22	7,450,825	4.85	0.1945	0.275	1.0
23	640,276	0.42	0.0167	0.275	1.0
24	2,944,216	1.92	0.0769	0.275	1.0
25	6,194,976	4.04	0.1618	0.275	1.0
26	320,373	0.21	0.0084	0.275	1.0
27	7,780,246	5.07	0.2031	0.275	1.0
28	3,903,846	2.54	0.1019	0.275	1.0
29	1,466,591	0.96	0.0383	0.275	1.0
30	2,153,260	1.40	0.0562	0.275	1.0
31	12,334,969	8.04	0.3221	0.275	1.0
32	13,013,742	8.48	0.3398	0.275	1.0
33	438,395	0.29	0.0114	0.275	1.0
34	2,636,562	1.72	0.0688	0.275	1.0
35	416,163	0.27	0.0109	0.275	1.0
36	527,610	0.34	0.0138	0.275	1.0
37	311,080	0.20	0.0081	0.275	1.0
38	339,124	0.22	0.0089	0.275	1.0
39	959,138	0.62	0.0250	0.275	1.0
40	81,534	0.05	0.0021	0.275	1.0
41	10,633,574	6.93	0.2777	0.275	1.0
42	8,326,289	5.42	0.2174	0.275	1.0

43	3,916,516	2.55	0.1023	0.275	1.0
44	13,871,541	9.04	0.3622	0.275	1.0
	153,501,843	**100**	**4.008**		

Hydrology Appendix B

HEC-HMS Model Input

Channel Reach Parameters

Reach:	A		Cross-section Coordinates		
Measurement Location:	Dowstream from confluence with reach O			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	10
Reach Length (ft) =	3920.2		2	200	7.5
Reach Top Elev. (ft) =	6.2	(top of bank elev.)	3	500	6.1
Reach Bottom Elev. (ft) =	6.0	(top of bank elev.)	4	502.5	1.1
Energy Slope (ft/ft) =	0.000051		5	517.5	1.1
Manning's N Value			6	520	6.1
Left Overbank	Channel	Right Overbank	7	600	7.5
0.025	0.035	0.025	8	700	10

Reach:	B		Cross-section Coordinates		
Measurement Location:	Approx. 50' south of Fairway Dr. on golf course			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	10
Reach Length (ft) =	2203.5		2	85	8
Reach Top Elev. (ft) =	6.5	(top of bank elev.)	3	175	6.35
Reach Bottom Elev. (ft) =	6.2	(top of bank elev.)	4	180	1.35
Energy Slope (ft/ft) =	0.000136		5	185	1.35
Manning's N Value			6	190	6.35
Left Overbank	Channel	Right Overbank	7	440	8
0.025	0.035	0.025	8	450	10

Reach:	C		Cross-section Coordinates		
Measurement Location:	Approx. 200' downstream from Fairway Dr. new box culverts			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	10
Reach Length (ft) =	866.1		2	25	9
Reach Top Elev. (ft) =	7.8	(top of bank elev.)	3	50	7.65
Reach Bottom Elev. (ft) =	6.5	(top of bank elev.)	4	52.5	2.65
Energy Slope (ft/ft) =	0.001501		5	62.5	2.65
Manning's N Value			6	65	7.65
Left Overbank	Channel	Right Overbank	7	115	8
0.025	0.035	0.025	8	160	10

Reach:	D		Cross-section Coordinates		
Measurement Location:	Approx. 250' upstream from Fairway Dr. new box culverts			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	14
Reach Length (ft) =	467.8		2	6	12
Reach Top Elev. (ft) =	8.2	(top of bank elev.)	3	18	8
Reach Bottom Elev. (ft) =	7.8	(top of bank elev.)	4	28	0.1
Energy Slope (ft/ft) =	0.000855		5	33	0.1
Manning's N Value			6	43	8
Left Overbank	Channel	Right Overbank	7	45	9
0.035	0.04	0.08	8	50	14

Reach:	E		Cross-section Coordinates		
Measurement Location:	Approx. 100' upstream from junction of reaches D and M			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	20
Reach Length (ft) =	2738.8		2	150	12
Reach Top Elev. (ft) =	12.0	(top of bank elev.)	3	210	10.1
Reach Bottom Elev. (ft) =	8.2	(top of bank elev.)	4	220	0.1
Energy Slope (ft/ft) =	0.001387		5	225	0.1
Manning's N Value			6	235	10.1
Left Overbank	Channel	Right Overbank	7	335	12
0.035	0.04	0.035	8	375	20

Reach:	**F**		**Cross-section Coordinates**		
Measurement Location:	Approx. 1000' upstream from Campton Rd.			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	20
Reach Length (ft) =	1369.3		2	25	18
Reach Top Elev. (ft) =	18.0	(top of bank elev.)	3	195	15
Reach Bottom Elev. (ft) =	12.0	(top of bank elev.)	4	200	10
Energy Slope (ft/ft) =	0.004382		5	205	10
Manning's N Value			6	210	15
Left Overbank	Channel	Right Overbank	7	350	18
0.035	0.04	0.035	8	375	20

Reach:	**G**		**Cross-section Coordinates**		
Measurement Location:	Hiked in from north end of Brogan St.			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	30
Reach Length (ft) =	794.2		2	30	24
Reach Top Elev. (ft) =	27.0	(top of bank elev.)	3	50	22.5
Reach Bottom Elev. (ft) =	18.0	(top of bank elev.)	4	51	17.5
Energy Slope (ft/ft) =	0.011332		5	54	17.5
Manning's N Value			6	55	22.5
Left Overbank	Channel	Right Overbank	7	95	24
0.05	0.05	0.04	8	400	30

Reach:	**H**		**Cross-section Coordinates**		
Measurement Location:	At O st. crossing above (3) 24" dia cmp's			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	44
Reach Length (ft) =	1332.0		2	20	40
Reach Top Elev. (ft) =	42.0	(top of bank elev.)	3	125	34.5
Reach Bottom Elev. (ft) =	27.0	(top of bank elev.)	4	128	30.5
Energy Slope (ft/ft) =	0.011261		5	131	30.5
Manning's N Value			6	135	34.5
Left Overbank	Channel	Right Overbank	7	230	40
0.04	0.05	0.04	8	250	44

Reach:	**I**		**Cross-section Coordinates**		
Measurement Location:	Approx. 1000' above junction with reaches B and C			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	10
Reach Length (ft) =	2676.1		2	20	8
Reach Top Elev. (ft) =	6.8	(top of bank elev.)	3	155	6.65
Reach Bottom Elev. (ft) =	6.5	(top of bank elev.)	4	157.5	1.65
Energy Slope (ft/ft) =	0.000112		5	167.5	1.65
Manning's N Value			6	170	6.65
Left Overbank	Channel	Right Overbank	7	380	8
0.025	0.035	0.025	8	420	10

Reach:	**J**		**Cross-section Coordinates**		
Measurement Location:	NO ACCESS - Assumed average of reaches I and K			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	12
Reach Length (ft) =	1435.4		2	55	10
Reach Top Elev. (ft) =	9.0	(top of bank elev.)	3	150	8.5
Reach Bottom Elev. (ft) =	6.5	(top of bank elev.)	4	151	5.5
Energy Slope (ft/ft) =	0.001742		5	158	5.5
Manning's N Value			6	160	8.5
Left Overbank	Channel	Right Overbank	7	230	10
0.035	0.04	0.035	8	285	12

Reach:	K		Cross-section Coordinates		
Measurement Location:	450' south of Lowell St. and 50' downstream of 4' dia. con. pipe			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	30
Reach Length (ft) =	1568.4		2	70	22
Reach Top Elev. (ft) =	22.0	(top of bank elev.)	3	175	15.5
Reach Bottom Elev. (ft) =	9.0	(top of bank elev.)	4	176	14
Energy Slope (ft/ft) =	0.008289		5	180	14
Manning's N Value			6	181	15.5
Left Overbank	Channel	Right Overbank	7	270	22
0.06	0.04	0.06	8	300	30

Reach:	L		Cross-section Coordinates		
Measurement Location:	At Union St. crossing			Station (ft)	Elevation (ft)
HMS Routing Method:	Musk. Cunge 8-pt.		1	0	20
Reach Length (ft) =	1778.0		2	35	10
Reach Top Elev. (ft) =	10.0	(top of bank elev.)	3	125	8
Reach Bottom Elev. (ft) =	6.5	(top of bank elev.)	4	125.5	6
Energy Slope (ft/ft) =	0.001969		5	130.5	6
Manning's N Value			6	131	8
Left Overbank	Channel	Right Overbank	7	225	10
0.035	0.04	0.035	8	250	20

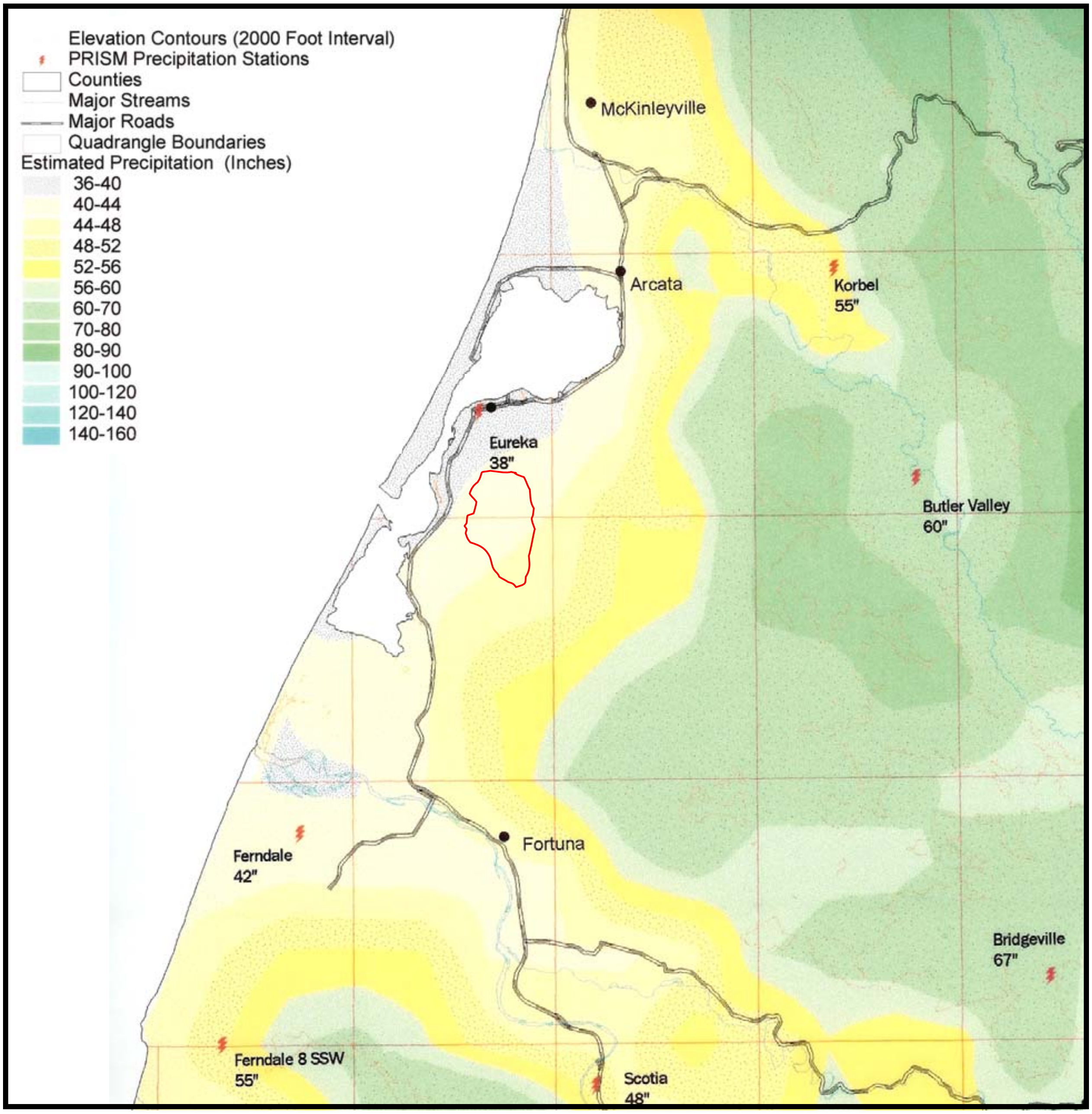
Reach:	M
Measurement Location:	Approx. 50' above junction with reaches D abd E (no active floodplain)
HMS Routing Method:	Kinematic Wave
Reach Length (ft) =	2383.8
Reach Top Elev. (ft) =	40.0
Reach Bottom Elev. (ft) =	8.2
Energy Slope (ft/ft) =	0.013340
Cross Section Shape =	Trapezpoid
Bottom Width (ft) =	3
Side Slope (xH:1V) =	0.5
Manning's n =	0.045

Reach:	N
Measurement Location:	Hiked in on skid road from east end of Lundblade Dr. (no active floodplain)
HMS Routing Method:	Kinematic Wave
Reach Length (ft) =	1537.4
Reach Top Elev. (ft) =	64.0
Reach Bottom Elev. (ft) =	40.0
Energy Slope (ft/ft) =	0.015611
Cross Section Shape =	Trapezpoid
Bottom Width (ft) =	3
Side Slope (xH:1V) =	0.5
Manning's n =	0.045

Reach:	O
Measurement Location:	At 11th Tee on golf course
HMS Routing Method:	Kinematic Wave
Reach Length (ft) =	6555.9
Reach Top Elev. (ft) =	150.0
Reach Bottom Elev. (ft) =	6.2
Energy Slope (ft/ft) =	0.021934
Cross Section Shape =	Trapezpoid
Bottom Width (ft) =	4
Side Slope (xH:1V) =	0.375
Manning's n =	0.045

Hydrology Appendix C

Isopluvial Map of Mean Annual Precipitation
In the Humboldt Bay Region

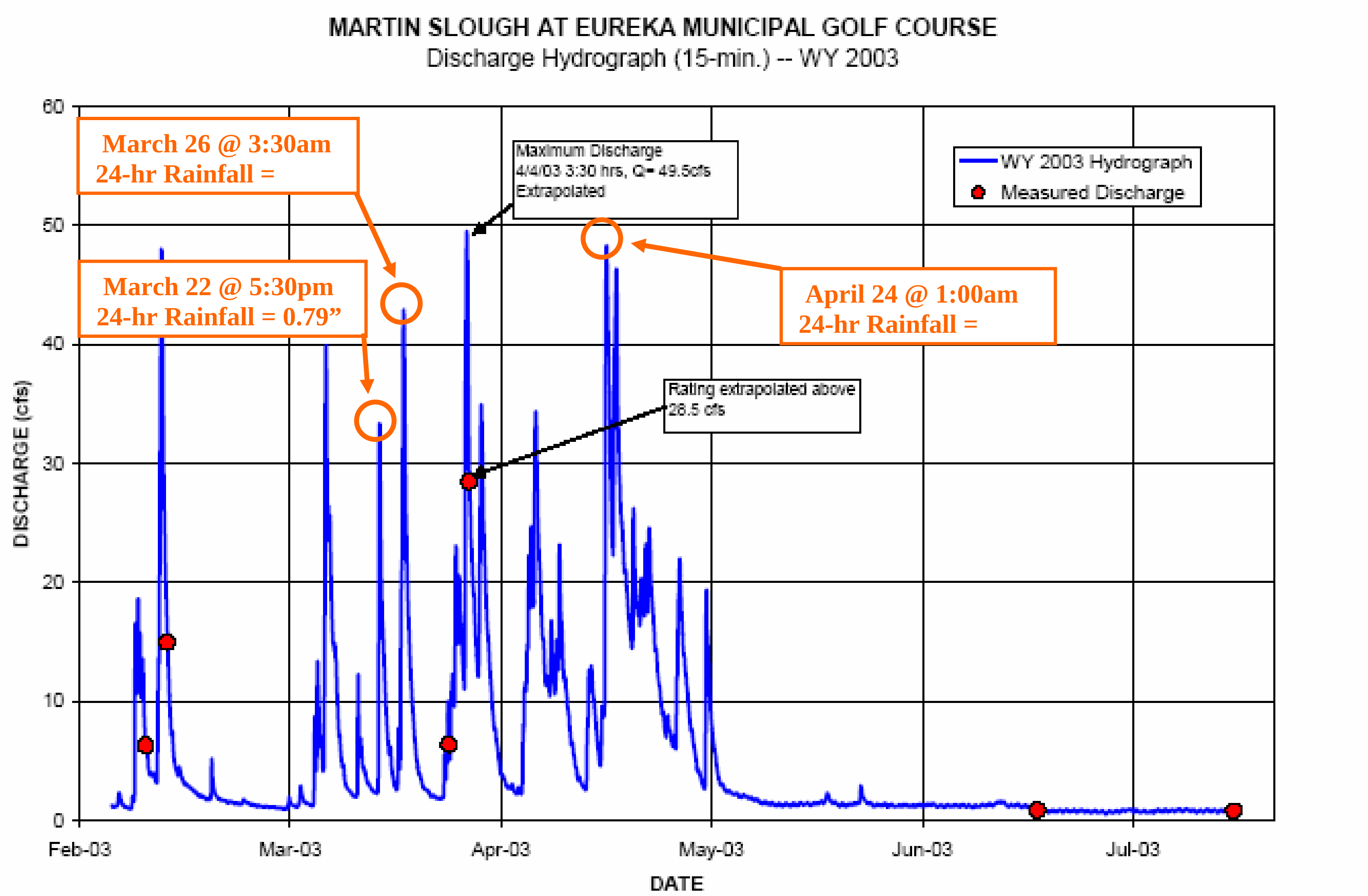


Isopluvial map for Mean Annual Precipitation developed from National Weather Services (Cooperation Service) by Oregon State University using PRISM, and mapped by NRCS February 2001, Arcata.

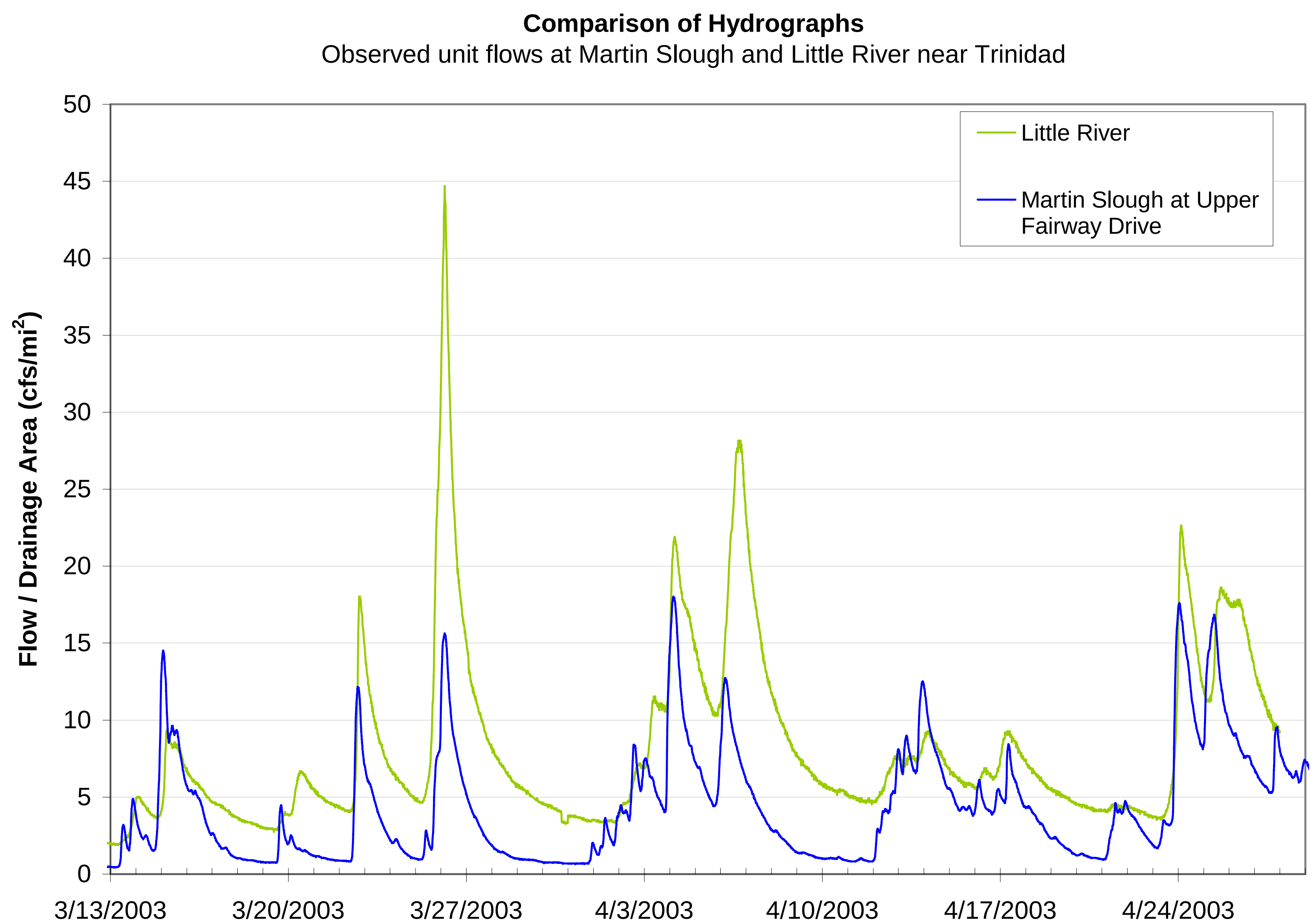
Hydrology Appendix D

Recorded stream flow at upper Fairway Drive crossing during Water Year 2003, including the peak flow events used for model calibration and validation

Flow Events Used to Calibrate HEC-HMS Model – All Corresponding Rainfall Events are Considered AMCII



PROJECT: MARTIN SLOUGH HYDROLOGIC MONITORING STUDY – WY 2003	GMA GRAHAM MATTHEWS & ASSOCIATES Hydrology • Geomorphology • Stream Restoration P.O. Box 1516 Weaverville, CA 96093-1516 (530) 623-5327 ph (530) 623-5328 fax	FIGURE 3



Comparing shape of hydrographs observed at Martin Slough (DA = 2.75 mi^2) and Little River (DA = 40.5 mi^2) [USGS Station No. 11481200]. Even though Little River's unit area peaks were higher than Martin Slough, they both have similar shaped recession limbs.

Hydrology Appendix E

Parameters Used in Final HEC-HMS Model

Parameters used for final HEC-HMS model simulations.

Sub-basin	Subbasin Area (mi^2)	Composite CN (AMC II)	Scaled Initial Loss (in.)	Non-physical Based Coefficient (Ct, unitless)	Snyder Lag Time (tp, hrs)	Non-physical Based Coefficient (Cp, unitless)	Scaled Baseflow with example of 2.73-cfs at Fairway Dr. (cfs)	Baseflow Recession Constant	Baseflow Recession Threshold Flow (ratio to peak)
1	0.1800	80	0.1160	2.5	2.31	0.45	0.1789	0.275	1
2	0.2193	63	0.1420	2.5	2.06	0.45	0.2179	0.275	1
3	0.1016	71	0.1437	2.5	1.82	0.45	0.1010	0.275	1
4	0.1215	74	0.1238	2.5	1.91	0.45	0.1207	0.275	1
5	0.1228	75	0.1320	2.5	1.34	0.45	0.1220	0.275	1
6	0.0232	64	0.1720	2.5	1.51	0.45	0.0230	0.275	1
7	0.0232	64	0.1720	2.5	1.12	0.45	0.0230	0.275	1
8	0.0924	78	0.1059	2.5	1.73	0.45	0.0918	0.275	1
9	0.0181	64	0.1774	2.5	1.34	0.45	0.0180	0.275	1
10	0.2380	77	0.1023	2.5	1.67	0.45	0.2365	0.275	1
11	0.0336	63	0.1698	2.5	1.20	0.45	0.0334	0.275	1
12	0.0928	68	0.1470	2.5	1.41	0.45	0.0922	0.275	1
13	0.0636	67	0.1609	2.5	1.64	0.45	0.0632	0.275	1
14	0.0036	64	0.1940	2.5	0.79	0.45	0.0036	0.275	1
15	0.0098	63	0.1722	2.5	0.83	0.45	0.0097	0.275	1
16	0.0583	79	0.0988	2.5	1.63	0.45	0.0579	0.275	1
17	0.1704	74	0.1295	2.5	2.13	0.45	0.1694	0.275	1
18	0.1365	74	0.1208	2.5	1.83	0.45	0.1356	0.275	1
19	0.1603	74	0.1250	2.5	1.89	0.45	0.1593	0.275	1
20	0.0087	65	0.1858	2.5	0.89	0.45	0.0086	0.275	1
21	0.0177	62	0.1811	2.5	0.86	0.45	0.0176	0.275	1
22	0.2673	71	0.1308	2.5	1.25	0.45	0.2656	0.275	1
23	0.0230	63	0.1810	2.5	1.34	0.45	0.0228	0.275	1
24	0.1056	70	0.1345	2.5	1.91	0.45	0.1049	0.275	1
25	0.2222	70	0.1334	2.5	2.10	0.45	0.2208	0.275	1
26	0.0115	68	0.1410	2.5	1.03	0.45	0.0114	0.275	1
27	0.2791	68	0.1443	2.5	2.10	0.45	0.2773	0.275	1
28	0.1400	63	0.1648	2.5	1.49	0.45	0.1391	0.275	1
29	0.0526	62	0.1658	2.5	1.75	0.45	0.0523	0.275	1
30	0.0772	67	0.1420	2.5	1.37	0.45	0.0767	0.275	1
31	0.4425	65	0.1528	2.5	2.52	0.45	0.4396	0.275	1
32	0.4668	50	0.1933	2.5	2.67	0.45	0.4638	0.275	1
33	0.0157	55	0.1940	2.5	1.08	0.45	0.0156	0.275	1
34	0.0946	57	0.1940	2.5	1.77	0.45	0.0940	0.275	1
35	0.0149	82	0.0955	2.5	0.73	0.45	0.0148	0.275	1
36	0.0189	58	0.1940	2.5	0.91	0.45	0.0188	0.275	1
37	0.0112	58	0.1940	2.5	0.70	0.45	0.0111	0.275	1
38	0.0122	85	0.0679	2.5	0.86	0.45	0.0121	0.275	1
39	0.0344	85	0.0679	2.5	1.14	0.45	0.0342	0.275	1
40	0.0029	85	0.0679	2.5	0.41	0.45	0.0029	0.275	1
41	0.3814	49	0.1940	2.5	2.45	0.45	0.3790	0.275	1
42	0.2987	48	0.1926	2.5	1.74	0.45	0.2968	0.275	1
43	0.1405	45	0.1940	2.5	2.36	0.45	0.1396	0.275	1
44	0.4976	54	0.1892	2.5	2.90	0.45	0.4944	0.275	1

Appendix B

Hydraulic Report

Hydraulic Analysis for Martin Slough

Prepared by:

Adele Milletello, PhD, Coastal Analysis LLC

With assistance from:

Michael Love, Michael Love & Associates
Steven Allen, Winzler & Kelly

July 18, 2005

HYDRAULIC ANALYSIS FOR
MARTIN SLOUGH

HYDRAULIC ANALYSIS

A computational model that calculates water level and current velocity was developed to evaluate and compare project alternatives. This section provides the goals of the modeling effort together with descriptions of the model and development for the Martin Slough site.

Goals and Description of Model

Goals of the hydraulic modeling are to evaluate and compare project alternatives in terms of inundation levels, inundation duration, and sediment transport for 2-yr and 10-yr flood events.

Hydraulic modeling was conducted with the two-dimensional finite-element model ADCIRC (Luettich et al 1992). This model was selected for the analysis because of its great flexibility in representation of bathymetric and topographic features, robust wetting and drying capabilities, representation of discharge and stage inputs, and proven performance for overland flooding calculations. ADCIRC calculates water-surface elevation and two horizontal components of current velocity on a finite-element mesh. This type of mesh allows for great detail to be specified where needed, such as the streams and ditches in the Martin Slough study area, and for coarser resolution in regions where detailed calculations are not needed, such as the higher-elevation areas of the subject study area.

Model Development and Approach

Development of the ADCIRC model for Martin Slough required topographic information to represent the stream, ditches, ponds, and upland areas. Two sources of topographic information were implemented in the model. A digital terrain model containing 2-ft contours, from the City of Eureka, provided wide-area topographic information. A stream and bank survey conducted for the present study provided detailed cross-sectional elevations at selected locations in the study area. Topographic information from both of these sources was combined and applied to generate the computational mesh for the existing condition. All action alternatives involved modification of the existing condition mesh.

Topographic data were provided with the vertical datum being NAVD 88 and horizontal coordinates of California State Plane Zone 1 with all units in feet. For application within ADCIRC, these data were converted to the vertical datum of Mean Sea Level at North Spit Coast Guard Station in units of meters and geographic horizontal coordinates (longitude and latitude) in units of decimal degrees. All calculations and grids shown herein have been converted back to NAVD 88 (ft).

The modeling approach taken in this study was to conduct simulations for the various alternatives in which the model results could be directly compared in terms of inundation area and inundation duration. Inundation events were specified as tributary input for 2-yr and 10-yr design storms. The watershed model was applied to calculate tributary discharges from the four primary drainages entering the Martin Slough study area. These discharges were then provided to the hydraulic model as upstream input.

Downstream input varied among the alternatives according to the specific configuration. Input was specified as water-surface elevation, and if tidegates were present, discharges combined with water-surface elevation. For simulations representing tidegates, the discharges and water-surface elevations were calculated by a tidegate model.

The computational mesh for the No-Action Alternative is shown in Figure 1. This mesh is comprised of 6,586 nodes and 12,624 elements. The downstream boundary, located at the confluence of Martin Slough and Swain Slough represents the tidegate configuration presently in place.

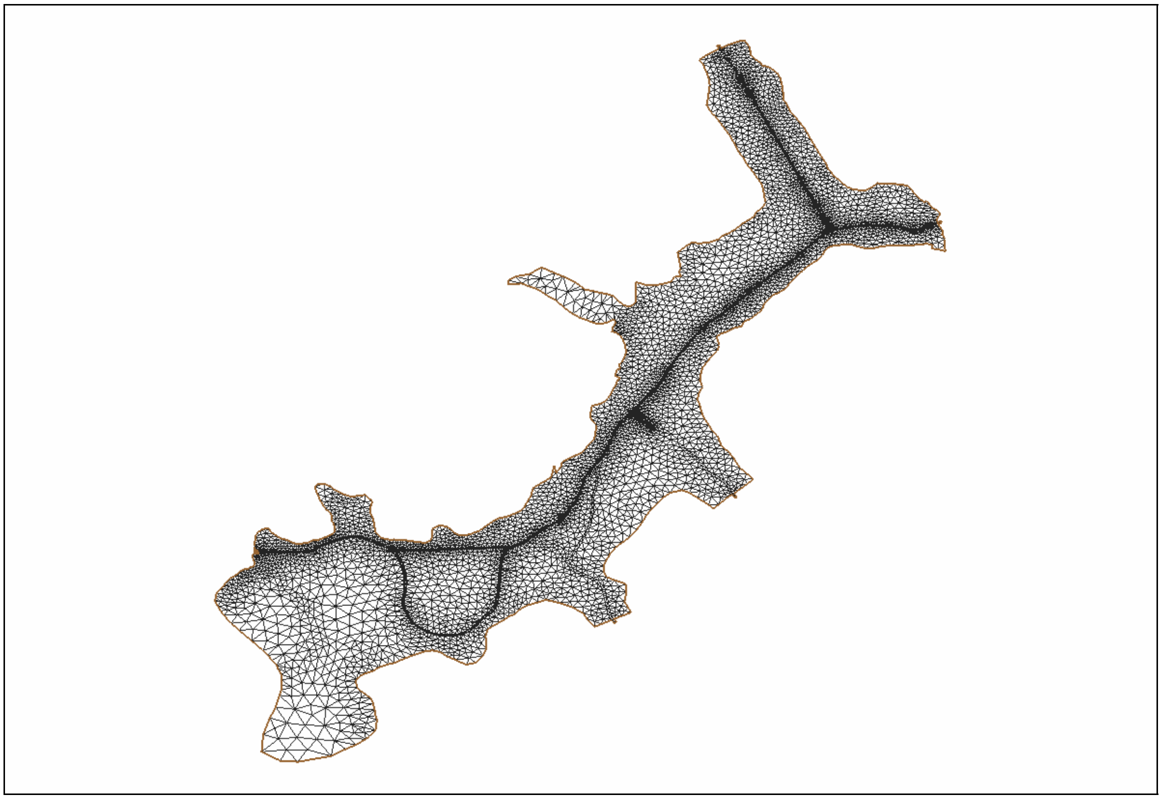


Figure 1. Computation mesh for No-Action, Alternative #1 (Existing Condition)

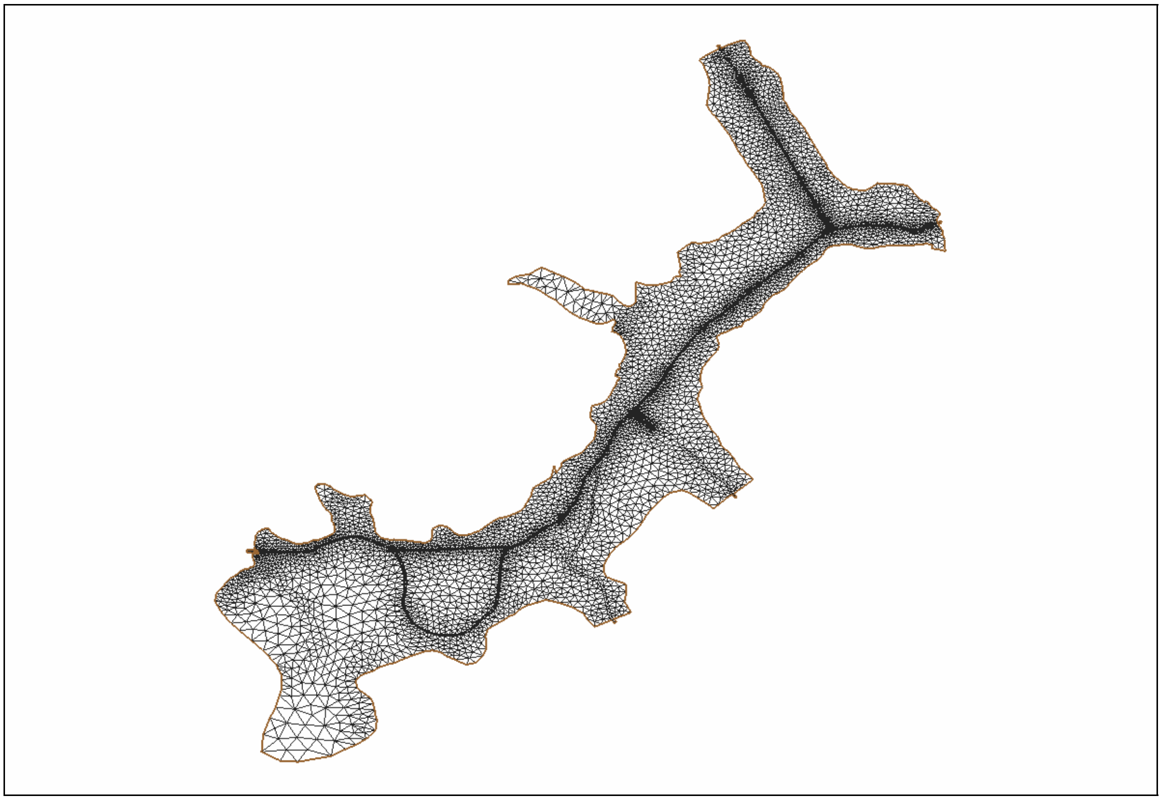


Figure 2. Computation mesh for No Tidegates Tidally-Influenced, Alternative #2

The computational mesh for the Modified Tidegates and Additional Storage Pond Alternative is shown in Figure 3. This mesh is comprised of 7,010 nodes, 13,464 elements. Downstream tidegate boundaries are located at the confluence of Martin Slough and Swain Slough, and at the proposed tidegate location.

The computational mesh for the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative is shown in Figure 4. This mesh is comprised of 7,706 nodes, 14,861 elements. Downstream tidegate boundaries are located at the confluence of Martin Slough and Swain Slough, and at the proposed tidegate location.

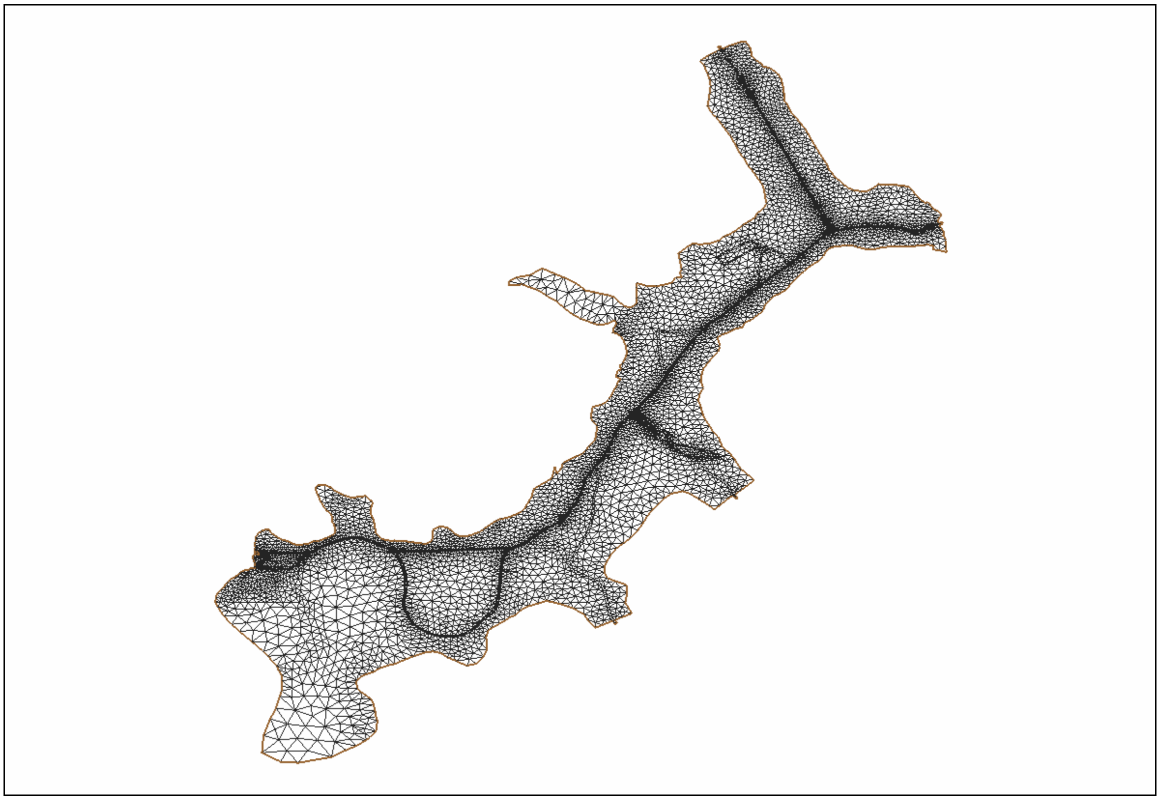


Figure 3. Computation mesh for Modified Tidegates and Additional Storage Pond, Alternative #3

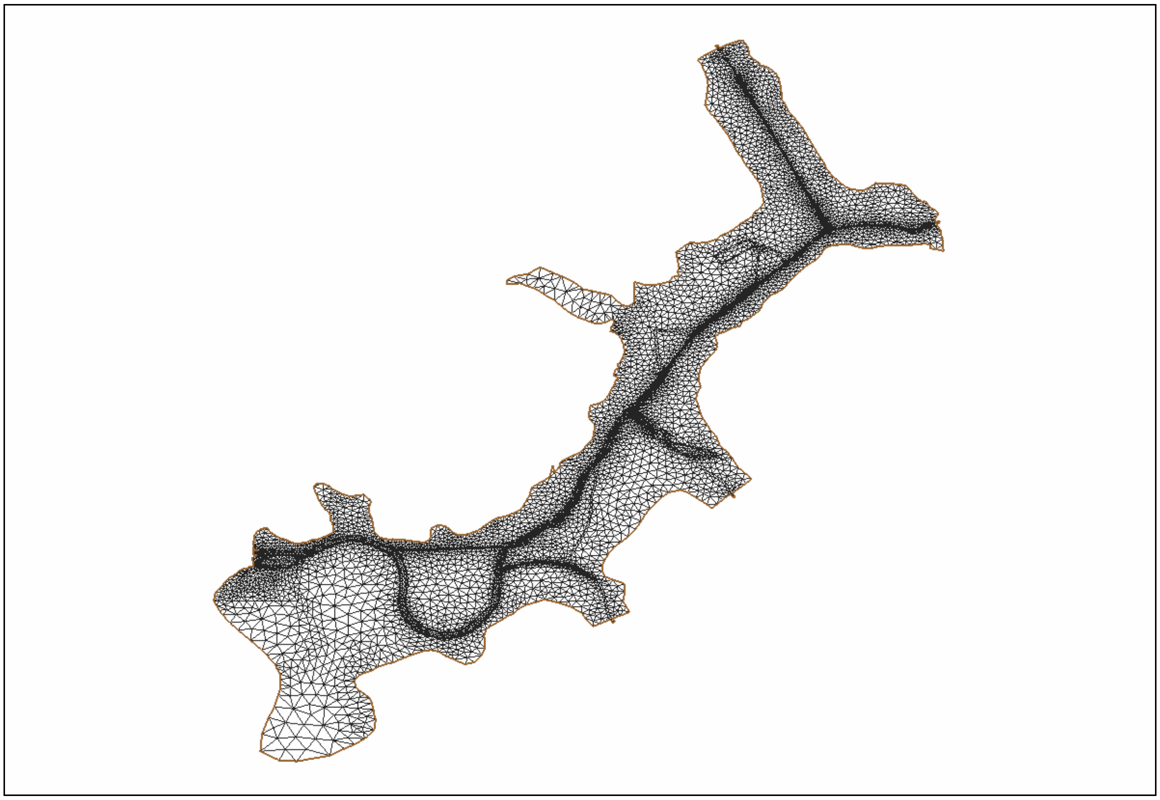


Figure 4. Computation mesh for Modified Tidegates, Additional Storage Ponds, and Modified Channel, Alternative #4

Details of each mesh at the downstream end are provided to show individual configurations to treat the boundaries there. Figure 5 shows the downstream mesh region for the No-Action Alternative. Figure 6 shows the downstream mesh region for the No Tidegate Tidally-Influenced Alternative. For this alternative, the boundary has been extended into Swain Slough. Figure 7 shows the downstream mesh region for the Modified Tidegates and Additional Storage Pond Alternative. Figure 8 shows the downstream mesh region for the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative.

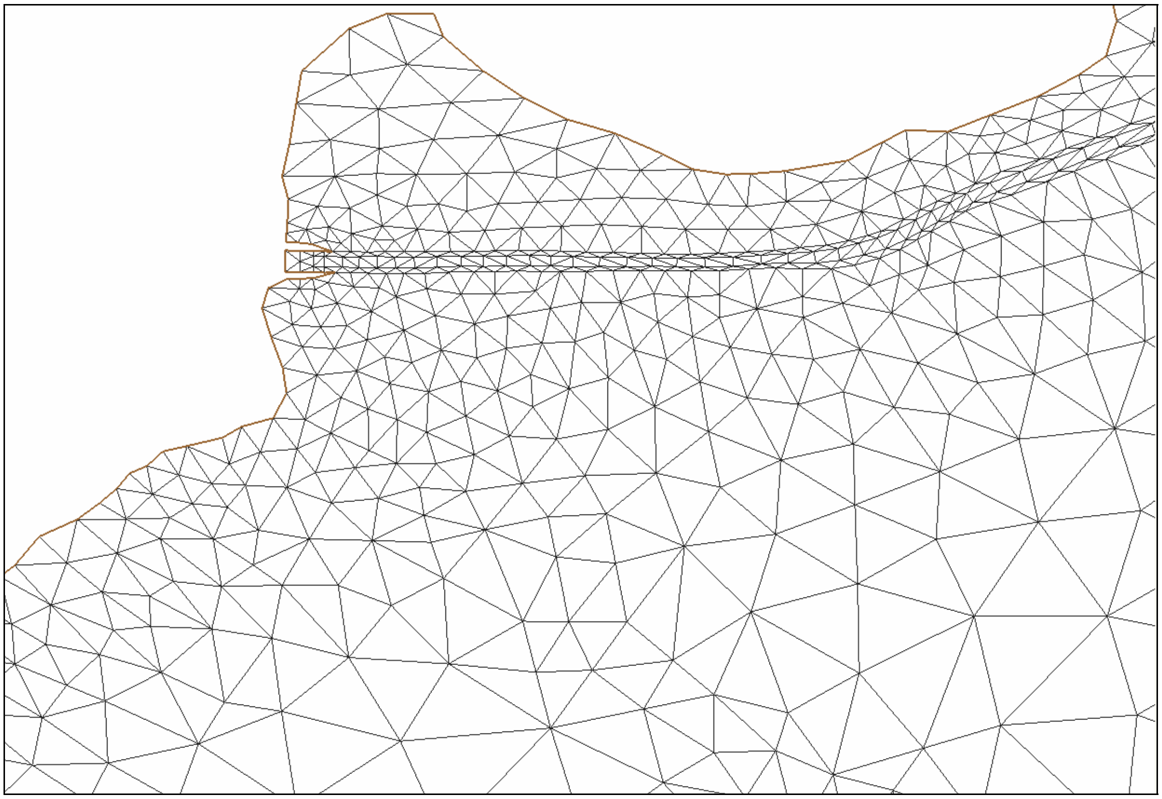


Figure 5. Downstream mesh detail for No-Action, Alternative #1 (Existing Condition).

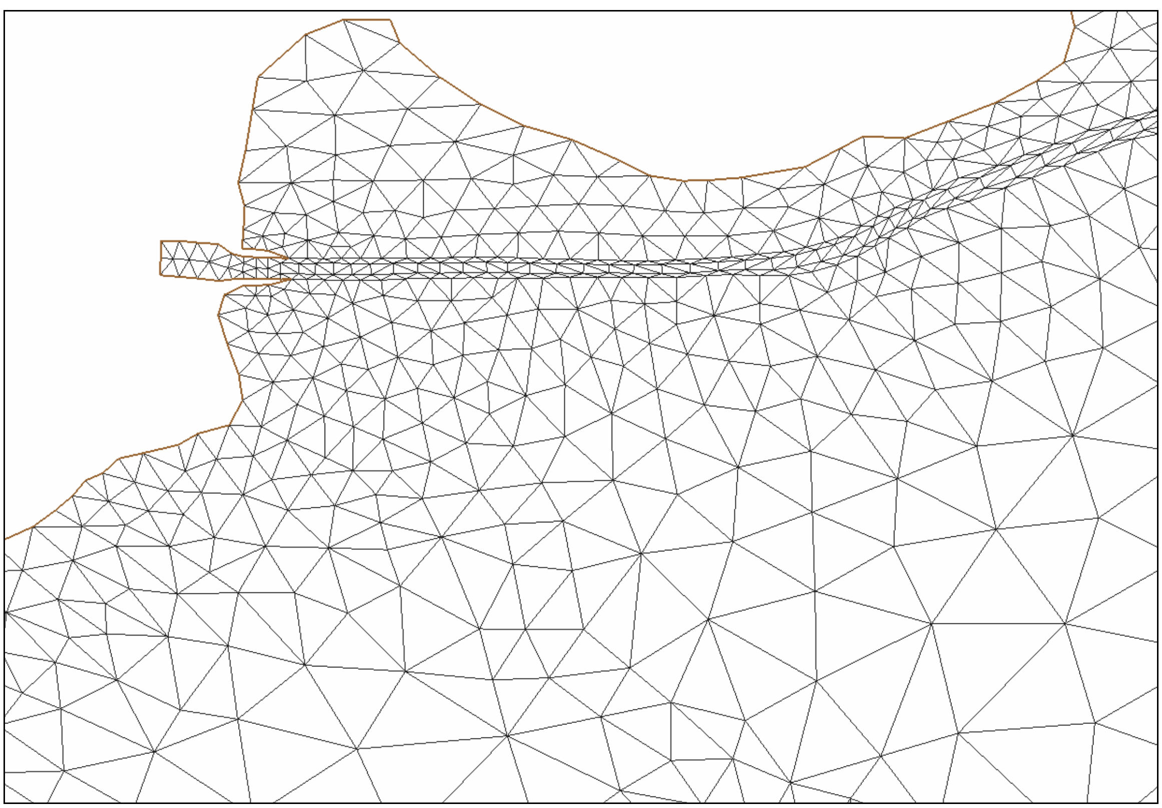


Figure 6. Downstream mesh detail for No Tidegates Tidally-Influenced, Alternative #2

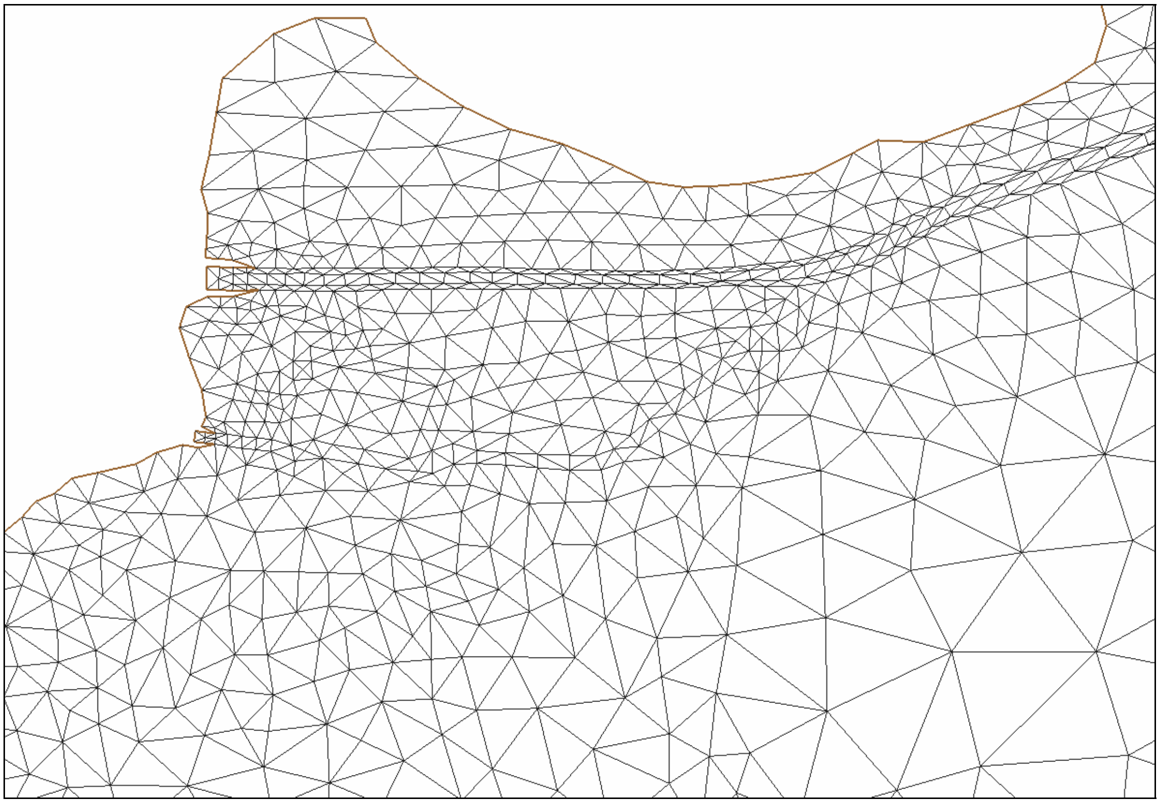


Figure 7. Downstream mesh detail for Modified Tidegates and Additional Storage Pond, Alternative #3

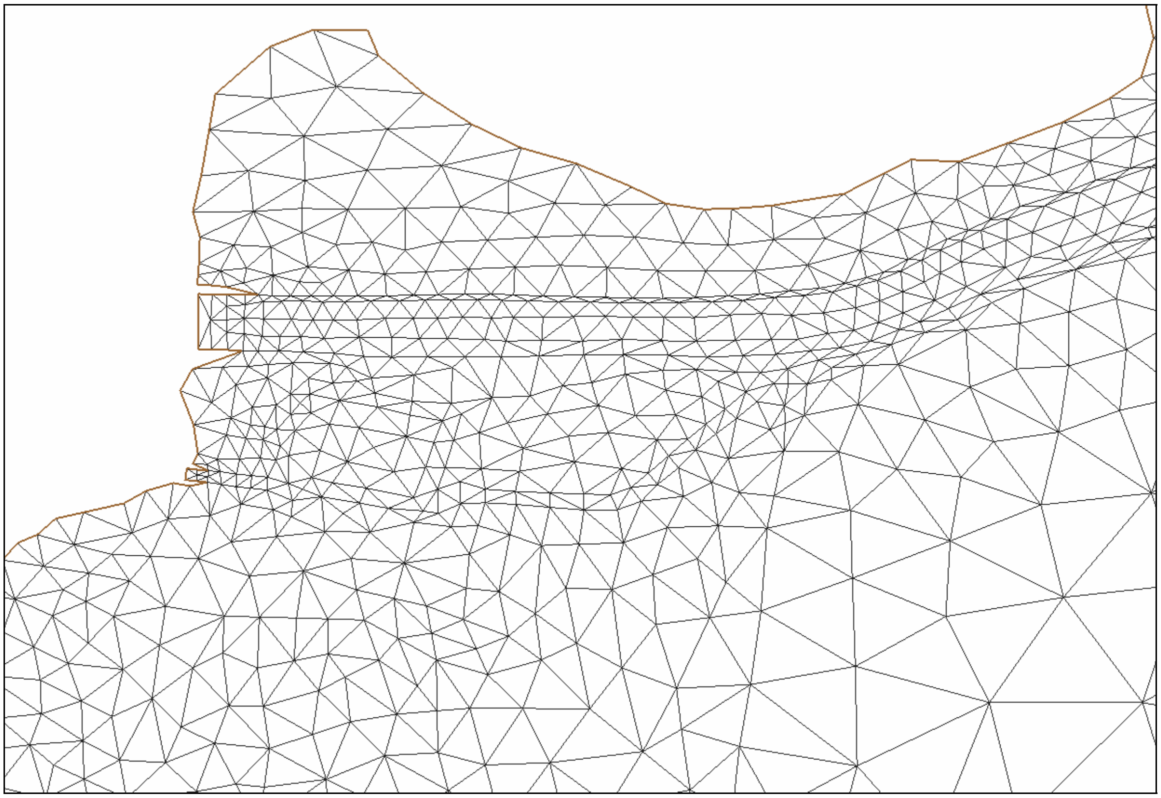


Figure 8. Downstream mesh detail for Modified Tidegates, Additional Storage Ponds, and Modified Channel, Alternative #4

Contour plots of mesh topography are shown in Figures 9, 10, 11, and 12 for the No-Action Alternative, No Tidegates Tidally-Influenced Alternative, Modified Tidegates and Additional Storage Pond Alternative, and Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative, respectively. Figures 9 and 10 are almost identical because the only difference between them is at the downstream boundary where Martin Slough enters Swain Slough. Figure 11 shows the additional storage provided by new ponds and increased area of existing ponds. Figure 12 shows the additional storage provided by the new ponds, increased area of existing ponds, and the widened and deepened Martin Slough channel together with a new channel extending west from the southernmost tributary and entering Martin Slough in the large channel bend.

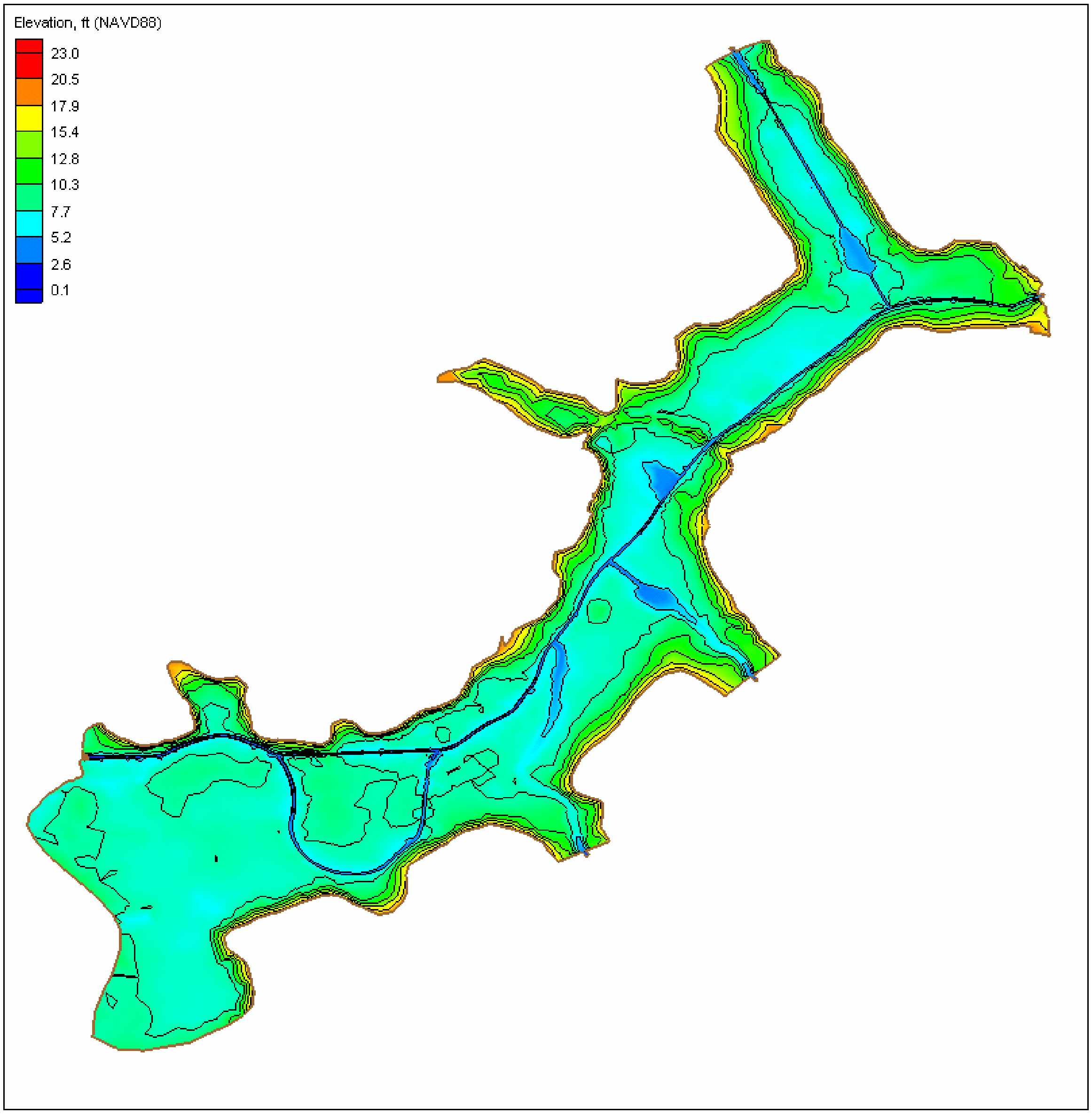


Figure 9. Mesh topographic surface for the No-Action Alternative (Existing Conditions), Alternative #1

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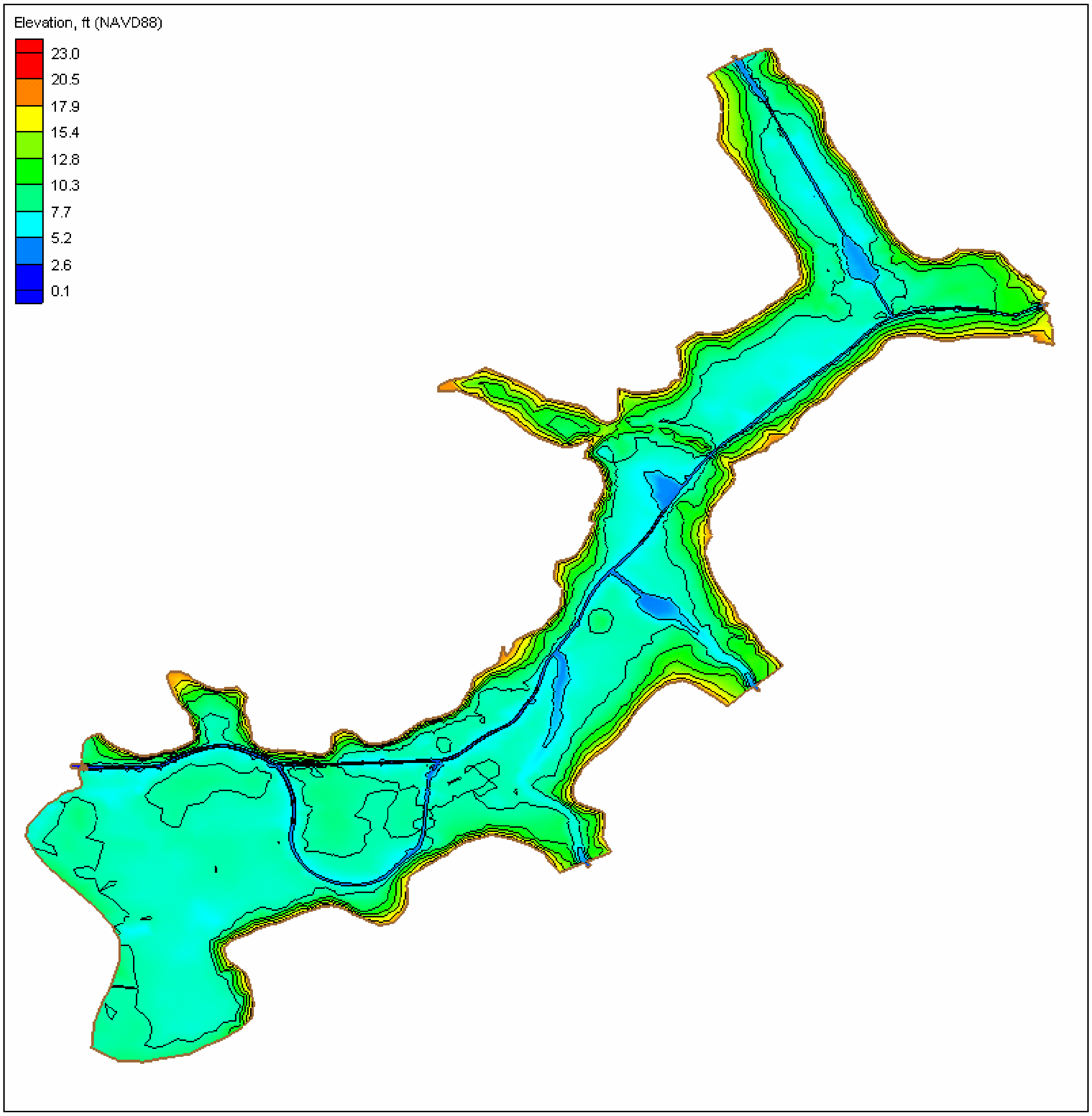

Figure 10. Mesh topographic surface for the No Tidegates Tidally-Influenced, Alternative #2

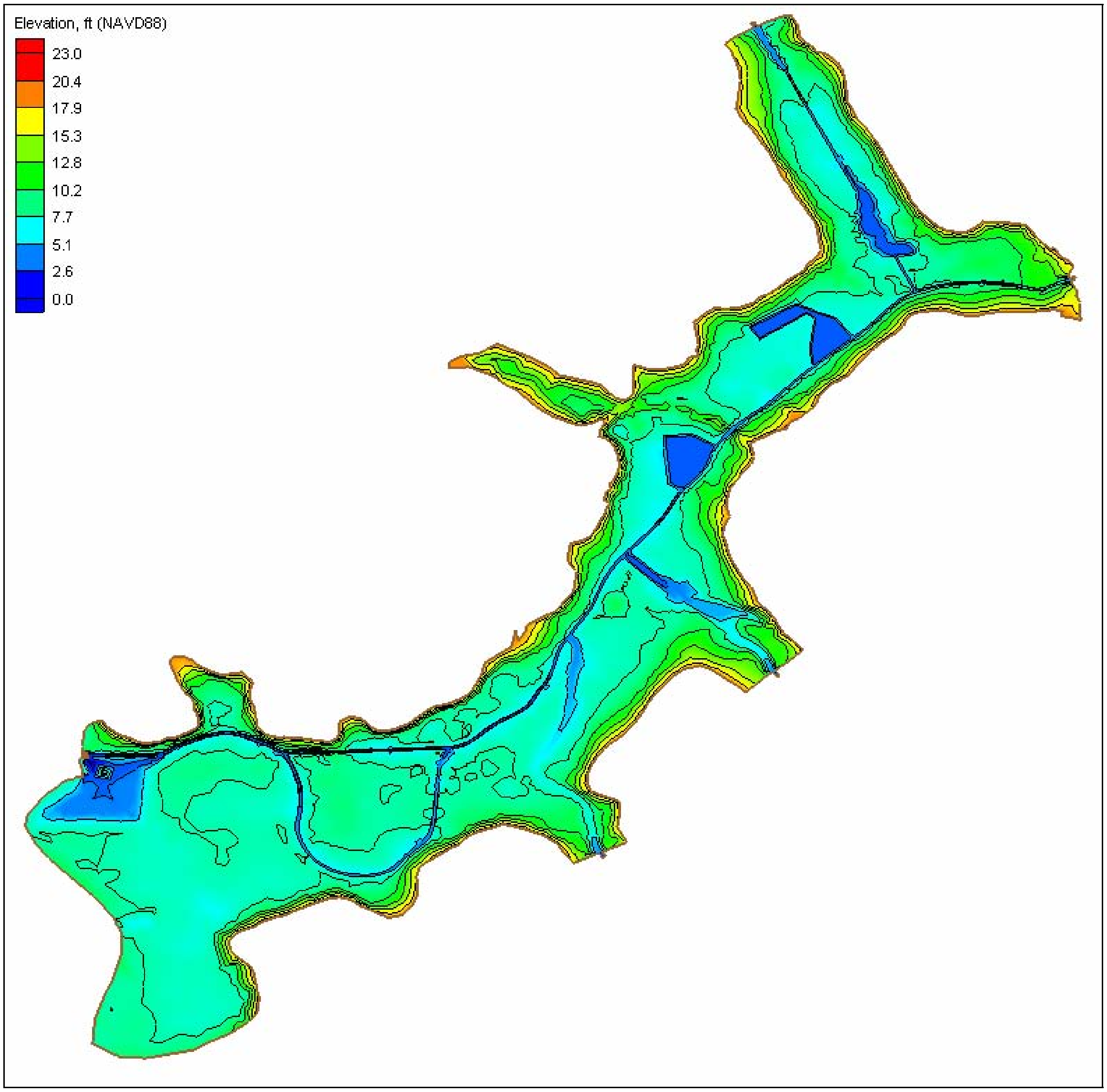

Figure 11. Mesh topographic surface for Modified Tidegates and Additional Storage Ponds, Alternative #3

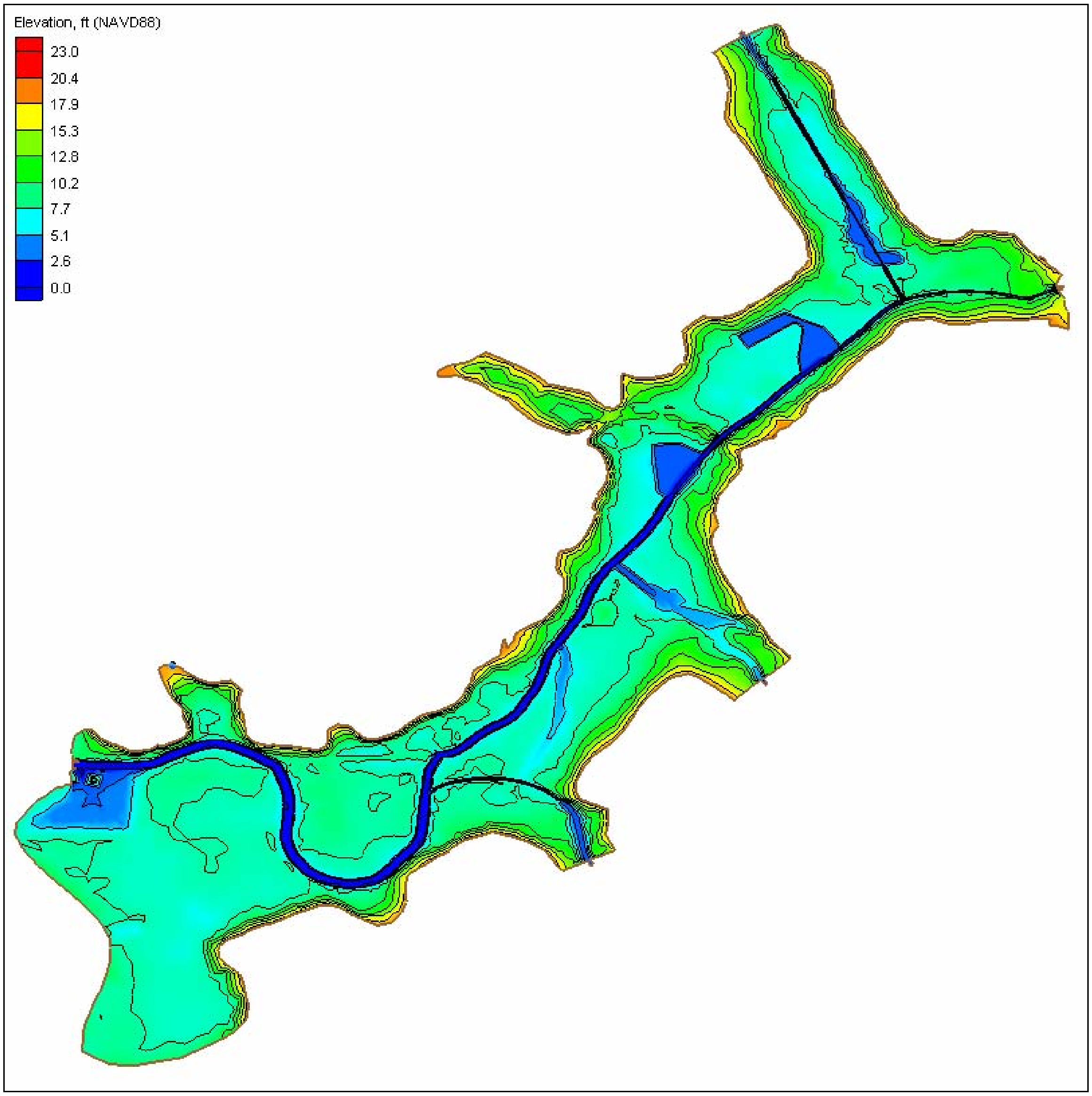

Figure 12. Mesh topographic surface for Modified Tidegates, Additional Storage Ponds, and Modified Channel, Alternative #4

Forcing of the boundary conditions in the hydraulic model for the Martin Slough study area was specified as tributary discharges, tidegate discharges, and downstream water-surface elevation values. Discharges were calculated by the watershed model and tidegate model for alternatives in which tidegates are present. Downstream boundaries were forced with water-surface elevation values. For Alternatives 3 and 4 with tidegates, boundary forcing included utilizing water-surface elevations predicted by the TIDEGATE model at nodes just upstream of the tidegate location, and water discharge volumes calculated by the TIDEGATE model at the tidegates.

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Simulations of flood inundation were computed for 2-yr and 10-yr events for each alternative. All simulations were conducted with the following computational specifications: time step = 0.02 s, friction coefficient = 0.007 (dimensionless), eddy viscosity coefficient = 5 m^2/s. Resolution along the Martin Slough channel has typical along-channel spacing of about 18 ft, with spacing ranging from approximately 14 to 40 ft. Maximum node spacing in the mesh is about 160 ft.

RESULTS OF MODELED ALTERNATIVES

Calculated water levels for each alternative for the 2-yr and 10-yr events are presented and discussed. Results of the No-Action Alternative provide a basis from which the action alternatives can be evaluated in terms of improvement of inundation area and duration. Plan-view plots of inundation are provided at the peak inundation, followed by 1, 2, 3, 5, and 7 days after the start of the simulation to show relative extent of inundation and time for water to drain from the upland areas. Plots showing time-series of water level at four stations are also provided to directly compare inundation levels and duration.

No-Action Alternative (Existing Conditions), Alt. #1

Plan view plots of inundation for the No-Action Alternative (Existing Conditions) are given in Figures 13 through 18 for the peak inundation and elapsed times of 1, 2, 3, 5, and 7 days, respectively, for the 2-year event. Most flooding for this event take place upstream of the large channel meander. Inundated areas downstream tend to dry before those upstream. The site remains strongly inundated through 2 days of simulation, but inundation is substantially reduced by day 3. Reduction in inundation is gradual from day 3 to day 7 with the remaining water being located primarily near existing ponds and in low lying areas.

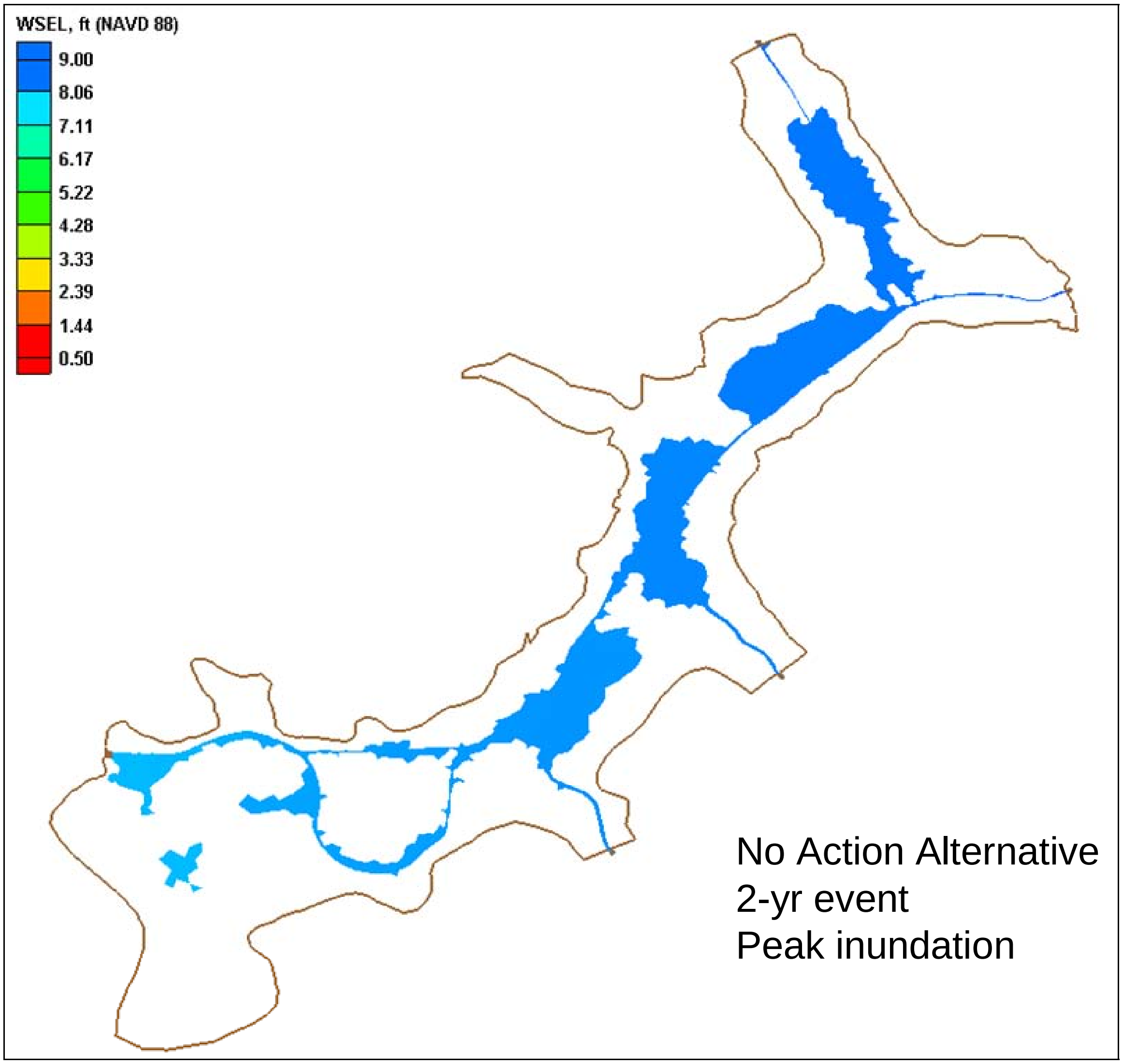

Figure 13. Peak inundation for the 2-yr event, No Action, Alternative #1

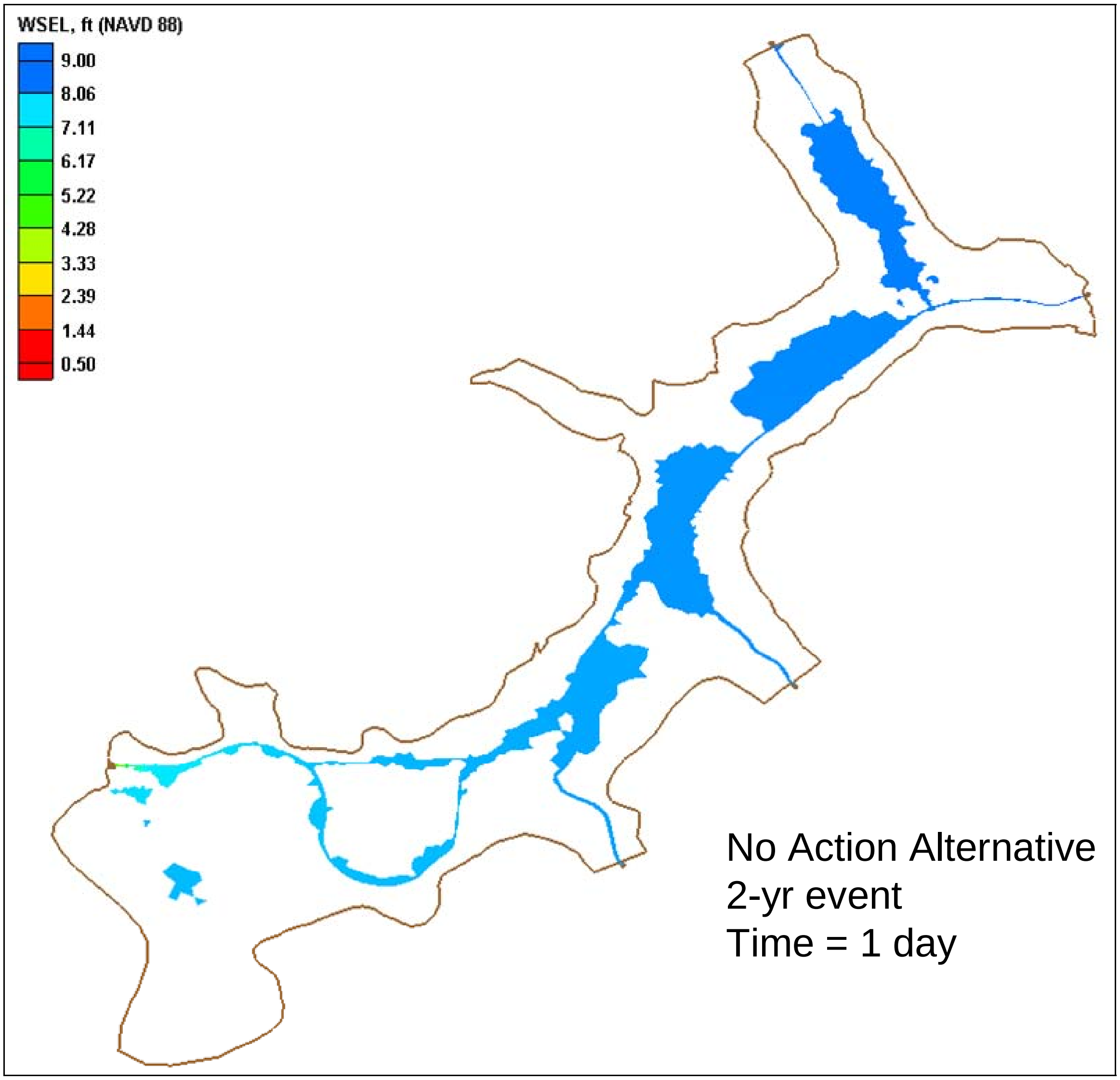

Figure 14. Inundation at time = 1 day for the 2-yr event, No Action, Alternative #1

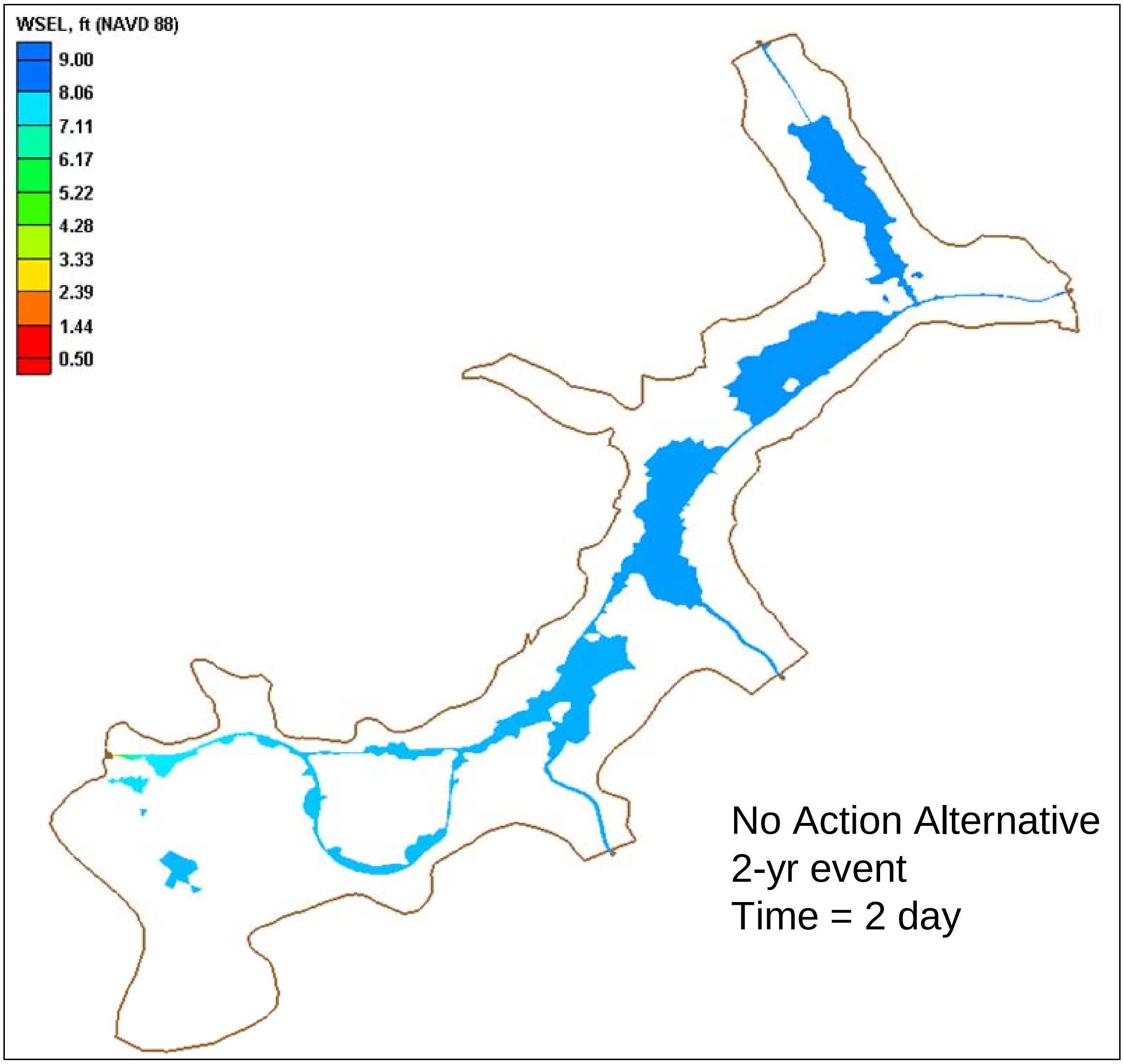

Figure 15. Inundation at time = 2 day for the 2-yr event, No Action, Alternative #1

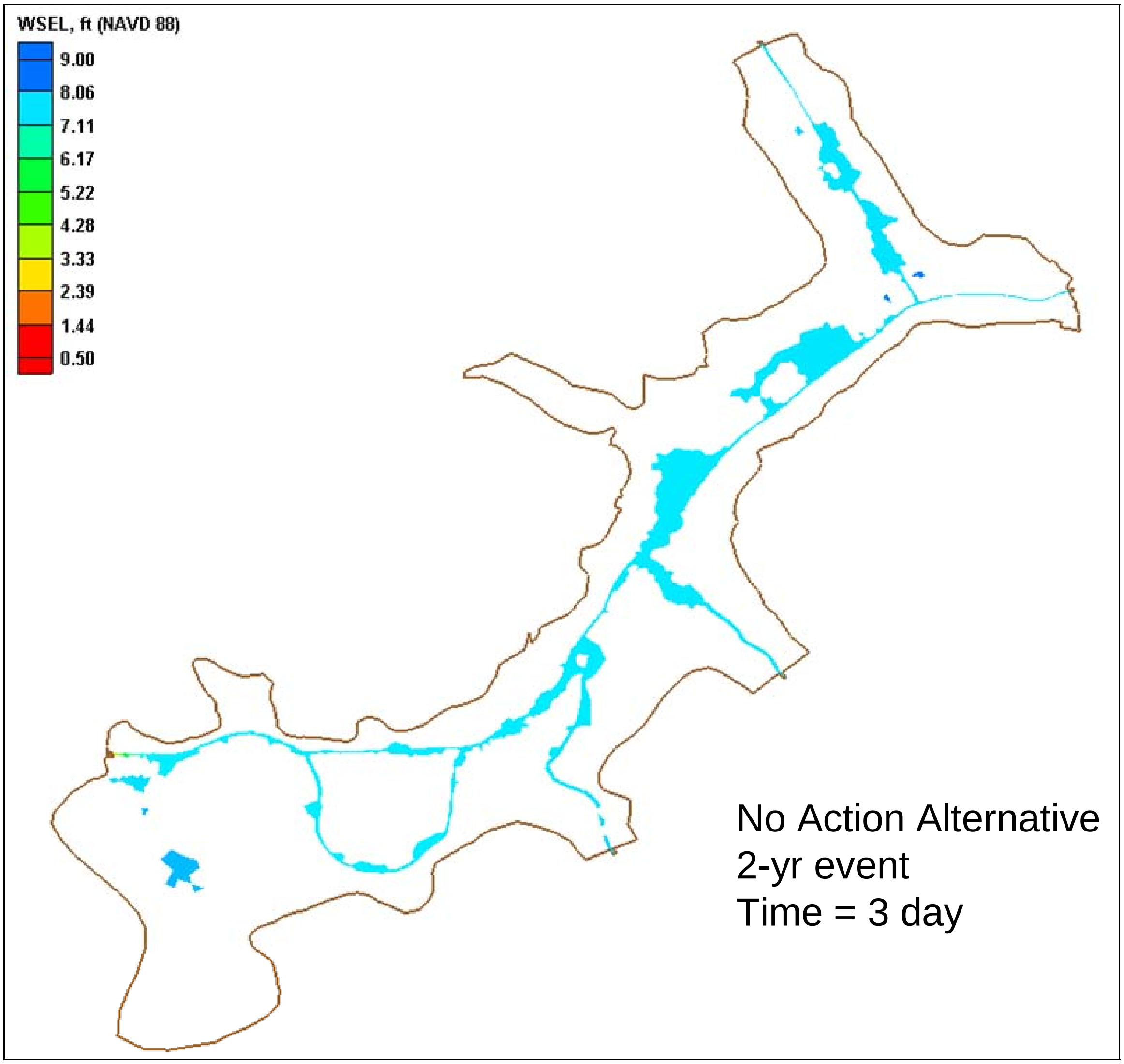

Figure 16. Inundation at time = 3 day for the 2-yr event, No Action, Alternative #1

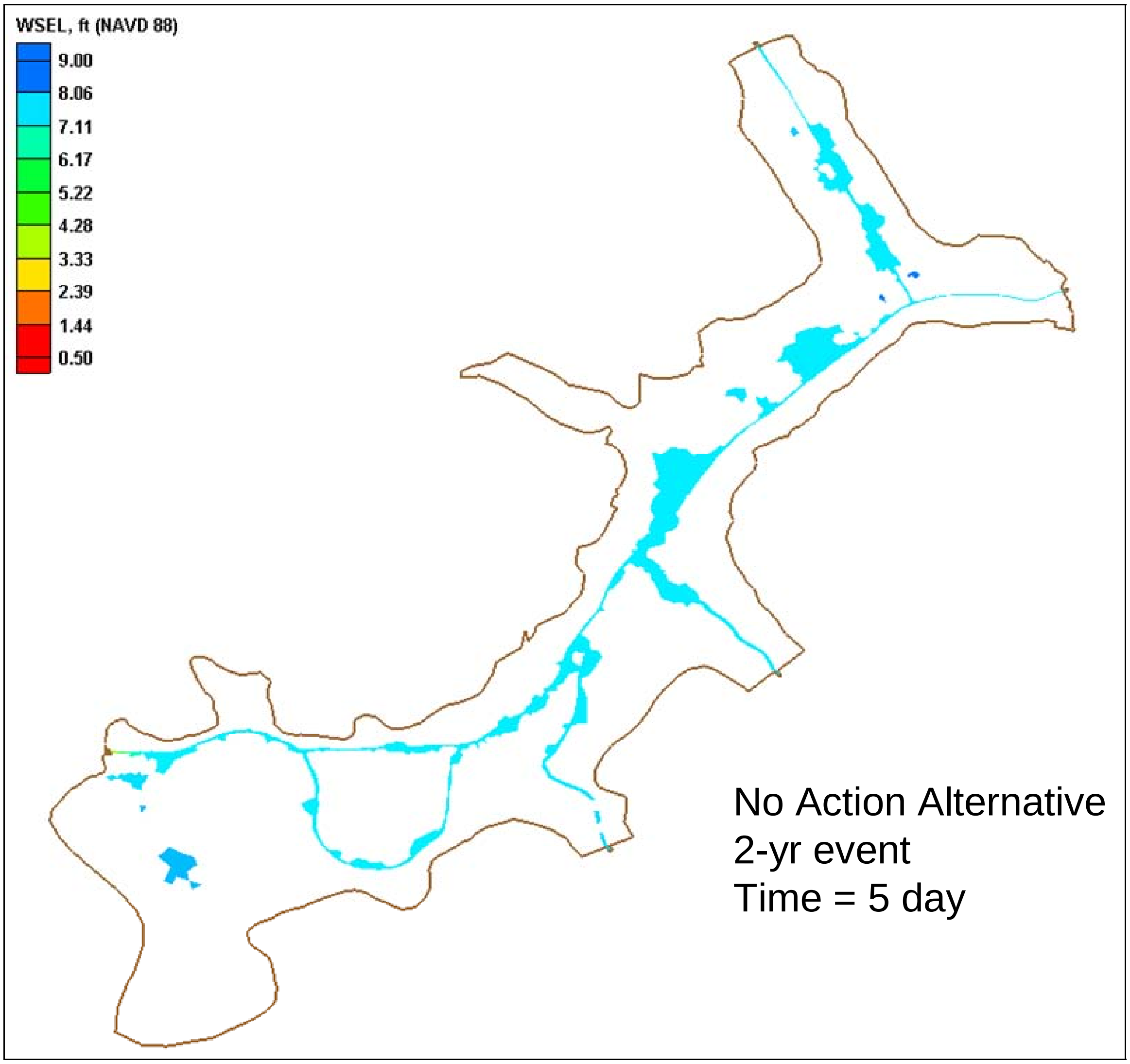

Figure 17. Inundation at time = 5 day for the 2-yr event, No Action, Alternative #1

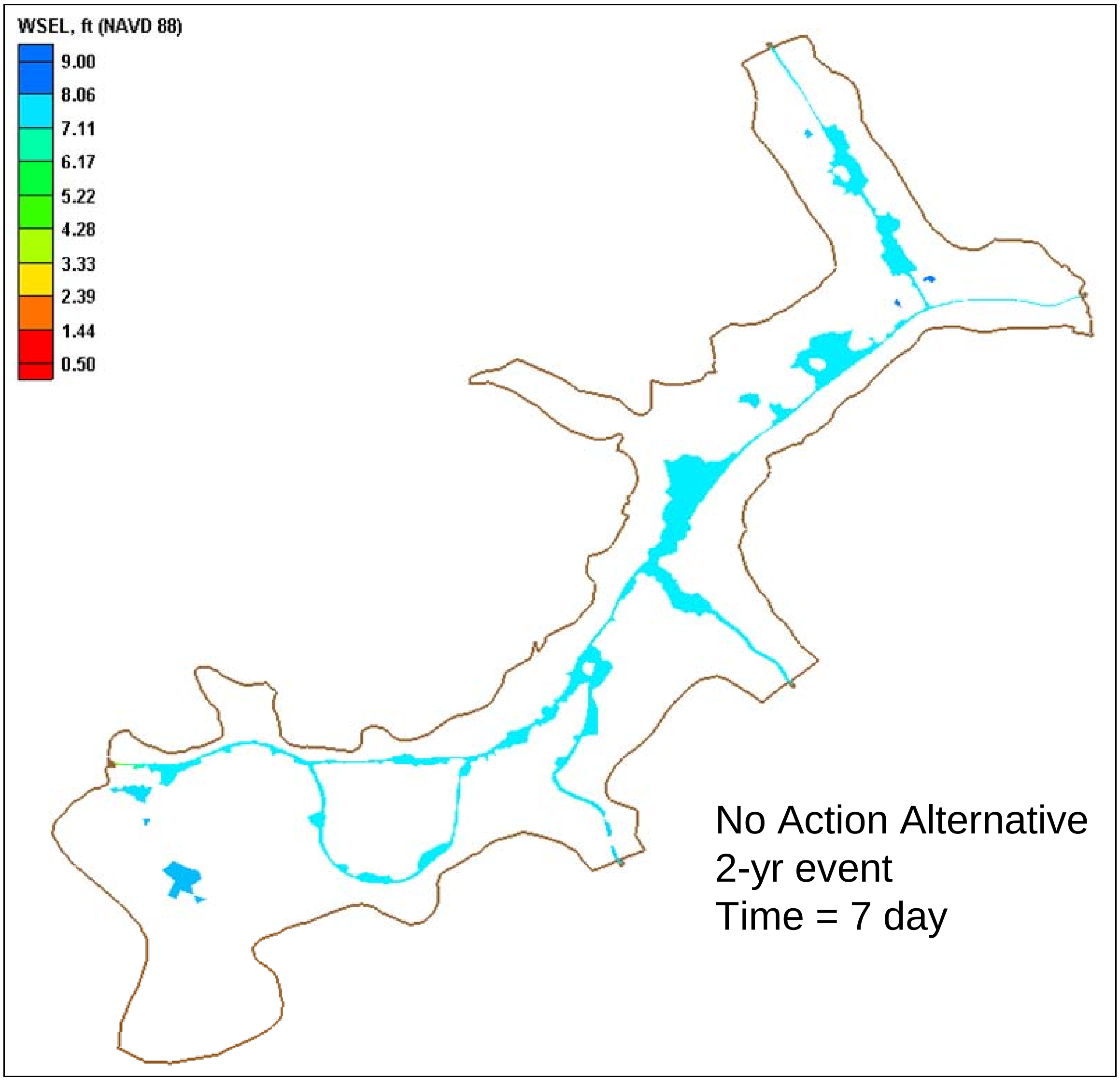

Figure 18. Inundation at time = 7 day for the 2-yr event, No Action, Alternative #1

Plan view plots of inundation for the No-Action Alternative (Existing Conditions) are given in Figures 19 through 24 for the peak inundation and elapsed times of 1, 2, 3, 5, and 7 days, respectively, for the 10-year event. Flooding for this event takes place over most of the study area, and inundating a significantly larger area than the 2-year event, particularly in the southwestern portion (pasture). Inundated areas downstream tend to dry before those upstream. The upstream portions of the site remain strongly inundated throughout the simulation, and reduction in inundation there is greatest between the peak and 3 days, then slowing over the remaining four days.

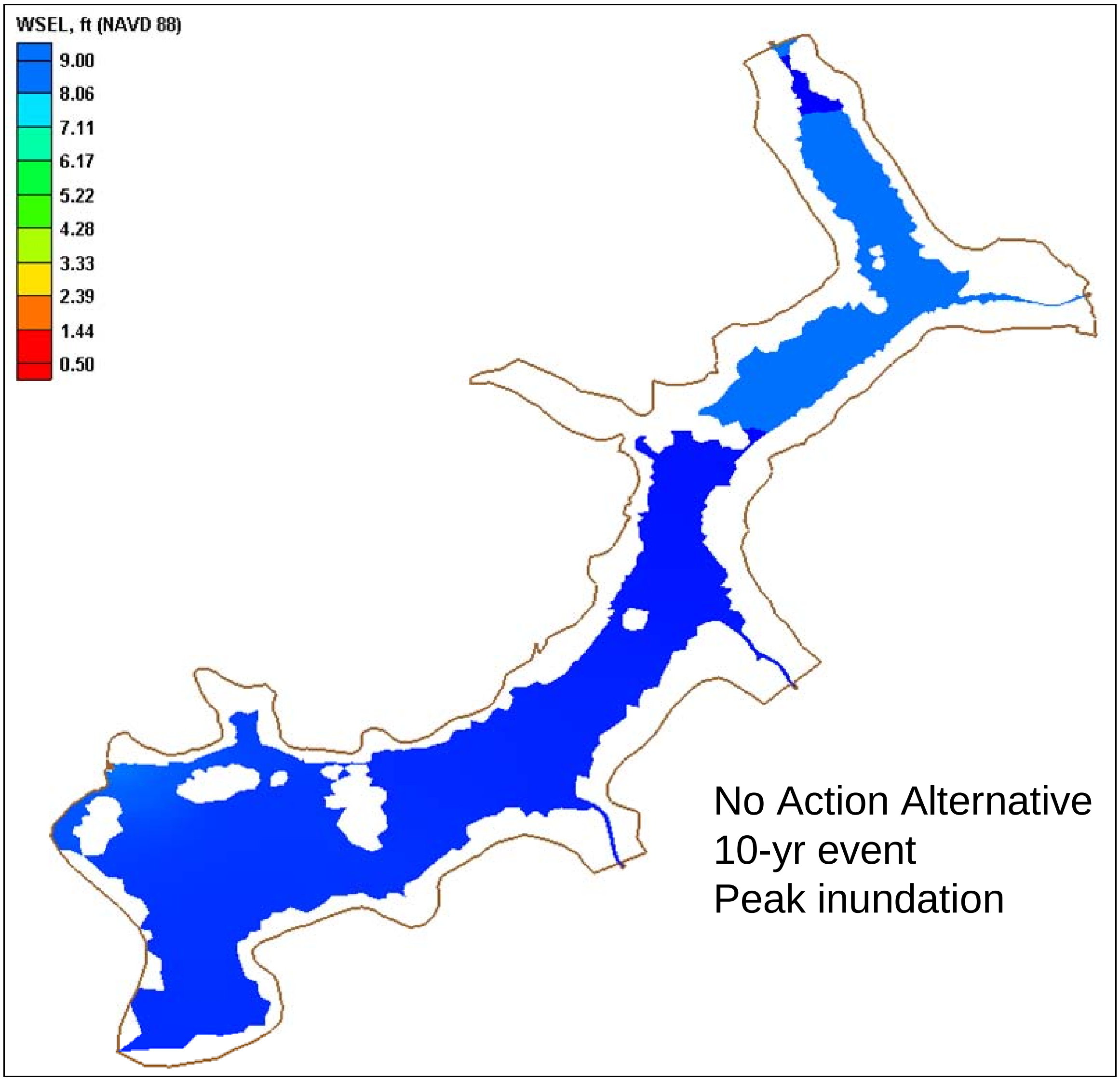

Figure 19. Peak inundation for the 10-yr event, No Action, Alternative #1

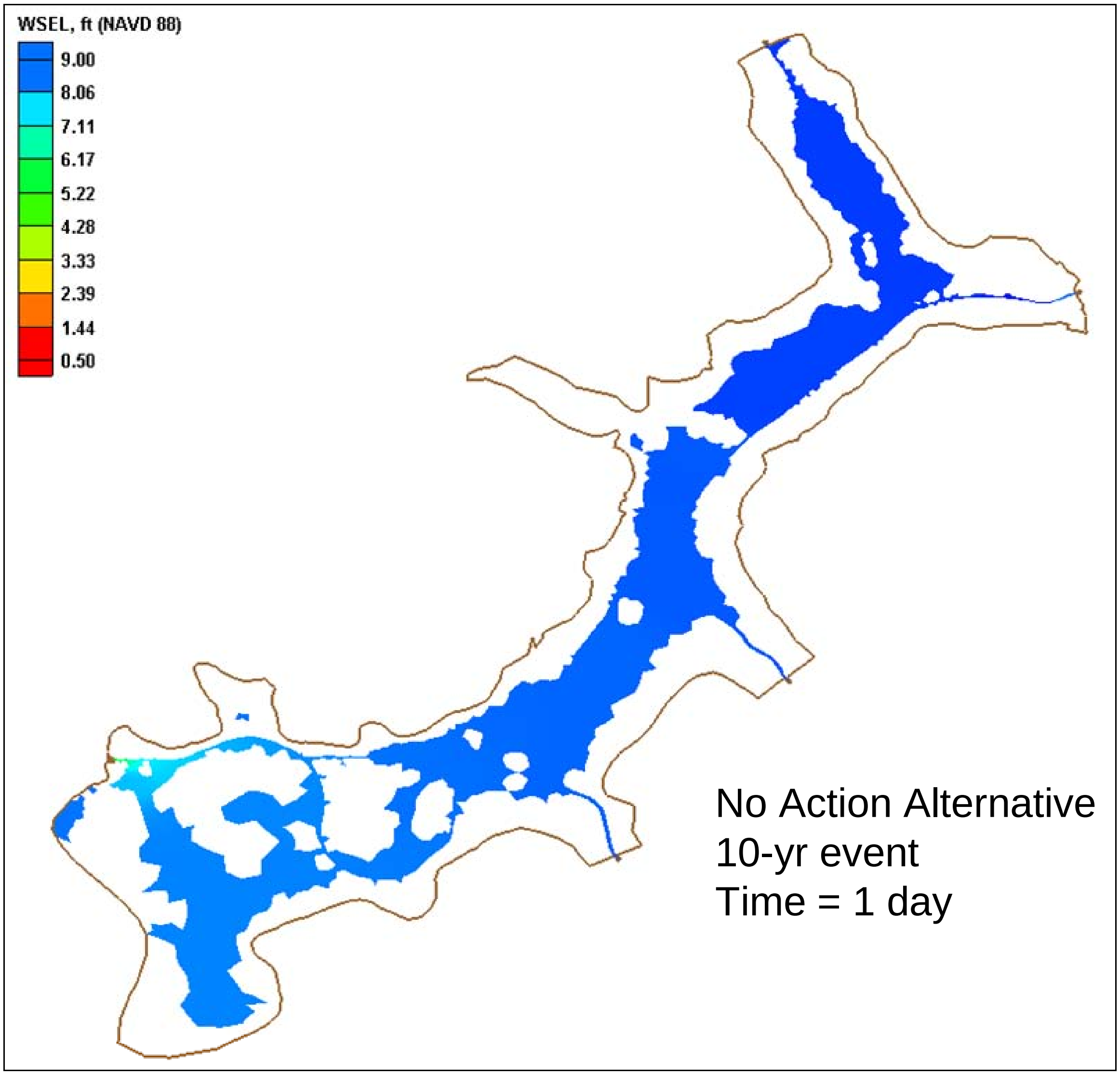

Figure 20. Inundation at time = 1 day for the 10-yr event, No Action, Alternative #1

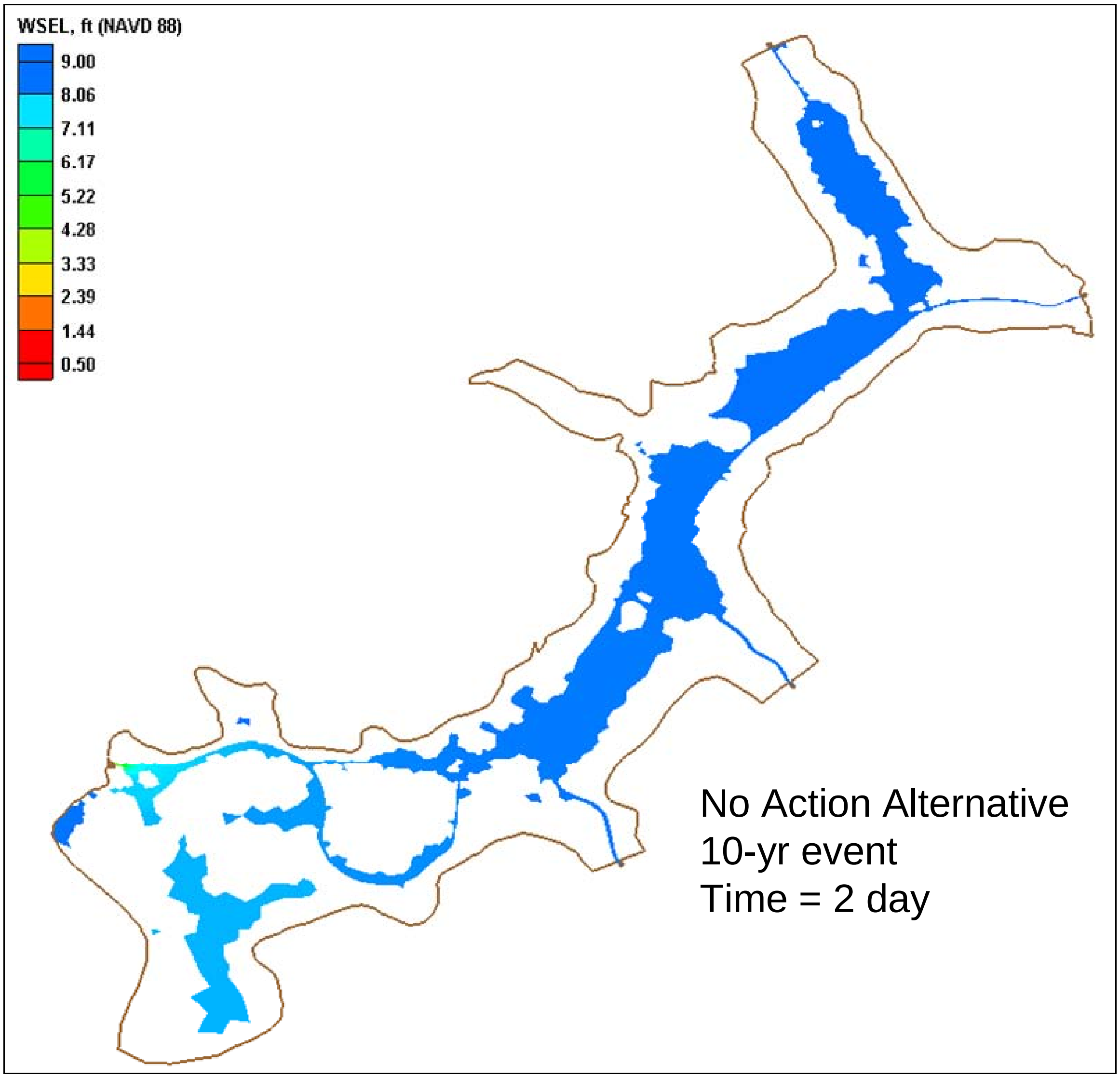

Figure 21. Inundation at time = 2 day for the 10-yr event, No Action, Alternative #1

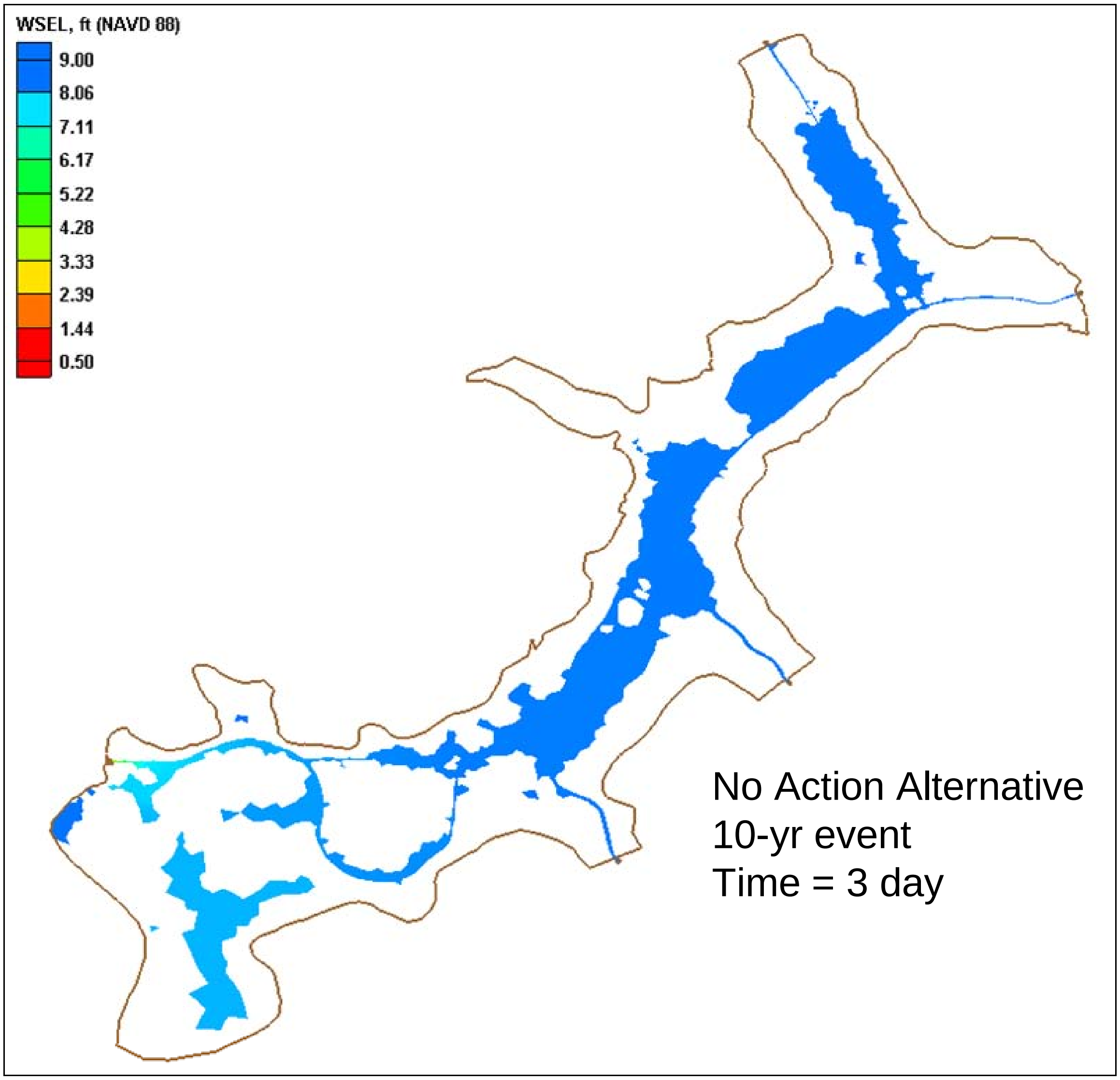

Figure 22. Inundation at time = 3 day for the 10-yr event, No Action, Alternative #1

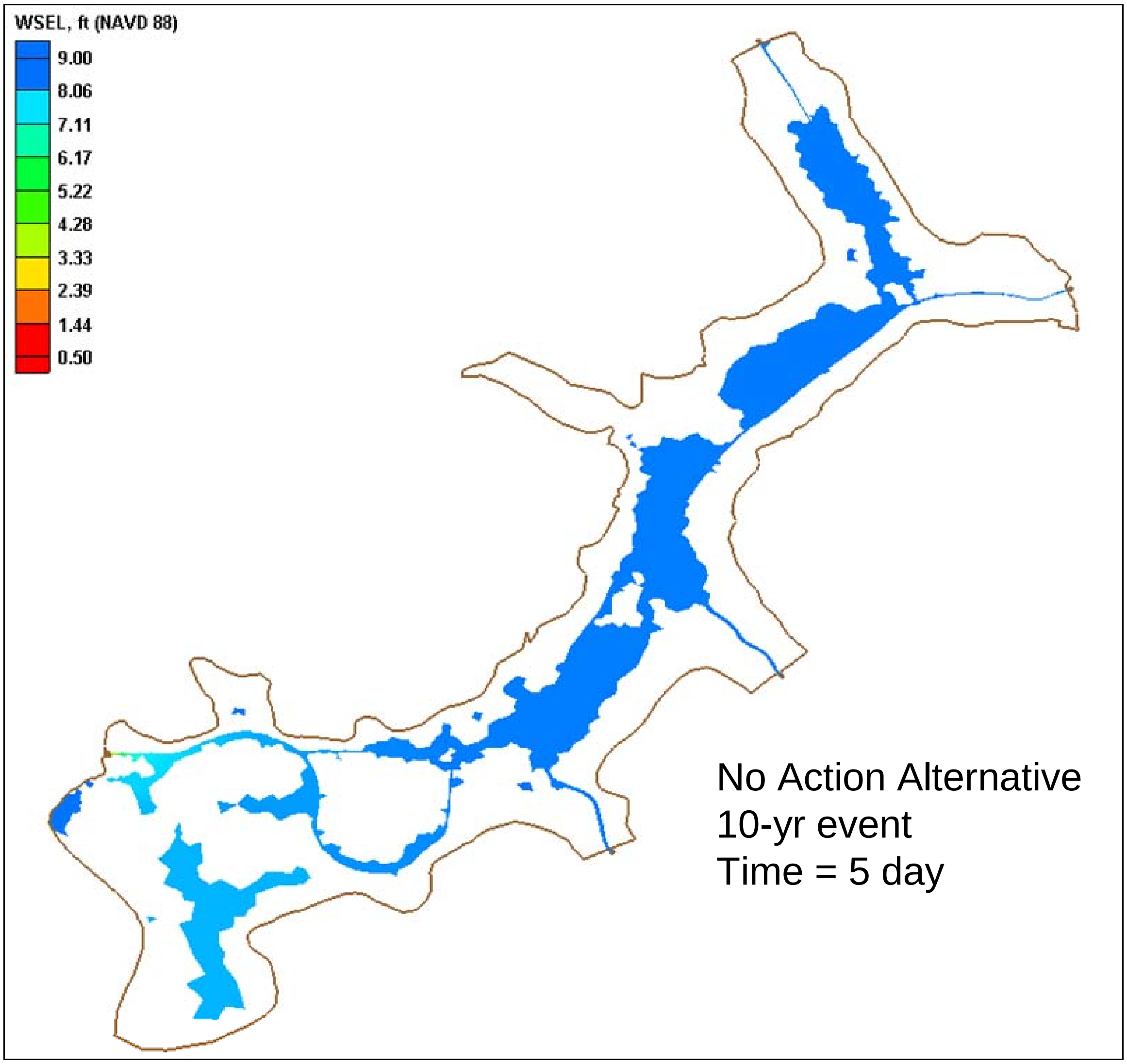

Figure 23. Inundation at time = 5 day for the 10-yr event, No Action, Alternative #1

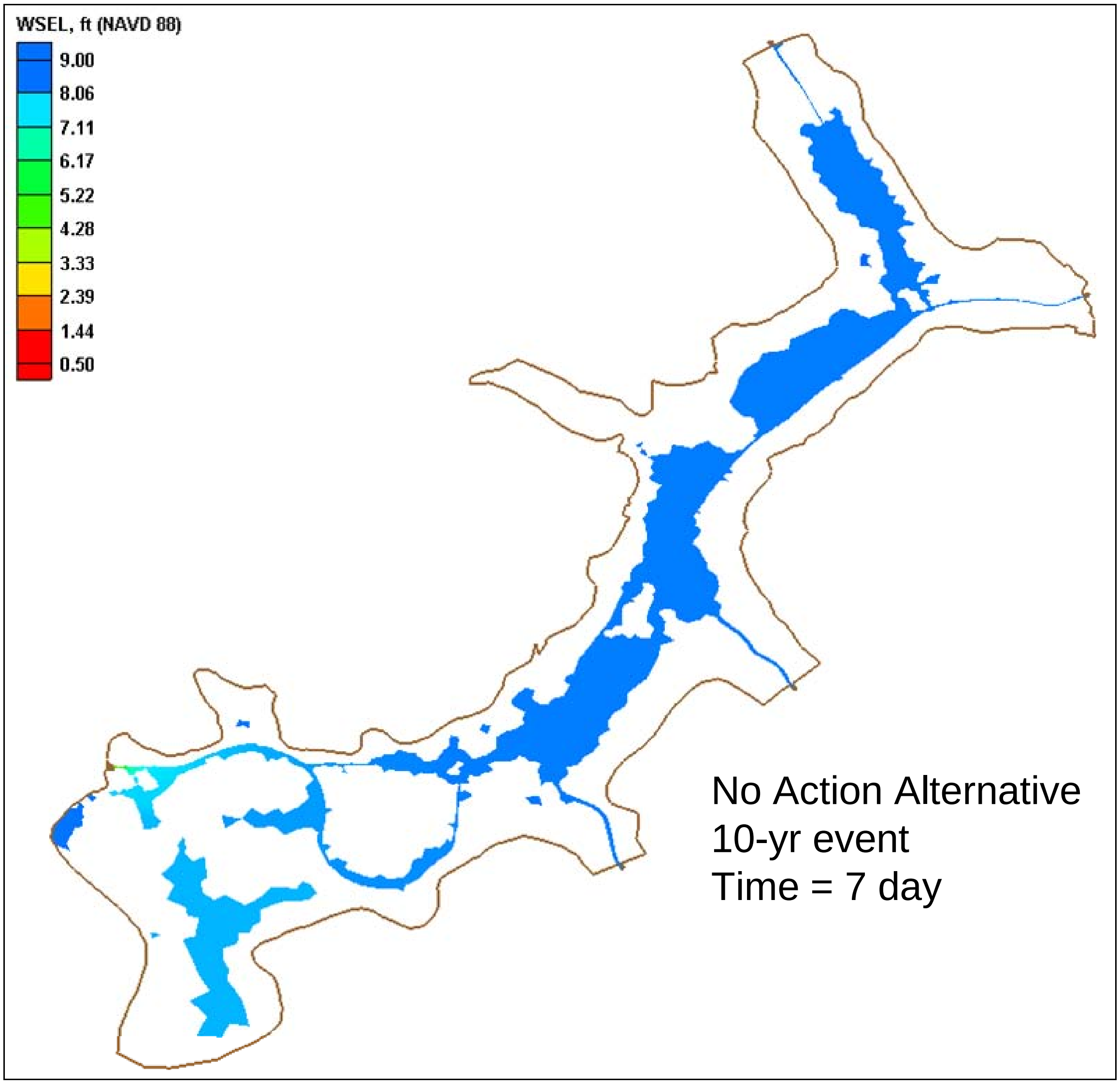

Figure 24. Inundation at time = 7 day for the 10-yr event, No Action, Alternative #1

No Tidegates Tidally-Influenced, Alternative #2

Plan view plots of inundation for the No Tidegates Tidally-Influenced Alternative are given in Figures 25 through 30 for the peak inundation and elapsed times of 1, 2, 3, 5, and 7 days, respectively, for the 2-year event. During peak inundation, flooding for this event takes place primarily upstream of the large channel meander and in the pasture downstream of the meander. Over the 7-day simulation interval, overall water level is minimally reduced. Free propagation of the tidal wave into the study area, combined with higher tidal elevations resulted in prolonged inundation over all areas that experienced flooding. Inundation expanse and duration are greater than that for the No Action Alternative for the 2-year event.

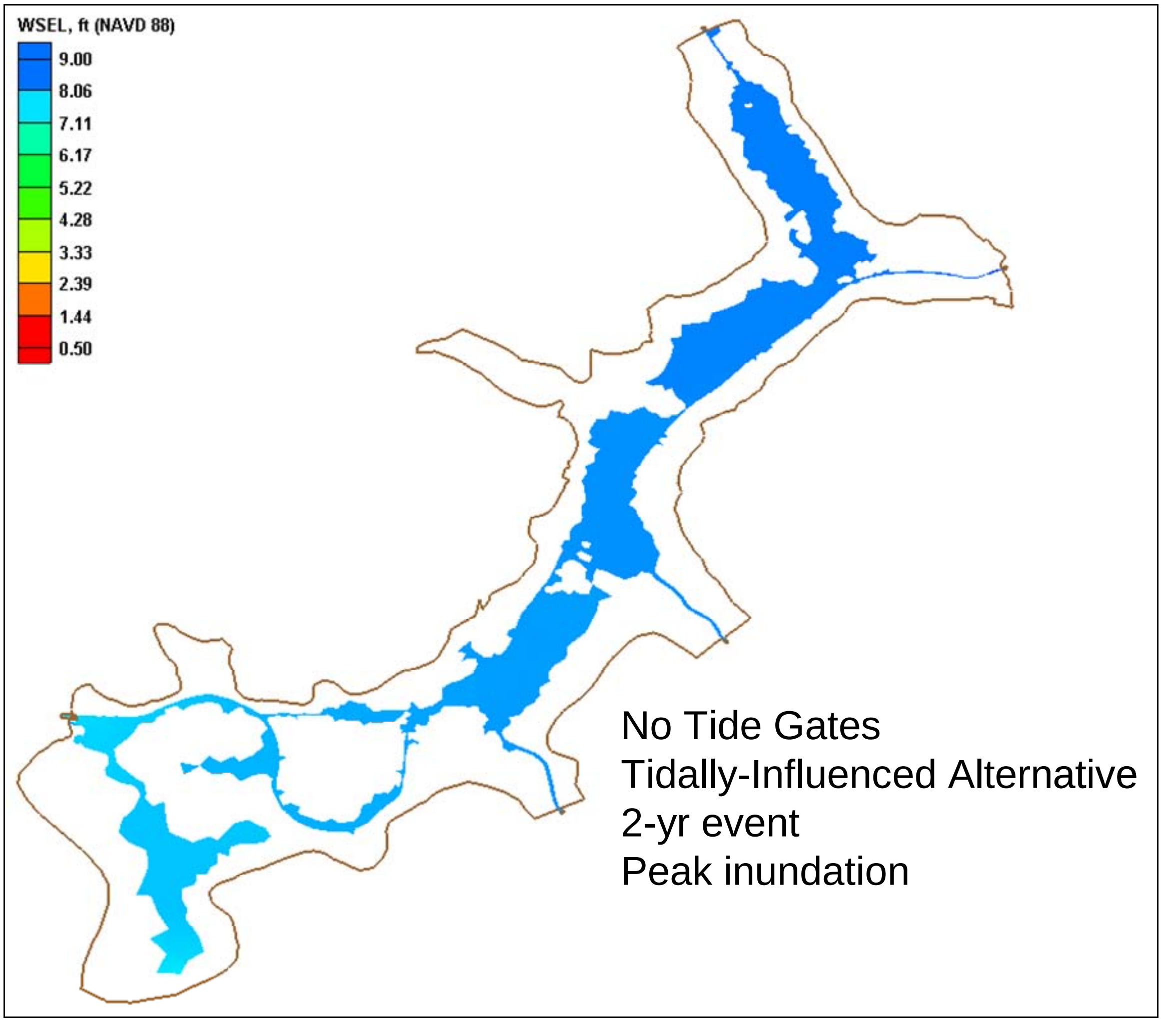

Figure 25. Peak inundation for the 2-yr event, No Tidegates Tidally-Influenced, Alternative #2

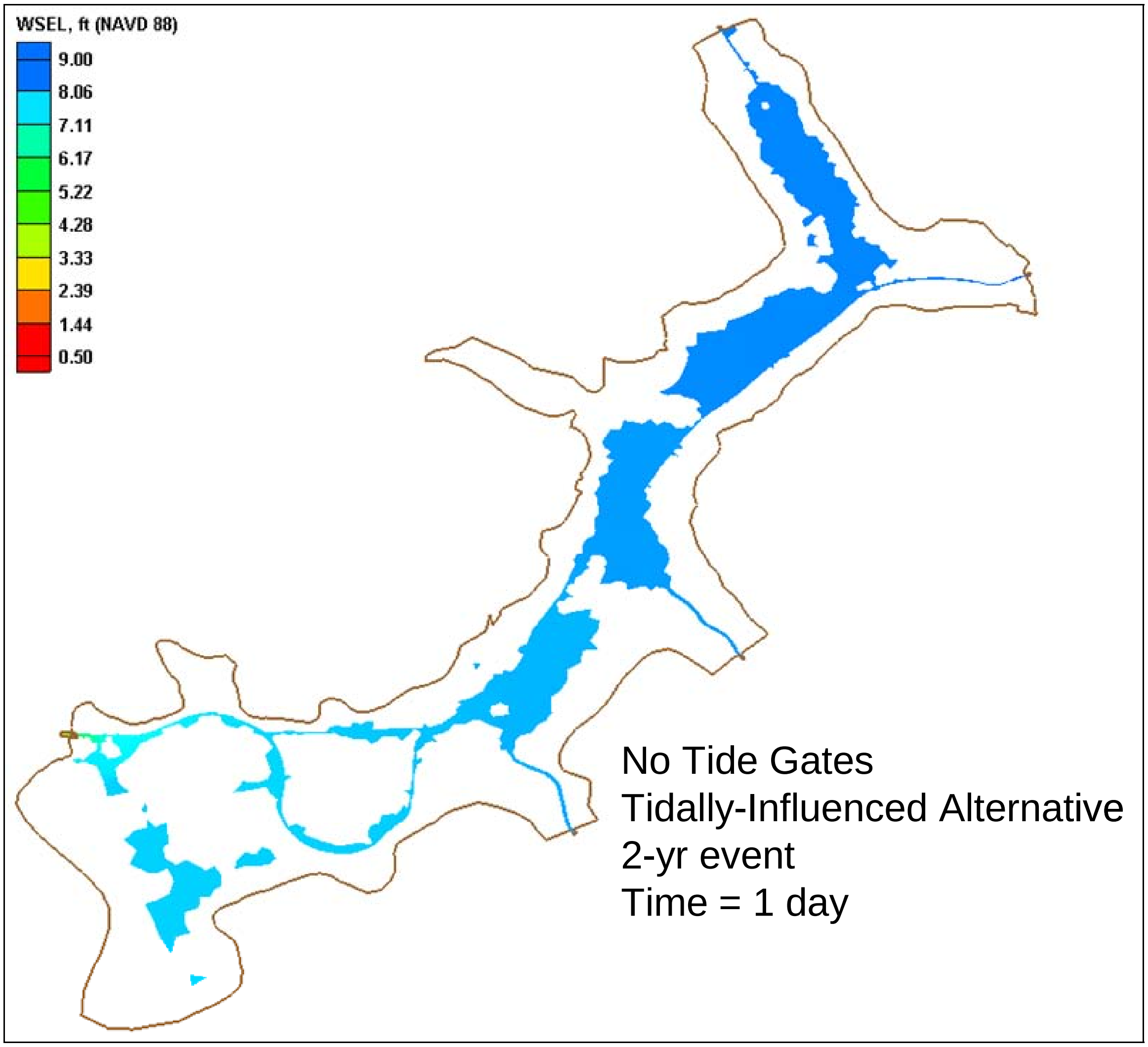

Figure 26. Inundation at time = 1 day for the 2-yr event, No Tidegates Tidally-Influenced, Alternative #2

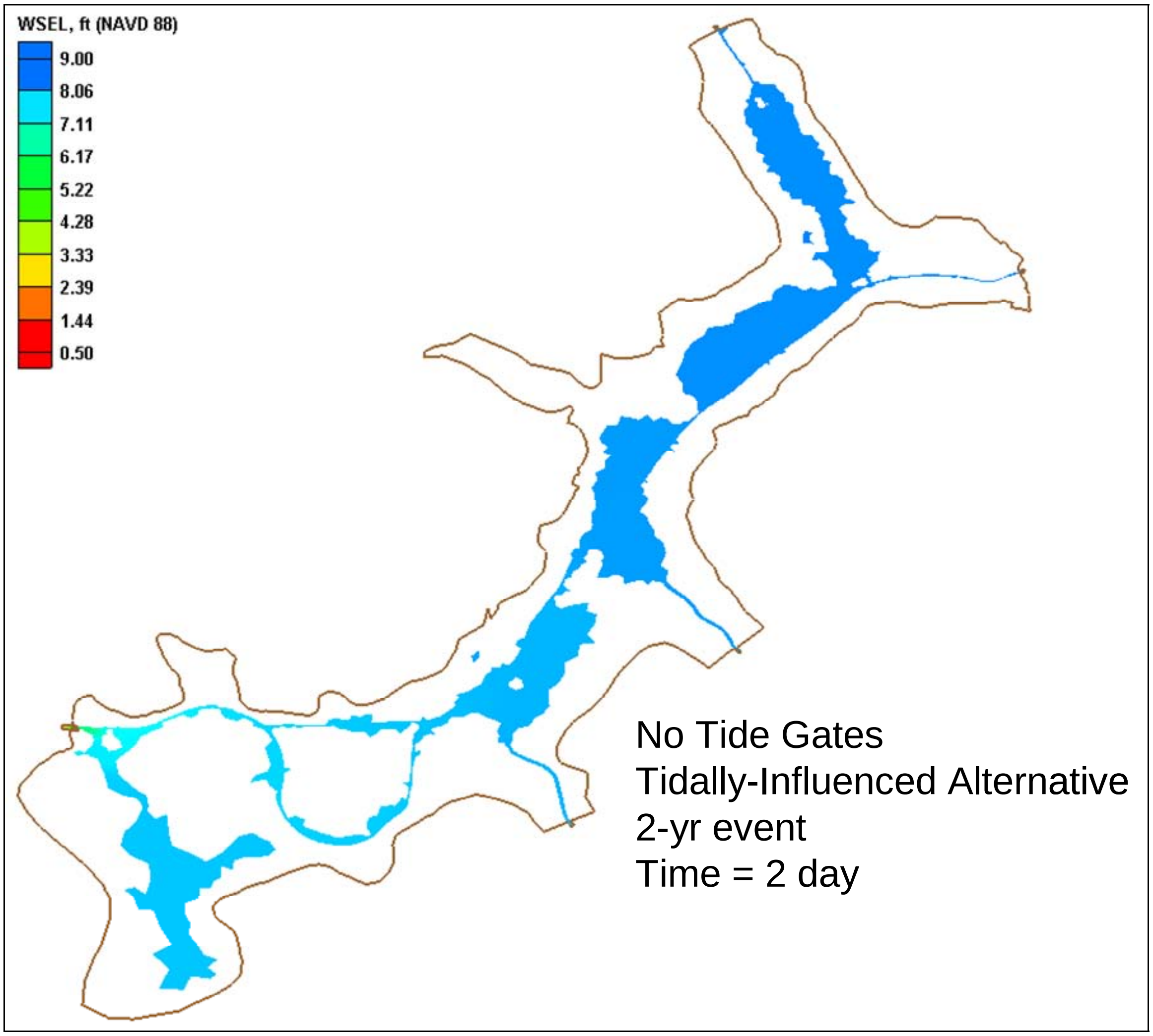

Figure 27. Inundation at time = 2 day for the 2-yr event, No Tidegates Tidally-Influenced, Alternative #2

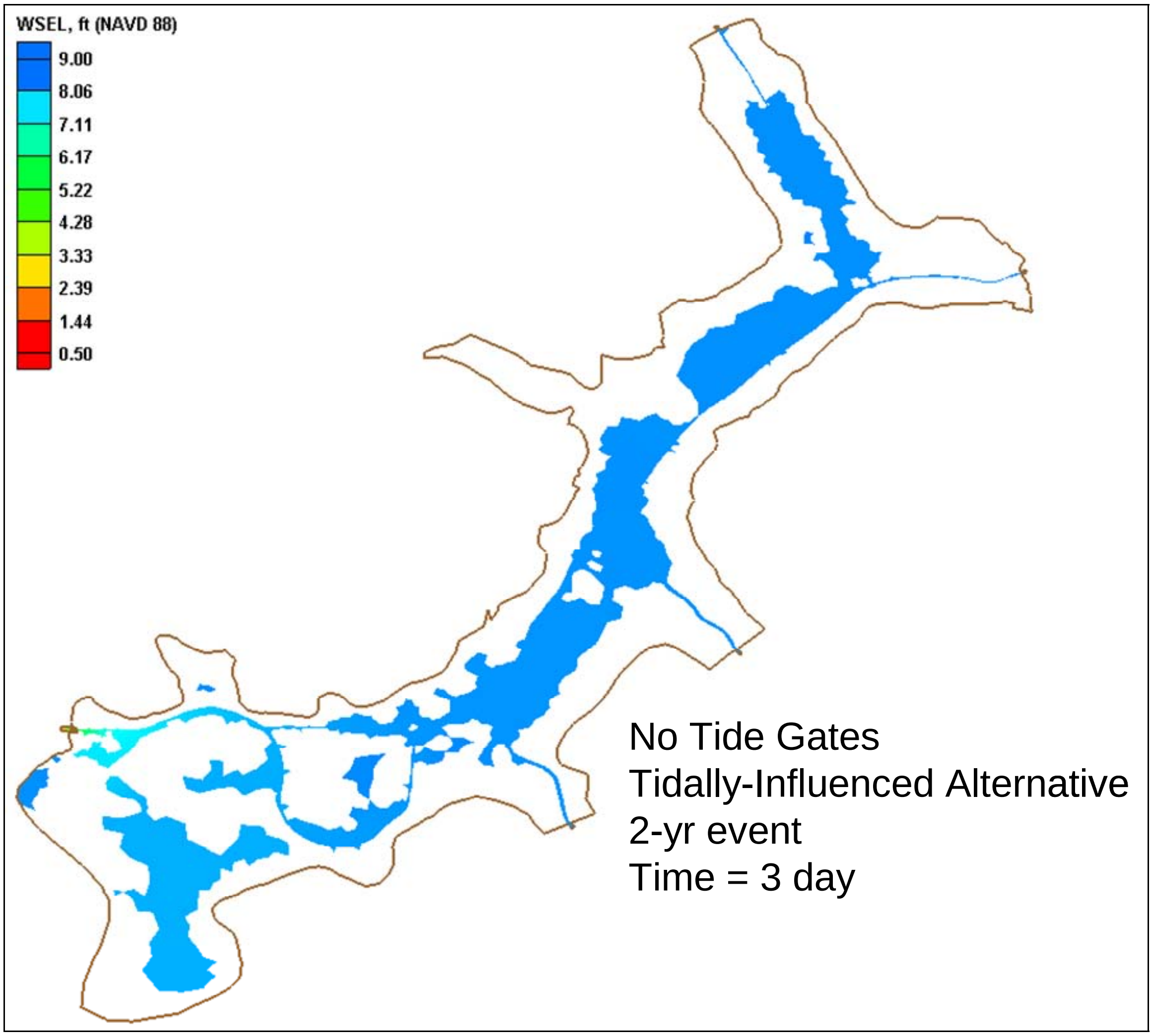

Figure 28. Inundation at time = 3 day for the 2-yr event, No Tidegates Tidally-Influenced, Alternative #2

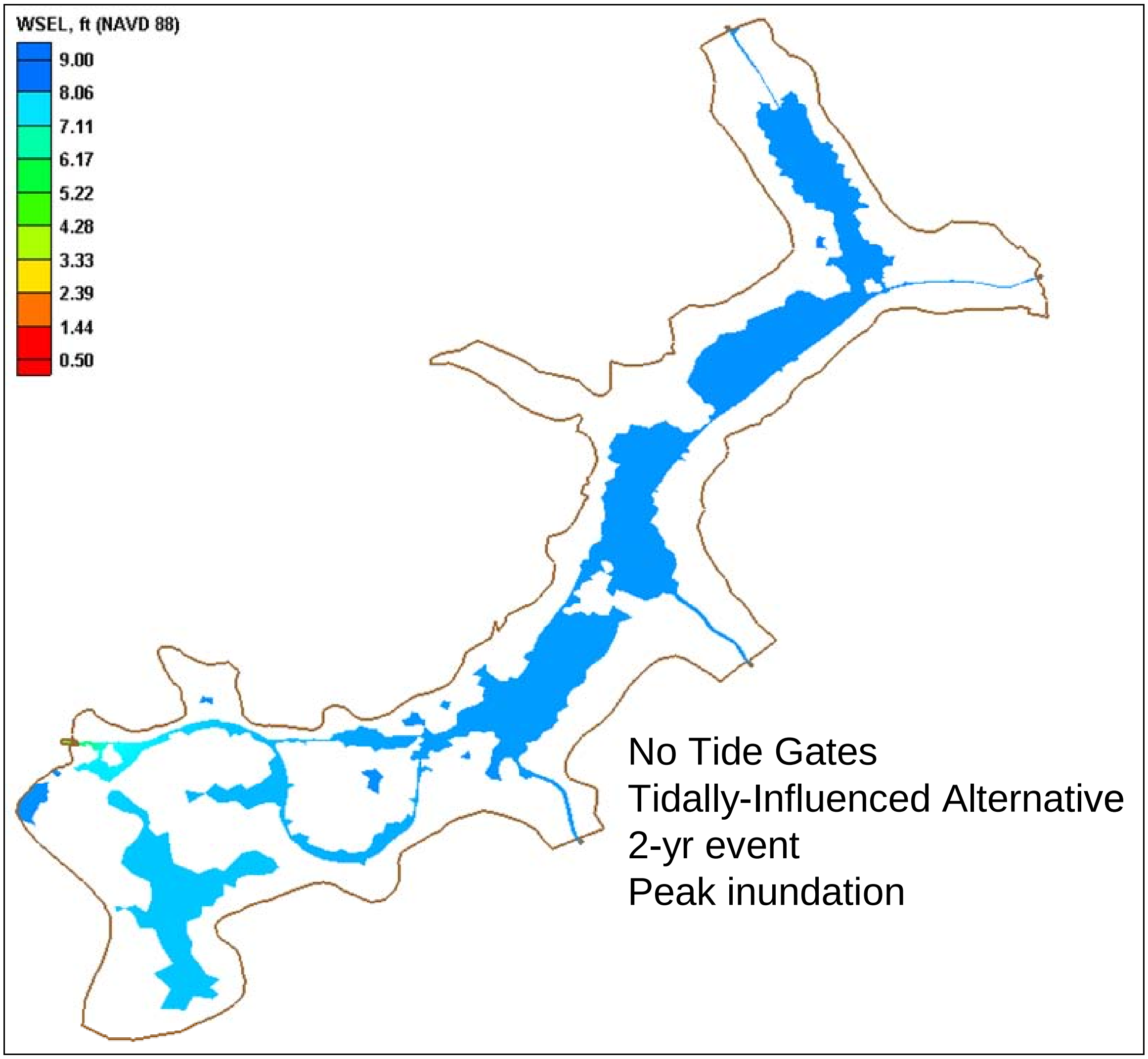

Figure 29. Inundation at time = 5 day for the 2-yr event, No Tidegates Tidally-Influenced, Alternative #2

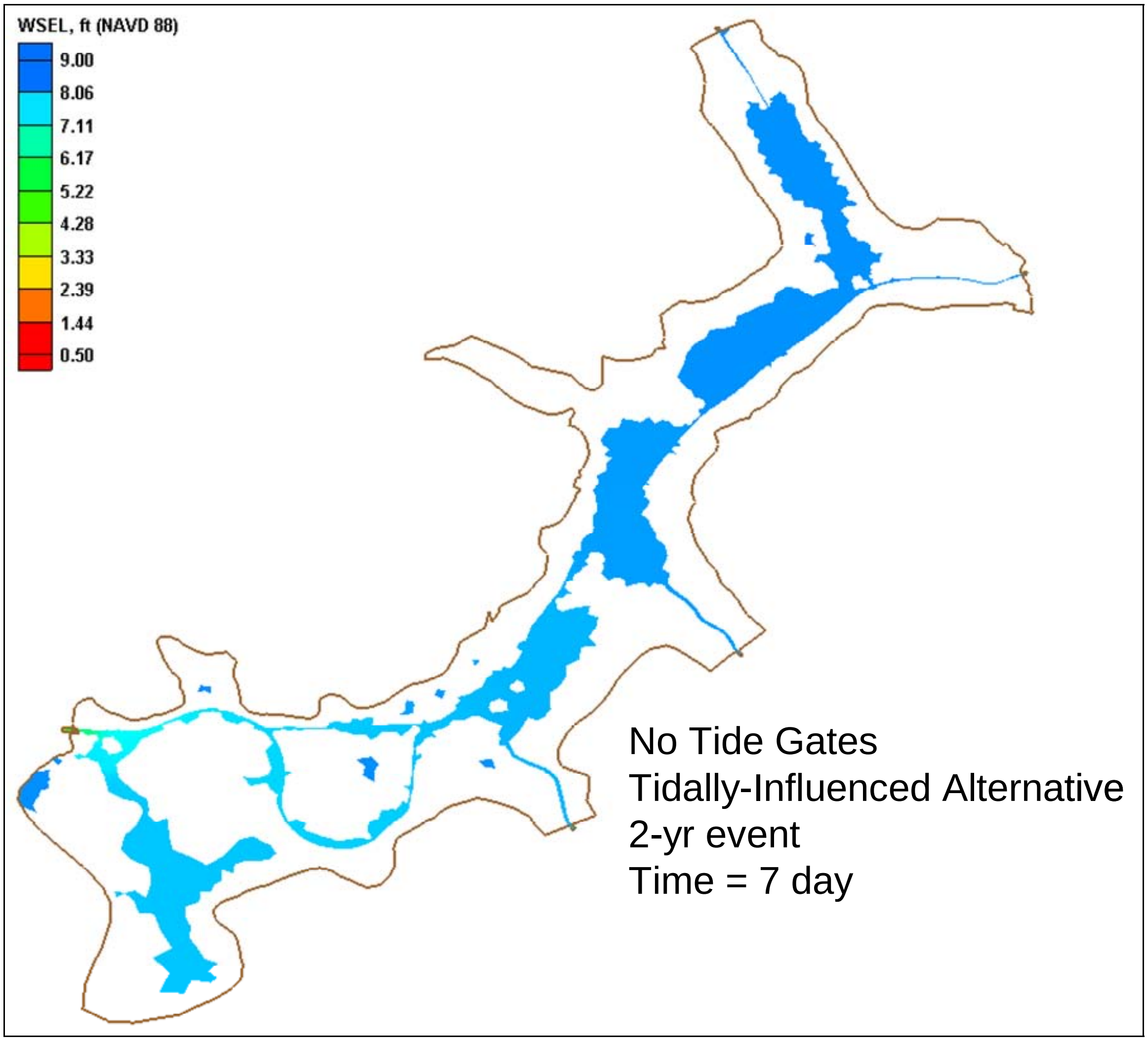

Figure 30. Inundation at time = 7 day for the 2-yr event, No Tidegates Tidally-Influenced, Alternative #2

Plan view plots of inundation for the No Tidegates Tidally-Influenced Alternatives are given in Figures 31 through 36 for the peak inundation and elapsed times of 1, 2, 3, 5, and 7 days, respectively, for the 10-year event. During peak inundation, flooding for this event takes place over most of the study area. Inundation levels are notably reduced after 2 days, but significant standing water remains in the upstream areas and downstream pasture at 7 days. Free propagation of the tidal wave into the study area, combined with higher tidal elevations resulted in prolonged inundation over all areas that experienced flooding. . Inundation expanse and duration are greater than that for the No Action Alternative for the 10-year event.

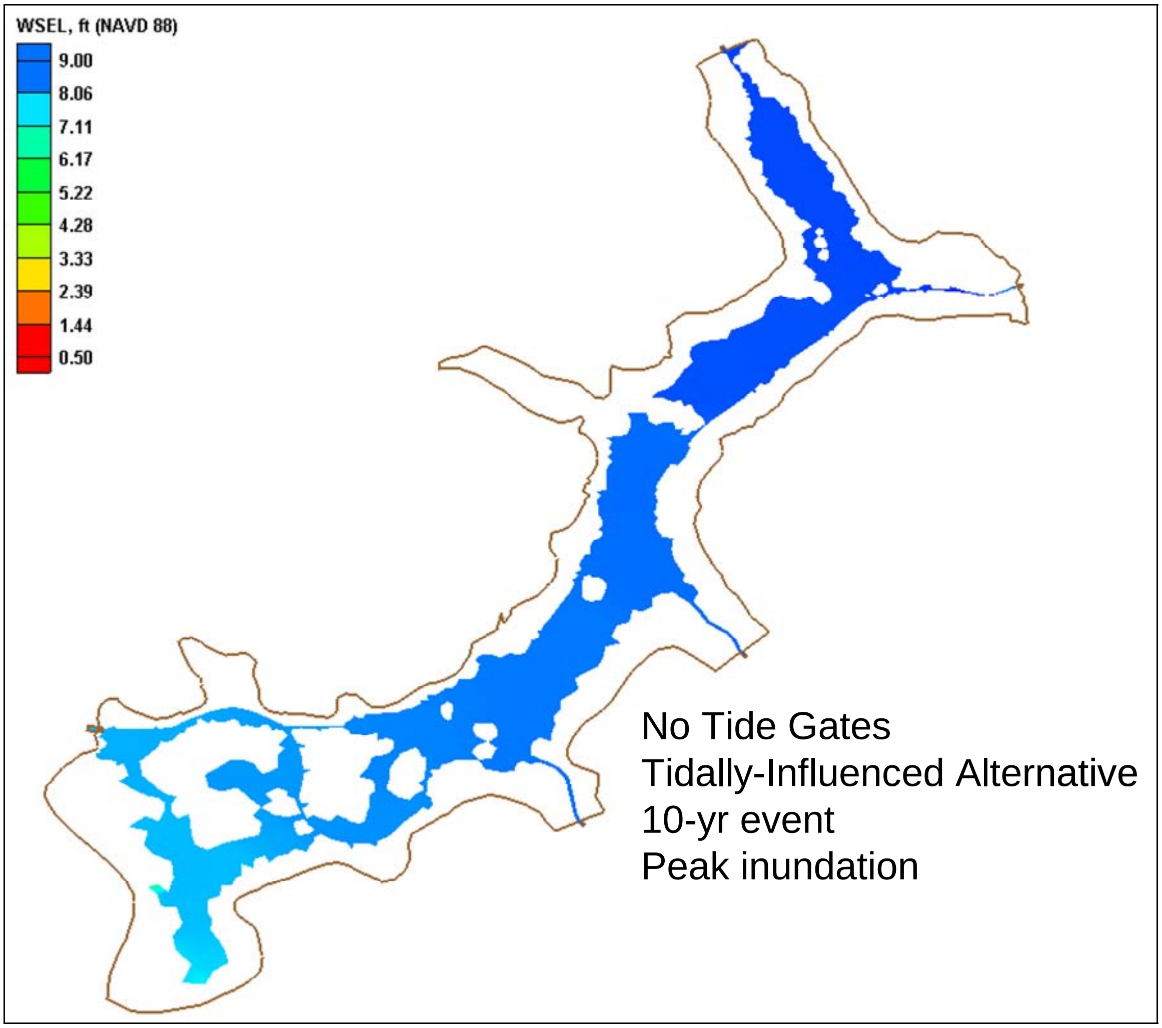

Figure 31. Peak inundation for the 10-yr event, No Tidegates Tidally-Influenced, Alternative #2

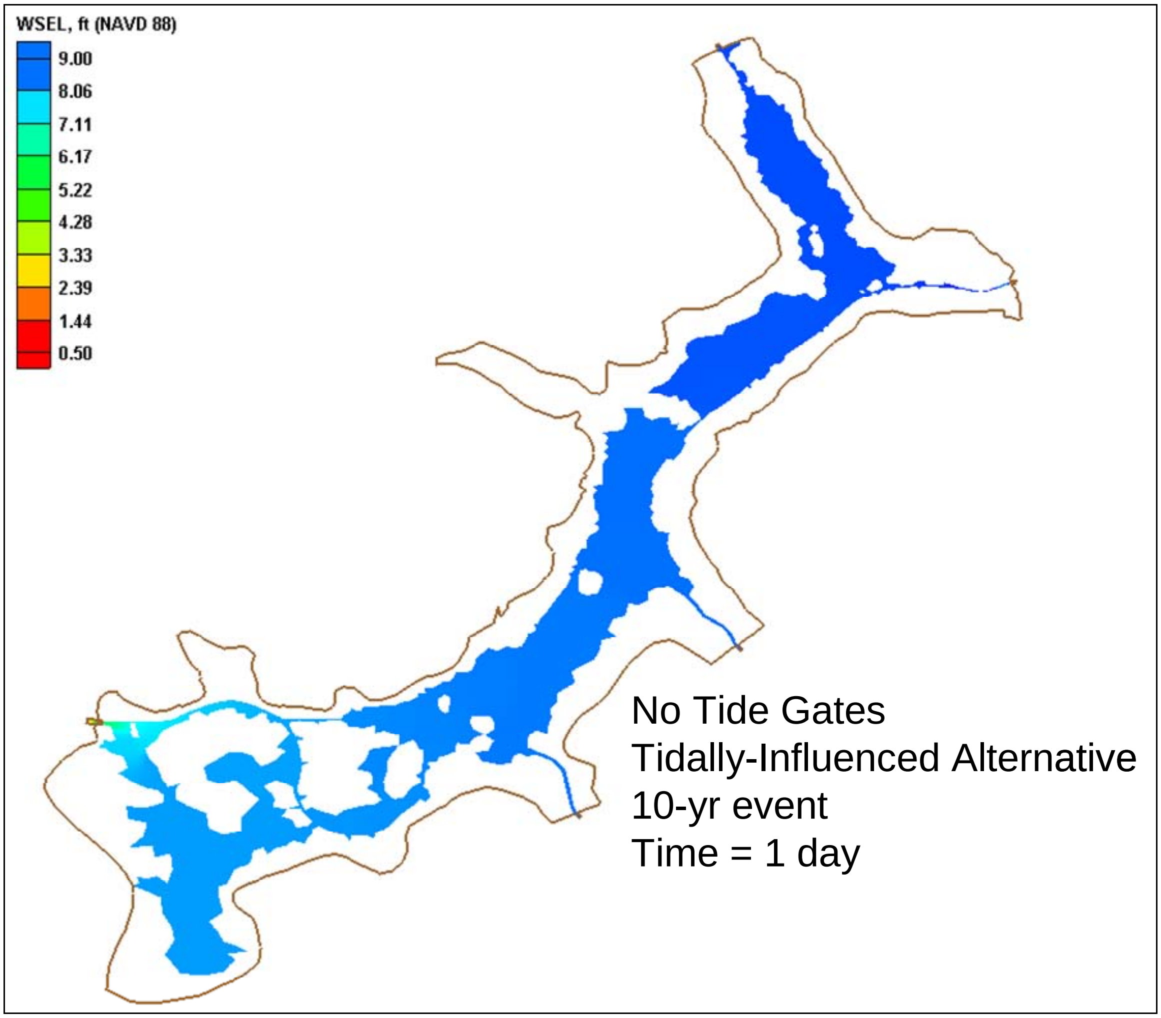

Figure 32. Inundation at time = 1 day for the 10-yr event, No Tidegates Tidally-Influenced, Alternative #2

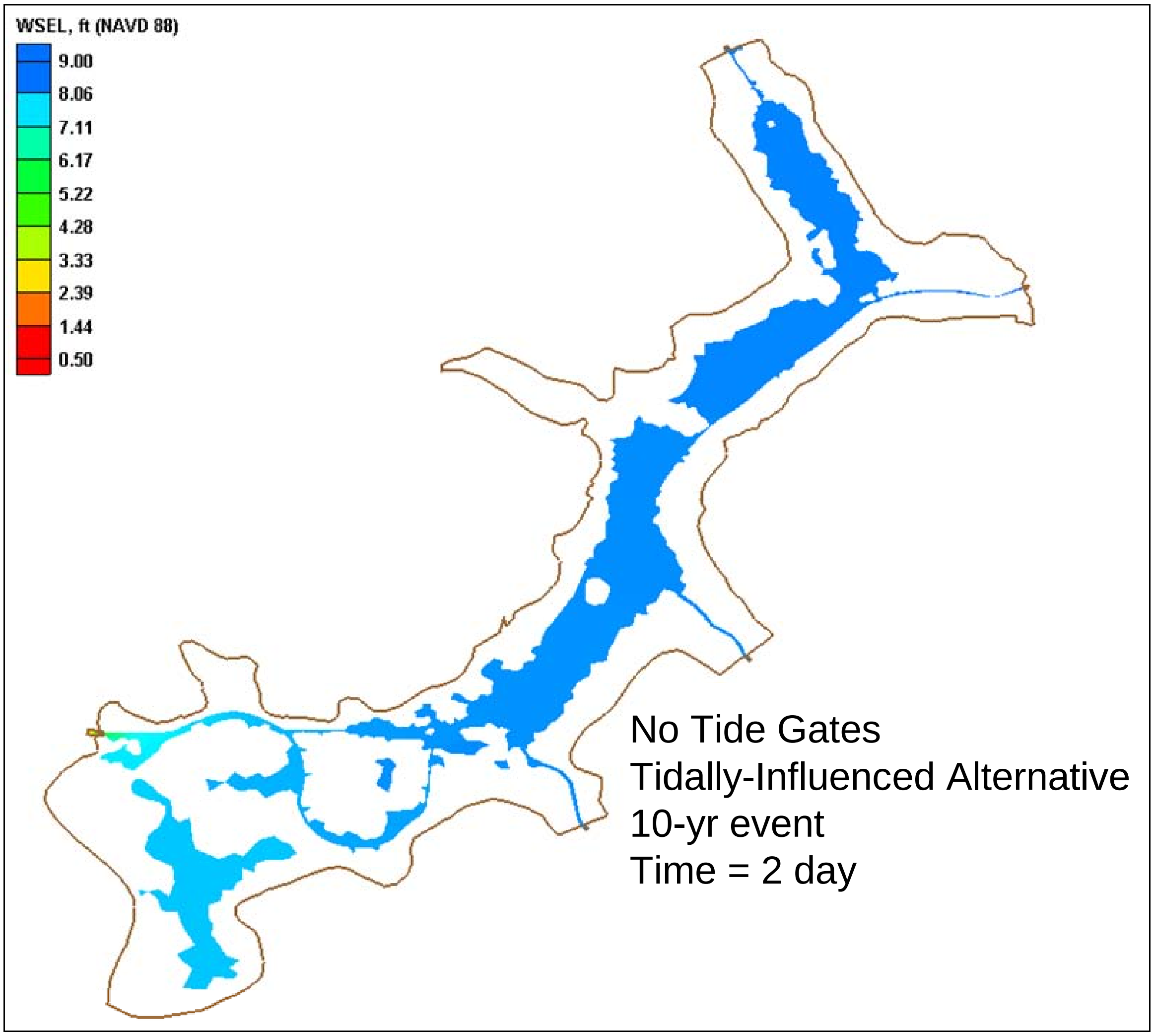

Figure 33. Inundation at time = 2 day for the 10-yr event, No Tidegates Tidally-Influenced, Alternative #2

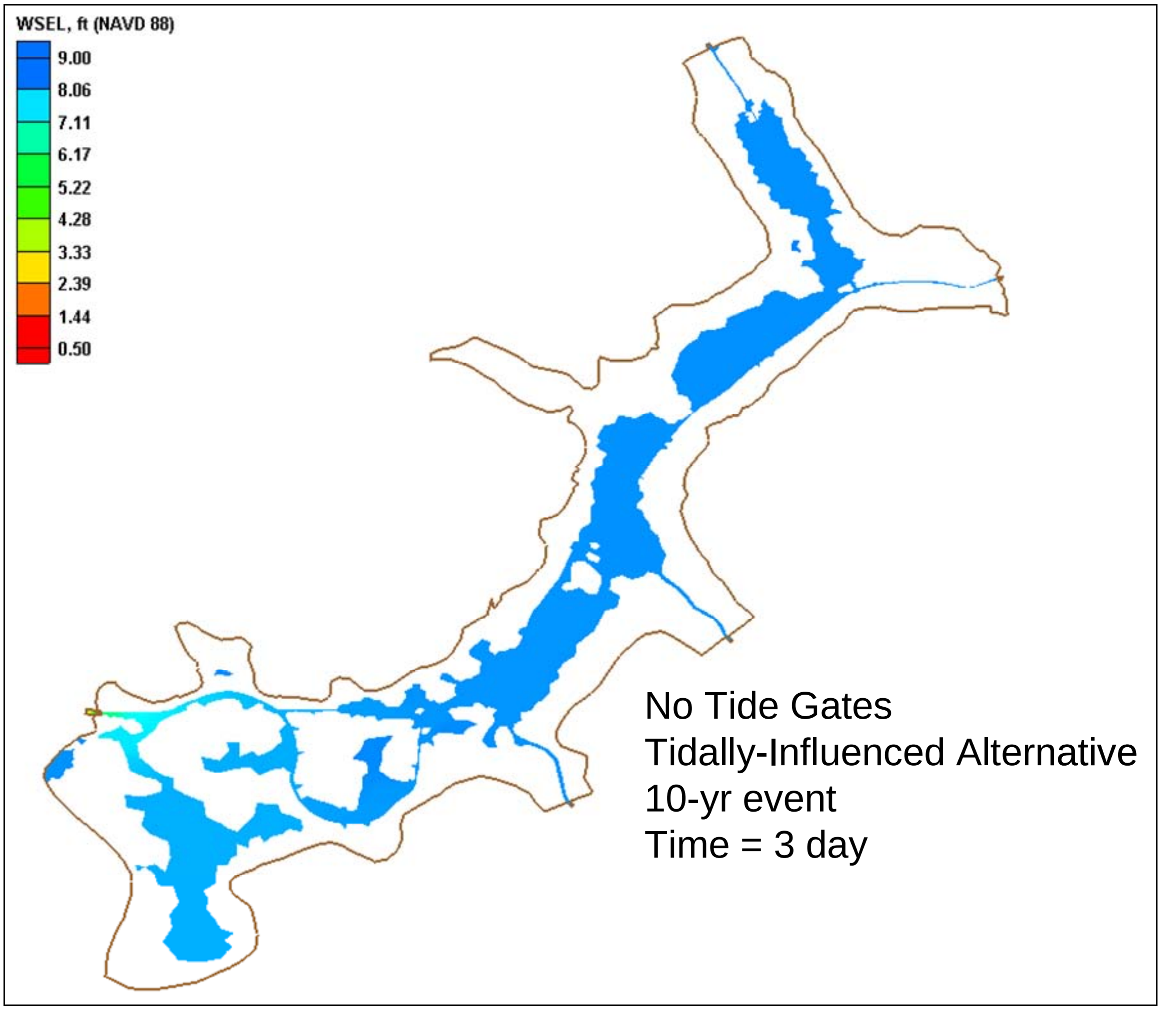

Figure 34. Inundation at time = 3 day for the 10-yr event, No Tidegates Tidally-Influenced, Alternative #2

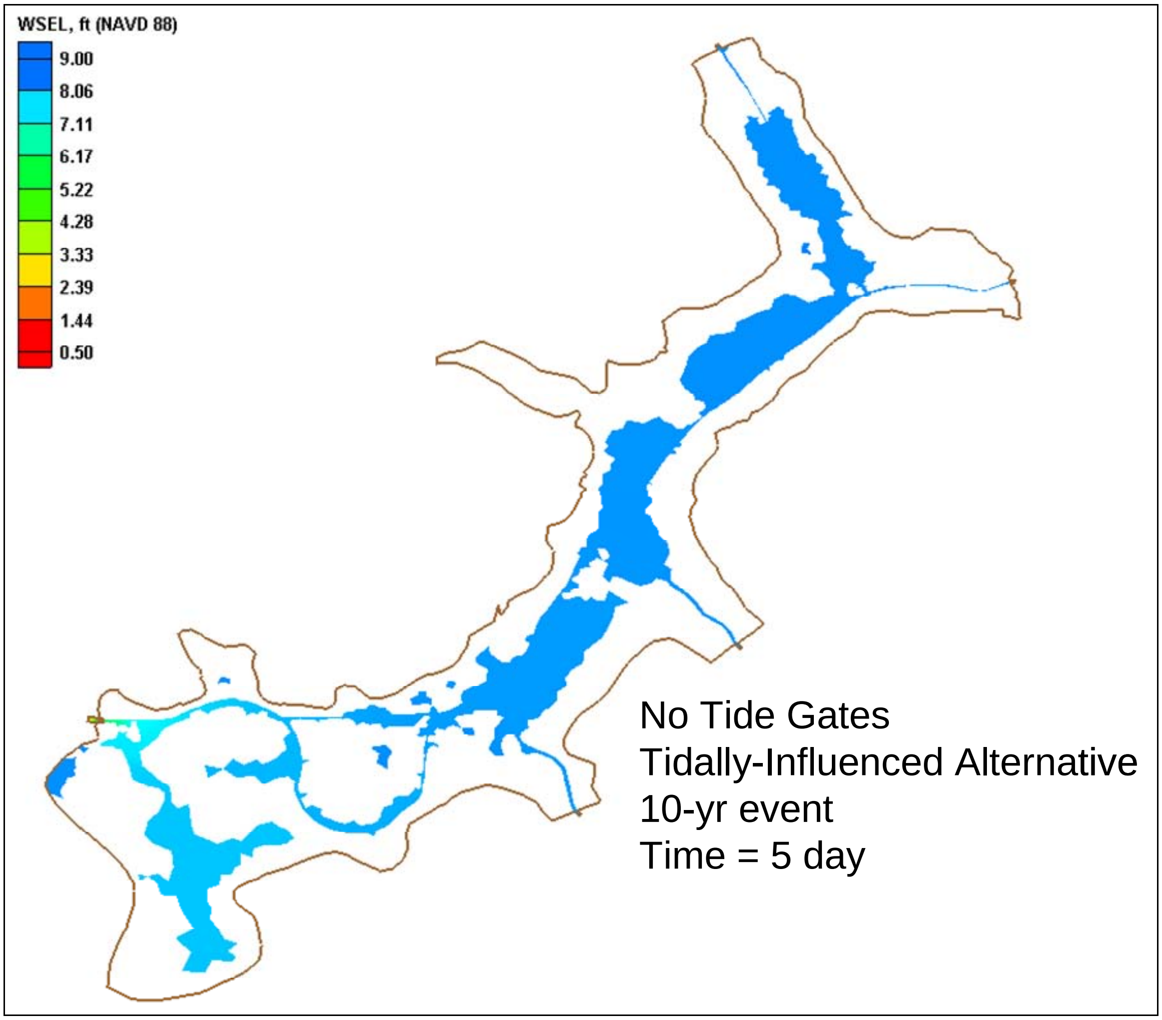

Figure 35. Inundation at time = 5 day for the 10-yr event, No Tidegates Tidally-Influenced, Alternative #2

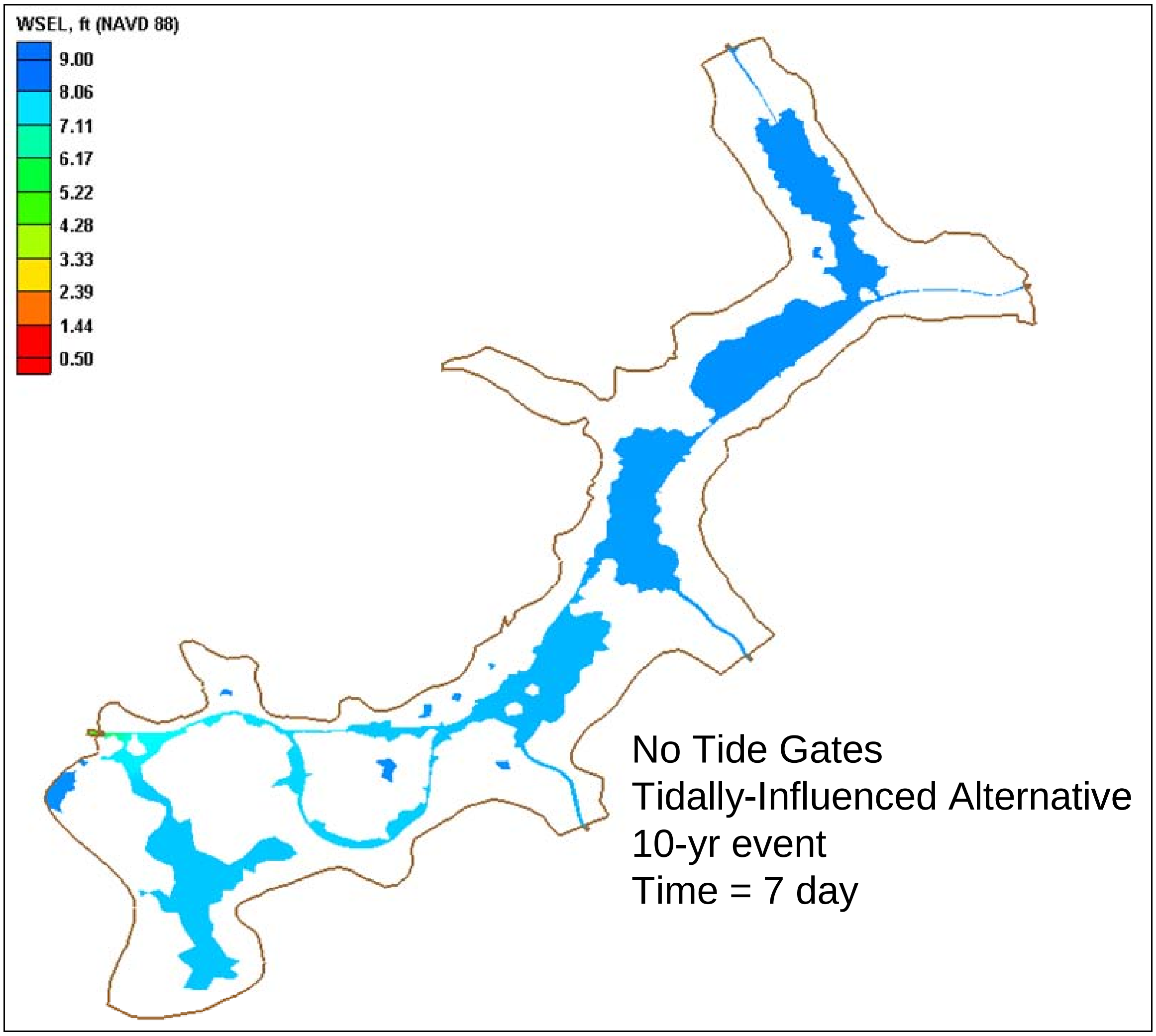

Figure 36. Inundation at time = 7 day for the 10-yr event, No Tidegates Tidally-Influenced, Alternative #2

Modified Tidegates and Additional Storage Ponds, Alternative #3

Plan view plots of inundation for the Modified Tidegates and Additional Storage Ponds Alternative are given in Figures 37 through 42 for the peak inundation and elapsed times of 1, 2, 3, 5, and 7 days, respectively, for the 2-year event. During peak inundation, flooding for this event takes place primarily upstream of the large channel meander and in the pasture downstream of the meander. After 1 day, there is a notable reduction in inundation just upstream of the meander. After 2 days, inundation has been reduced significantly in all areas with the exception of the downstream pasture. Strong inundation reduction continues up to Day 3, and then tapers off such that there is minimal change through Day 7. Inundation expanse and duration are comparable to the No-Action Alternative for the 2-year event, except that the downstream pasture experiences greater flooding for this alternative. This increase in flooding in the downstream pasture could probably be alleviated by digging a channel connecting the inundated area to the downstream pond to improve drainage.

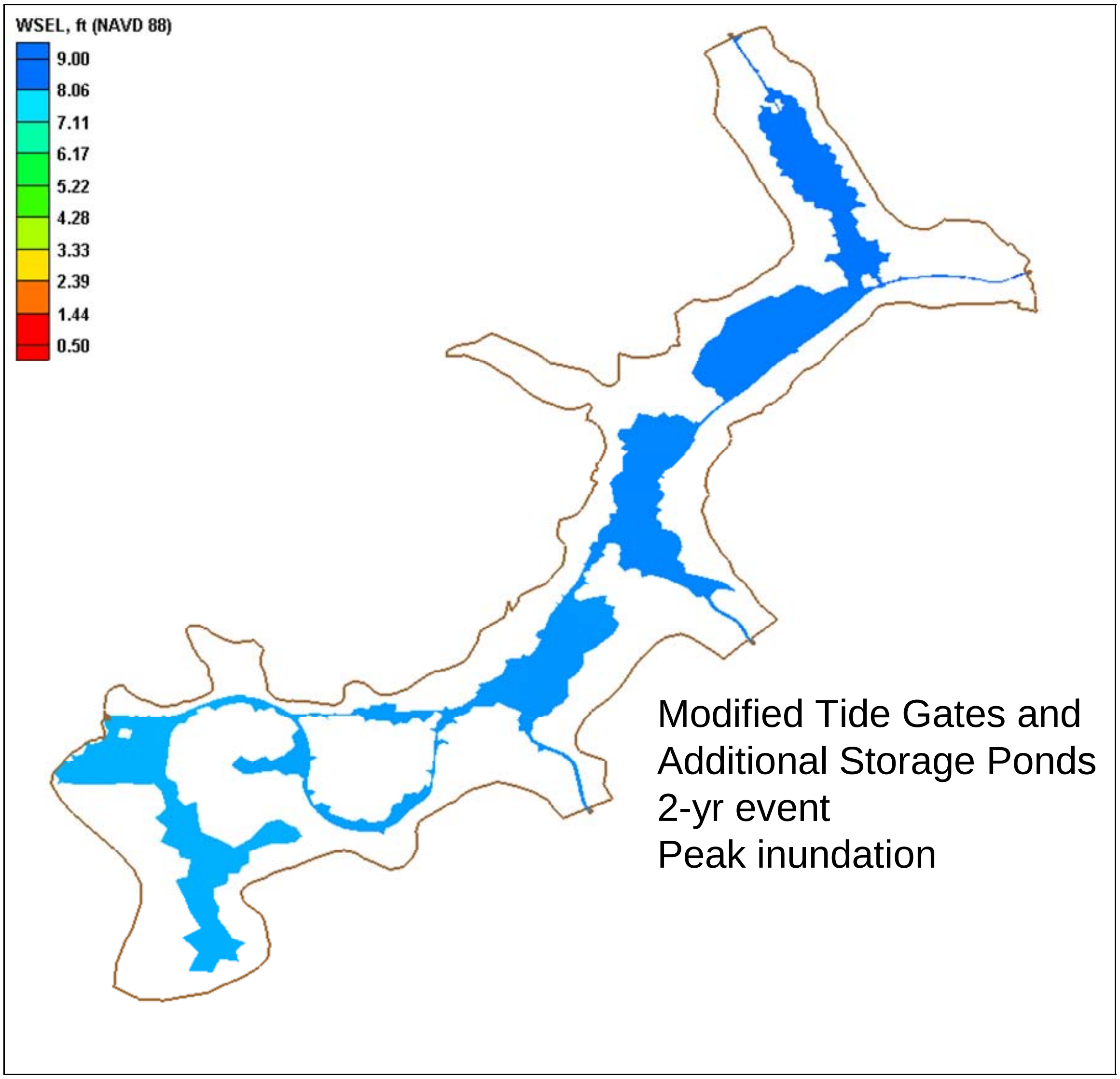

Figure 37. Peak inundation for the 2-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

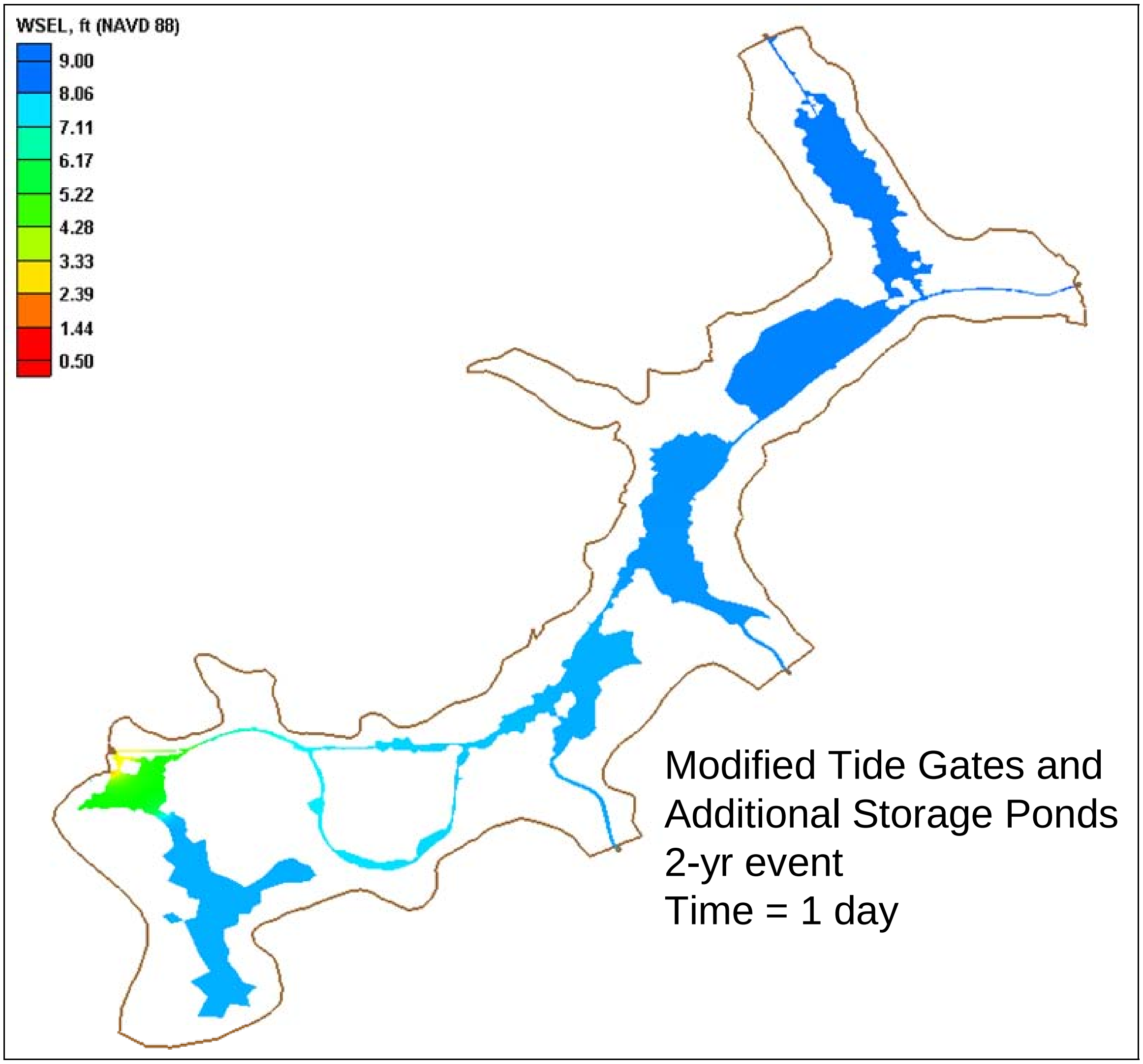

Figure 38. Inundation at time = 1 day for the 2-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

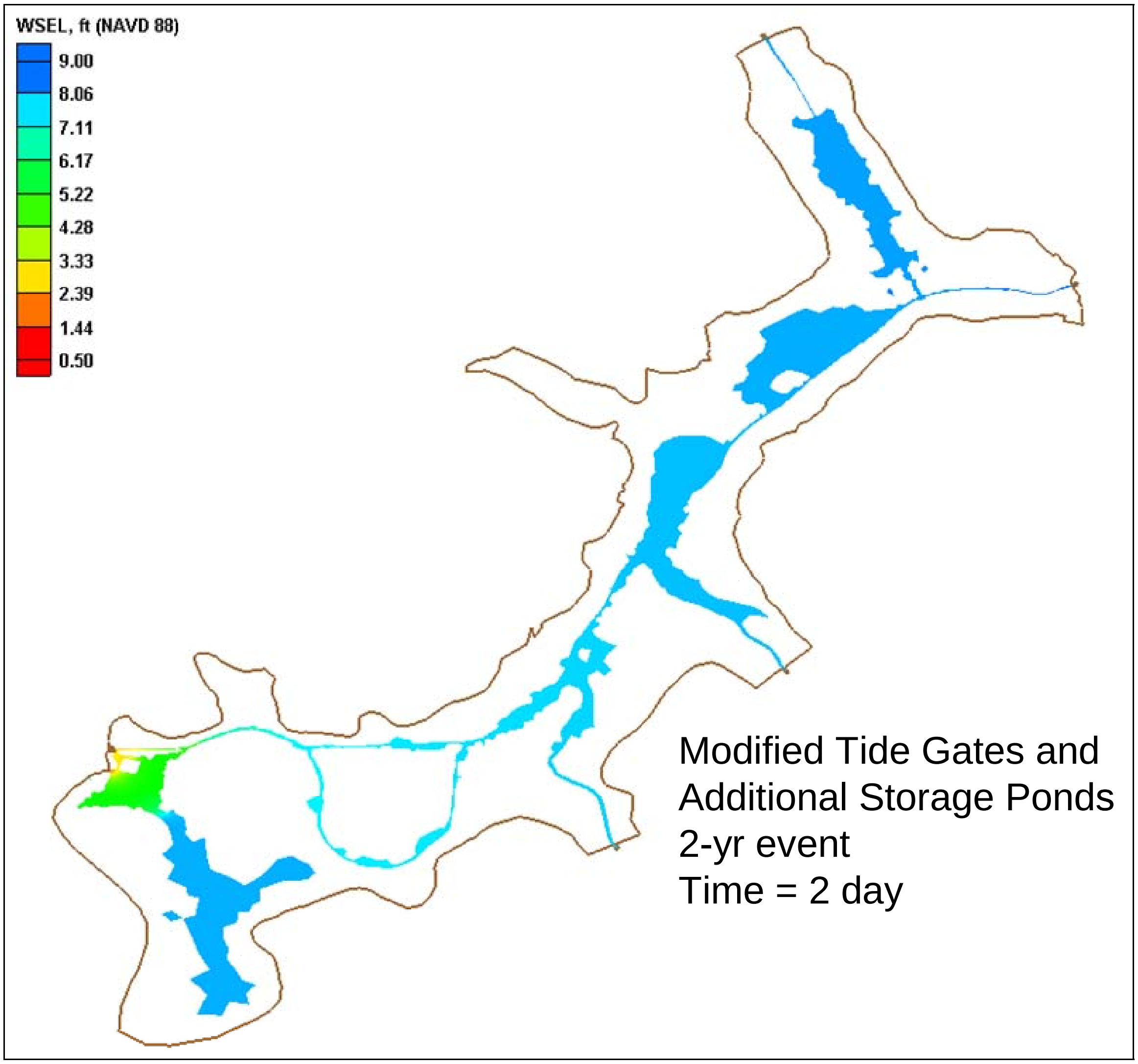

Figure 39. Inundation at time = 2 day for the 2-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

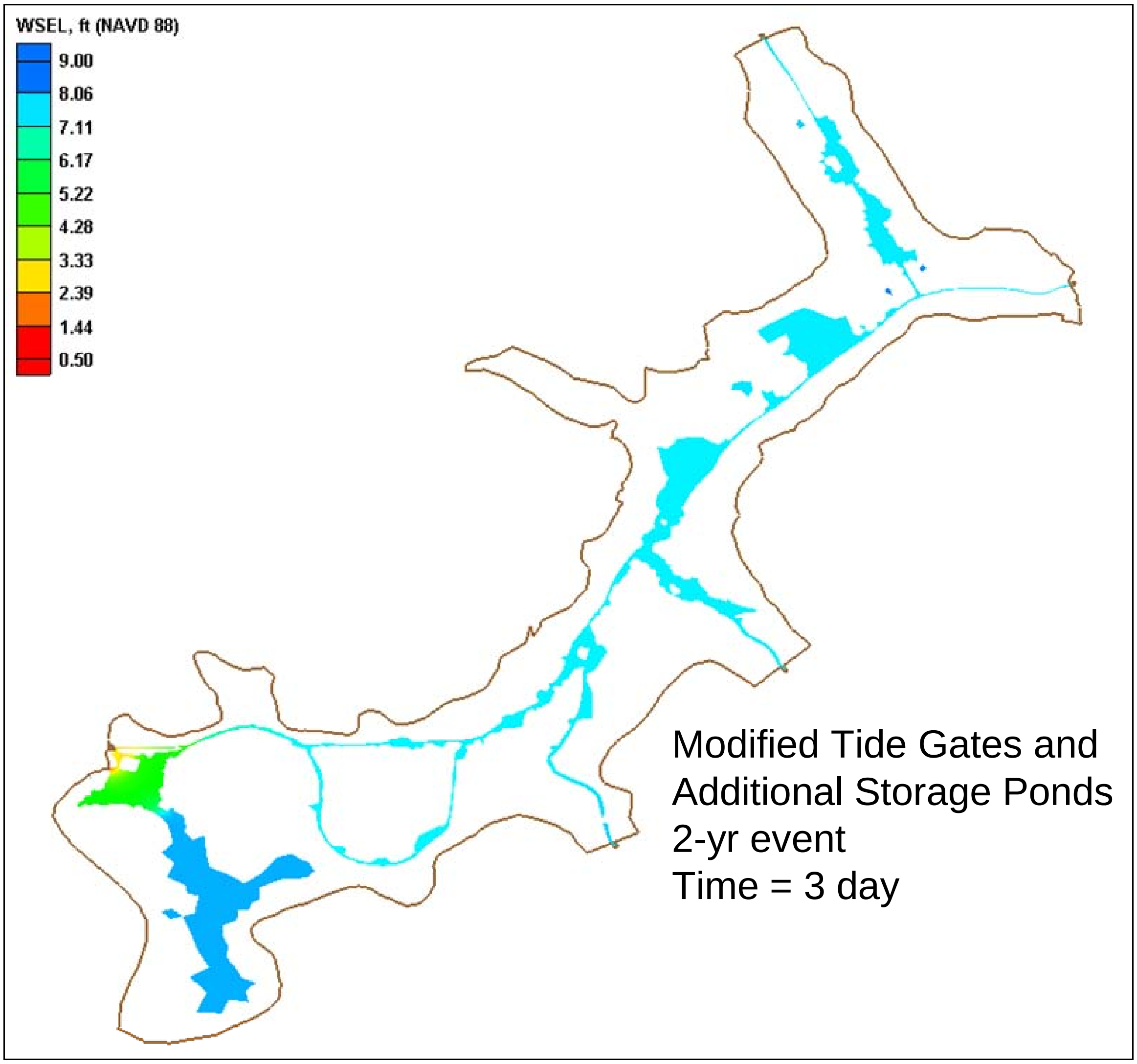

Figure 40. Inundation at time = 3 day for the 2-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

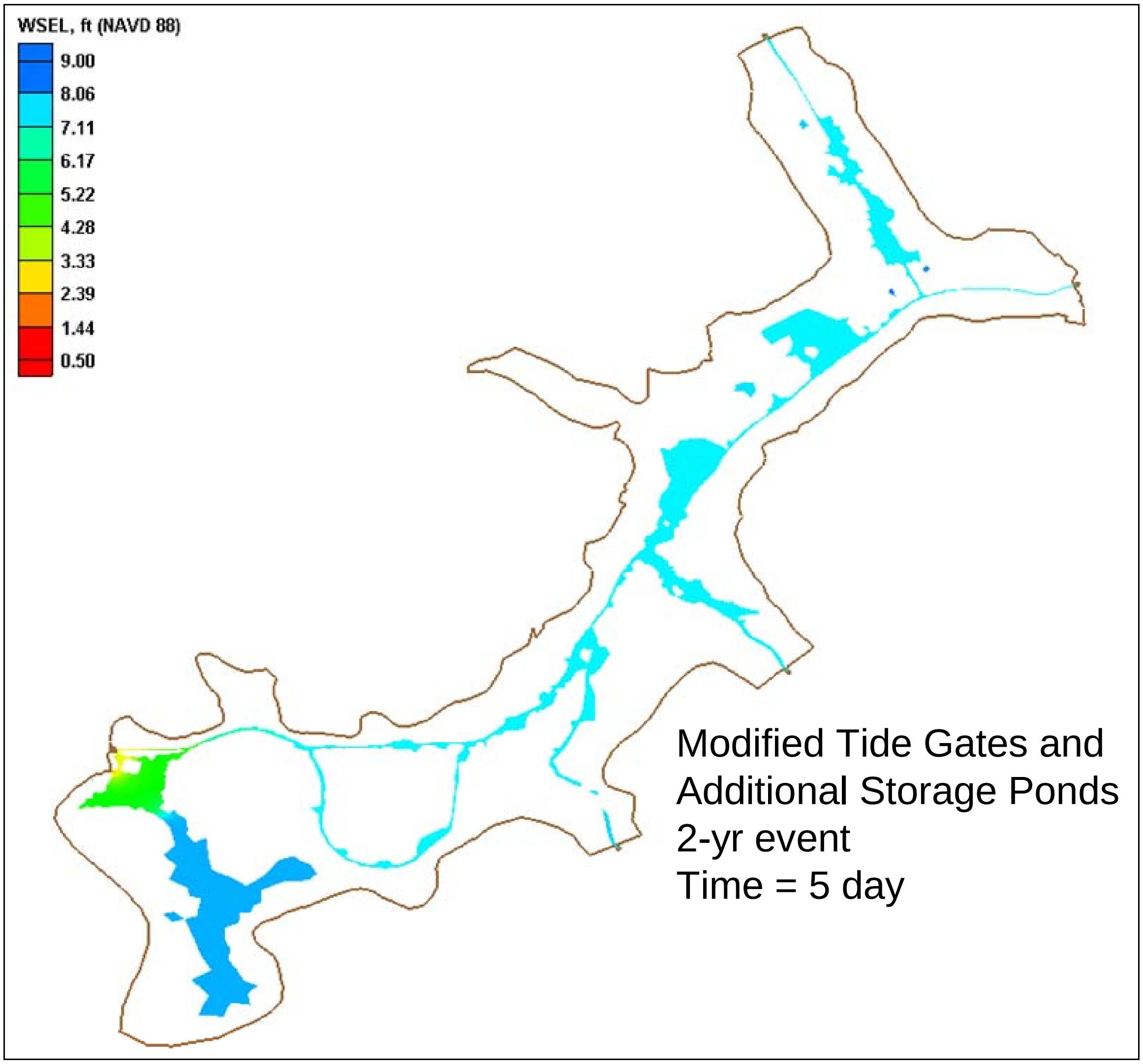

Figure 41. Inundation at time = 5 day for the 2-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

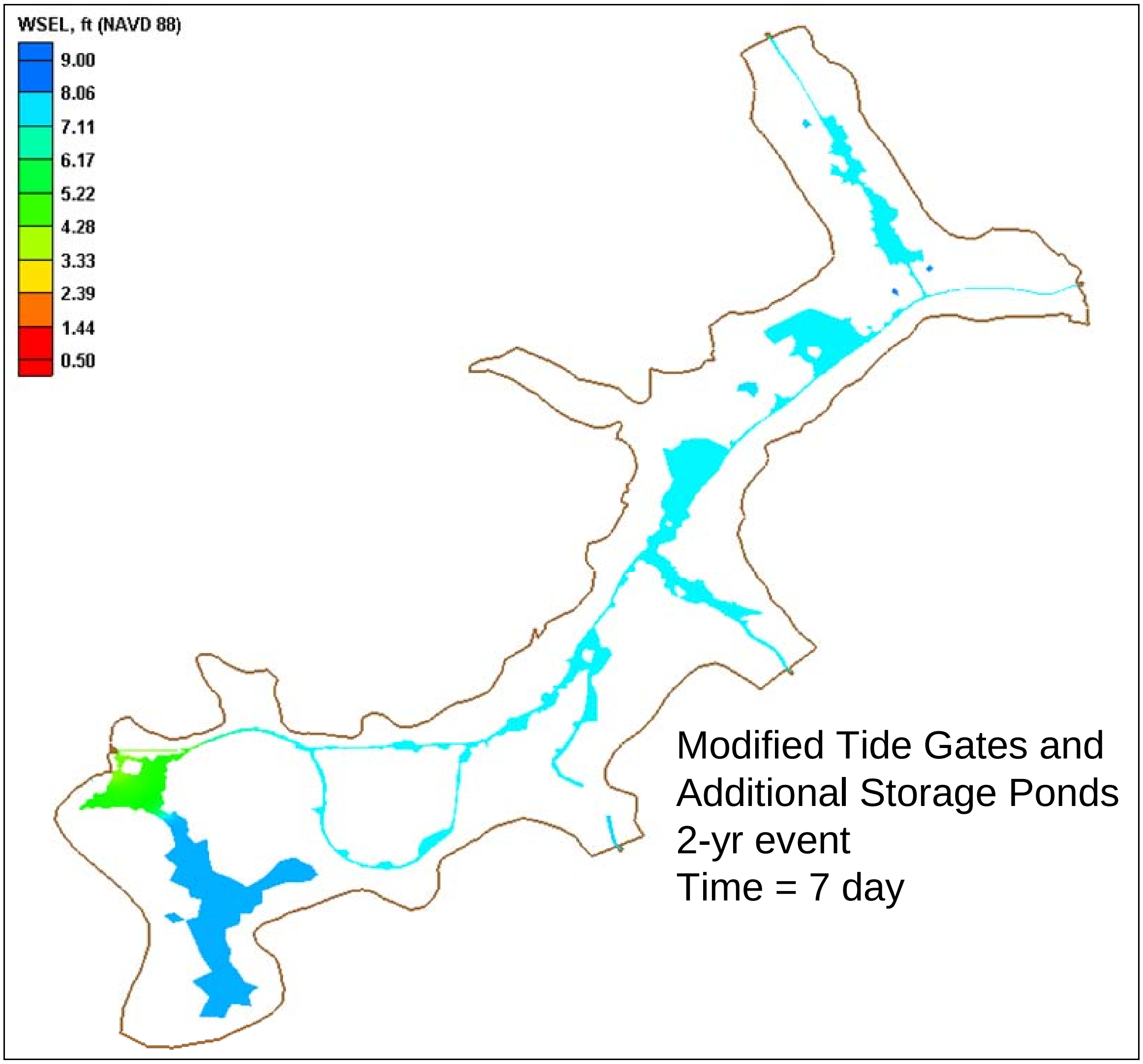

Figure 42. Inundation at time = 7 day for the 2-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

Plan view plots of inundation for the Modified Tidegates and Additional Storage Ponds Alternative are given in Figures 43 through 48 for the peak inundation and elapsed times of 1, 2, 3, 5, and 7 days, respectively, for the 2-year event. During peak inundation, flooding for this event takes place upstream of the large channel meander and in the pasture downstream of the meander. After 1 day, there is a notable reduction in inundation near to and downstream of the meander. After 2 days, inundation has been reduced in all areas. Between Days 3 and 7, inundation is slowly reduced. Inundation expanse and duration are generally comparable to the No-Action Alternative for the 10-year event, except that the downstream pasture and vicinity of the channel meander experience significantly less flooding for this alternative. Upstream of the channel meander, this alternative produces a slightly smaller inundation area, as compared to the No-Action Alternative, but the relative difference is not significant.

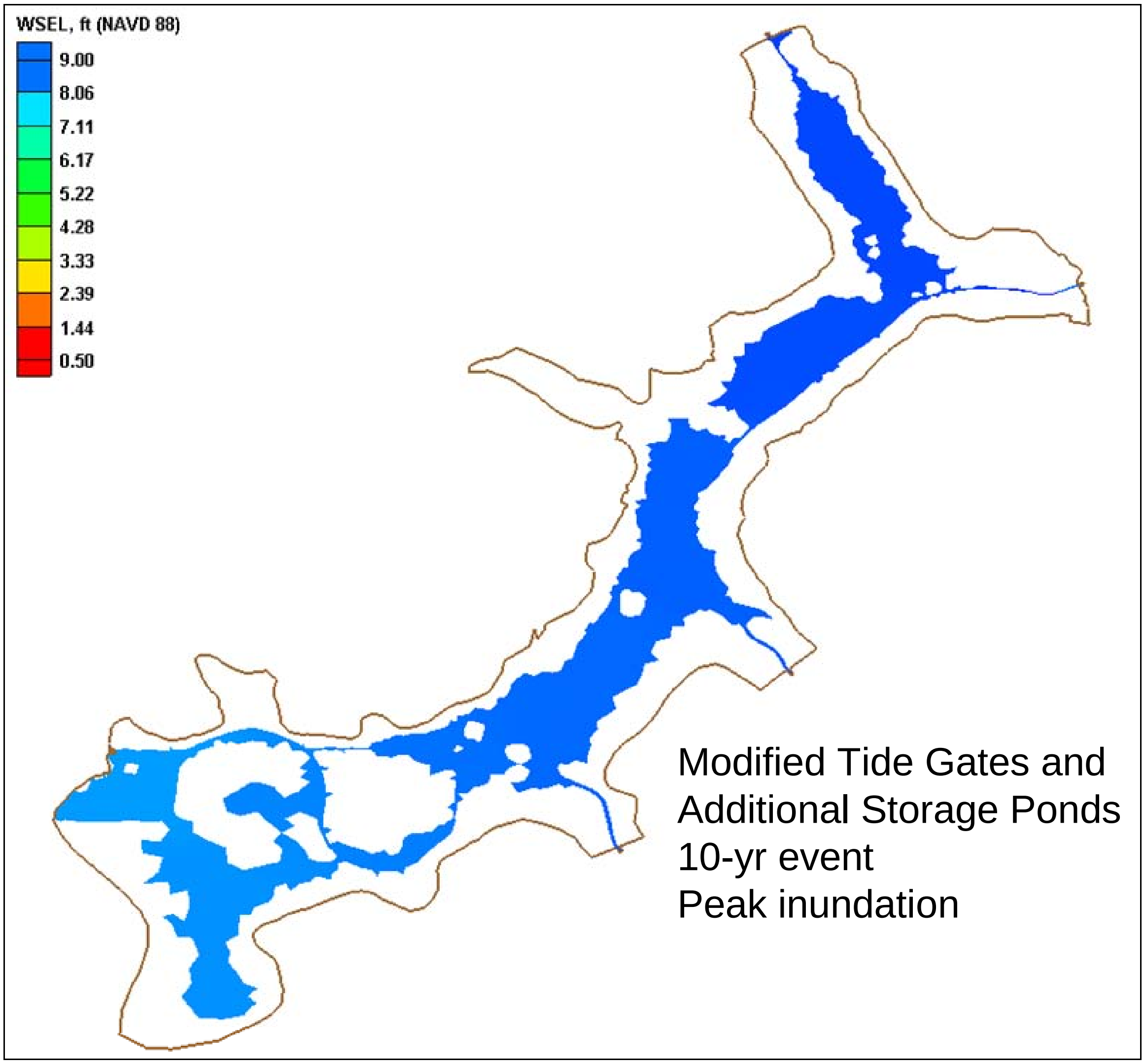

Figure 43. Peak inundation for the 10-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

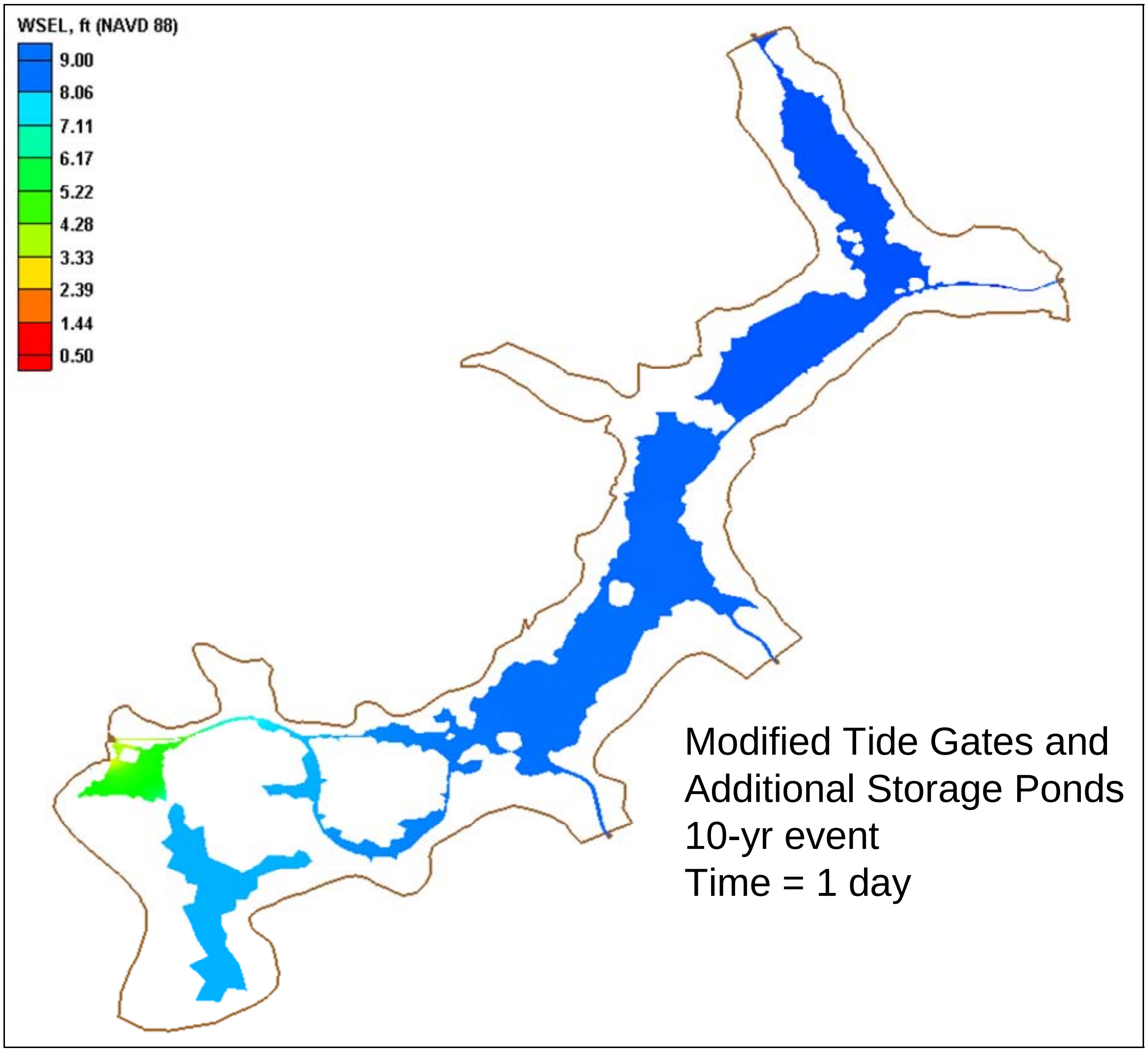

Figure 44. Inundation at time = 1 day for the 10-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

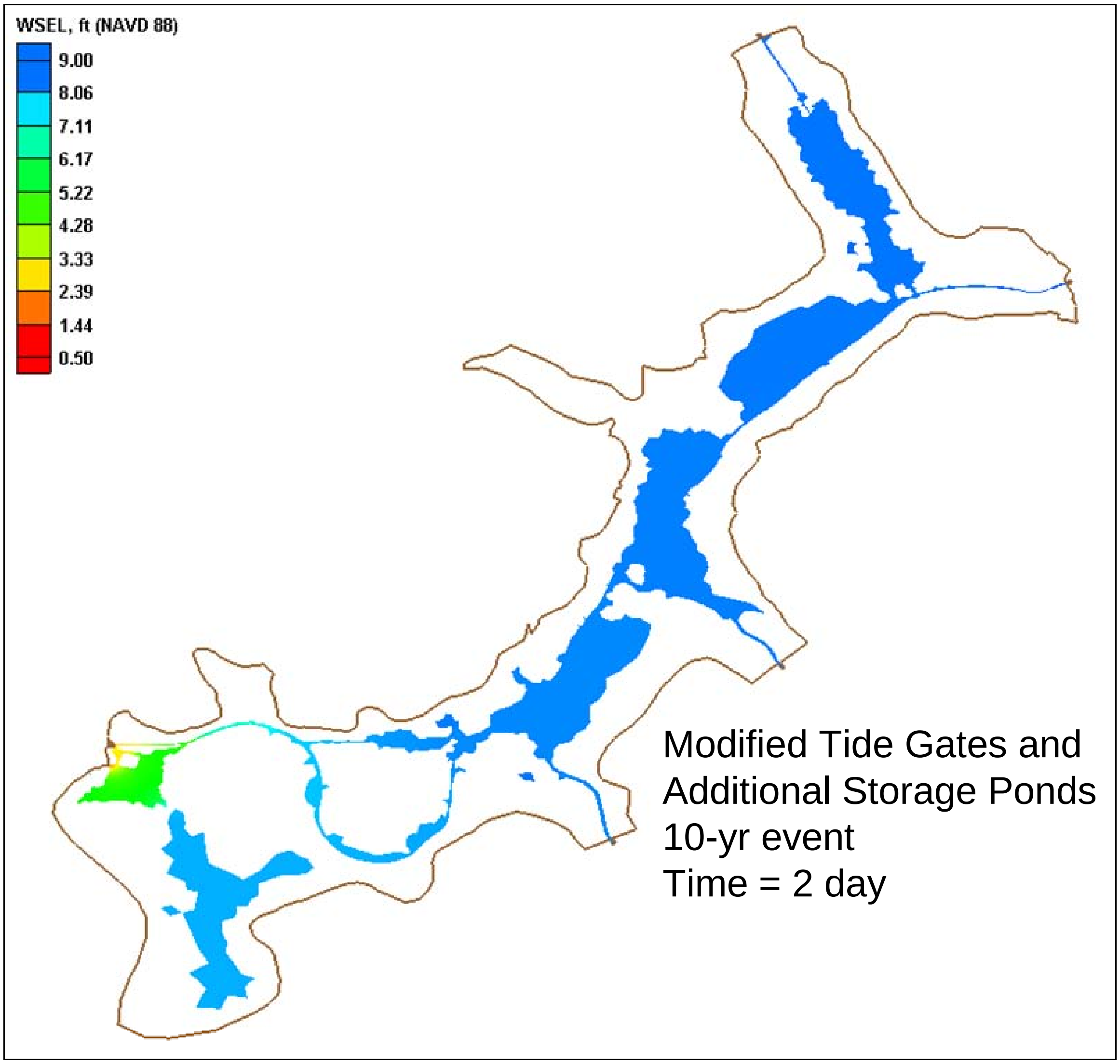

Figure 45. Inundation at time = 2 day for the 10-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

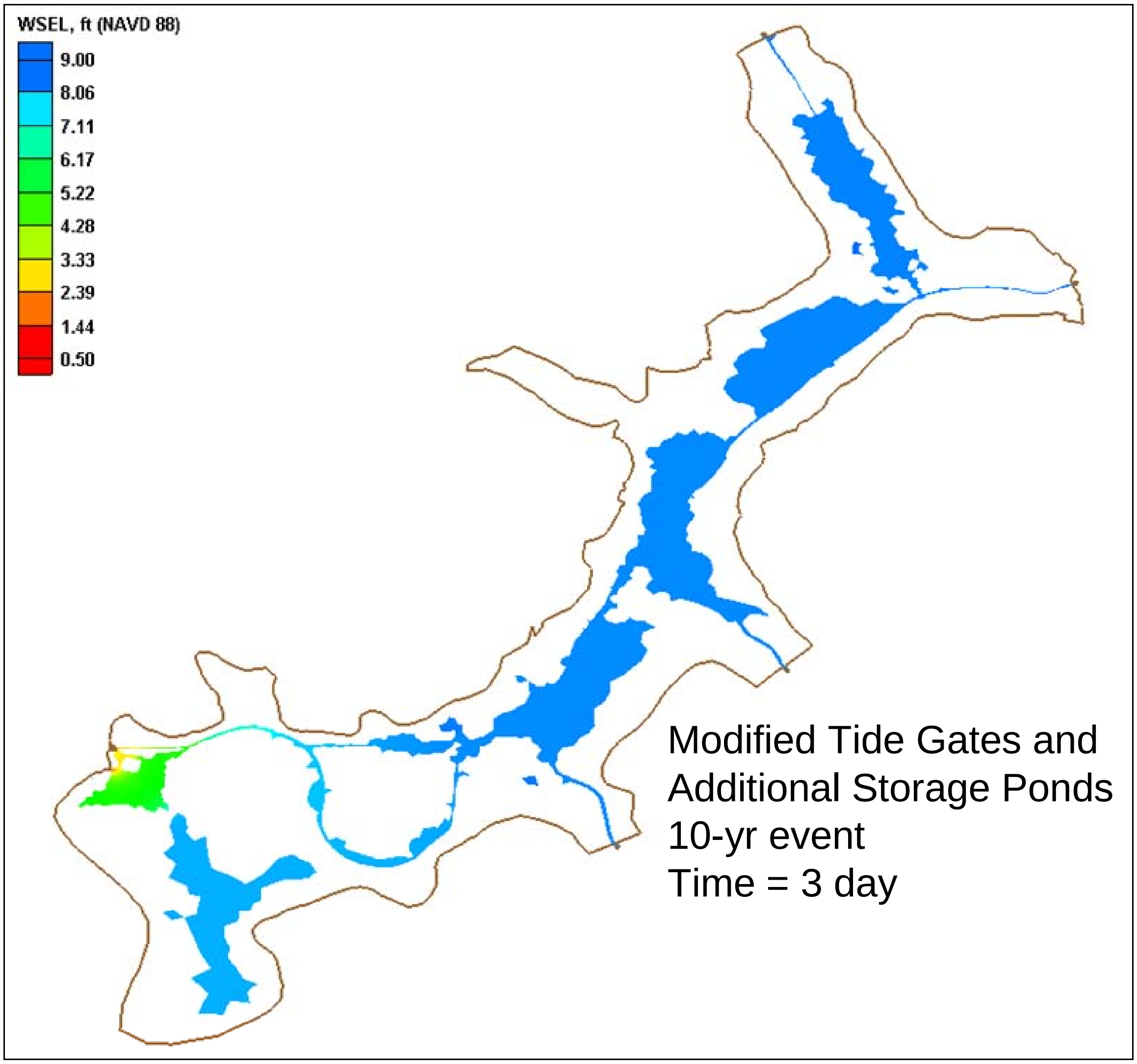

Figure 46. Inundation at time = 3 day for the 10-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

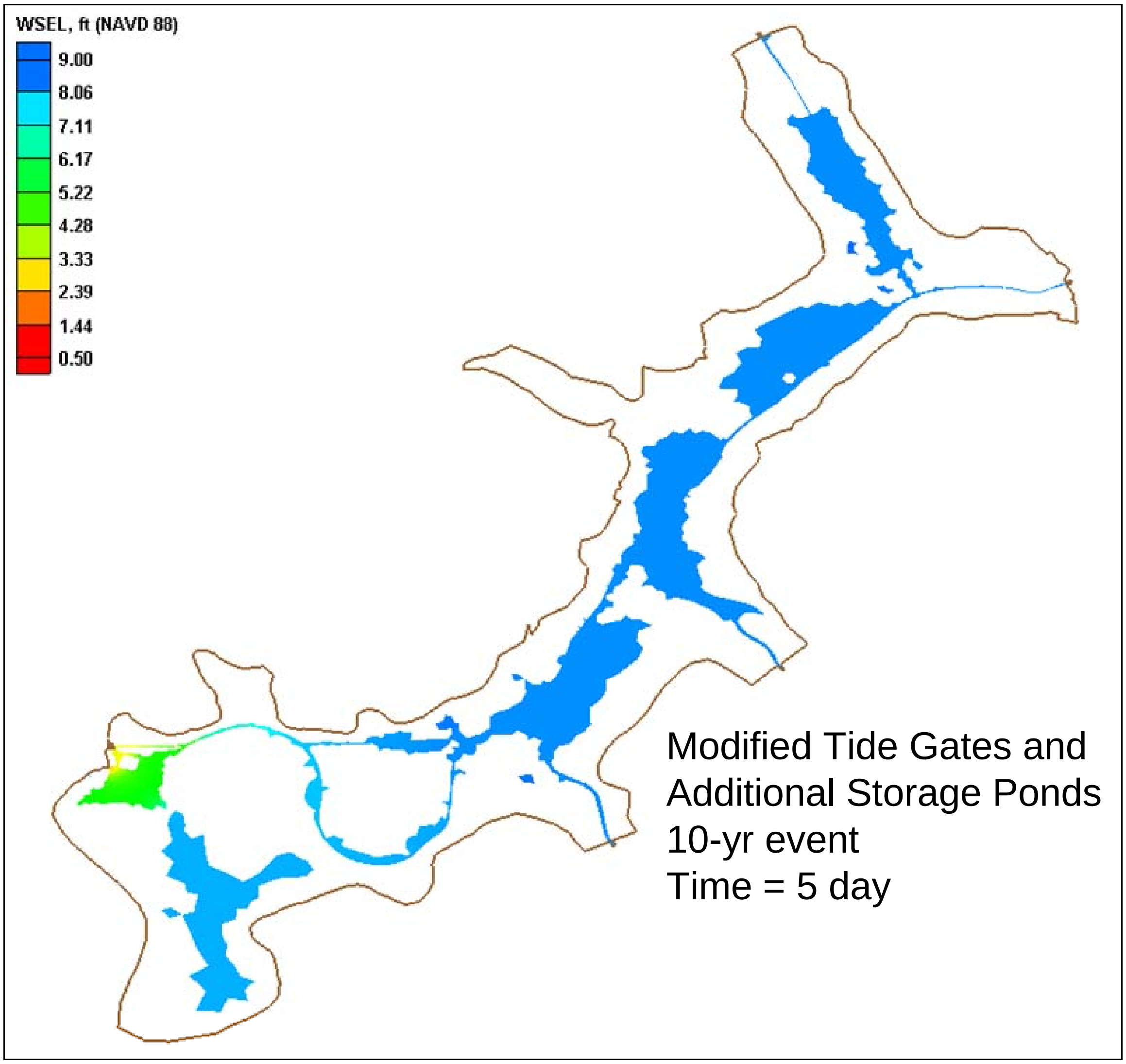

Figure 47. Inundation at time = 5 day for the 10-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

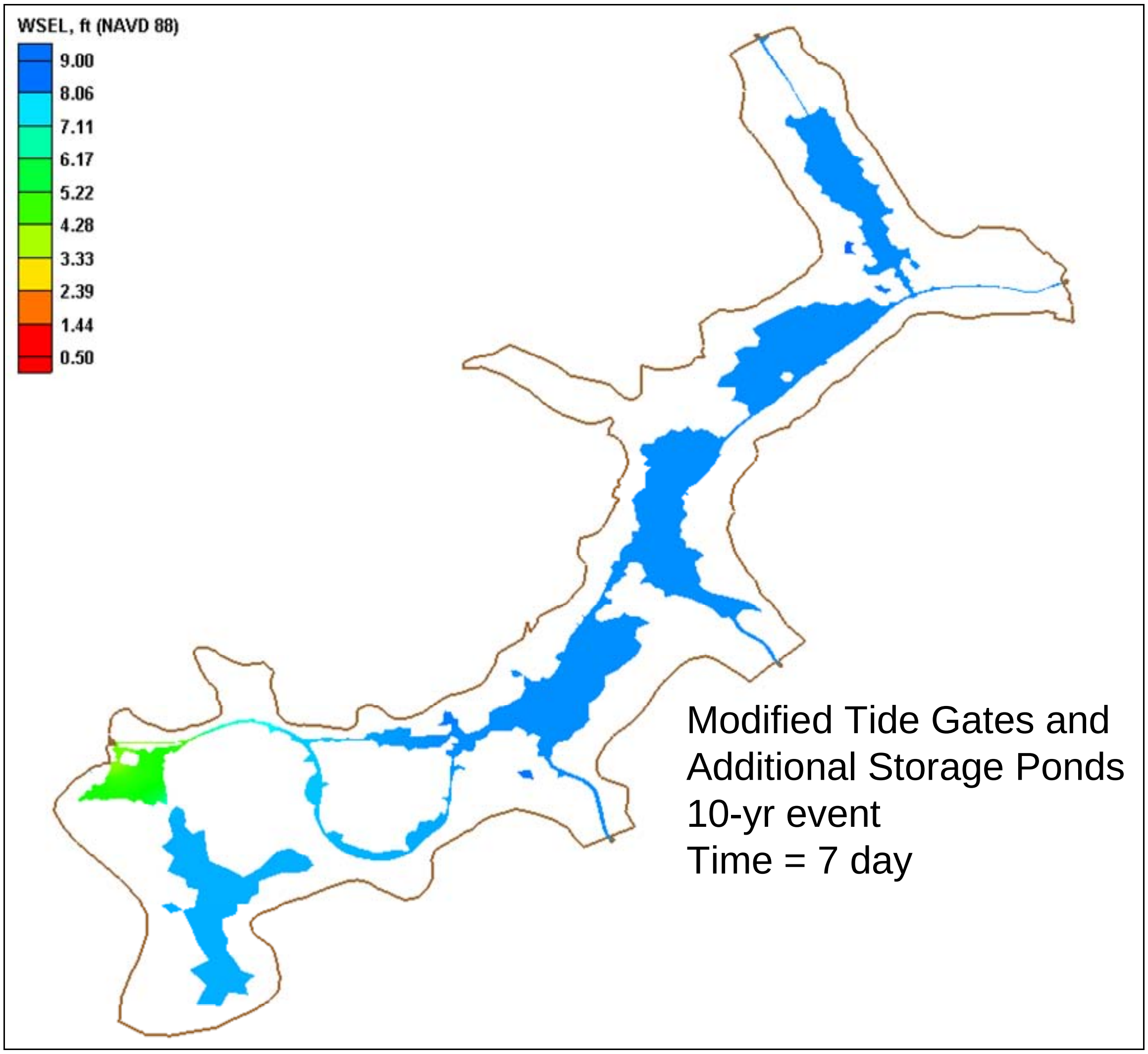

Figure 48. Inundation at time = 7 day for the 10-yr event, Modified Tidegates and Additional Storage Ponds, Alternative #3

Modified Tidegates, Additional Storage Ponds, and Modified Channel, Alternative #4

Plan view plots of inundation for the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative are given in Figures 49 through 54 for the peak inundation and elapsed times of 1, 2, 3, 5, and 7 days, respectively, for the 2-year event. During peak inundation, flooding for this event takes place primarily upstream of the large channel meander with some flooding in the pasture downstream of the meander. After 1 day, all areas experiencing flooding have been drained with the exception of parts of the downstream pasture. The downstream pasture retains standing water throughout the 7 days. Draining of this pasture could be improved by constructing a channel to route water to the pond located adjacent to the tidegates. Inundation expanse at peak flooding is less than that for the No-Action Alternative, with the exception of the downstream pasture, which experiences greater flooding. Inundation duration is greatly improved with this alternative, with water levels over most of the study site returning a non-inundation level within 1 day.

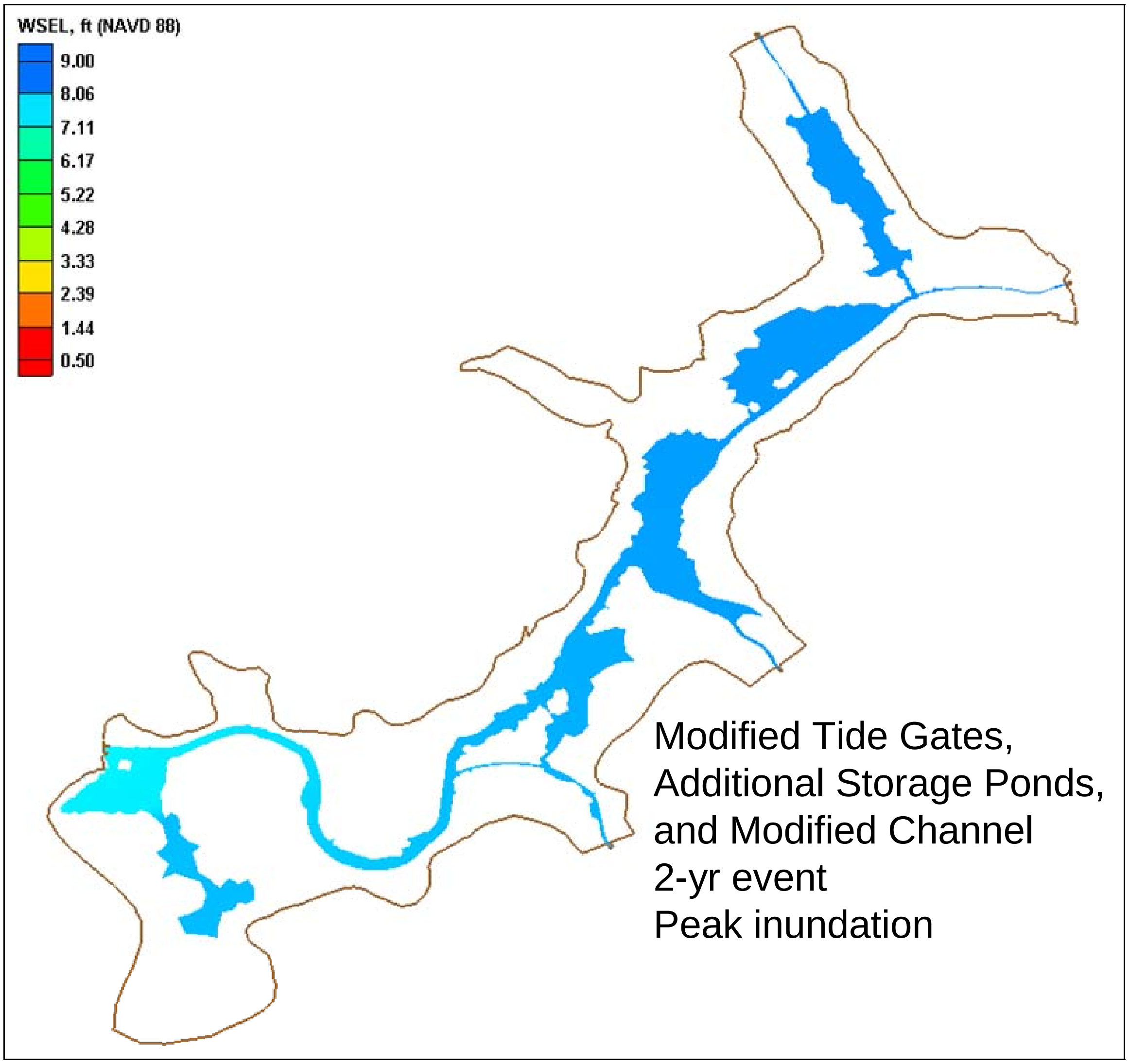

Figure 49. Peak inundation for the 2-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

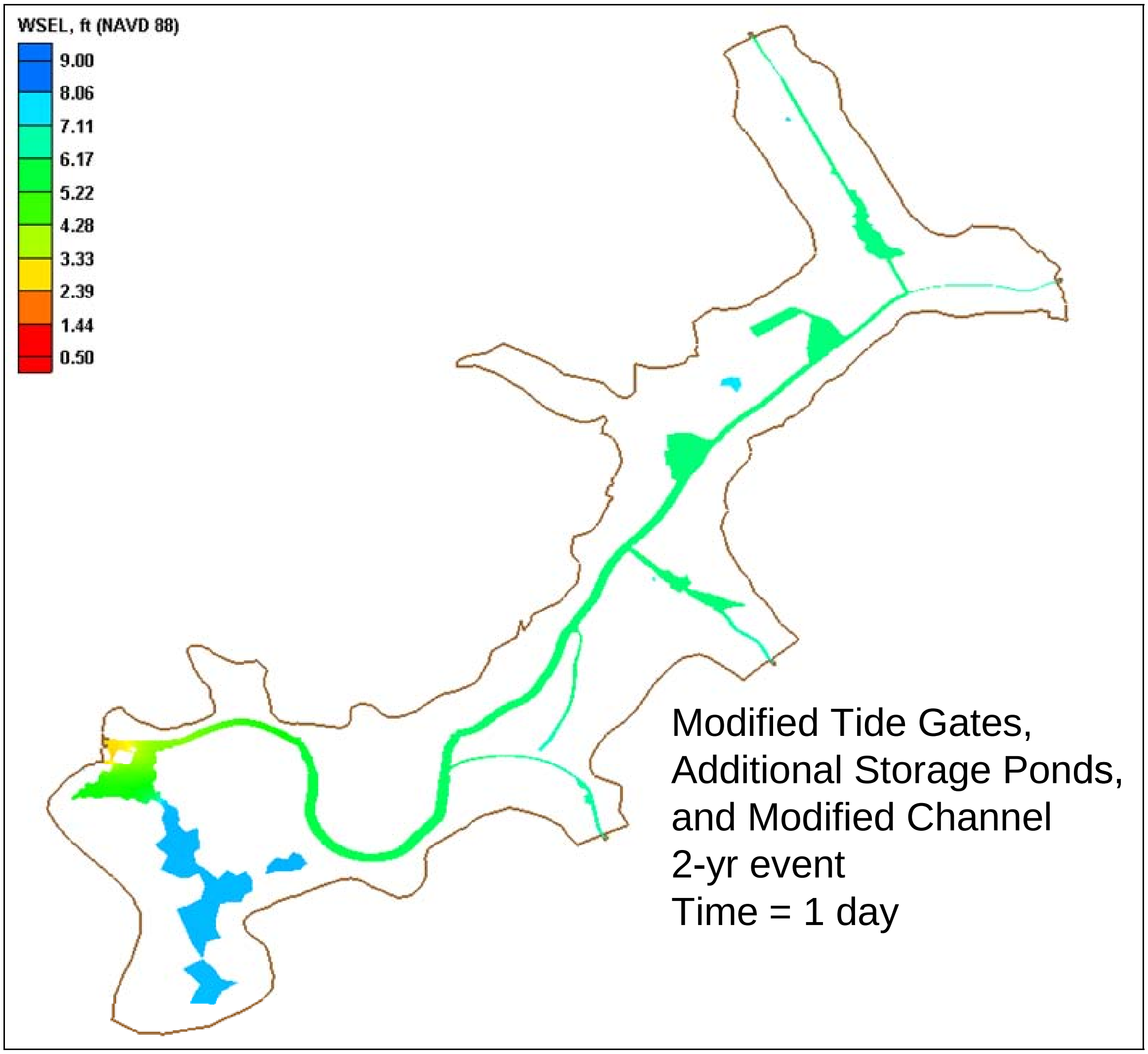

Figure 50. Inundation at time = 1 day for the 2-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

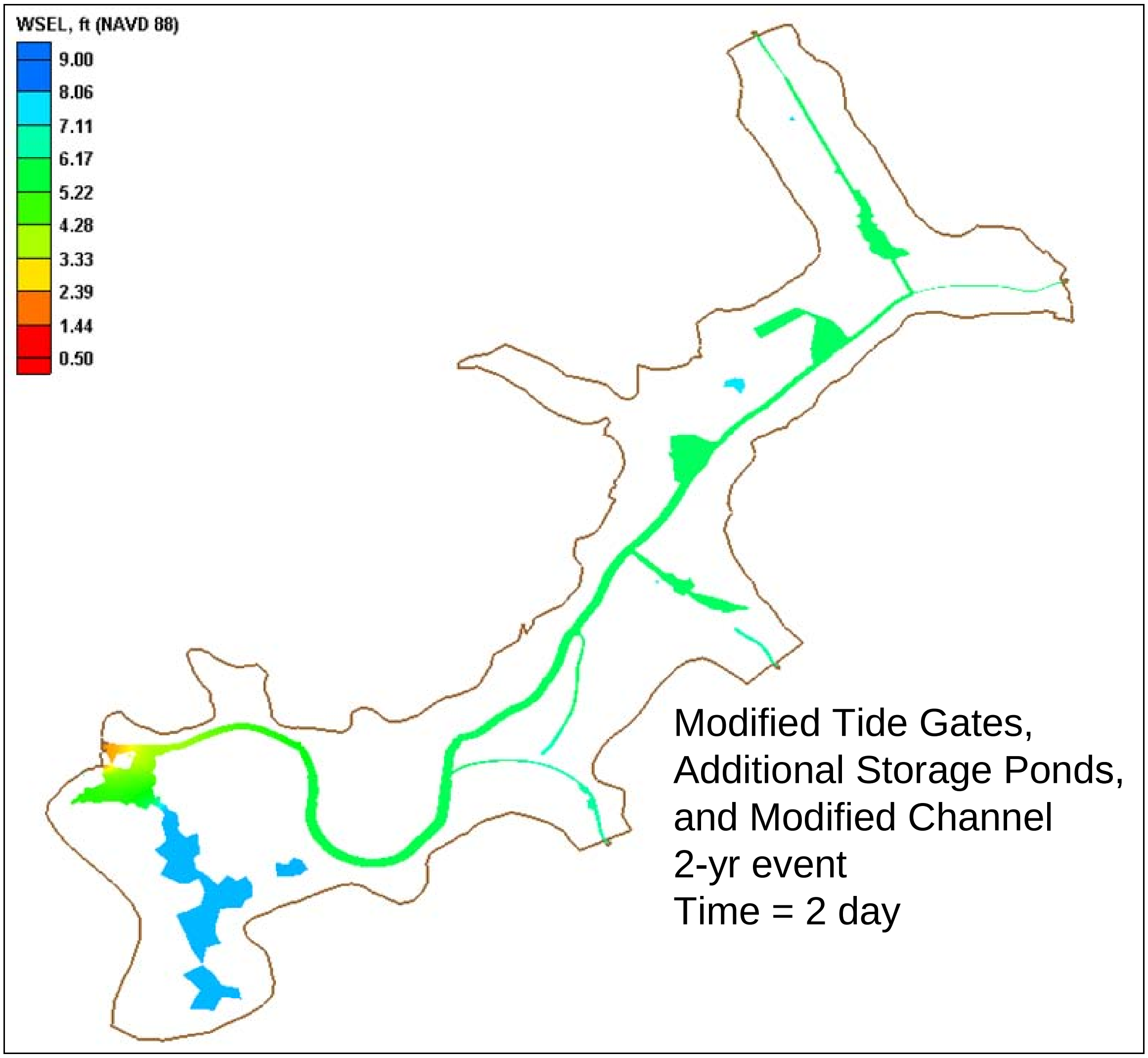

Figure 51. Inundation at time = 2 day for the 2-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

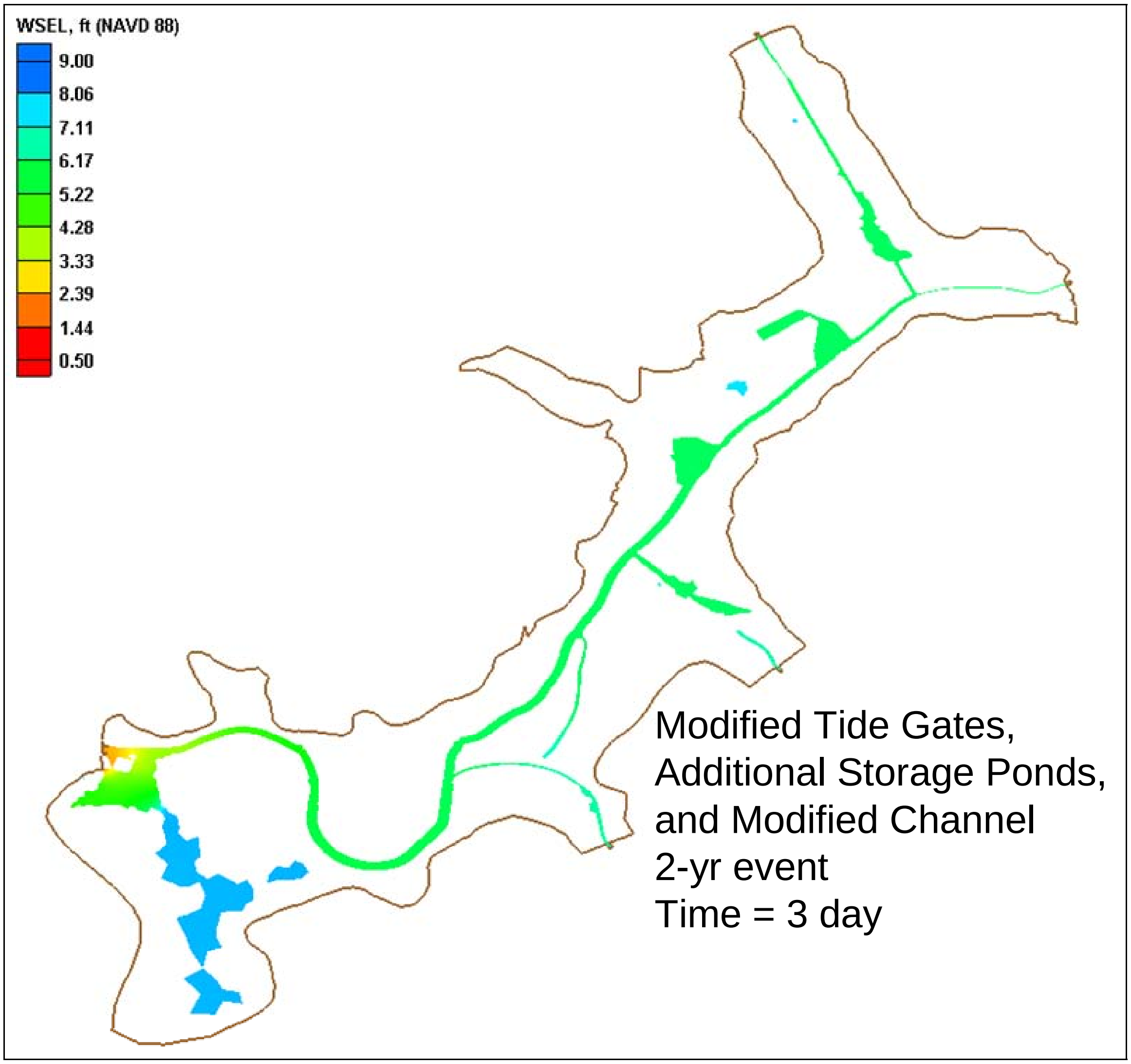

Figure 52. Inundation at time = 3 day for the 2-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

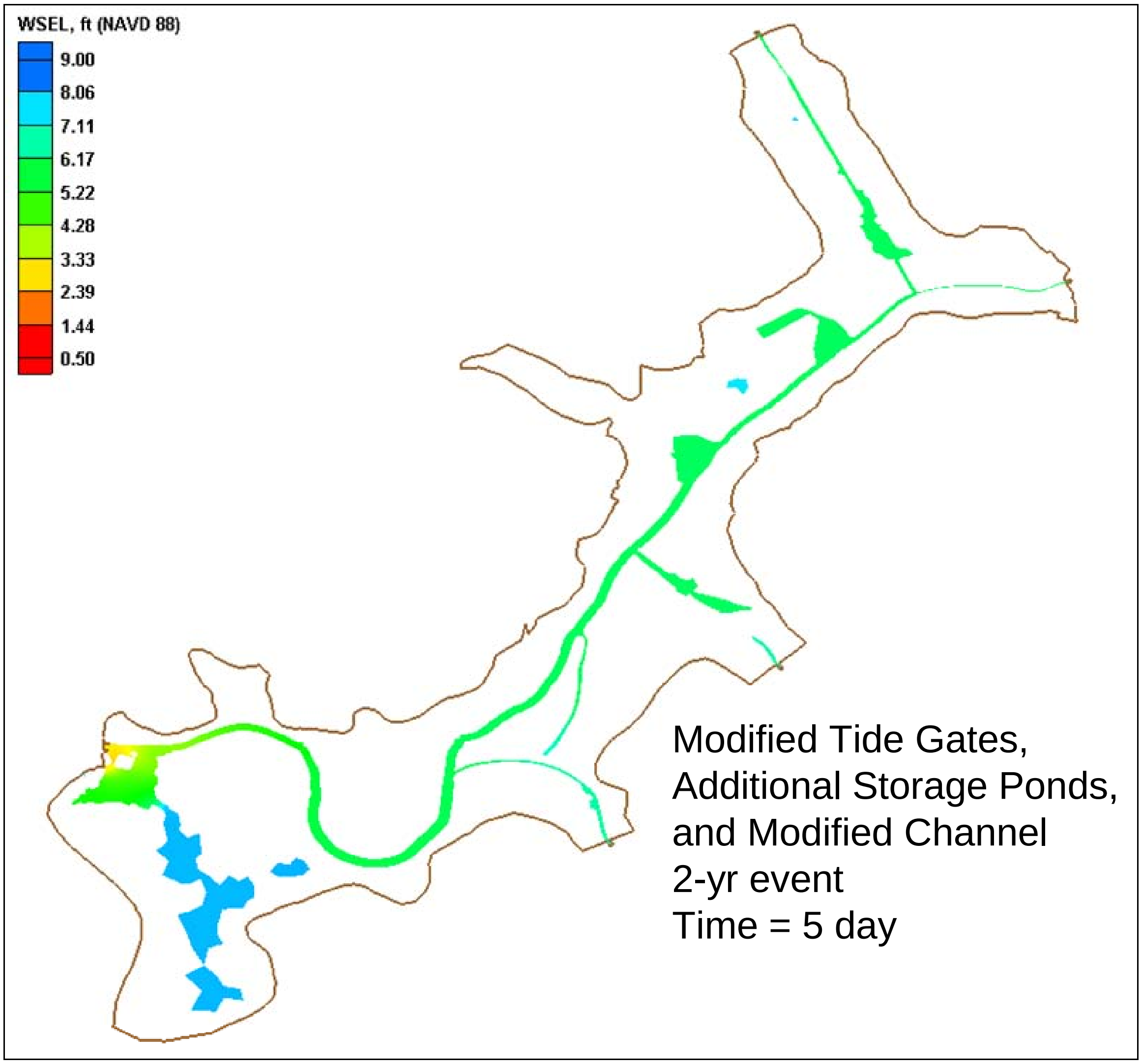

Figure 53. Inundation at time = 5 day for the 2-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

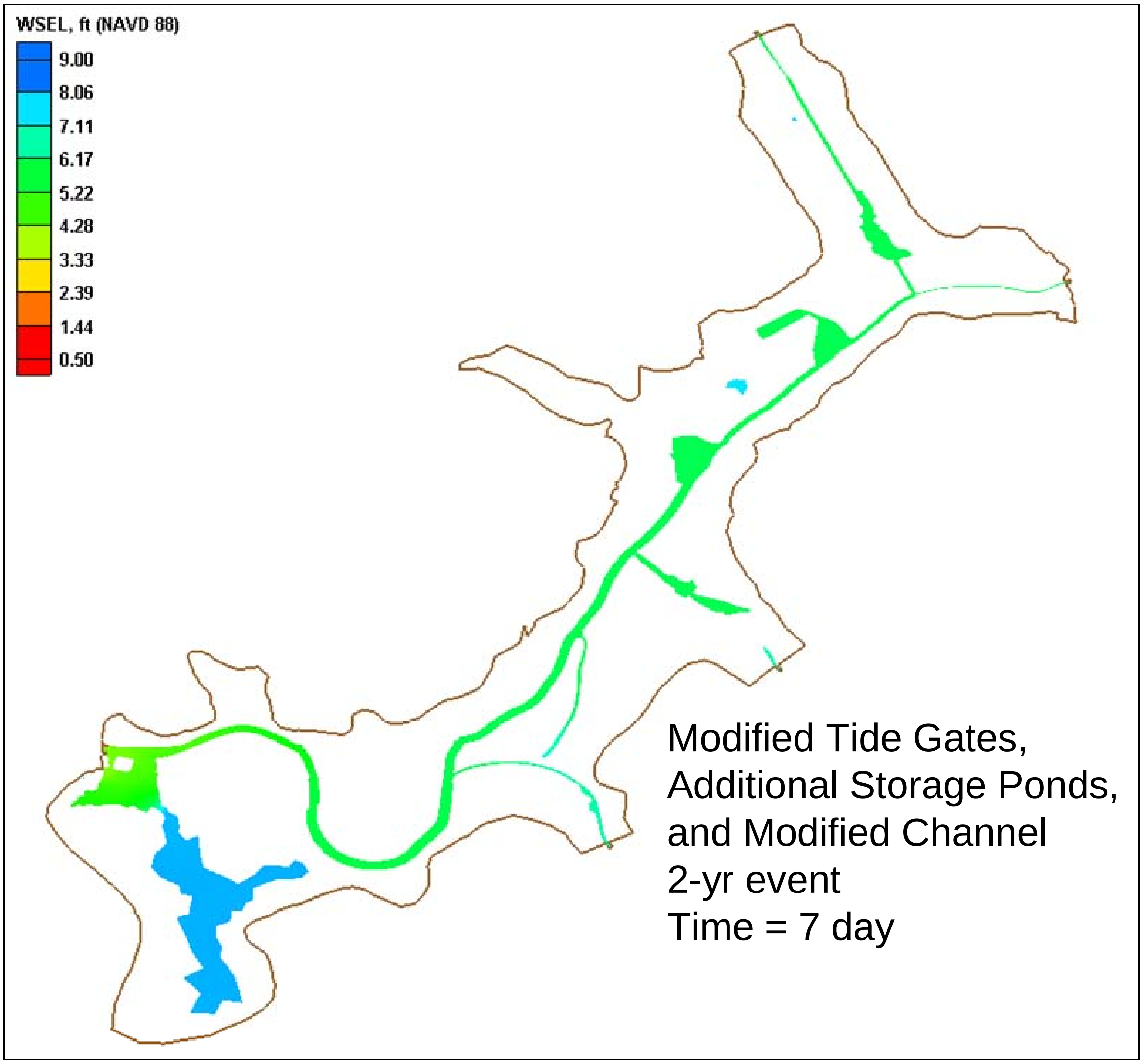

Figure 54. Inundation at time = 7 day for the 2-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

Plan view plots of inundation for the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative are given in Figures 55 through 60 for the peak inundation and elapsed times of 1, 2, 3, 5, and 7 days, respectively, for the 10-year event. During peak inundation, flooding for this event takes place upstream of the large channel meander and in the pasture downstream of the meander. After 1 day, the inundation is substantially reduced, particularly upstream of the meander. Bay Day 2, all areas experiencing flooding have been drained with the exception of parts of the downstream pasture. The downstream pasture retains standing water throughout the 7 days. Draining of this pasture could be improved by constructing a channel to route water to the pond located adjacent to the tidegates. Inundation expanse at peak flooding is significantly less than that for the No-Action Alternative over the entire study site. Inundation duration is greatly improved with this alternative, as compared to the No-Action Alternative, with most water draining off of the flooded areas within 1 day, and a return to a no-inundation state by Day 2, with the exception of the downstream pasture.

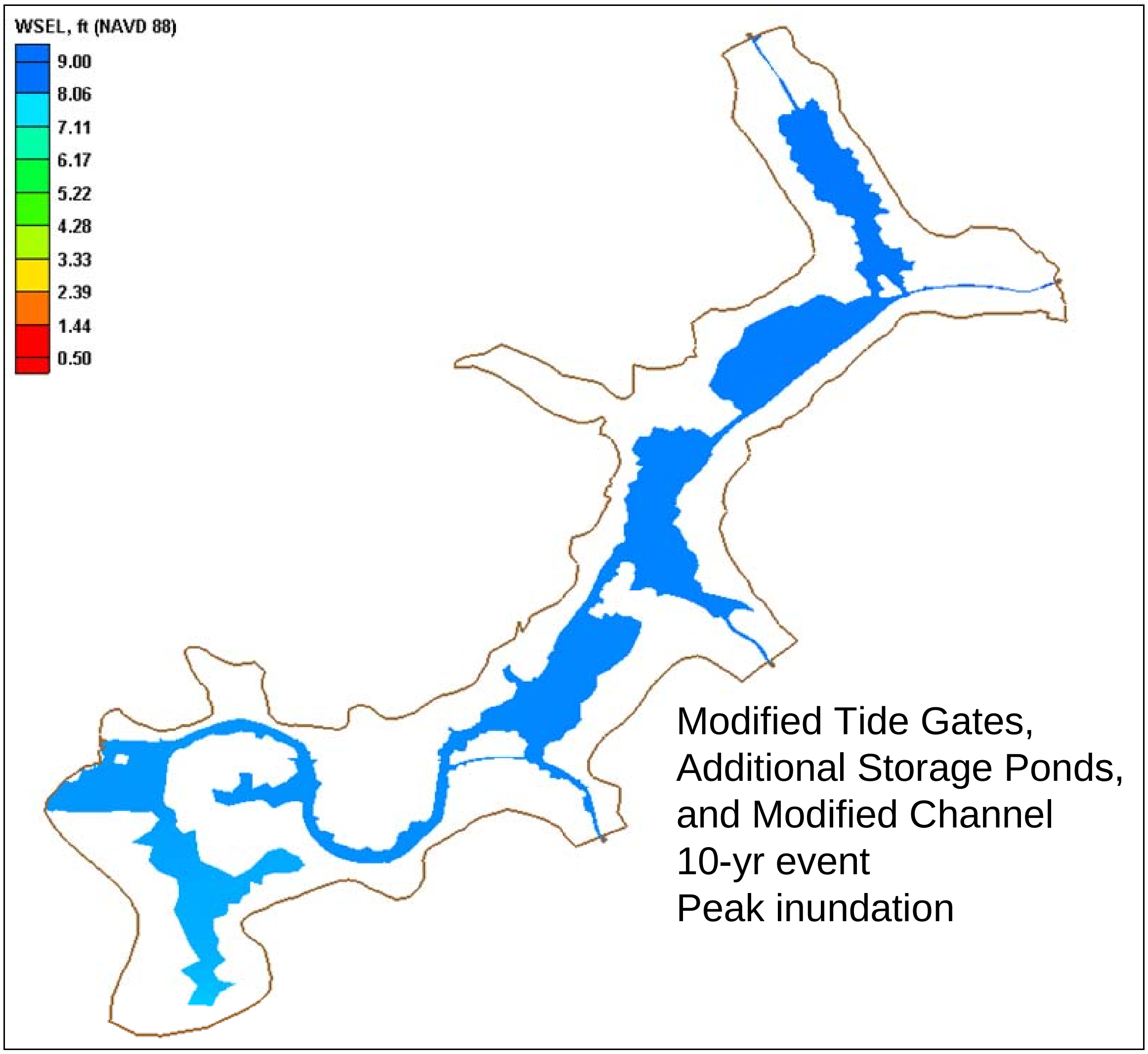

Figure 55. Peak inundation for the 10-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

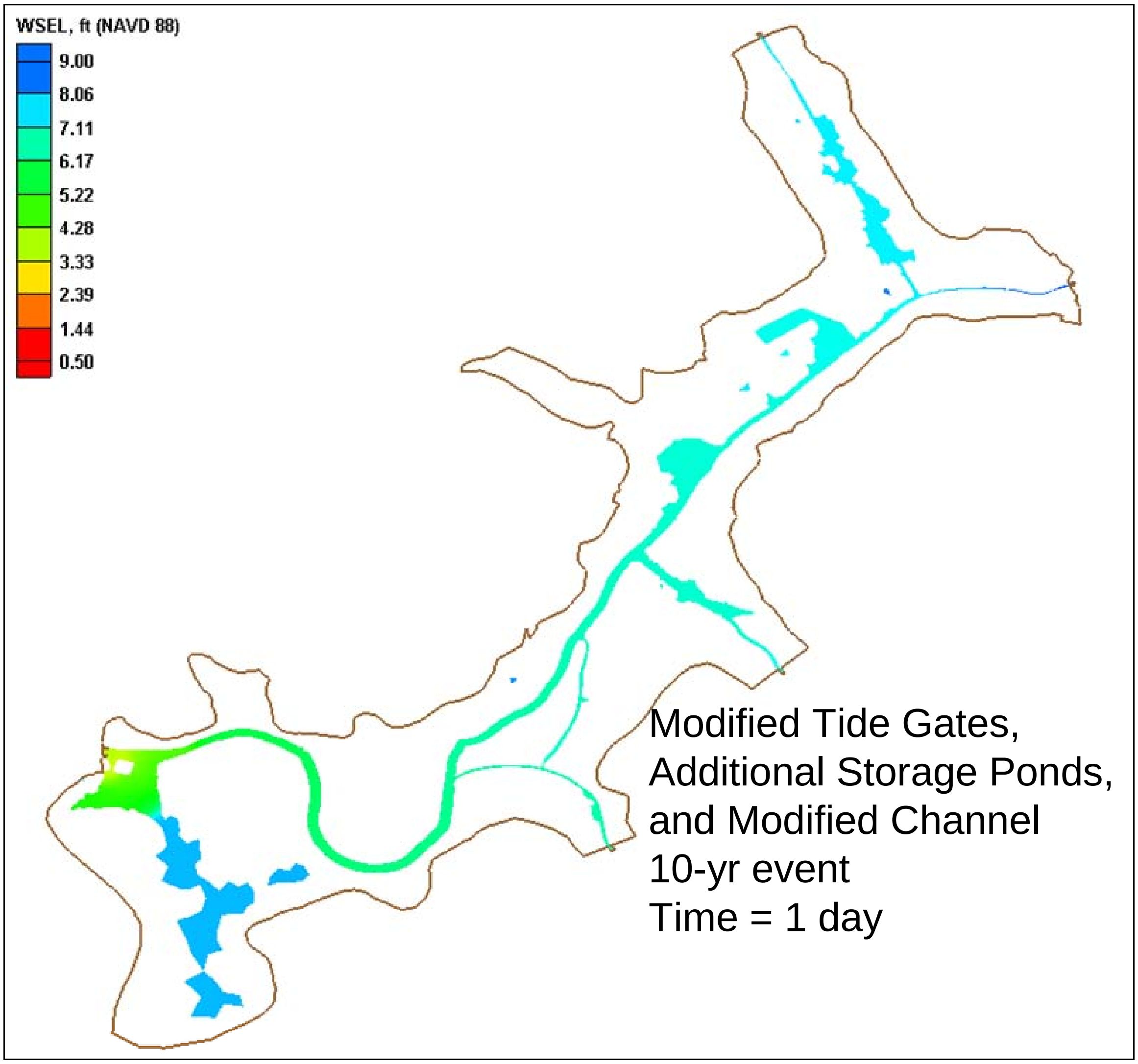

Figure 56. Inundation at time = 1 day for the 10-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

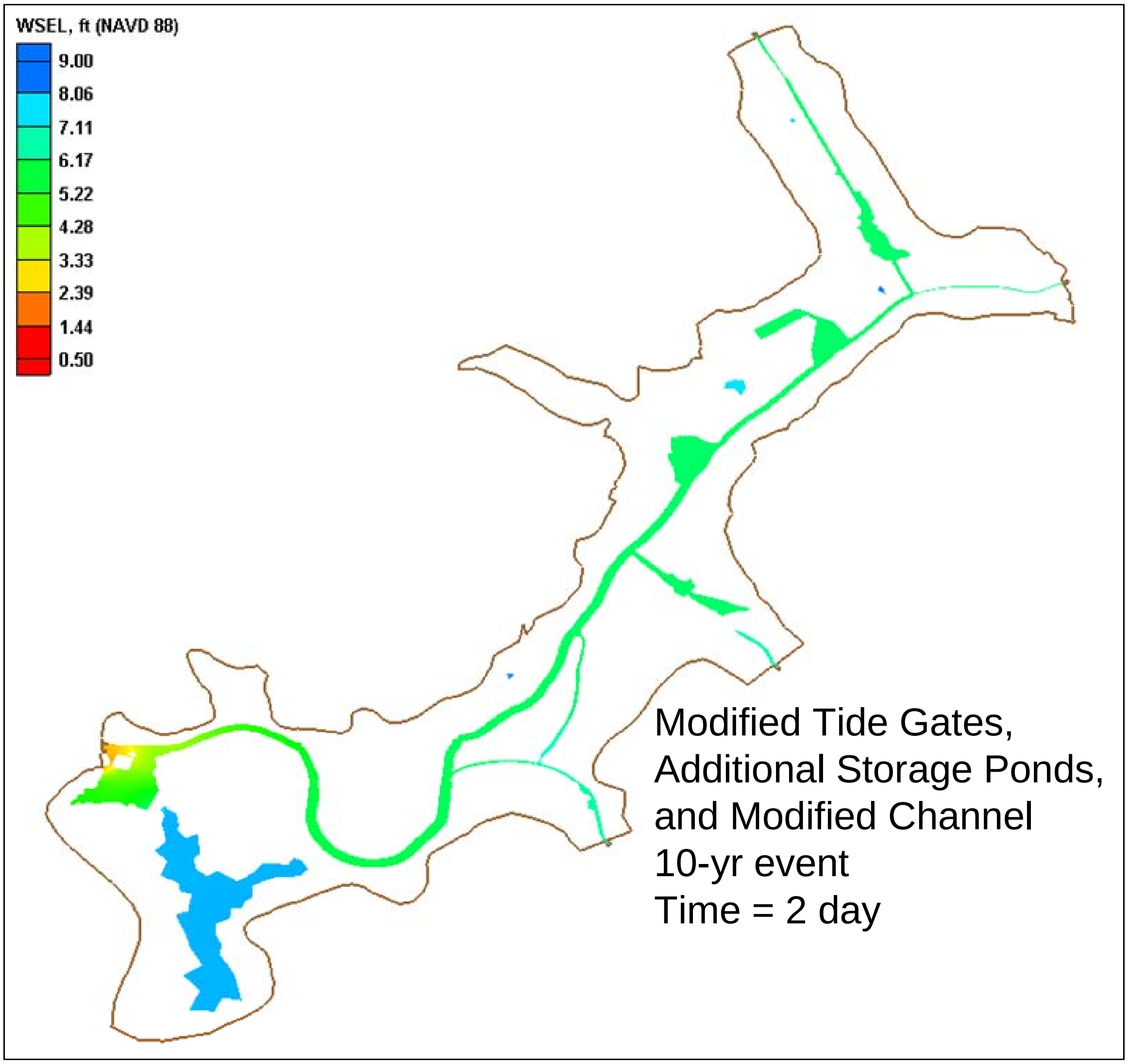

Figure 57. Inundation at time = 2 day for the 10-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

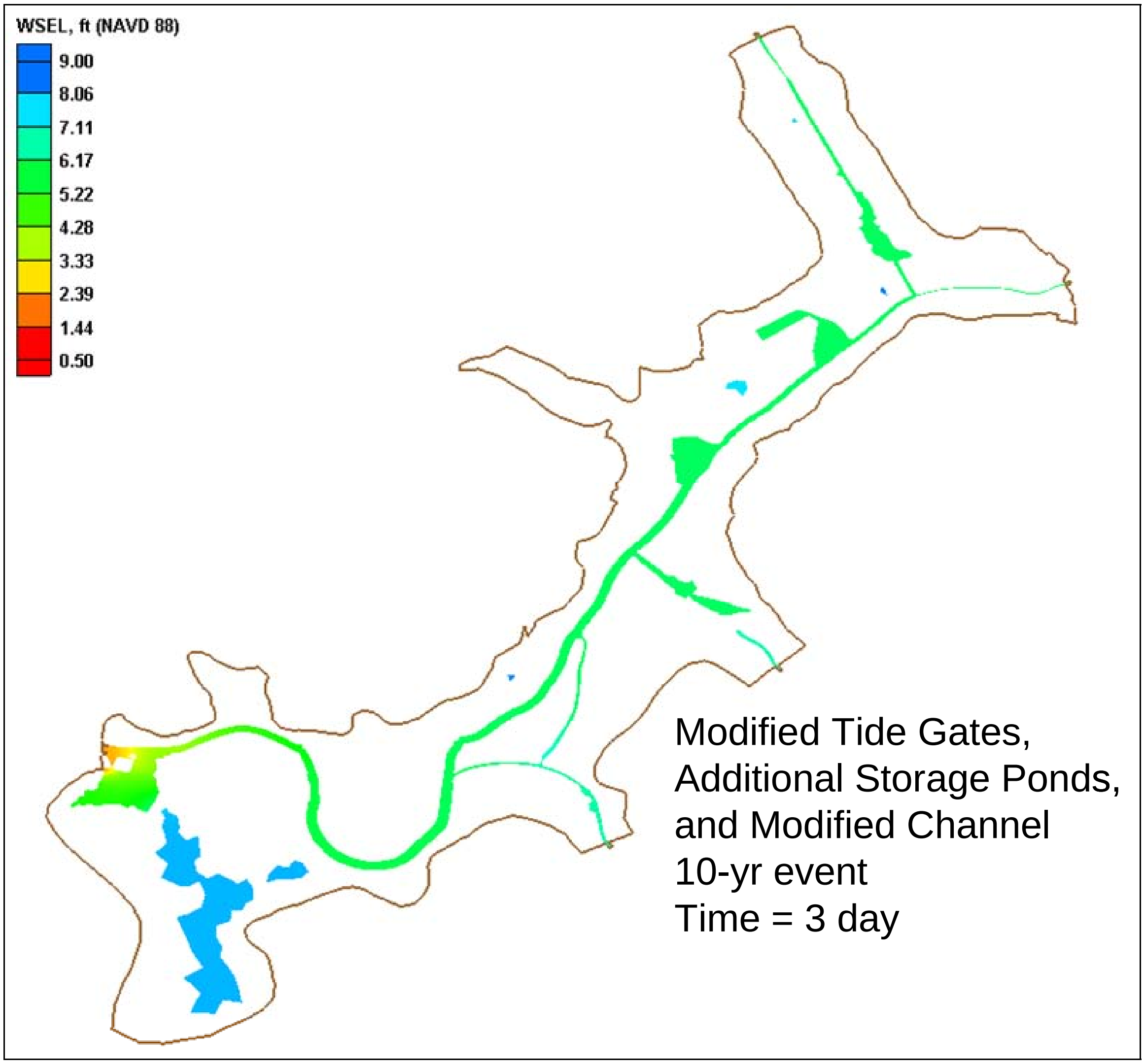

Figure 58. Inundation at time = 3 day for the 10-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

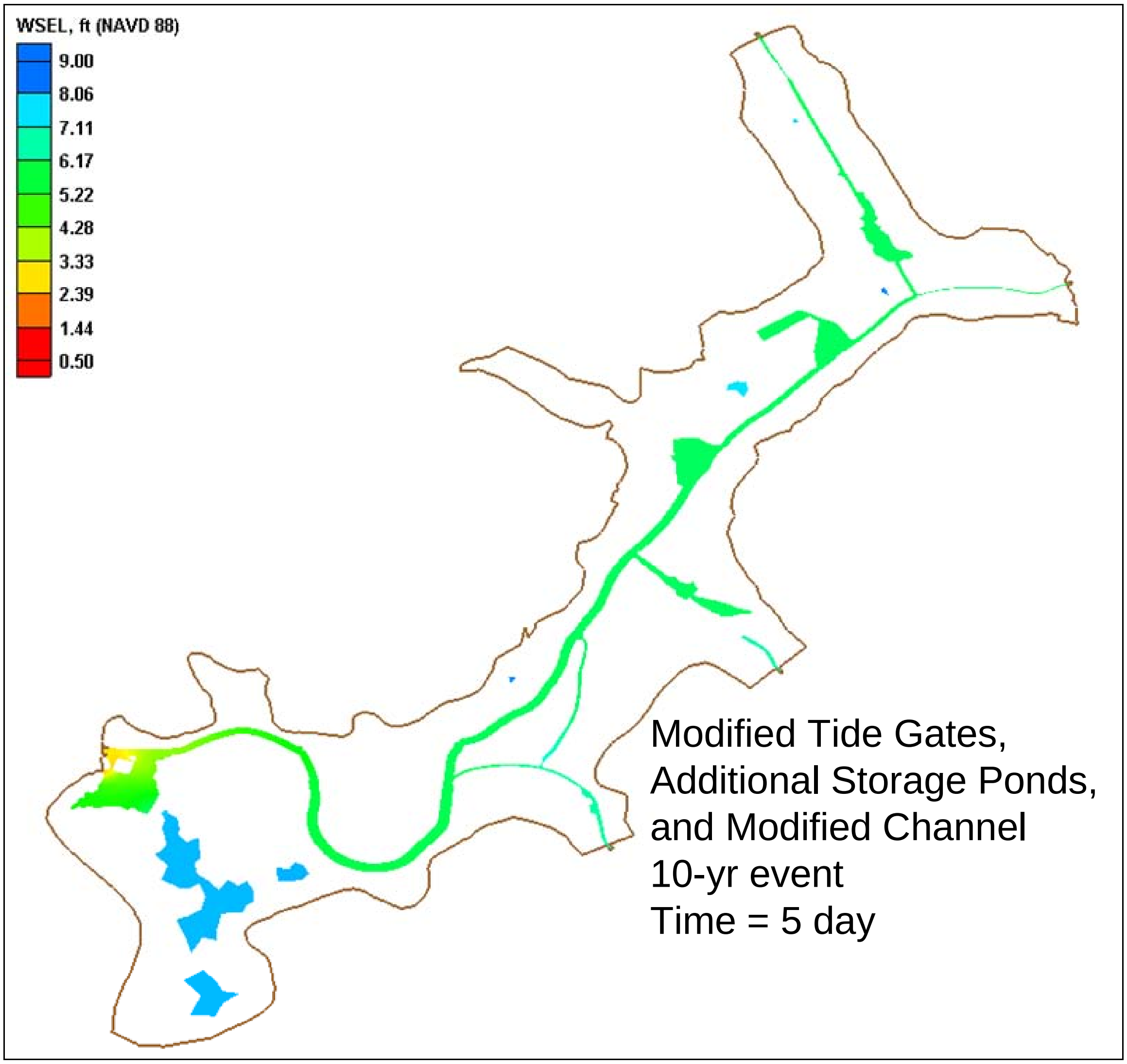

Figure 59. Inundation at time = 5 day for the 10-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

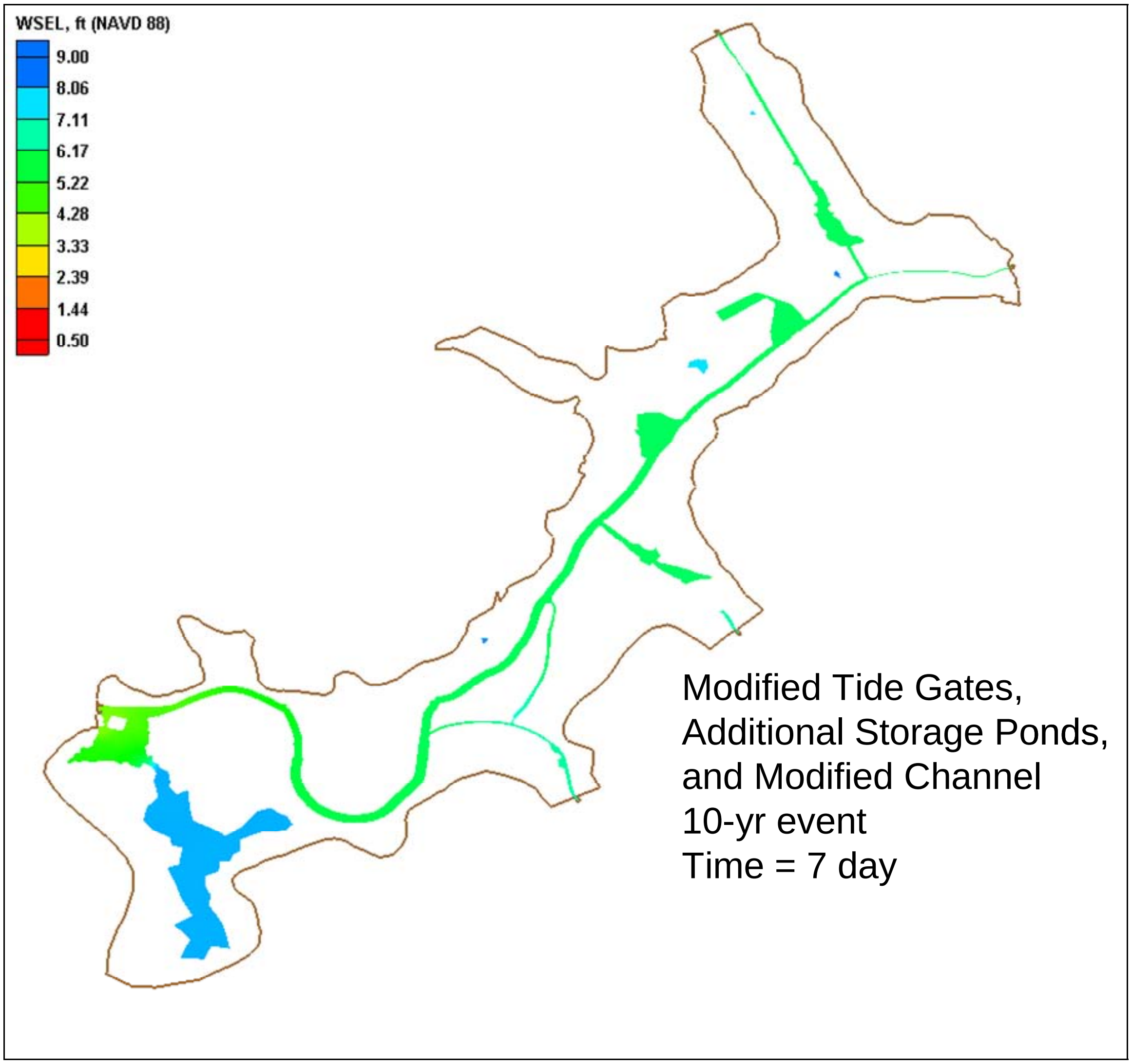

Figure 60. Inundation at time = 7 day for the 10-yr event, Modified Tidegates, Additional Storage Ponds, and Channel Modification, Alternative #4

Comparison of Water-Level Time Series at Four Stations

Four stations were selected for comparison of inundation levels and duration. These stations are located in areas prone to inundation during both minor and major events. Figure 61 shows the locations of the four stations, which are distributed along parts of the study area having different inundation properties. For each station and event, a plot showing time series of water-surface elevation for each alternative has been developed so that direct comparison among the alternatives can be made.

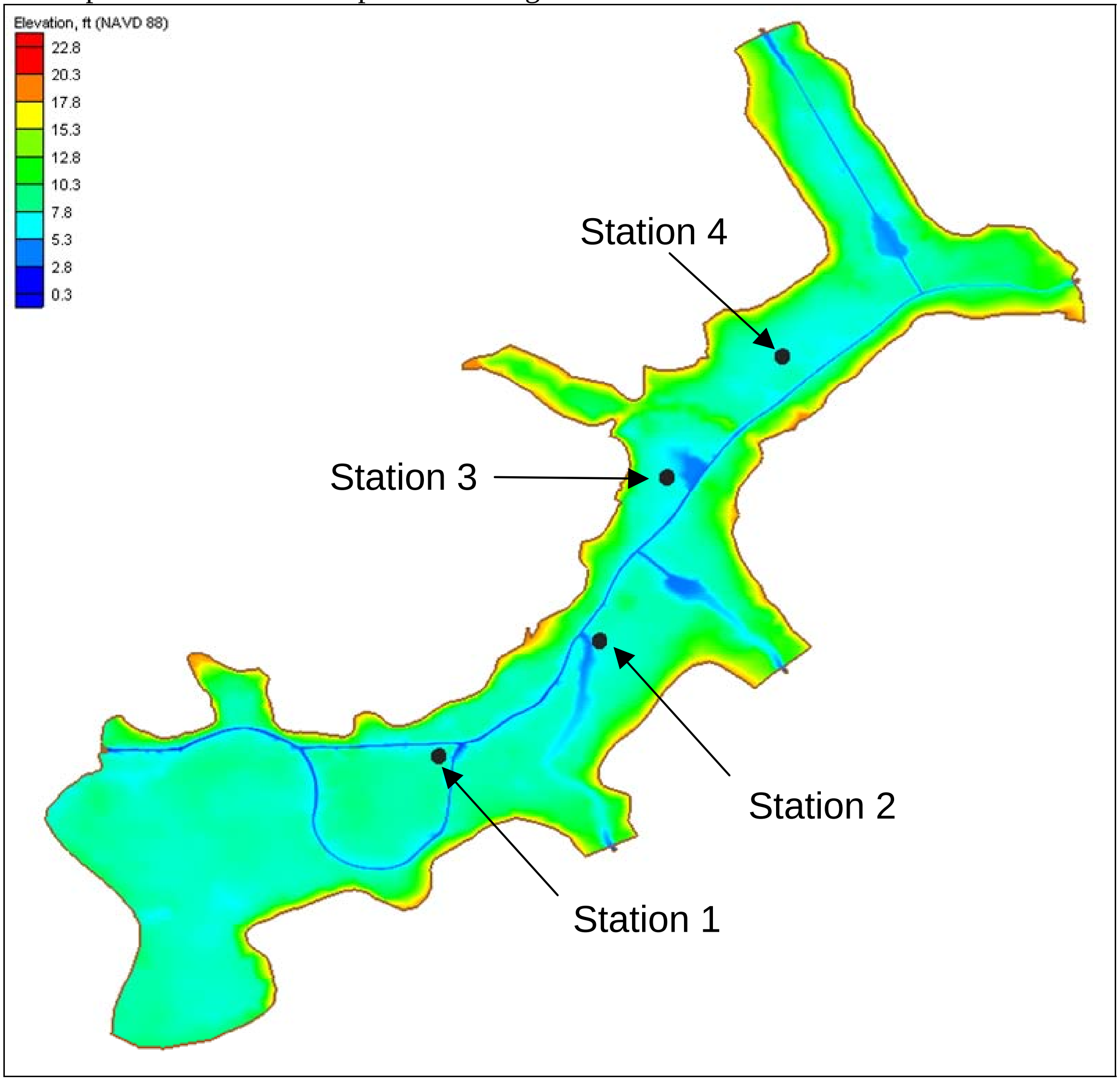

Figure 61. Location of Stations at which inundation levels are compared.

Time series of water-surface elevation at Station 1 for the 2-year event are shown in Figure 62. The No-Action Alternative and the Modified Tidegates and Additional Storage Ponds alternatives show similar inundation properties, both having short-duration flooding at Station 1. The No Tidegates Tidally-Influenced Alternative shows much longer inundation and that inundation contains tidal periodicity. The Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative does not flood at Station 1 for the 2-year event. Thus, at Station 1 for the 2-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative provides the most effective design to reduce inundation.

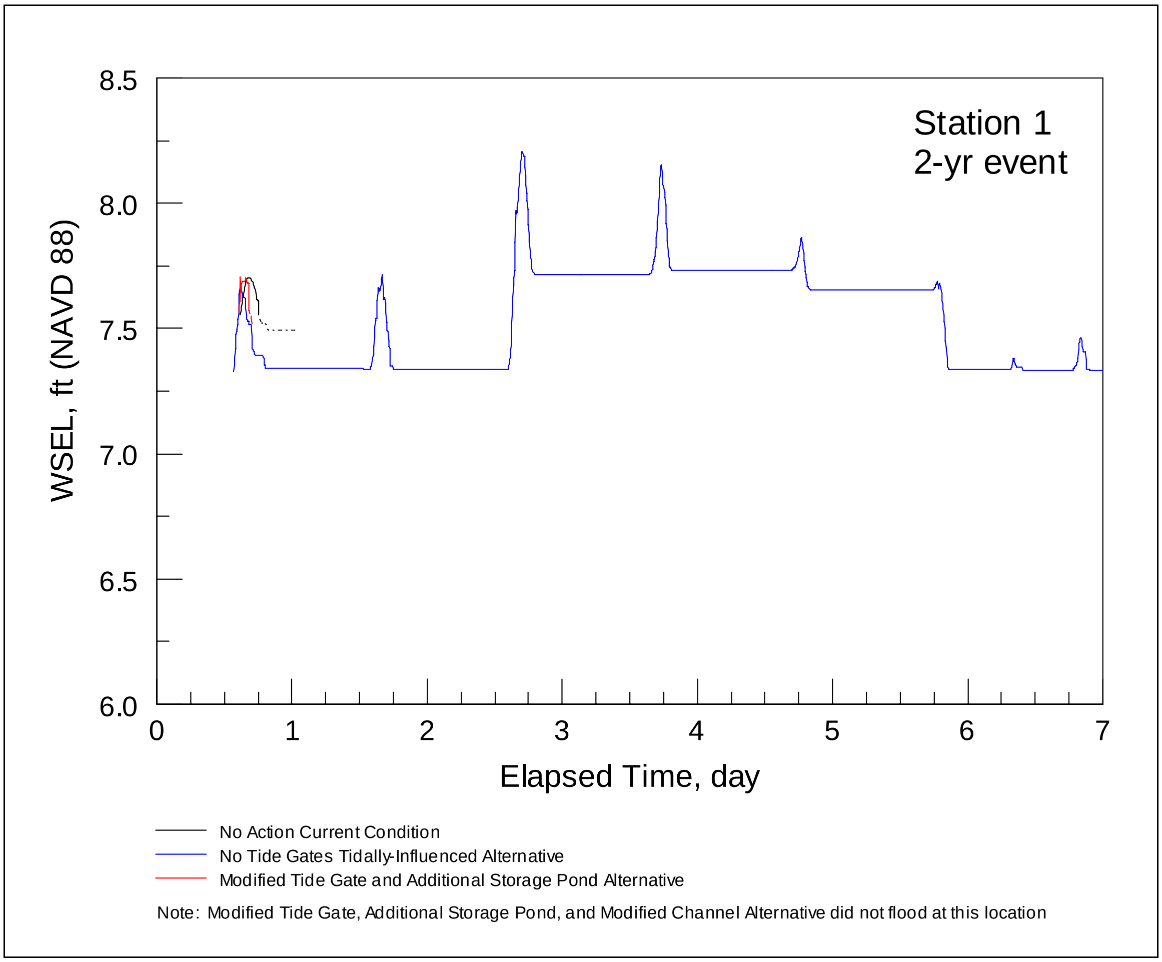

Figure 62. Time series of water-surface elevation at Station 1 for the 2-year event.

Time series of water-surface elevation at Station 2 for the 2-year event are shown in Figure 63. The Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative floods during peak inundation, but quickly drains off. The No-Action Alternative and the Modified Tidegates and Additional Storage Ponds alternatives show similar inundation peaks, but the No-Action Alternative remains flooded for about 4.5 days and the the Modified Tidegates and Additional Storage Ponds Alternative drains much faster, in about 2.5 days. The No Tidegates Tidally-Influenced Alternative shows much longer inundation and that inundation contains tidal periodicity. Thus, at Station 2 for the 2-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative provides the most effective design to reduce inundation because, even though this station was flooded, water drained away quickly.

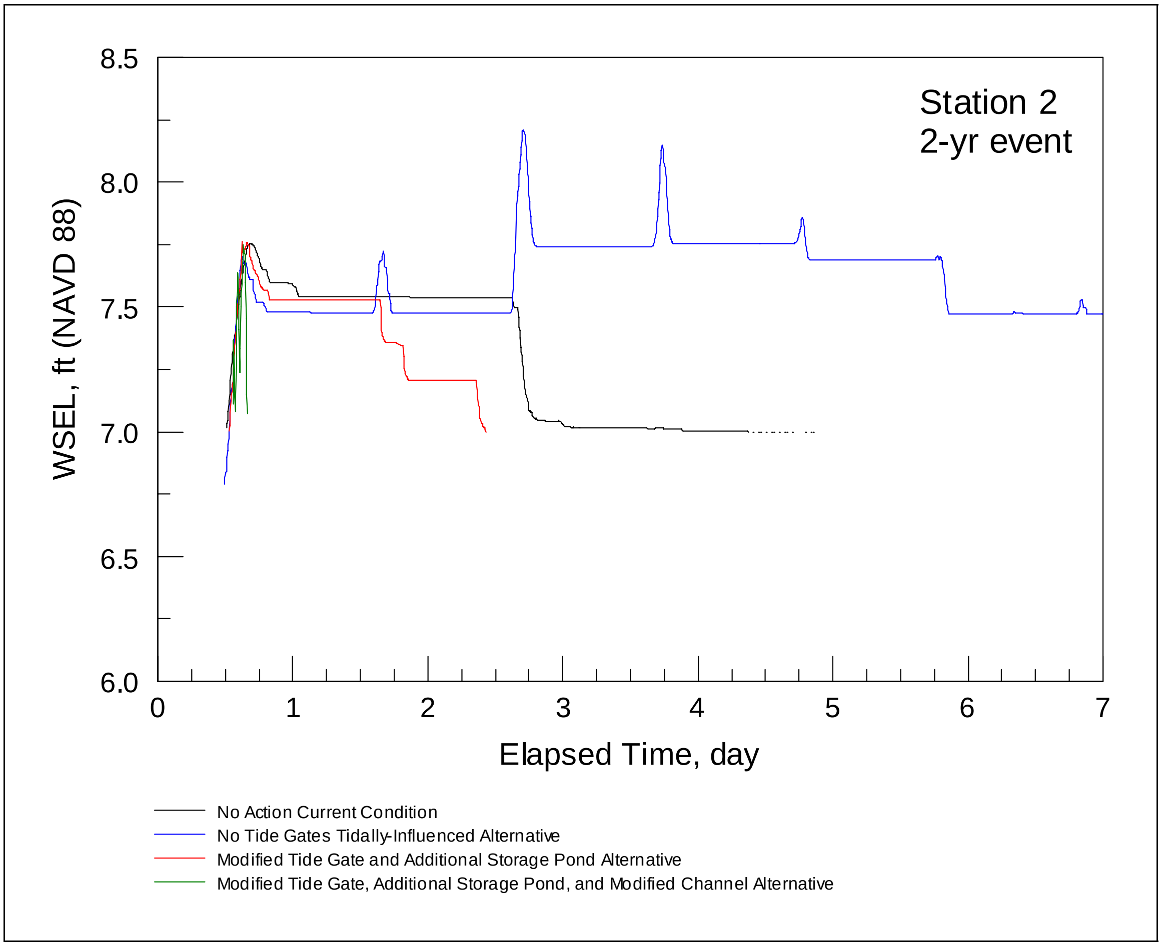

Figure 63. Time series of water-surface elevation at Station 2 for the 2-year event.

Time series of water-surface elevation at Station 3 for the 2-year event are shown in Figure 64. The Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative floods during peak inundation, but quickly drains off. The No-Action Alternative and the Modified Tidegates and Additional Storage Ponds alternatives show similar inundation peaks and both remain flooded for the 7-day time interval. The overall water level does drop more rapidly for the Modified Tidegates and Additional Storage Ponds Alternative than for the No-Action Alternative. The No Tidegates Tidally-Influenced Alternative shows inundation for the entire 7 days and that inundation contains tidal periodicity. Thus, at Station 3 for the 2-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative provides the most effective design to reduce inundation because water drained away quickly after the inundation took place.

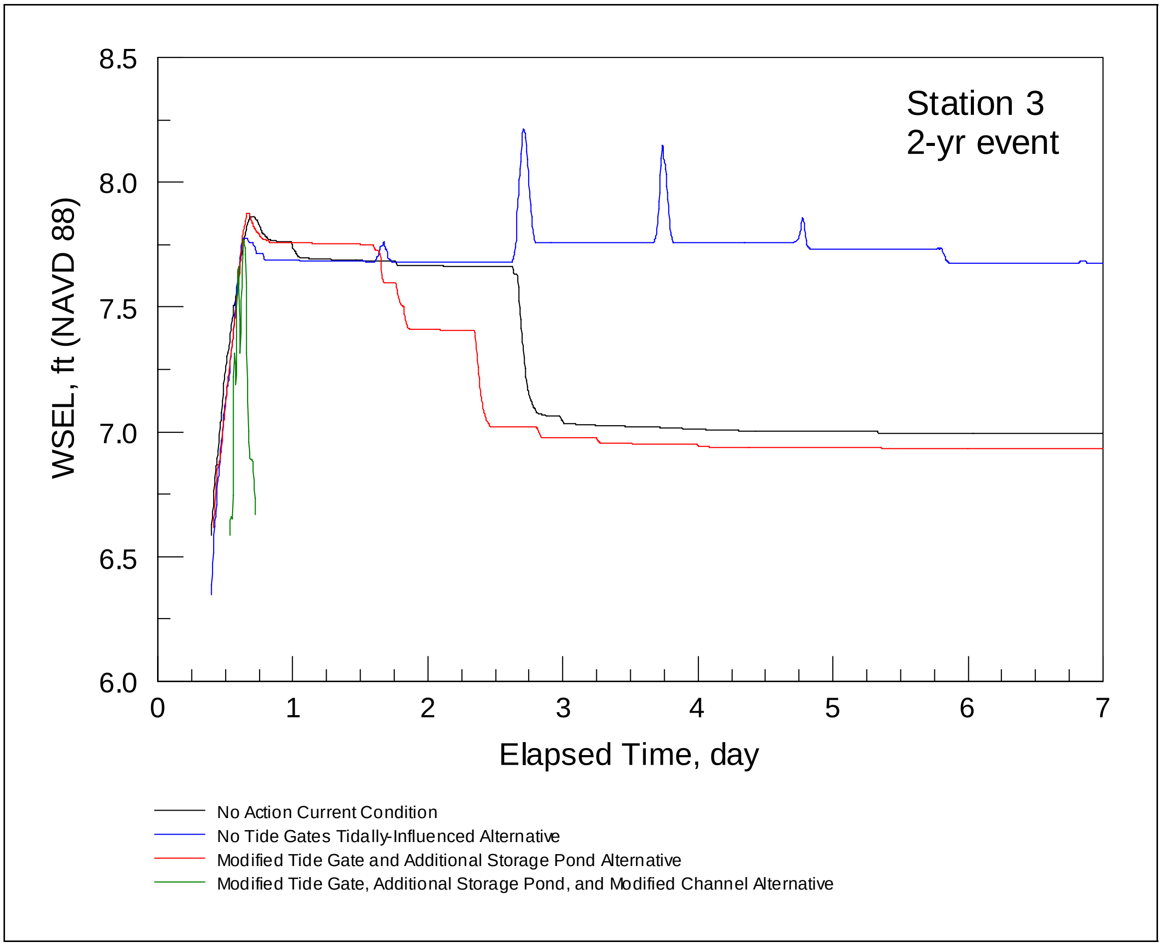

Figure 64. Time series of water-surface elevation at Station 3 for the 2-year event.

Time series of water-surface elevation at Station 4 for the 2-year event are shown in Figure 65. The Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative floods during peak inundation, but quickly drains off. The No-Action Alternative and the Modified Tidegates and Additional Storage Ponds alternatives show similar inundation peaks and both remain flooded for the 7-day time interval. The overall water level does drop more rapidly for the Modified Tidegates and Additional Storage Ponds Alternative than for the No-Action Alternative. The No Tidegates Tidally-Influenced Alternative shows inundation for the entire 7 days and that inundation contains tidal periodicity. Thus, at Station 4 for the 2-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative provides the most effective design to reduce inundation because water drained away quickly after the inundation took place.

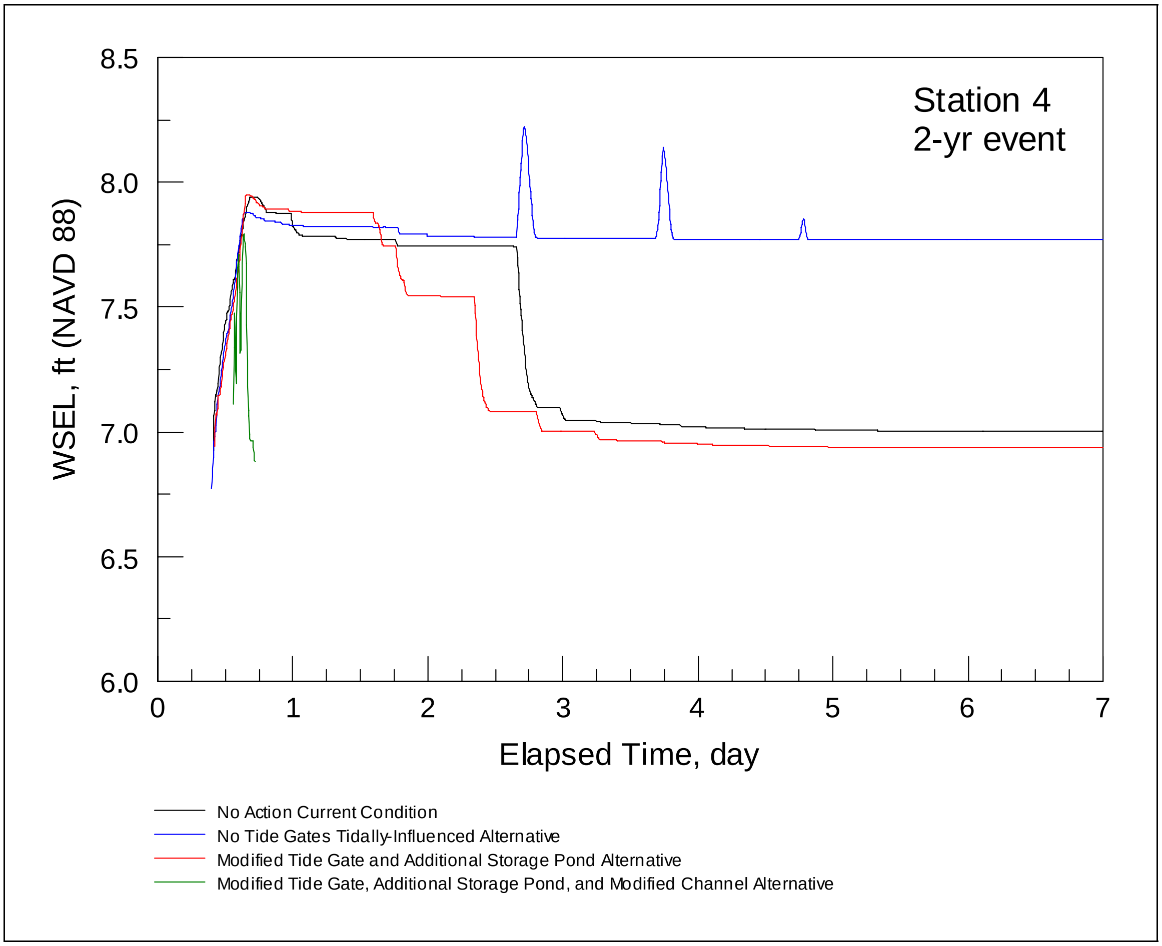

Figure 65. Time series of water-surface elevation at Station 4 for the 2-year event.

Time series of water-surface elevation at Station 1 for the 10-year event are shown in Figure 66. The No-Action Alternative and the Modified Tidegates and Additional Storage Ponds alternatives show similar inundation properties, both having a peak water level that tapers off to an almost-constant level. The No-Action Alternative, however, has a significantly larger peak value, which diminishes rapidly. The No Tidegates Tidally-Influenced Alternative shows a slightly reduced peak inundation, as compared to the Modified Tidegates and Additional Storage Ponds Alternative, contains tidal periodicity that diminishes over time. The Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative does not flood at Station 1 for the 10-year event. Thus, at Station 1 for the 10-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative provides the most effective design to reduce inundation.

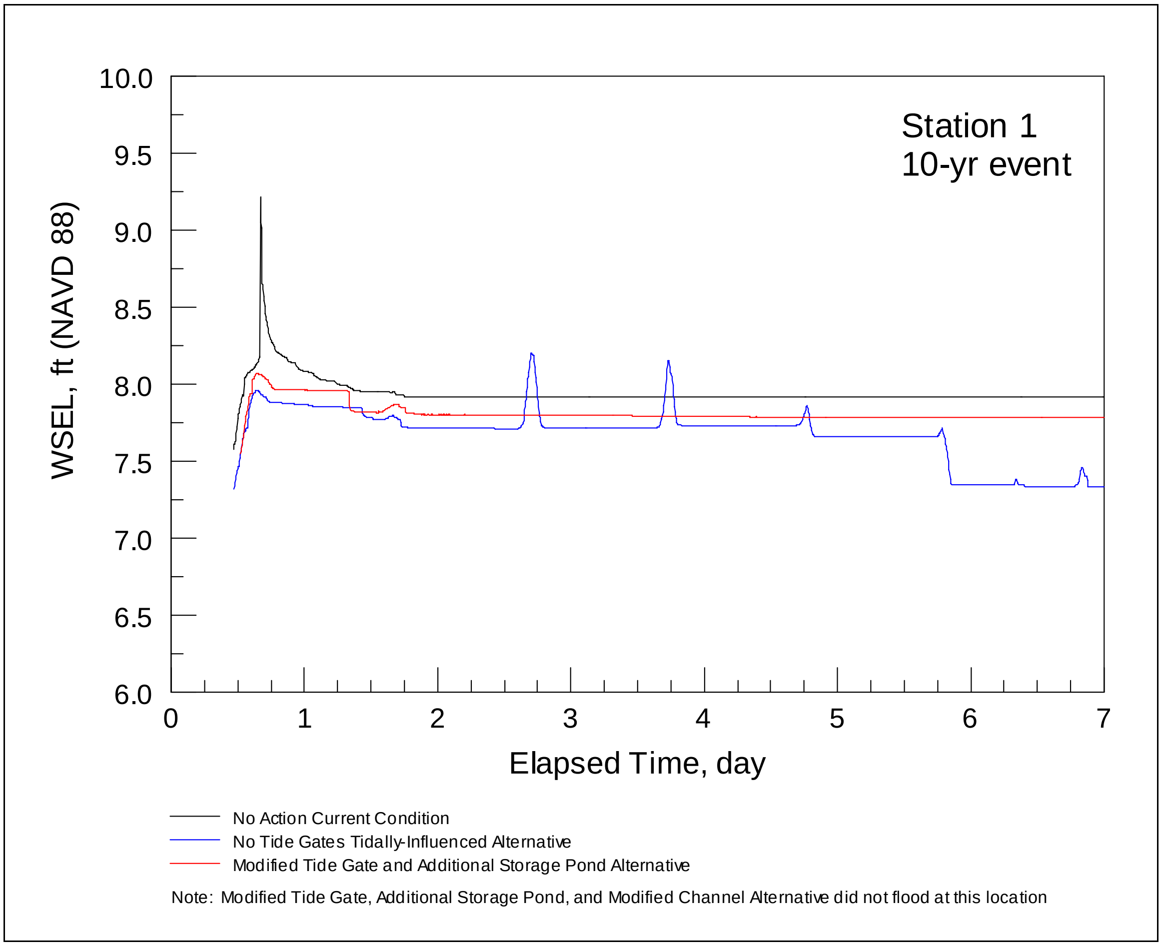

Figure 66. Time series of water-surface elevation at Station 1 for the 10-year event.

Time series of water-surface elevation at Station 2 for the 10-year event are shown in Figure 67. The No-Action Alternative, Modified Tidegates and Additional Storage Ponds Alternative, and the No Tidegates Tidally-Influenced Alternative show similar inundation properties, all having an initial peak water level that tapers off. Of these three alternatives, the No-Action Alternative has the greatest peak inundation and greatest retained water level over seven days. The Modified Tidegates and Additional Storage Ponds Alternative and the No Tidegates Tidally-Influenced Alternative show similar water-surface elevations, with the exception of tidal peaks in the No Tidegates Tidally-influenced Alternative and slightly higher overall water level in the Modified Tidegates and Additional Storage Ponds, Alternative #3 The Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative has the minimum peak water level, which drains off rapidly. It has a second flooding interval, which also drains off quickly. Thus, at Station 2 for the 10-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative provides the most effective design to reduce inundation.

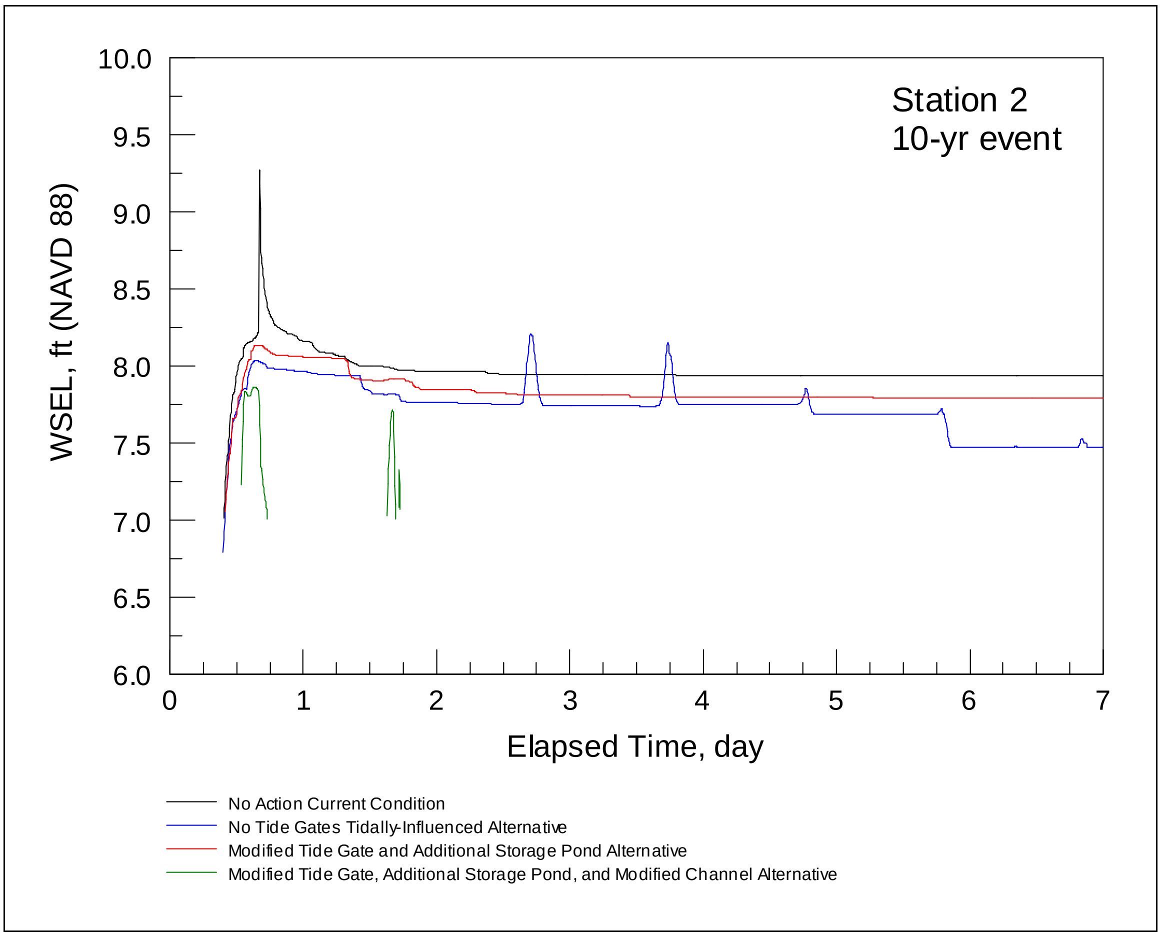

Figure 67. Time series of water-surface elevation at Station 2 for the 10-year event.

Time series of water-surface elevation at Station 3 for the 10-year event are shown in Figure 68. The No-Action Alternative, Modified Tidegates and Additional Storage Ponds Alternative, and the No Tidegates Tidally-Influenced Alternative show similar inundation properties, all having an initial peak water level that tapers off. Of these three alternatives, the No-Action Alternative has the greatest peak inundation and greatest retained water level over seven days. The Modified Tidegates and Additional Storage Ponds Alternative and the No Tidegates Tidally-Influenced Alternative show similar water-surface elevations, with the exception of tidal peaks in the No Tidegates Tidally-influenced Alternative and slightly higher overall water level in the Modified Tidegates and Additional Storage Ponds, Alternative #3 The Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative has the minimum peak water level, which drains off rapidly. It has a second flooding interval, which also drains off quickly. Thus, at Station 3 for the 10-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative provides the most effective design to reduce inundation.

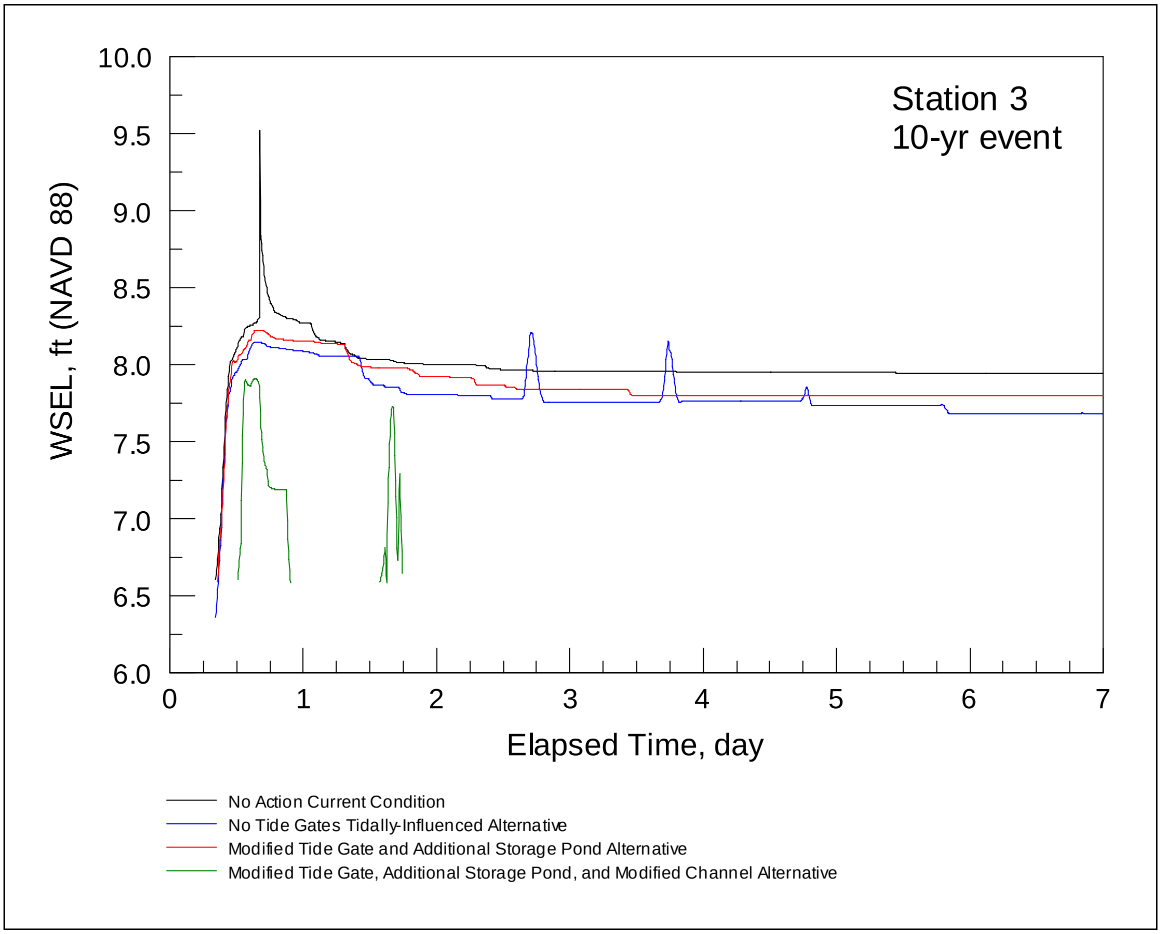

Figure 68. Time series of water-surface elevation at Station 3 for the 10-year event.

Time series of water-surface elevation at Station 4 for the 10-year event are shown in Figure 69. The No-Action Alternative, Modified Tidegates and Additional Storage Ponds Alternative, and the No Tidegates Tidally-Influenced Alternative show similar inundation properties, all having an initial peak water level that tapers off. Of these three alternatives, the No-Action Alternative has the greatest peak inundation and greatest retained water level over seven days. The Modified Tidegates and Additional Storage Ponds Alternative and the No Tidegates Tidally-Influenced Alternative show similar water-surface elevations, with the exception of tidal peaks in the No Tidegates Tidally-influenced Alternative and slightly higher overall water level in the Modified Tidegates and Additional Storage Ponds, Alternative #3 The Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative has the minimum peak water level, which reduces rapidly in elevation, and is followed by a second peak, which drains off quickly. Thus, at Station 4 for the 10-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative provides the most effective design to reduce inundation.

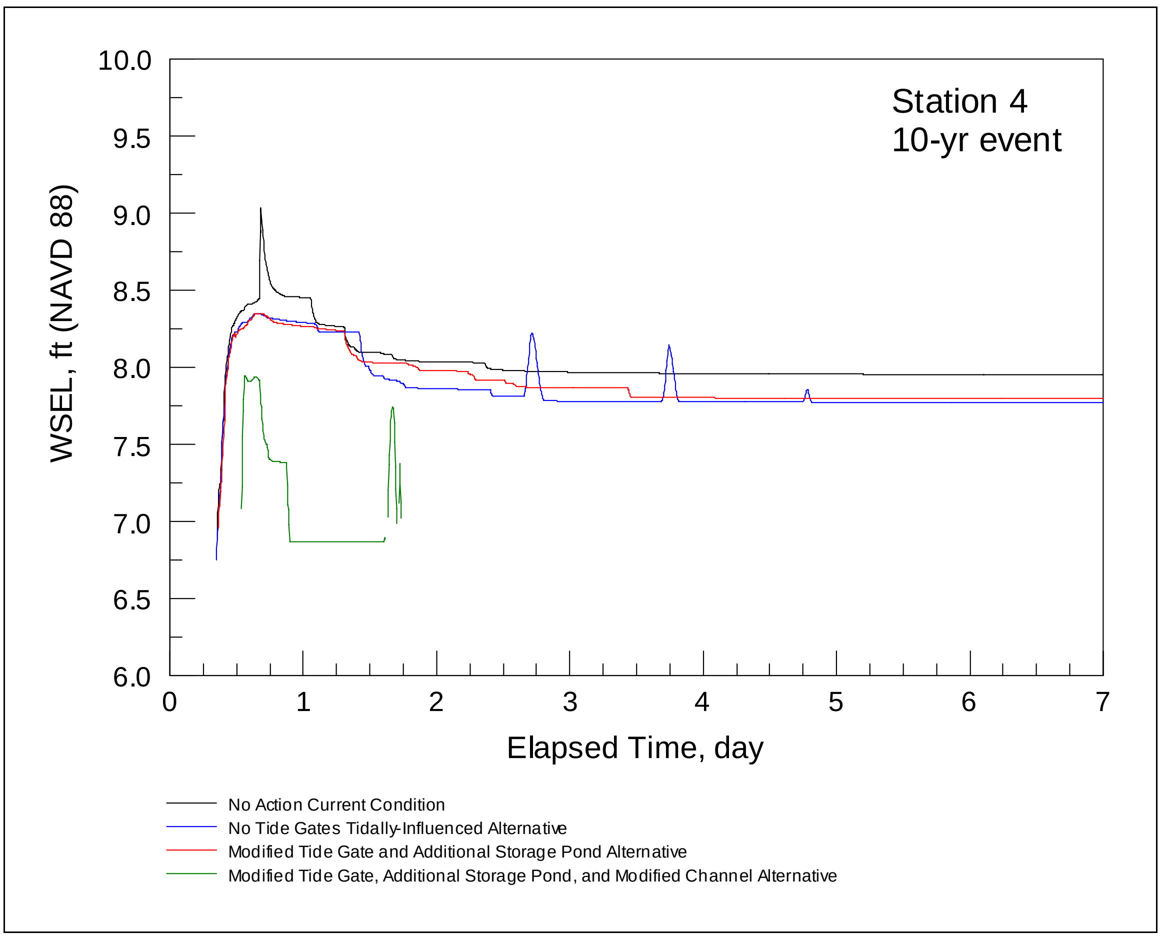

Figure 69. Time series of water-surface elevation at Station 4 for the 10-year event.

At the four stations for the 2-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative either did not experience flooding or inundation duration was very short with draining by the end of day 1. With the exception of Station 1, all other alternatives retained water for over 2 days, with durations increasing upstream. In many cases, inundation lasted for the entire simulation duration.

At the four stations for the 10-year event, the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative experienced inundation duration lasting less than 2 days, with the exception of Station 1, which did not flood for this alternative. All other alternatives were inundated for the entire 7 days at all four stations.

The significant improvement in inundation reduction for the Modified Tidegates, Additional Storage Ponds, and Modified Channel Alternative over the remaining alternatives owes to the greater conveyance capacity of the channel, together with rerouting of water from the southernmost tributary into the meander. By enlarging the channel, a greater volume of water can be transported downstream in a shorter period of time. Thus, water is more readily moved off of the upland areas, into the channel, and out of the system.

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Militello, A., and Kraus, N. C. 2001. *Shinnecock Inlet, New York, Site Investigation, Report 4, Evaluation of Flood and Ebb Shoal Sediment Source Alternatives for the West of Shinnecock Interim Project, New York*. Coastal Inlets Research Program Technical Report ERDC-CHL-TR-98-32. U. S. Army Engineer Research and Development Center, Vicksburg, MS.

Appendix C

Technical Advisory Committee

September 21, 2004
Martin Slough Technical Advisory Committee Meeting
Sign-in Sheet

Name	Organization	Phone	Address
Don Allan	RCAA - NRS	269-2063	904 G. St. Eureka 95501
Chuck Glasgow	NOAA Fisheries	825-5170	1655 Heindon Rd. Arcata 95521
Margaret Tauzer	NOAA Fisheries	825-5174	1655 Heindon Rd. Arcata 95521
Michelle Gilroy	DFG	441-5791	619 2nd St. Eureka 95501
Don Roller	Eureka Golf Course	443-1456	4750 Fairway Dr. Eureka 95501
Ray Davies	Course Co/Eureka	(650) 888-7120	1695 Foxwood Dr. Tracy, CA 95376-5315
Mike Zoppo	City of Eureka	441-4186	531 K St. Eureka 95501
Rob Burnett	Humboldt County - Public Works	445-7205	3033 H St. Eureka 95501
Alyson Hunter	Humboldt County - Planning	268-3731	3015 H St. Eureka 95501
Lisa Shikany	City of Eureka	268-5265	531 K St. Eureka 95501
Gene Senestraro	Land Owner	442-6396	510 Valley View, Eureka
Steven Allen	Winzler & Kelly	443-8326	633 Third St. Eureka 95501
Michele Copas	RCAA - NRS, GIS	269-2062	904 G. St. Eureka 95501

TAC Meeting No. 1

September 21, 2004

FINAL Project Criteria for

Martin Slough Enhancement Plan

1. Existing Land Uses
   - 1.1.1. Maximize Retention of Agricultural Land
   - 1.1.2. Maintain/Improve Eureka Municipal Golf Course
   - 1.1.3. Allow for full Build-out Potential for City/County
   - 1.1.4. Allow for Installation and Maintenance Access for City's Martin Slough Sewer Interceptor Project.

2. Flood Impacts
   - 2.1.1. Reduce Flood Inundation Area
   - 2.1.2. Reduce Frequency of Flooding
   - 2.1.3. Reduce Duration of Flooding

3. Fish Passage and Fish Access for Juveniles and Adults
   - 3.1.1. Maximize Migration Access at Tide Gates during Fish Migration Flows

4. Fish Habitat
   - 4.1.1. Maximize Estuarine Habitat
   - 4.1.2. Increase Channel Complexity

5. Riparian Corridor
   - 5.1.1. Increase Riparian Habitat
   - 5.1.2. Increase Riparian Canopy

6. Water Quality
   - 6.1.1. Decrease Nutrient Loading
   - 6.1.2. Decrease Sediment Load

7. Wetlands
   - 7.1.1. Improve Wetland Habitat
   - 7.1.2. Increase Area of Wetlands
   - 7.1.3. Increase Diversity of Wetland Types

8. Provide Realistic and Buildable Alternatives
   - 8.1.1. Minimize Required Earthwork
   - 8.1.2. Minimize Construction Costs
   - 8.1.3. Minimize Levee Footprint/Wetland Loss

TAC Meeting No. 2

November 30, 2004

Martin Slough Enhancement Plan

Project Status Report and Upcoming Schedule

1. Survey Complete
    1.1.1. Hydrologic sub-basins ground truth and digitized
    1.1.2. Limited physical survey of Martin Slough thalweg from Swain Slough through Eureka Municipal Golf Course to upper Fairway Drive complete

2. Hydrology
    2.1.1. Using HEC-HMS software
    2.1.2. Currently in calibration phase
    2.1.3. Having some difficulties with calibration based on limited stream gage data. We suspect gage near upper Fairway Drive was tidally influenced.

3. Hydraulics
    3.1.1. Hydraulic model ready to be started
    3.1.2. We have survey data but need hydrology results
    3.1.3. Hydraulic model alternatives to be developed include:
        1 - A "No Action" alternative (modeling existing conditions),
        2 - The 25 acre tidally influenced channel and wetland with modified tide gates as proposed by GMA,
        3 - A no-tide-gate tidally influenced channel and wetland option (approximately 10 acres in size),
        4 - A modified tide gate tidally influenced channel and wetland option to be developed (matching the same acreage and layout as the option above).

4. Upcoming Schedule

December	Finish hydrology; finish hydraulic model of existing conditions, start hydraulic model of three project alternatives
January	Conduct biological fieldwork, finish planning level base map of project area; finish draft hydraulic model of three project alternatives; develop draft results of hydraulic modeling to share with TAC; conduct TAC meeting to review planning level base map, review hydrology results, review draft results of hydraulic modeling, review draft report table of contents.
February	Further develop project alternatives, prepare draft report of findings; conduct TAC meeting to review draft report
March	Finalize report

Martin Slough Technical Advisory Committee Meeting Notes
January 27, 2005

The meeting started at 10 AM with welcome and introductions.
Those present for the meeting:

Don Allan – RCAA	Mike Zoppo – City of Eka.
Michele Copas- RCAA	Lisa Shikany – City of Eka.
Ray Davies - CourseCo	Michelle Gilroy - DFG
David Ammerman – USACE	Gary Boughton – City of Eka.
Margaret Tauzer – NOAA Fisheries	Keytra Meyes - NOAA Fisheries
Gene Senestraro – Property Owner	Bruce Perisho – Eka. Golf Course
Steve Allen – Winzler and Kelly	Don Roller – Eka. Golf Course
Michael Love, Michael Love and Associates	Alyson Hunter, Humboldt County Planning
Rob Burnett, Humboldt County Real Property	

After introductions, Don Allan passed around photos of the January 10, 2005 high tides, which the tide book listed as 8.4 feet. The photos drove home the need to have the Reardon property involved in the development of the enhancement plan. The extreme high tides are higher than the Swain Slough- Martin Slough levee and to prevent saltwater inundation during high tides the existing levee would need to be repaired and maintained at a higher elevation on both properties, unless a new levee was built on Gene's property and tied into the eastern valley wall on the golf course. That latter option could also require moving part of the creek onto Gene's property, assuming the property line is a fixed line (metes and bounds survey) and does follow the center line of the creek (the question as to which type of survey described Gene's boundary was discussed but Gene wasn't sure).

Don A. gave an update on landowner contacts. The attempts at contacting a representative of the Reardon Estate (Bill Thorington) were unsuccessful – Don will keep trying (Feb.3 note – Bill Thorington called on ????. Ed Frederickson, owner of the property between Gene's and the golf course, was contacted. He is generally supportive of the concept for the enhancement plan (reducing floodwater retention time/ improving flood and sediment routing), enhancing fish passage at the tide gates, and enhancing instream habitat but wasn't sure about the idea of levees. Mr. Frederickson sold one property immediately adjacent to the Slough – that landowner has not yet been contacted.

Mike Love passed around graphs of hydrology results and gave an explanation of the hydrology work done to date. Mike mapped all the drainages, gave the info to RCAA, Michele Copas digitized the land coverages by sub-watershed. Mike built a model (HMS – Army Corps) and calibrated the model using the data from the stream gage near the Fairway Drive crossing installed by Graham Matthews & Associates (GMA). Mike started with the standard SCS method. It is often used when no stream gauge data is available to calibrate to. The SCS method over-predicted the peak and had a quick drop off. Mike used data from the Little Rive gauge to compare results between Little River and Martin Slough. Little River had a similar pattern to Martin Slough and the flow dropped more slowly than the model predicted. The same occurred using data from the Elk River gauge. The SCS method was not accurate so Mike explored other methods. Mike did find a model that had a better fit to the data from the GMA stream gauge on Martin Slough. Mike is more concerned with the volume of the flow as opposed to the peak of the flow. Mike is modeling the hydrology of the watershed to estimate stream flow. The hydrology model does not take into account the hydraulics of the tidal effect – the hydraulic modeling is being done by Dr. Adele Militello of Coastal Analysis which does include tidal influences.

After studying the results of the hydrologic modeling, Mike believes the GMA stream gauge was influenced by the tides at lower flows – below 10 cubic feet per second (cfs). At higher flows, the gauge did not display backwater effects from high tides. Mike ran a couple of other models to see if they matched the gauged flow better and they did. Mike noted that most hydrologic modeling is done without real data to calibrate to. Fortunately our study does have real data and the model is matching the real data quite well. Mike ran the model for 2, 10, and 100 year events. The peak flows weren't as high as the SCS method predicted but the volumes are very high – there is a lot of storage in the gulches. The Oscar Larson and Associates report of 1989 used the SCS method which predicted a much higher peak flow than the calibrated method Mike is using. However, for this project, i.e., designing tide gates and levees, the volume is more important than the peak flow.

Adele is building a 2D hydraulic model for predicting the interaction of the tides and the stream flow.

Steve: we have data to calibrate to now and that is much better than trying to model without data.

Gary: the calibration at the Fairway Drive gauge location is different than at the tide gates.

Mike: we used the data at the Fairway Drive gauge to calibrate the model but the model can be applied to the whole watershed.

Gary: a good part of the watershed coming in below Fairway Drive is in timber.

Margaret: because the model is calibrated to the gauge, it can be applied to the entire watershed. How many rain gauges were used (in the GMA study)?

Mike – one – at the bottom of the Golf Course.

Steve: we are waiting for full build-out information from the County before modeling for the entire watershed. But, full buildout is not the key to the design because we are not modeling or designing for a 100 year event.

Steve passed around examples of the output from several modeling exercises and discussed the handouts.

We were using the Dynlet Model for the tidal modeling but Adele ran into errors in the program – it was not working properly. She contacted the person who developed the model and they found several more bugs, so we switched to a different model to keep this project moving ahead– the ADCIRC 2 D model. It uses different parameters and is not as easy to set up as Dynlet. The ADCIRC model is slow and bulky. It runs slower than real time – it took 7 days to model 5 days of real time. However, it does a better job at modeling flow through the tide gates at all tide levels. Design alternatives can be modeled to see how it affects the flood flows. The model also shows velocity vectors which helps with sediment transport - erosion problems and where sediment might settle out. Currently Adele is fine tuning the existing conditions with the model. Adele is getting a new computer with dual processors to speed up the modeling time and hopes to reduce the run time by 25%. We will also remove 2 northern tributaries from the surface model to help reducethe run time – it will still have the hydrologic flow input from the removed area. We have some data from the tide gates but it is very limited (Don -two gauges were installed – one on either side of the tide gates, and they were vandalized. One data collector was never found, the other one was – so we got a few days of data which were used to calibrate Martin Slough to the Humboldt Bay tide gauge).

Mike will talk about the tide gate configuration. He will do it in a spreadsheet and then put those preliminary results into the model, which will also speed up the modeling process.

Mike: the spreadsheet model looks at storage + flow in + flow out of the tide gates. The one week of data that we have to calibrate with is probably plenty. There is some leakage at the tide gates – the tides go up (on the inside of the tide gates) even when the gates are shut. (looking at the GMA graph) Whenever the green line is above the blue and red lines – it is high tide and the gates should be closed. The fact that the red and blue lines keep rising after the peak shows that the gates leak. (looking at page 7) – We can do quick iterations to find out what works best and then put that into the hydraulic model. There are lots of options for tide gates for flood routing and for fish passage.

Steve: regarding leaky gates – the new 2D hydraulic model does a better job of modeling them. Something we can’t deal with is levee overflow – as Don mentioned earlier, we need to assume that we will build up the levees along Swain Slough.

Don: I talked to Aldaron Laird who is working on a project on the Mad River Slough. (Don talked about planning for 100 years ahead, potential sea level rises due to global warming, the need to create a levee footprint big enough to allow for adding to the height of the levee in the future to account for a rise in the sea level – that the levee would become upland and could be added to in the future without filling in wetlands). Aldaron said that he was asking the Coastal Commission to allow 1:1 mitigation (and thought he might succeed) because he was creating a higher value wetland (salt/ brackish marsh) than the grazed wetland that he would be filling in. Don asked David what the Army Corps might allow.

David: one to one would be the minimum, it could be more.

A discussion ensued of the Mad River Slough - Lisa and Alyson expressed skepticism that 1:1 mitigation would be allowed and Don noted that Aldaron did not have his permits yet (that he was trying to get 1:1 mitigation. Follow-up note – Don checked the notes from his conversation with Aldaron. What Aldaron said was that the Federal Consistency Division of the Coastal Commission in San Francisco made a finding that restoring tidal marsh is a coastal dependent use and has a higher beneficial use than agriculture, which is not a coastal dependent use. Relocation of a dike to increase salt marsh is a restoration of a coastal dependent use that would qualify for a lower level of mitigation [i.e., 1:1]. Building a new dike to protect agricultural land is not a coastal dependent use and would require a higher level of mitigation. Several days after the meeting, Don talked with Jim Baskin at the Coastal Commission, gave him a quick update on the Martin Slough Project, and invited Jim to the next TAC meeting so Jim could explain the finding. Jim said he would try to attend.).

Gary: the USFWS had to do 2 or 3: 1 mitigation. The question is – is enhancement beneficial? NOAA Fisheries, DFG, USFWS – (need to weigh in)

Margaret: nothing is decided yet. There were discussions a few years ago regarding the grazing land.

Keytra: NOAA is consulting on a new project

Gene: were dikes breached?

Margaret: they let one island go back naturally – it is slowly degrading. It used to be pasture but they are letting it go back to salt marsh.

Mike: Salmon Creek is happening this year – it is a good test case.

Discussion (Keytra or Alyson or ?): Vance Dairy will be applying this summer. Rob's Duck hunting Club is the applicant. The Planning Commission approved the restoration – the pond – 50-60 acres- (in a non wetland area) will be grazed in the summer and will be a pond in the winter. (un-attributed comment) Vance Dairy is in litigation.

Goals for the next meeting:

- have the hydrologic model and hydraulic model of existing conditions completed
- look at alternatives – enhanced fish/ wildlife values, reduce flooding
- due to the speed of the hydraulic model, reluctant to state it as a goal, but – hoping to have reconnaissance level wetland mapping done if hydraulic modeling of alternatives is complete

Suggestion: try talk to Jim Baskin at the Coastal Commission.

The next Martin Slough Technical Advisory Committee Meeting was set for Tuesday, March 8, 10:30- 12:00, Room 207, Eureka City Hall.

Two workshops regarding estuary and fisheries restoration are occurring locally in the next couple of months – the Humboldt Bay Symposium at the Kate Buchanan Room (HSU, Arcata) March 14-15, and the Salmonid Restoration Federation Annual Conference at the River Lodge in

Fortuna, March 30 – April 2 (Don Allan is organizing a field tour of estuary restoration projects which will include stops at Salmon Creek, Martin Slough, possibly Rocky Gulch, Butcher Slough, and McDaniels Slough).

Adjourn.

Martin Slough Technical Advisory Committee

Tuesday, March 8, 2005, 10:30 AM

Room 207, City Hall, Eureka

1) Introductions

2) Recap of project goals and objectives for new attendees

3) Demonstration and discussion of hydraulic model; implications for design

4) Discussion of alternatives

5) Levee construction versus relocation/ reconstruction – permitting issues

6) Set goals for next meeting

7) Set next meeting date

8) Adjourn

Martin Slough Technical Advisory Committee Meeting Notes
March 8, 2005

Introductions and Project goals and objectives were skipped. Those were intended for those who were going to be attending their first Technical Advisory Committee meeting. Those in attendance were:

Michele Copas- RCAA-NRS	Margaret Tauzer – NOAA Fisheries
Keytra Meyer - NOAA Fisheries	Michelle Gilroy – DFG
Mike Zoppo – City of Eureka	Michael Love - Michael Love and Associates
Tom Hofweber (Hum. Co. Planning)	Don Roller – Eureka Golf Course
Gary Boughton – City of Eureka.	Steve Allen – Winzler and Kelly
Lisa Shikany – City of Eureka.	Rob Burnett – Hum. Co. Public Works
Don Allan – RCAA-NRS	

Start with item 3 on the agenda – demonstration of the hydraulic model.

Steve Allen – The hydraulic model is in the calibration phase based on the GMA (Graham Matthews and Associates) stream and tide gauging results to establish tidal predictions and stream flow. We are using the available low flow data for the calibration (based on the results of stream flow gauging which did not include any high flows during the period of gauging).

Currently, the model is running just under real time. It is a 2D finite element model, which models flow. It is being calibrated to lower tidal conditions because the GMA data was vandalized and did not produce enough high tide data to calibrate to. We only have 10 days worth of data, which is okay but of course it would have been ideal to have multiple days and more data with highs and lows. The water surface elevations measured at the gauge is not the same high tide as really high tide. Observations go with what we could expect high tide to be on both sides of the tide gates.
The high tide tops the dike at 8.0 ft.
We will start adding more input flows after the model is calibrated.
Photos were taken and are being used as visual qualitative data to compare to and look at the reasonableness of the model results at high tides.
The calibration phase should be done this week.

There are essentially 4 alternatives:

1) No Action – what we have now – these existing conditions will be useful for calibrating the model and for permitting purposes.

2) No Tide Gate Scenario –just remove the tide gates. This is useful for the pre-levee/ tide gate condition and for assessing the historical condition.

Lisa – Just so I am clear here, are you using a different type of modeling for different alternatives. Is it feasible to have no tide gates as an alternative? What is the purpose?

Steve – first it is the same model, and there are 2 reasons: 1) the SCC (State Coastal Conservancy) requested it for historical perspective; 2) it helps everyone to understand what effect removing the tidegates would have on the system

Some discussion followed as to what "no tide gates" means – does that mean no levees too?

Steve – no tide gates means we are looking to model no backwater effect from the tidegates. The goals for the no-tide-gate alternative are: 1) is it feasible – from a maintenance and flooding perspective; 2) what might the salt marsh have looked like historically

Margaret – it is best to remove all man-made topography.

Tom – there was some mapping of diked former tide lands in the Shapiro study in the early 80's (available at County Public Works).

Lisa – To clarify- are you planning to leave the levees in place?

Steve- Basically we are opening up the channel by removing the tide gates to define a new inundation area down there.

Margaret – You won't see any elevation change down there if you are leaving the levee there, with just the tide gates out.

Steve – We are pulling out enough of the levee in the model to remove the backwater effect -model for no levee influence and no tide gates. I'm not sure how the model will account for the levee. It is basically just a wall in the model right now.

A question was asked about how the model deals with the levee and how it would be modeled with no tide gates or levee influence

Mike Love – Not quite sure how the model works. I don't know if she (Adele) has taken that wall (boundary condition) out.

Margaret – County Public Works has old photos- talk to Andrew Glubzynski at 268-2687 – he's in the trailer on 2nd st.

Steve – The no tide gate alternative did not really come from Mike and I per se. Since we have model set-up it is good to look at options requested. Michael Bowen requested the project look at the historic perspective for permitting reasons.

Lisa – With the no tide gate alternative will you look what it would take to restore to natural state without taking out the levees?

Steve – we are looking at the level of inundation without the tide gates, not to fully restore the area to pre-levee conditions.

Tom – You have a significant levee with the railroad – you can model it but not easily make it go away.

Steve – Alternatives 3 and 4 are not nailed down yet but they are probably what everyone wants to see. The rough target is to see where the iterative process takes us – For alternative 3 we plan to take the existing channel condition, add some storage volume in the channel; then modify the tide gates to convey it out when the tide goes down. We need to have more storage for high tide without restoration of the whole channel. We can look at modifying tide gates so the design will work to help water move out at lower tide.

Don Allan – And we would be looking for a channel designed so outgoing tides carry sediment out (prevent sedimentation of the channel).

Don Roller – the existing ponds haven't been maintained in 5 years – aquatic vegetation is filling them in. They could have 2 benefits – 1) Flood Storage, and 2) Habitat.

Tom – There are a lot of assumed urban storm water activities. Maintenance in the upper part of watershed has got to happen.

Gary – even with storage, it is still going to flood due to a lack of relief.

Steve – the amount of flow through the tide gates also needs to increase to convey the water out at low tides.

Alternative 4 would be to modify the tide gates with the additional storage and also the channel using an iterative process.

Lisa – Permit every 5yrs. for pond maintenance. Keep it out of the Coastal Zone - not on Gene's property. Modeling system – storage is storage realistically and other maintenance issues arise.

Don Roller – we are willing to redesign golf course to make this work.

Don Allan – ponds within channel would be additional storage but would they become sediment traps and need maintenance?

Mike Love – Gene's property more salt aquatic vegetation less of an issue – more flushing regularly off channel.

Gary – John Murray used the Rational Channel and determined a 50-60ft wide channel was needed to convey the peak flow.

Steve – those methods over estimate the peak and under estimate the volume. It was a good first step and we hope to better define a new channel size with this model.

Don Allan –at high flows the valley is full of water and Swain Slough is not moving the water out.

Steve - modifications will not have an effect on a 100 year event; the modifications are to benefit the land owners, land use, habitat. We may be able to affect maybe a 2yr storm event.
It's all part of an iterative process- what happens at different flood events and what we can affect.

Don Allan - Elk River is open to tidal flushing

Gary – the railroad is a form of dike and there is the highway. You have a 90ft bridge that constricts an otherwise 1/2mile opening.

Steve – To improve the channel we do not have a set width. We are looking at the active channel rather than the flood plain channel for conveyance and velocity.

Margaret (?) – It comes down to low enough volume flood channel to flush it out, but do you have a target? You need to make sure channel is not too wide.

Mike Love – Adele's research is in intertidal influence and I am sure she is accounting for this …. it is her specialty.

Steve – We want to optimize flood conveyance and sediment transport out… Full sediment transport analysis is not part of the scope but can't be ignored. More or less it's velocities we can use to keep it clean….Like I said, it is an iterative process –to determine what the channel widths might be, what the tide gate volumes might be.

Tom – Redwood Creek Scenarios. They are modeling the set backs of levees and alternatives for estuary restoration.- re-configuring the mouth of Redwood Creek. It is tough to do and easy to point out the problems. The Regional Water Quality Control Board is doing a Lidar Flight of Elk River – not sure if it includes Martin Slough. Check with Adona at the regional board.

Steve – We have 2ft contours from the Interceptor project and we added some surveyed cross sections to ensure we had good data to use for the model.

Don Allan - What kind of accuracy does the Lidar have?

Mike Love – real good accuracy.

Margaret – I've heard good but also contradictions to that claim so it would be interesting to see.

Lisa – the HCP is long term.

Keytra - how long? I have to think about the HCP but I am currently consulting with the Corps and they typically only issue coverage for the extent of their permit.

Lisa – Section 10 …HCP?

Keytra – that sounds difficult.

Lisa – an individual permit is lengthy. Consider down the road problems that permitting can create.

Don Allan – We want to design this with the goal to have a self-maintaining system without dredging.

Steve – A low maintenance system has multiple benefits to land owners, cost and habitat

Tom – With permitting, the benefits of the enhancement/ restoration project need to be clear for the coastal commission perspective because if the proposed project just addresses flood control, then you can forget about it.

Don Allan – if the channel is routed through the old meander, the old channel (ditch) is needed to drain the hill side. We have talked about maybe creating a marsh in the current channel by the barn (the ditch) which could filter runoff from the barn area. The old channel is currently clogged with vegetation but riparian enhancement could create more shade which would prevent the channel from being choked by vegetation.

Lisa – What man made purpose was there in creating the channel?

Don Allan – To increase conveyance by straightening channel was just the old way of thinking.

Margaret – They just forgot about elevation.

Don Allan – I may have brought up the topic pre-maturely at the last meeting and I need to be careful about the words I use, but I looked back at the notes I made when I was talking to Aldaron and then talked about it with Jim Baskin. What Aldaron said was that the Federal Consistency Division of the Coastal Commission in San Francisco had made a finding that salt marsh is a coastal dependent use, whereas agriculture is not, and conversion of agricultural lands to wetlands is a higher beneficial use of coastal resources, therefore a lower level of mitigation would be required. But part of the issue is whether you are building new dikes or re-locating an existing dike – new dikes would require a higher level of mitigation but re-locating a dike could qualify for the lower level of mitigation

Keytra - when building new levees there is higher mitigation versus removing or relocating levees where there is decreased mitigation.

Steve – We are a lot better off if we do not have to deal with the levees.

Don Allan – I thought maybe the dredging was used to build up the levees but there is not really a levee along the creek.

Mike Love – to contain a 2-year flow is probably infeasible.

Steve – There is very little existing storage. To contain a 2 year storm we need a very large volume, or get it out in 1-2 tidal cycle versus 5-6 (as it is now).

Keytra – Follow-up on permitting, I am consulting on a plan in Rocky Gulch including maintenance as part of the proposed action. They are proposing maintenance every 5 years. I have a call into the Corps and will get back to you on it – we are still figuring out the term length of the agreement.

Don Allan – I wish Jim and Bill Thorington were here. I have had a chance to discuss this with him (Bill) and he is in favor of reducing the flooding. He is open to negotiation and has full authority over the estate which owns ½ of the old meander.

Michelle Gilroy – 1600 agreements – longer term (than one year) could be possible. I do not have all the details. I have calls into Redding.

Don Allan – I have heard of people asking for formal consultation versus informal consultation because with a formal, it starts the clock, but informal is open ended and you don't know how long it will take.

Keytra – Informal consultation does not require a biological assessment. Formal consultation starts the clock but a biological assessment needs to be complete and accepted.

Lisa (? – not sure if she said this or someone else) – when will we select the preferred alternative, what kind of environmental document will be needed, and who is doing the permitting

Steve – We are presenting data and a report with information so the group can choose. The idea is to present a conceptual analysis of different alternatives so a preferred alternative can be chosen; then find funding to go to full design; and then do the permitting.

Goals for next meeting:

- get Jim Baskin and Bill Thorington to attend the meeting
- have the hydrologic assessment completed

- work with Lisa and Tom to discuss what full build out in the watershed might be
- Michelle Gilroy will look into getting longer term 1600 agreements
- have the first two modeling scenarios completed
- work on developing the 3rd and 4th output alternatives (they won't be finished but we should have some idea of what they will look like).

Next Meeting Date - Tuesday April 19th at 10 AM in Room 207, City Hall, Eureka.

Martin Slough Technical Advisory Committee

Thursday, June 30, 2005, 10:00 AM

Room 207, City Hall, Eureka

1) Introductions

2) Discussion of study and modeling results

3) Review handouts

4) Upcoming schedule

5) Adjourn

Appendix D

Previous Work

MARTIN SLOUGH
HYDROLOGICAL MONITORING STUDY
WATER YEAR 2003

Prepared for:

Natural Resource Services
Redwood Community Action Agency
901G Street
Eureka, CA

Prepared by:

Graham Matthews and Associates
P.O. Box 1516
Weaverville, CA 96093
(530) 623-5327

November 2003

MARTIN SLOUGH HYDROLGICAL MONITORING STUDY – WATER YEAR 2003

Introduction

The purpose of this project was to collect hydrologic data at several locations on Martin Slough in Water Year (WY) 2003. The data was collected as part of the proposed Martin Slough restoration project. Uses of this data may include design criteria for creek and marsh restoration, flood routing and detention, and calibration of hydrologic and hydraulic models that may be used in the restoration planning and design phases.

This study was prepared for Redwood Community Action Agency (RCAA) by Graham Matthews and Associates (GMA) and with input and oversight by Randy Klein.

Scope and Objectives

The scope of this project was to collect hydrological data on lower Martin Slough. The WY 2003 work consisted of collecting field data through completion of the following tasks:

- Establish continuous stage monitoring stations upstream and downstream of the Swain Slough tidegates and at the upper end of the project area,
- Perform wading discharge measurements to develop a stage-discharge relationship at the upper continuous stage monitoring site,
- Install crest stage gages and/or stage measurements at four locations along lower Martin Slough,
- Install a rain gauge in the project area,
- Regularly download data collectors and inspect for proper functioning,
- Check crest stage gages at each site visit and record current water surface elevations.

Methods

Discharge Measurements

Streamflow measurements were taken at the upstream gaging site using standard or modified USGS protocols (Figure 1). All measurements were performed by wading near the gage location. Streamflow equipment for wading measurements included a 4 foot top-set wading rod, JBS Instruments AquaCalc 5000 -Advanced Stream Flow Computer, and magnetic head Price AA or Pygmy current meters.

Continuous Stage Recorders

Global Water continuous stage recorders were installed at three locations on Martin Slough (Figure 1). The furthest upstream recorder is located 50 feet downstream of the double concrete culverts that pass beneath Herrick Avenue. Two other recorders were located immediately upstream and downstream of the tidegates at Swain Slough to obtain paired data. Global Water Level Loggers are of a pressure transducer type, utilizing a silicon diaphragm and have an approximate 15 foot range. The pressure transducer at each site was downloaded on a monthly schedule using a laptop computer.

Rainfall Recorder

A rain gauge was installed beside the maintenance station at the Eureka Municipal golf course (Figure 1). The gauge was a tipping bucket rain gauge connected to a HOBO data logger. The unit measures rainfall in discrete events marked by time and date. Each event indicates 1/100 inch of rainfall. The instrument was downloaded regularly with a laptop computer.

Crest Stage Gages

Three crest stage gages were installed inside of the golf course in the Martin Slough channel (Sheet 1). Stage readings were also taken upstream and downstream of the culvert located on the lower portion of Martin Slough, just upstream of the tidegates (Sheet 1). A crest stage gage consists of a 2 inch PVC pipe mounted on a fence post. The pipe has holes in the top and bottom that allows water to enter as the stage rises. A piece of wood is placed inside of the pipe along with a small amount of granulated cork. The cork floats as water rises within the pipe and marks the maximum water surface elevation during a storm. The crest stage gages were surveyed to a common datum and subsequently linked together to provide a series of water surface elevations at discrete times.

Hydrologic Data Results

All water surface elevations presented in this study are referenced to the NAVD 1988 (NAVD88) datum, unless otherwise noted. All hydrologic data collected as part of this study are included on the attached CDROM.

Streamflow

Six wading streamflow measurement were taken between 17 February and 22 July 2003. The storm events that occurred following the establishment of the gaging stations were less significant than the storms occurring during the month of December. The measured discharges and observed staff heights were used to develop a stage-discharge rating curve for the uppermost site (Figure 2). A power function was used to establish the stage-discharge relationship ($R^2 = 0.9999$). To improve the power function fit, a stage offset of 1.9 feet was used.

Martin Slough Hydrograph

The discharge hydrograph for the gaging station (gage near Fairway Drive) is shown in Figure 3. All discharge values on the hydrograph above 28.5cfs are extrapolations of the stage-discharge relationship. The highest discharge estimated by the power equation was 49.5cfs, which occurred on 4 April 2003. If possible, high flow measurements should be taken next season to extend the rating curve, and to verify the extrapolated discharge values. Further investigation is also required to determine the stage which causes overbank flow and subsequent flooding onto the golf course and lower floodplain. Since overbank discharge would be difficult to measure along Martin Slough, the rating curve could be extended above bankfull discharge using a hydraulic model (e.g. HEC-RAS).

Precipitation

Figure 4 shows the cumulative rainfall recorded from 13 February to 22 July 2003. The data indicates a moderate to low amount of rainfall in February followed by consistent rainfall throughout most of March and April.

Paired Tidegate Data

The water surface elevations for the paired tidegate stage recorders are shown on Figure 5. Due to vandalism, the period of record is only from 12 to 20 February 2003. Also plotted on Figure 5 is the tidal stage for the North Spit Tidal Station (Station No. 9418767) corrected to NAVD88.

It should be noted that the tidegate stage recorders were vandalized shortly after installation, so only a partial record was obtained. One unit was completely destroyed and the other subsequently removed from the location.

Crest Stage and Measured Water Surface Elevation Data

Table 1 lists the observed crest stage gage elevations and/or measured water surface elevations at the stage reference sites. Refer to Figure 1 for locations and name references for each crest stage gage.

Recommendations for Future Work

The following recommendations are based on the information collected with regards to Martin Slough for WY 2003:

1. Continue the upstream gaging station for WY 2004,

2. Collect high flow discharge to extend the rating table to bankfull or slightly above in WY 2004,

3. Continue to collect peak stage values at the crest stage locations for WY 2004,

4. Continue to collect onsite rainfall for WY 2004.

Station	Date/ Time	WSE ft-NAVD88	Date/ Time	WSE ft-NAVD88	Date/ Time	WSE ft-NAVD88	Date/ Time	WSE ft-NAVD88	Date/ Time	WSE ft-NAVD88	Date/ Time	WSE ft-NAVD88
Gaging Station												
Gage Height	2/12/2003 13:39	6.52	2/17/2002 10:47	6.33	2/20/03 11:41	7.00	4/1/02 14:03	6.40	4/4/03 12:35	7.85	8/6/2003	5.70
PEAK			2/16/03 8:45	7.26	2/19/03 17:00	8.97	3/26/03 3:15	8.70	4/4/03 3:30	9.05		
#1 CSG	2/12/2003 14:00		2/16/03 8:45	7.14	2/19/03 17:00	8.53	3/26/03 3:15	8.34	4/4/03 3:30	8.70		
#1 Tpost	2/12/2003 14:00	5.19	2/17/2002 10:38	6.20	2/20/03 11:41	7.10	4/1/02 15:00	6.12	4/4/03 12:40	7.75	8/6/2003	5.29
#2 CSG	2/12/2003 14:17		2/16/03 8:45	7.07	2/19/03 17:00	7.76	3/26/03 3:15	7.46	4/4/03 3:30	8.01		
#2 Tpost	2/12/2003 14:17	4.67	2/17/2002 10:17	6.15	2/20/03 11:35	7.05	4/1/02 15:00	5.76	4/4/03 12:54	7.58	8/6/2003	4.55
#3 CSG	2/12/2003 14:38		2/16/03 8:45	7.17	2/19/03 17:00	7.71	3/26/03 3:15	7.48	4/4/03 3:30	8.02		
#3 Tpost	2/12/2003 14:38	4.68	2/17/2002 10:28	6.27	2/20/03 11:24	6.75	4/1/02 14:42	5.53	4/4/03 13:10	7.25	8/6/2003	4.58
Barn Culvert US			2/17/2002 10:08	6.11	2/20/03 11:04	6.38	4/1/02 14:30	5.26				
Barn Culvert DS			2/17/2002 10:08	6.33	2/20/03 11:04	6.03	4/1/02 14:30	5.33				
Tidegate GH US	2/12/2003 17:52	3.35	2/17/2002 10:00	6.27	2/20/03 10:59	6.21	4/1/02 14:23	5.44	4/4/03 13:24	6.77	8/6/2003	3.64
Tidegate GH DS	2/12/2003 17:41	0.53	2/17/2002 10:00	5.29	2/20/03 10:50	3.44	4/1/02 14:23	2.81	4/4/03 13:24	4.01	8/6/2003	3.13

WSE = water surface elevation
CSG = crest stage gage

PROJECT:

MARTIN SLOUGH HYDROLOGIC MONITORING STUDY – WY 2003

GMA
GRAHAM MATTHEWS & ASSOCIATES
Hydrology • Geomorphology • Stream Restoration
P.O. Box 1516 Weaverville, CA 96093-1516
(530) 623-5327 ph (530) 623-5328 fax

TABLE

1

		DATE	REV. DATE	PREPARED FOR	OWNED BY	
PREPARED BY	KB	8/03				**MARTIN SLOUGH HYDROLOGIC MONITORING STUDY, WY 2003** **Humboldt County, California**
REVIEWED BY	JKA	8/03				

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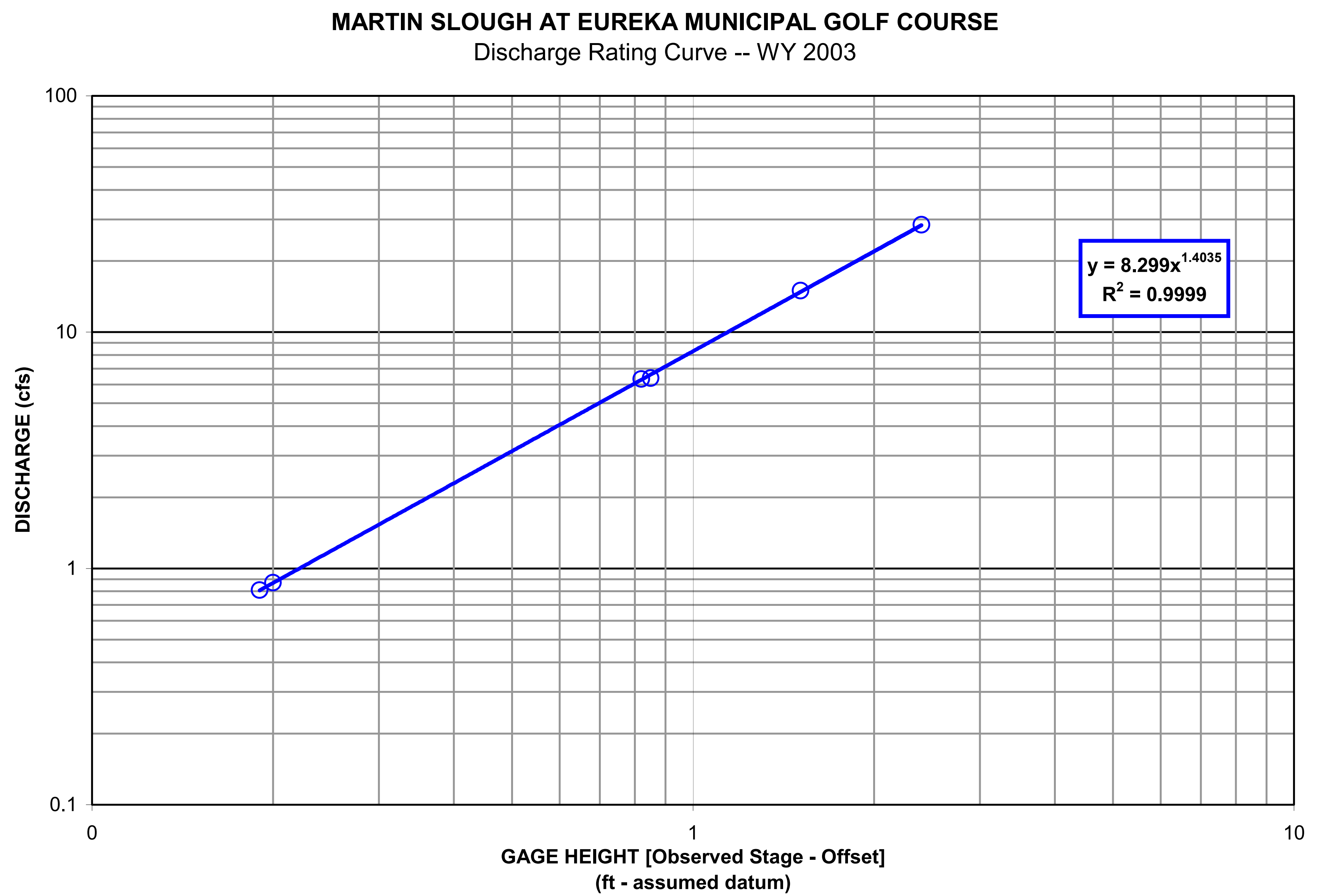
N

SCALE
0 200

FIGURE
1



PROJECT:

MARTIN SLOUGH HYDROLOGIC MONITORING STUDY – WY 2003

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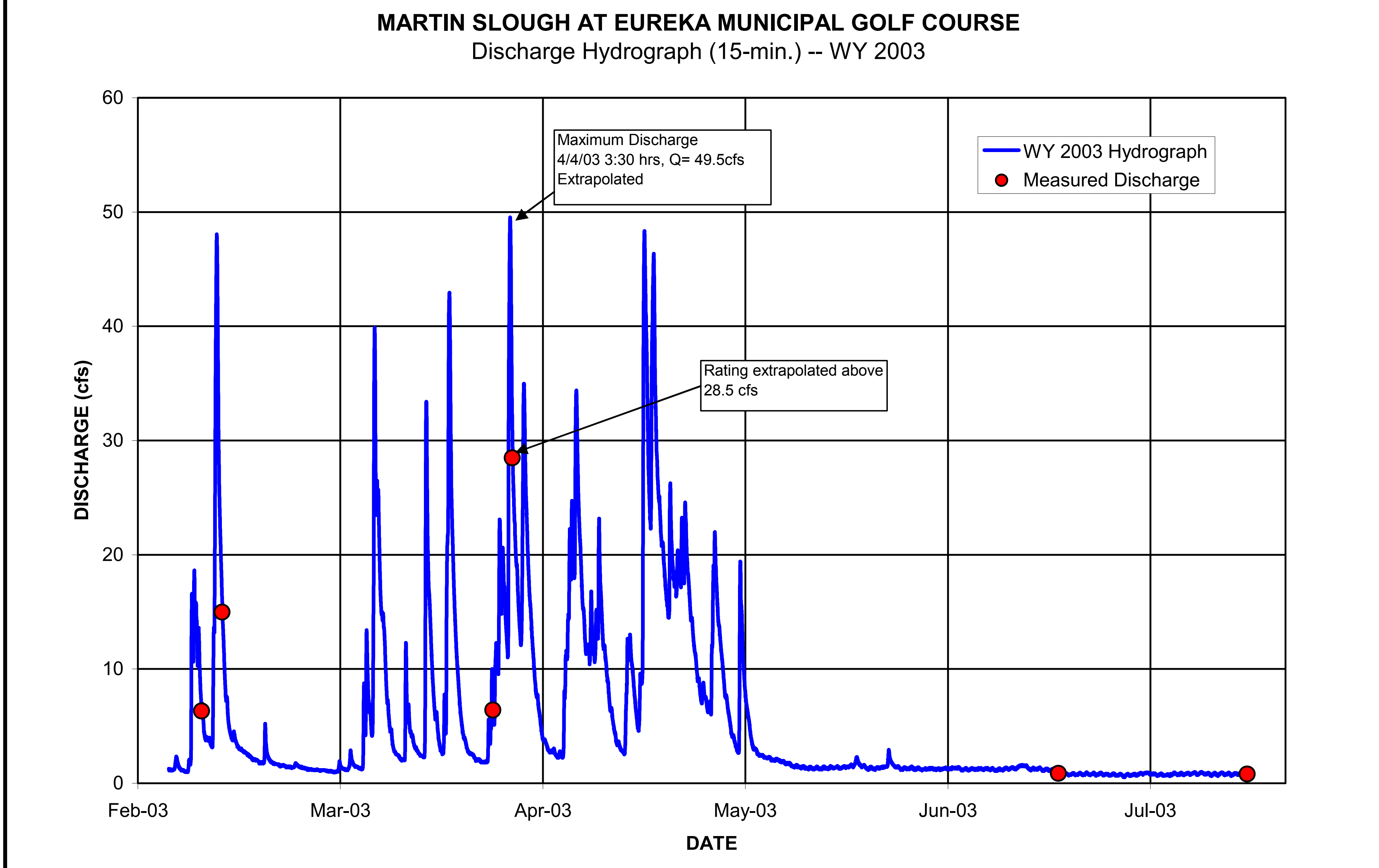
FIGURE

2



PROJECT: MARTIN SLOUGH HYDROLOGIC MONITORING STUDY – WY 2003	GMA GRAHAM MATTHEWS & ASSOCIATES Hydrology • Geomorphology • Stream Restoration P.O. Box 1516 Weaverville, CA 96093-1516 (530) 623-5327 ph (530) 623-5328 fax	FIGURE 3

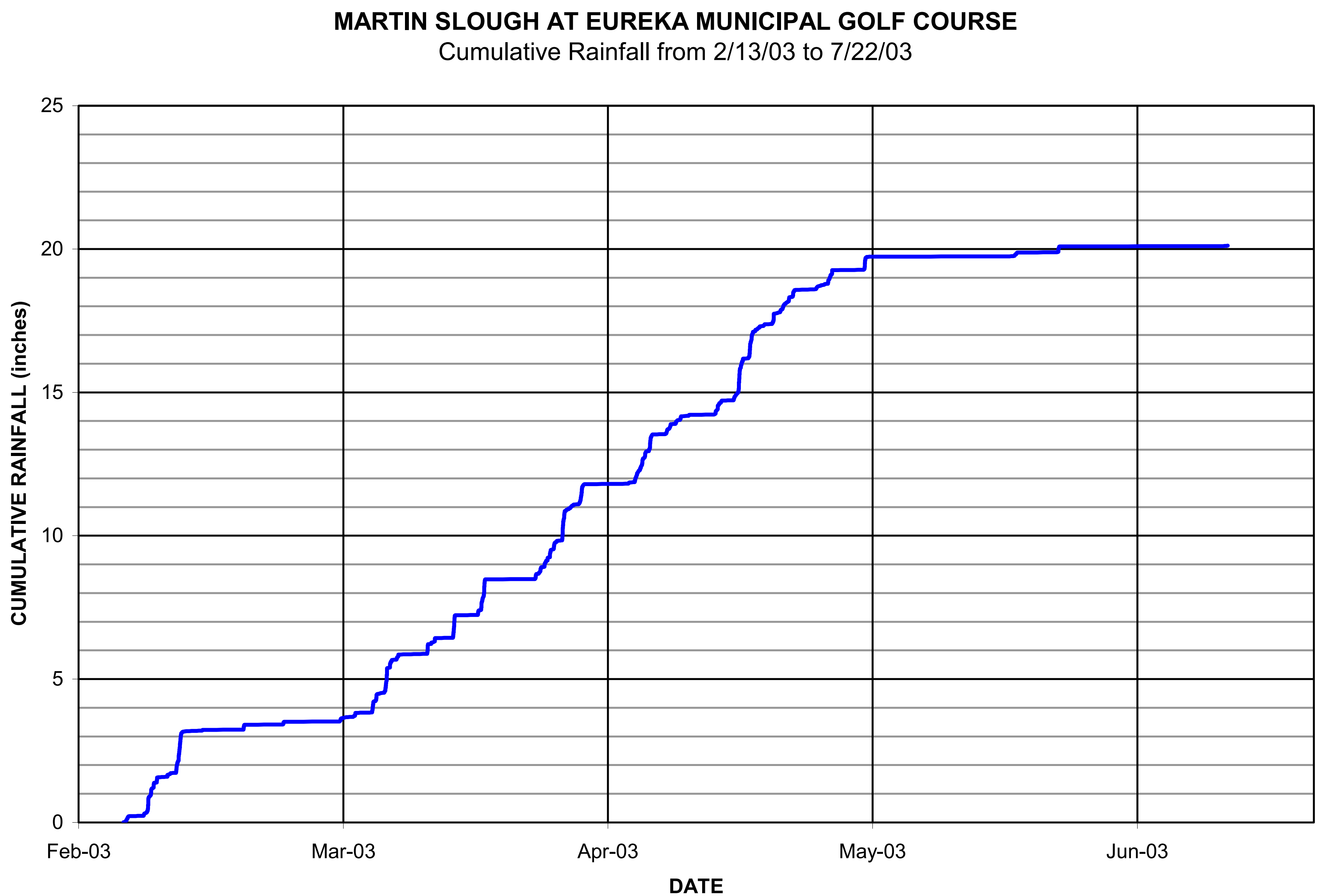


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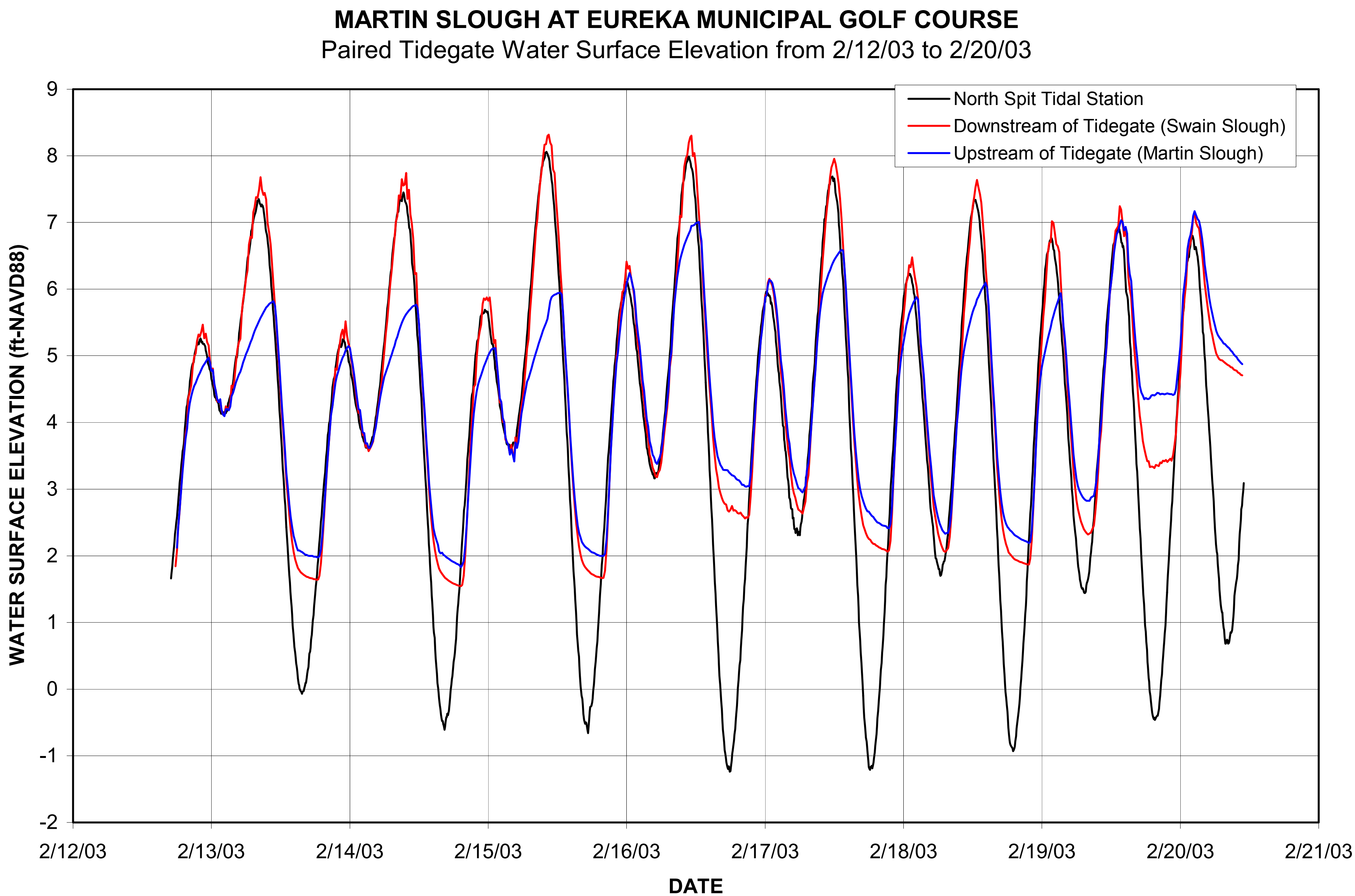
FIGURE

4



PROJECT: MARTIN SLOUGH HYDROLOGIC MONITORING STUDY – WY 2003	GMA GRAHAM MATTHEWS & ASSOCIATES Hydrology • Geomorphology • Stream Restoration P.O. Box 1516 Weaverville, CA 96093-1516 (530) 623-5327 ph (530) 623-5328 fax	FIGURE 5

Appendix E

Opinion of Probable Costs

Martin Slough Enhancement Plan
Order of Magnitude Estimate of Probable Construction Cost

Alternative 2 - No Tidegates Tidally Influenced System

Item No.	Item Name	Quantity	Unit	Unit Price	Total Line Item Price
1	Mobilization/Demobilization Costs	1	LS	$ 8,000	$ 8,000
2	Traffic Control	1	LS	$ 5,000	$ 5,000
3	Fisheries Biologist for Fish Removal	1	LS	$ 2,500	$ 2,500
4	Erosion and Sediment Control	1	LS	$ 3,000	$ 3,000
5	Demolition of Existing Tidegate and Removal of Levee Section	1	LS	$ 80,000	$ 80,000
6	Hauling and Disposal of Excavated Materials	1	LS	$ 15,000	$ 15,000
				Subtotal:	$ 113,500
				Estimating Contingency@ 30%:	$ 34,050
				New Subtotal:	$ 147,550

Engineers Opinion of Probable Construction Costs: $ 150,000

(Opinion of probable construction costs based on conceptual level plans on July 15, 2005)

Assumptions:

Assume Fish Removal Efforts for Fisheries Biologist and Technician for 1 fish rescue effort
Demolition of Tidegate and Breaching of Levee Includes Control of Water

Martin Slough Enhancement Plan
Order of Magnitude Estimate of Probable Construction Cost

Alternative 3 - New Tidegates and Storage Ponds

Item No.	Item Name	Quantity	Unit	Unit Price	Total Line Item Price
1	Mobilization/Demobilization Costs	1	LS	$ 95,000	$ 95,000
2	Traffic Control	1	LS	$ 30,000	$ 30,000
3	Fisheries Biologist for Fish Removal	1	LS	$ 7,500	$ 7,500
4	Erosion and Sediment Control	1	LS	$ 15,000	$ 15,000
5	Demolition of Existing Tidegate and Installation of New Tidegates	1	LS	$ 350,000	$ 350,000
6	Temporary Haul Roads	1	LS	$ 70,000	$ 70,000
7	Temporary Construction Fence	1	LS	$ 15,000	$ 15,000
8	Control of Water (Dewater Ponds)	1	LS	$ 10,000	$ 10,000
9	Excavation (Ponds)	90,000	CY	$ 5	$ 450,000
10	Hauling and Disposal of Excavated Materials	90,000	CY	$ 6	$ 540,000
11	Riparian Fencing	3,000	LF	$ 15	$ 45,000
12	Swain Slough Levee Repair/Maintenance	1	LS	$ 150,000	$ 150,000
13	Re-Vegetation (Ponds)	5	Acre	$ 5,000	$ 25,000
14	Year 1 Monitoring and Maintenance	1	LS	$ 15,000	$ 15,000

Subtotal:	$ 1,817,500
Estimating Contingency@ 30%:	$ 545,250
New Subtotal:	$ 2,362,750
Engineers Opinion of Probable Construction Costs:	**$ 2,400,000**

(Opinion of probable construction costs based on conceptual level plans on July 15, 2005)

Assumptions:

2 mile one way hauling distance for disposal of excavated materials (unknown disposal site)
Assume Fish Removal Efforts for Fisheries Biologist and Technician for 3 fish rescue efforts
Tidegate Item Includes Demolition and Control of Water

Martin Slough Enhancement Plan
Order of Magnitude Estimate of Probable Construction Cost

Alternative 4 - New Tidegates, Storage Ponds, and Channel Improvements

Item No.	Item Name	Quantity	Unit	Unit Price	Total Line Item Price
1	Mobilization/Demobilization Costs	1	LS	$ 125,000	$ 125,000
2	Traffic Control	1	LS	$ 50,000	$ 50,000
3	Fisheries Biologist for Fish Removal	1	LS	$ 15,000	$ 15,000
4	Erosion and Sediment Control	1	LS	$ 50,000	$ 50,000
5	Demolition of Existing Tidegate and Installation of New Tidegates	1	LS	$ 350,000	$ 350,000
6	Temporary Haul Roads	1	LS	$ 150,000	$ 150,000
7	Temporary Construction Fence	1	LS	$ 25,000	$ 25,000
8	Control of Water (Dewater Ponds)	1	LS	$ 10,000	$ 10,000
9	Control of Water (In Creek Channel)	1	LS	$ 75,000	$ 75,000
10	Excavation (Ponds)	90,000	CY	$ 5	$ 450,000
11	Excavation (Channel)	50,000	CY	$ 6	$ 300,000
12	Hauling and Disposal of Excavated Materials	140,000	CY	$ 6	$ 840,000
13	Retrofit Existing Bridge Footings at Fairway Drive	1	LS	$ 75,000	$ 75,000
14	Riparian Fencing	10,000	LF	$ 15	$ 150,000
15	New Bridge for Farmer Access	2	LS	$ 125,000	$ 250,000
16	Swain Slough Levee Repair/Maintenance	1	LS	$ 150,000	$ 150,000
17	Lower Utilities that Cross Creek Channel	1	LS	$ 60,000	$ 60,000
18	Re-Vegetation (Ponds)	5	Acre	$ 5,000	$ 25,000
19	Re-Vegetation (Channel)	5	Acre	$ 7,000	$ 35,000
20	Year 1 Monitoring and Maintenance	1	LS	$ 40,000	$ 40,000

Subtotal: $ 3,225,000
Estimating Contingency@ 30%: $ 967,500
New Subtotal: $ 4,192,500

Engineers Opinion of Probable Construction Costs: $ 4,200,000
(Opinion of probable construction costs based on conceptual level plans on July 15, 2005)

<u>Assumptions:</u>
2 mile one way hauling distance for disposal of excavated materials (unknown disposal site)
Assume Fish Removal Efforts for Fisheries Biologist and Technician for 7 fish rescue efforts
Tidegate Item Includes Demolition and Control of Water
Assume 10 feet of Revegetation on Each Side of Channel

Appendix F

Fisheries and Water Quality Sampling

Field Note

Martin Slough, Thence Swain Slough, Thence Elk River, Thence Humboldt Bay
Eureka Golf Course Property

Redwood Community Action Agency is leading a planning effort to develop alternatives for enhancement of the Martin Slough channel from upper Fairway Drive to the tide gates at Swain Slough. Fisheries and water quality sampling ((with a Yellow Springs Instruments (YSI) meter)) was conducted within the golf course property of Martin Slough to document the current water quality conditions and to gain additional data regarding salmonid use within the stream and ponds. Sampling was conducted on July 7th by DFG, July 14th by DFG and RCAA, and August 12th, 2005 by DFG.

July 7th, 2005

Pond 1 – Tributary Pond (5th and 6th Fairways)

Air and water temperatures at 1100 were 18.9°C and 17.8°C, respectively.
A total of 10 baited minnow traps were set along the right bank (as looking downstream) of downstream end of pond. Traps were placed 2 to 5 feet from the bank at depths ranging from 2 to 4 feet deep. Five of the traps were set in an area without riparian vegetation and five were set amongst riparian. Individual traps were fished for 20 to 30 minutes each. A total of sixteen stickleback were captured.

Martin Slough – downstream of upper Fairway Drive culvert (9th Fairway)

Air and water temperatures at 1215 were 16.5°C and 15.5°C, respectively.
Two baited minnow traps were set mid-channel in depths of 2 to 3 feet for 45 minutes each. A total of 4 coho salmon and 15 stickleback were captured. The coho salmon lengths were 70, 76, 82, and 84 millimeters.

Pond 2 – Martin Slough Pond (Downstream of lower Fairway Drive bridge – between 17th and 18th Fairways)

Air and water temperatures at 1245 were both 18.0°C.
Three baited minnow traps were set 3-4 feet from the right bank (looking downstream) of the pond amongst riparian vegetation for 35 minutes each. Water depths were 2 to 3 feet. A total of 3 stickleback were captured.

July 14th, 2005

Pond 1 - Tributary Pond (5th and 6th Fairways)

Only water quality measurements were taken within Pond 1. It was determined that seining within this pond would be difficult due to the amount of vegetation present. A future trip to set minnow traps and perform water quality measurements from a small boat in the middle of the pond may be worthwhile. The following are the water quality measurements recorded:

Downstream End of Pond 1:
Time: 0810

Location	Depth (ft)	Water Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)***	Conductivity (µS/cm)
Surface	1.0	18.4°C	0.1	3.72	219.3
Middle	4.0	17.4°C	0.2	0.19	360.1
Bottom	5.0	18.5°C	0.1	3.82	216.0

Upstream End of Pond 1:
(2 western pond turtles were observed at this site)

Location	Depth (ft)	Water Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)***	Conductivity (µS/cm)
Surface	<0.5	18.4°C	0.1	5.80	212.4
Bottom	N/A	17.7°C	0.2	0.54	314.3

Creek Channel Upstream of Pond 1:

Location	Depth (ft)	Water Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)***	Conductivity (µS/cm)
Surface	<0.5	18.7°C	0.1	2.07	219.0

Pond 2 - Martin Slough Pond (Downstream of lower Fairway Drive bridge – between 17th and 18th Fairways):

Middle of Pond 2:
Maximum depth of 4.4 feet

Time: 0923

Location	Depth (ft)	Water Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)***	Conductivity
Surface	<0.5	17.6°C	0.1	3.01	210.7μS/cm
Middle	2.2	17.1°C	0.1	3.01	210.8μS/cm
Bottom	4.0	18.9°C	3.2	0.28	5.74mS/cm

Downstream End of Pond 2:
Maximum depth of 4 feet

Location	Depth (ft)	Water Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)***	Conductivity (μS/cm)
Surface	<0.5	17.8°C	0.1	3.47	202.2
Middle	2.0	16.9°C	0.1	2.95	187.4
Bottom	4.0	18.5°C	1.4	11.87	4663

Seining was conducted from a small skiff at the downstream end of Pond 2 with a 100' long by 5' deep seine (1/8" bag mesh and ¼" net mesh).

One 160 mm cutthroat trout, 147 coho salmon, as well as numerous stickleback were captured. 30 coho salmon were measured and 117 were tallied. Fork lengths (in millimeters) of the 30 measured coho salmon are listed in the table below:

65	70	64
95	87	75
80	97	67
85	103	74
74	85	107
77	72	87
80	86	73
72	72	85
81	90	83
78	80	84

At 1120, one 30' seine pull was also conducted at the upstream end of Pond 2.
Air temperature 17°C; water temperature 18.9°C
Numerous stickleback, tadpoles, and macroinvertebrates (numerous water boatman) were captured.

Martin Slough (downstream of Pond 2):

4 baited minnow traps were placed in Martin Slough downstream of Pond 2 (in between Pond 2 and the downstream end of the golf course property) for approximately an hour each (~1140 to1240). Only stickleback were captured.

Tributary to Martin Slough (between 16th and 17th Fairways):

2 baited minnow traps were placed in the tributary for approximately an hour each (~1155-1255). One trap captured one 62mm coho salmon and one stickleback (the trap was placed 200 to 300 feet upstream of Martin Slough). The 2nd trap, placed just upstream of the small, heavily vegetated pond, captured stickleback only.

Problems with the water quality meter occurred at this site, therefore, only air and water temperatures are reported here:
Time: 1349
Air temperature: 22°C
Water temperature: 18.1°C

Martin Slough (downstream end of golf course property at first golf cart bridge upstream of property fence line):

Conducted one 30 foot seine pull.
2 prickly sculpin and numerous stickleback (approximately 24) were captured.

Time: 1155

Location	**Depth (ft)**	**Water Temperature (°C)**	**Salinity (ppt)**	**Dissolved Oxygen (mg/l)*** **	**Conductivity (µS)**
Surface	<0.5	18.5°C	0.1	4.0 mg/l	210

Martin Slough (just downstream of upper Fairway Drive)

One 30' seine pull was conducted---3 foot deep maximum

Time: 1318
Air temperature: 20°C

Location	**Depth (ft)**	**Water Temperature (°C)**	**Salinity (ppt)**	**Dissolved Oxygen (mg/l)*** **	**Conductivity (µS)**
Surface	<0.5	17.1°C	0.1ppt	2.62	164.9

Prickly sculpin, stickleback, newt, pacific giant salamander, and a coho salmon (62mm) were captured.

***** It is possible that dissolved oxygen measurements taken on July 14, 2005 were inaccurate. For this reason, water quality measurements for Pond 2 area (where coho salmon were captured) were repeated August 12, 2005 and data is reported below).**

August 12, 2005
Water Quality Sampling

Ten water quality measurements were recorded on August 12, 2005 within Pond 2 (where coho salmon were captured on July 14, 2005) to further document water quality conditions and compare suspect measurements taken on July 14, 2005.

Water quality measurements for Sites One through 8 were taken within Pond 2 – Martin Slough Pond. Sites 4 and 6 were located in the area of the pond where seining on July 14, 2005 resulted in the capture of coho salmon and cutthroat trout.

Site 9 was located near where 2 minnow traps were placed on July 14th (downstream of first bridge downstream of Pond 2), and Site 10 was located at the lower Fairway Drive bridge.

The results of the water quality sampling are as follows:

Pond 2 - Martin Slough Pond (Downstream of lower Fairway Drive bridge – between 17th and 18th Fairways):

Site 1:
Time: 1005

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Surface	~0.5	16.3	0.1	3.00	253.3
Middle	1.0	15.1	0.1	2.90	224.0
Bottom	2.0	15.2	0.2	3.00	288.5

Site 2:
Time: 1015

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Surface	~0.5	15.5	0.1	2.55	221.9
Middle	1.5	14.9	0.1	2.11	218.0
Bottom	3.0	16.3	0.8	3.03	1206.0

Site 3:
Time: 1028

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Surface	~0.5	15.3	0.1	2.93	191.4
Middle	1.5	15.0	0.1	2.53	184.1
Bottom	3.0	16.3	0.8	3.09	1368.0

Site 4:
Time: 1035

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Surface	~0.5	15.4	0.1	2.19	182.9
Middle	1.00	14.9	0.1	3.01	179.4
Bottom	2.25	14.6	0.1	2.37	211.2

Site 5:
Time: 1040

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Surface	~0.5	15.8	0.1	3.87	186.1
Bottom	>1.0	N/A	N/A	N/A	N/A

Site 6:
Time: 1045

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Middle	1.00	15.1	0.1	2.55	184.5
Bottom*	1.75	N/A	N/A	N/A	N/A

* A bottom measurement was not taken, so another water quality bottom measurement should be taken to compare with data taken on July 14th.

Site 7:
Time: 1055

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Middle	1.0	15.2	0.1	2.68	190.9
Bottom	2.0	15.0	0.2	4.33	252.7

Site 8:
Time: 1105

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Surface	~0.5	16.6	0.1	2.93	219.6
Middle	1.25	15.3	0.2	2.33	274.5
Bottom	2.5	16.2	0.5	1.16	815.0

Upstream of Pond 2 at Fairway Drive:
Site 9:
Time: 1120

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Bottom	0.5	15.2	0.1	3.46	201.8

Downstream of Pond 2:
Site 10:
Time: 1130

Location	Depth (ft)	Temperature (°C)	Salinity (ppt)	Dissolved Oxygen (mg/l)	Conductivity (µS)
Bottom	0.5	14.3	0.1	2.08	154.7

Further water quality and fisheries sampling is recommended in order to acquire additional baseline data prior to restoration project implementation. This would be an ideal opportunity to begin a year-round biological sampling program to document pre-project conditions for several years prior to restoration work that may occur in the future.

Photographs of the Martin Slough sample sites are attached. Photographs were taken by DFG and Don Allan of Redwood Community Action Agency.

Written by:
Michelle M. Gilroy
District Watershed Biologist
California Department of Fish and Game
Eureka
August 26, 2005