GENERAL RE-EVALUATION REPORT
APPENDIX C: ENGINEERING
SAVANNAH HARBOR EXPANSION PROJECT
Chatham County, Georgia and Jasper County, South Carolina

January 2012

US Army Corps of Engineers
Savannah District
South Atlantic Division

This report is **Appendix C**
to the **General Re-Evaluation Report** for
the **Savannah Harbor Expansion Project**
located in Chatham County, Georgia and
Jasper County, South Carolina.

Acronyms and Abbreviations

AFB	Alternative Formulation Briefing
AIWW	Atlantic Intracoastal Waterway
ANN	Artificial Neural Networks
ASA	Assistant Secretary of the Army
ATM	Applied Technology and Management, Inc.
ATR	Agency Technical Review
BL	Base/Bilge Line
BR	Back River
CADET	Channel Analysis & Design Evaluation Tool
CB	Container Berth
CDF	Confined Disposal Facilities
CEDEP	Corps of Engineers Dredge Estimating Program
CH	High liquid limit clay
CHL	Coastal and Hydraulics Laboratory
CL	Low liquid limit clay
CWCCIS	Civil Works Construction Cost Index System
CWE	Current Working Estimate
CZM	Coastal Zone Management
DMCA	Dredged Material Containment Areas
DMMP	Dredged Management and Material Plan
DO	Dissolved Oxygen
DOT	Department of Transportation
DOTS	Dredging Operations Technical Support
DQC	District Quality Control
EFDC	Environmental Fluid Dynamics Computer Code
EIS	Environmental Impact Statement
EN	Engineering
EPA	Environmental Protection Agency
ERDC	Engineering Research and Development Center
EWT	Effective Work Time
FEMA	Federal Emergency Management Agency
FIS	Flood Insurance Study
GAEPD	Georgia Environmental Protection Division
GIS	Geographic Information System
GP	Poorly graded gravels
GPA	Georgia Ports Authority
GRR	General Reevaluation Report
HH&C CoP	Hydrology, Hydraulics and Coastal Community of Practice
HTRW	Hazardous, Toxic, and Radioactive Waste
IEPR	Independent External Peer Review

IP	International Paper
IWR	Institute for Water Resources
JOI	Jones/Oysterbed Island
KITB	King's Island Turning Basin
LNG	Liquefied Natural Gas
LTMS	Long Term Management Strategy
M2M	Model-to-Marsh
MCACES	Microcomputer Aided Cost Estimating System
MGD	Million Gallons per Day
MH	High liquid limit silt
MHW	Mean High Water
ML	Low liquid limit silt
MLLW	Mean Lower Low Water
MLW	Mean Low Water
MR	Middle River
MSM	Marsh-succession Model
MSL	Mean Sea Level
MTL	Mean Tide Level
MTRG	Modeling Technical Review Group
NAVD	North American Vertical Datum
NED	National Economic Development
NGVD	National Geodetic Vertical Datum
NMFS	National Marine Fisheries
NOAA	National Oceanic and Atmospheric Administration
NSBL&D	New Savannah Bluff Lock and Dam
O&M	Operations and Maintenance
ODMDS	Ocean Dredged Material Disposal Site
OH	Organic silts
OSHA	Occupational Safety and Health Administration
OSI	Ocean Surveys, Incorporated
PAH	Polycyclic Aromatic Hydrocarbons
PCB	Polychlorinated Biphenyls
PD	Planning Division
PDT	Project Design Team
PED	Planning, Engineering, and Design
PIANC	World Association for Waterborne Transport Infrastructure
RMSE	Root Mean Square Error
S&A	Supervision and Administration
SAD	South Atlantic Division
SAS	Savannah District
SC	Clayey sands
SCDHEC	South Carolina Department of Health and Environmental Control

SEG	Stakeholders Evaluation Group
SET	Scientific and Engineering Technology
SHEP	Savannah Harbor Expansion Project
SM	Silty sands
SNG	Southern Natural Gas
SNS	Shortnose sturgeon
SOD	Sediment Oxygen Demand
SP	Poorly graded sands
SPT	Standard Penetration Test
SRBA	Savannah River Below Augusta
STS	Ship Tow Simulator
TMDL	Total Maximum Daily Load
TPCS	Total Project Cost Summary
TS	Technical Services
USACE	United States Army Corps of Engineers
USCG	United States Coast Guard
USFWS	United States Fish and Wildlife Service
USGS	United States Geological Survey
VE	Value Engineering
WASP	Water quality Analysis Simulation Package

Table of Contents

List of Figures

List of Tables

Attachments

ATTACHMENT 1
Channel Design Drawings and Typical Sections

ATTACHMENT 2
MCACES-MII Cost Estimate and Total Project Cost Summary (TCPS) Agency Technical Review (ATR) Certification

ATTACHMENT 3
Engineering Investigations Supplemental Materials (CD)

ATTACHMENT 4
Project Cost and Schedule Risk Analysis Report

1.0 INTRODUCTION

The purpose of this appendix is to present the results of engineering studies, investigations, and analyses that have been performed in developing the recommended project improvements for the Savannah Harbor Expansion Project (SHEP).

The current Savannah Harbor Expansion project authorization included in Section 101 of the Water Resources Development Act of 1999 authorized a deep draft navigation project up to a depth of 48 ft below MLLW subject to further evaluation by the agencies and concurrence by the Secretaries of the Army, Commerce, and Interior and the Administrator of the Environmental Protection Agency (EPA). Given the unique authorization of this project, any final recommendation of a preferred plan must meet the requirements of the legislation. Therefore, the engineering evaluations were performed for project depth alternatives ranging from 44 to 48 ft below MLLW, a maximum of 6 ft below the currently authorized navigation channel at 42 ft below MLLW. **Table 1.0-1** shows the plan descriptions for each depth alternative.

Within this range of feasible depths, an NED plan of 47 ft below MLLW was selected, which complies with Army policy. Prior to release of the Draft GRR and EIS for agency and public comment, the State of Georgia asked the Corps to consider the 48-foot depth alternative as the Locally Preferred Plan. As a result of comments received and subsequent discussions with the sponsor, the Corps declined to select the 48-foot alternative for implementation.

Conclusions presented in this appendix are based on previous studies, studies performed specifically for this project, field investigations, laboratory analyses, numerical modeling, available data and engineering experience in the project area over the full range of project alternatives (44 ft to 48 ft below MLLW). Technical studies were performed by the USACE Savannah, Charleston, Mobile and Wilmington Districts, the USACE Engineering Research and Development Center (ERDC) Waterways Experiment Station, other government agencies, such as the US Geological Survey and the US Fish and Wildlife Service, and contract technical consultants. Input from project users and the Stakeholders Evaluation Group (SEG) was also considered in evaluating alternatives.

The major design elements evaluated in this phase of the project are the refinement of the proposed channel alignment; impacts to the groundwater aquifer, water quality, marine habitat, and barrier islands; placement of the dredged material and disposal area capacity; and the effect deepening may have on water surface elevations, current velocity, and sedimentation in the navigation channel. USACE guidance used in preparation of this appendix includes:

ER 1110-2-1150, *Engineering and Design for Civil Works Projects*, 31 August 1999
ER 1110-2-1403, *Studies by Coastal, Hydraulic, and Hydrologic Facilities and Others*, 1 January 1998
EM 1110-2-1613, *Hydraulic Design of Deep-Draft Navigation Projects*, 31 May 2006
ER 1110-2-1302, *Civil Works Cost Engineering*, 15 September 2008
EC 1165-2-212, *Sea-Level Change Considerations for Civil Works Programs*, 1 October 2011
EM 1110-2-1100, *Coastal Engineering Manual*, 2002
EM 1110-2-2504, *Design of Sheet Pile Walls*, 31 March 1994
EM 1110-2-1601, *Hydraulic Design of Flood Control Channels*, 30 June 1994
EM 110-2-1607, *Tidal Hydraulics*, 15 March 1991

The results from the vast spectrum of engineering investigations completed for this project are outlined and summarized within this document. The related reports, drawings, and correspondence that went into the development of this document are provided in full in the Engineering Investigations Supplemental Materials in electronic format (Attachment 3). See Section1.1 for a complete list of these reference documents.

Table 1.0-1: Project Alternatives and Plan Description

Project Alternative	**Plan Description**
44 Foot Depth	Channel deepening to a controlling depth of 44 feet below MLLW. Construction of: 2 meeting areas; flow-altering mitigation Plan 6B; Dissolved Oxygen injection system; striped bass hatchery funding; New Savannah Bluff Lock and Dam fish passage structure; and Back River boat access alternative; CSS Georgia archaeological data recovery. Mitigation monitoring.
45 Foot Depth	Channel deepening to a controlling depth of 45 feet below MLLW. Construction of: 2 meeting areas; flow-altering mitigation Plan 6A; Dissolved Oxygen injection system; striped bass hatchery funding; New Savannah Bluff Lock and Dam fish passage structure, Back River boat access alternative; CSS Georgia archaeological data recovery. Mitigation land acquisition 1,643 acres. Mitigation monitoring.
46 Foot Depth	Channel deepening to a controlling depth of 46 feet below MLLW. Construction of: 2 meeting areas; flow-altering mitigation Plan 6A; Dissolved Oxygen injection system; striped bass hatchery funding; New Savannah Bluff Lock and Dam fish passage structure, Back River boat access alternative; CSS Georgia archaeological data recovery. Mitigation land acquisition 2,188 acres. Mitigation monitoring.
47 Foot Depth	Channel deepening to a controlling depth of 47 feet below MLLW. Construction of: 2 meeting areas; flow-altering mitigation Plan 6A; Dissolved Oxygen injection system; striped bass hatchery funding; New Savannah Bluff Lock and Dam fish passage structure, Back River boat access alternative; CSS Georgia archaeological data recovery. Mitigation land acquisition 2,245 acres. Mitigation monitoring.
48 Foot Depth	Channel deepening to a controlling depth of 48 feet below MLLW. Construction of: 2 meeting areas; flow-altering mitigation Plan 6A; Dissolved Oxygen injection system; striped bass hatchery funding; New Savannah Bluff Lock and Dam fish passage structure, Back River boat access alternative; CSS Georgia archaeological data recovery. Mitigation land acquisition 2,683 acres. Mitigation monitoring.

1.1 ENGINEERING INVESTIGATIONS SUPPLEMENTAL MATERIALS

1	BTG Inc.	Revised 35% Design Construction Estimate New Savannah Bluff Dam Fish Passage Facility Richmond County, Georgia	December 2002
2	Concord Project Consulting, Inc.	Value Engineering Study Summary Report Savannah Harbor Expansion Project	June 2008
3	FRAMATOME ANP DE&S, Inc.	New Savannah Bluff Lock and Dam Project Savannah River Georgia and South Carolina Fish Passage Facility Engineering Report	December 2002
4	Tetra Tech, Inc.	Oxygen Injection Design Report Savannah Harbor Expansion Project	October 2010
5	Tetra Tech, Inc.	Development of the Hydrodynamic and Water Quality Models for the Savannah Harbor Expansion Project	January 2006
6	Tetra Tech, Inc.	Habitat Impacts of the Savannah Harbor Expansion Project	October 2006
7	Tetra Tech, Inc.	Savannah Harbor Expansion Project – Chloride Data Analysis and Model Development	November 2006
8	Tetra Tech, Inc.	Water Quality Impacts of the Savannah Harbor Expansion Project	February 2007
9	US Geological Survey	Simulations of Water Levels and Salinity in the Rivers and Tidal Marshes in the Vicinity of the Savannah National Wildlife Refuge, Coastal South Carolina and Georgia	June 2006
10	USACE ERDC/CHL	DOTS Program Savannah District Request for Technical Assistance Dredge Vertical Construction Accuracy	March 2006
11	USACE ERDC/CHL	Memorandum Subject: Savannah Harbor Entrance Channel Simulations 2010 Report	March 2010
12	USACE ERDC/CHL	Memorandum Subject: Savannah Harbor Simulations Study 2009	March 2009
13	USACE ERDC/CHL Dennis Webb	Navigation Study for Savannah Harbor Channel Improvements	September 2004

14	USACE ERDC/CHL Jane McKee Smith, Donald K. Stauble, Brian P. Williams, and Raymond Chapman	Impacts of Savannah Harbor Expansion Project	October 2006
15	USACE ERDC/CHL Joseph Z. Gailani, S. Jarrell Smith, Layla Raad, and Bruce Ebersole	Savannah Harbor Entrance Channel: Nearshore Placement of Dredged Material Study	July 2003
16	USACE ERDC/CHL Michael J. Briggs and William G. Henderson	Vertical Ship Motion Study for Savannah, GA Entrance Channel	June 2011
17	USACE ERDC/CHL Stephen T. Maynord	Ship Forces on the Shoreline of the Savannah Harbor Project	August 2006
18	USACE Savannah District SAS	Correspondence between USACE & Federal/State Agencies Regarding Hydrodynamic & Water Quality Model Acceptability	2005/2006
19	USACE Savannah District SAS-EN	Channel Extension Boring Locations	April 2010
20	USACE Savannah District SAS-EN	Chloride Impact Evaluation Impacts of Harbor Deepening Only	February 2007
21	USACE Savannah District SAS-EN	Hurricane Surge Modeling	September 2005
22	USACE Savannah District SAS-EN	Memorandum Subject: Material Fines for Savannah Harbor Sediments, Station +103+000 thru -85+000 and KITB	August 2003
23	USACE Savannah District SAS-EN	Review and Costs for Supplemental Water Supply City of Savannah Intake at Abercorn Creek	September 2009
24	USACE Savannah District SAS-EN	Savannah Harbor Expansion Bank Erosion Study	July 2010
25	USACE Savannah District SAS-EN	Savannah Harbor Expansion Bank Erosion Study Update	June 2011
26	USACE Savannah District SAS-EN	Savannah Harbor Expansion Bank Stability Report Analysis and Reevaluation Summary	July 2010
27	USACE Savannah District SAS-EN	Savannah Harbor Expansion General Reevaluation Report Dredged Material Physical Analysis Report	2002

28	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Chloride Impacts with Proposed Mitigation Plan	December 2007
29	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Fishery Habitat Impacts with Proposed Mitigation Plan	January 2010
30	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Hurricane Surge Impacts with Proposed Mitigation Plan	December 2007
31	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Marsh/Wetland Impacts with Proposed Mitigation Plan	November 2007
32	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Water Quality Impacts with Proposed Mitigation Plan	September 2009
33	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Mitigation Evaluation for Marsh/Wetland Impacts	November 2007
34	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Sensitivity Analysis of Proposed Navigation Meeting Areas	September 2009
35	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Sensitivity Analysis of Proposed Sill on Middle River	September 2009
36	USACE Savannah District SAS-EN	Supplemental Studies to Determine Potential Groundwater Impacts to the Upper Floridan Aquifer Savannah Harbor Expansion Project	June 2007
37	USACE Savannah District SAS-EN	Wetland/Marsh Impact Evaluation	February 2007
38	USACE Savannah District SAS-PD	Savannah Harbor Data Analysis & Modeling Expectations of Federal Agencies	November 2001
39	USACE Savannah District SAS-PD	Savannah Harbor Expansion Project Dredged Material Management Plan	January 2012
40	USACE Savannah District SAS-PD	Savannah Harbor Expansion Project Impacts to O&M	January 2012
41	USACE Savannah District SAS-PD	Savannah Harbor Long Term Management Strategy	August 1996

42	USACE Savannah District SAS-PD	Sediment Quality Evaluation	January 2012
43	USACE Savannah District SAS-PM	Correspondence Between USACE and Utilities Regarding Pipeline Crossings	May 2008
44	USACE Wilmington District SAW-TS	Sedimentation Analysis	July 2009
45	USACE Mobile District SAM	Savannah Harbor Deepening Project ATM Marsh Succession Model Marsh/Wetland Impact Evaluation	May 2007
46	USACE Mobile District SAM	Savannah Harbor Deepening Project USGS/USFWS Marsh Succession Model Marsh/Wetland Impact Evaluation	June 2007
47	MACTEC	Identification and Screening Level Evaluation of Measures to Improve Dissolved Oxygen in the Savannah River Estuary	June 2005
48	MACTEC	Savannah Harbor Reoxygenation Demonstration Project	January 2008
49	MACTEC	Savannah Harbor Reoxygenation Demonstration Project Supplemental Data Evaluation Report	August 2009
50	Tetra Tech Inc.	Modeling of GPA's Oxygen Injection Demonstration Project	July 2009
51	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Adult SNS (Summer) Habitat Impacts with Proposed Mitigation Plan	March 2011
52	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Adult SNS (Winter) Habitat Impacts with Proposed Mitigation Plan	March 2011
53	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Juvenile SNS (Winter) Habitat Impacts with Proposed Mitigation	March 2011
54	Tetra Tech, Inc. & Advanced Data Mining Services, LLC	Chloride Modeling Savannah Harbor Expansion Project	December 2010
55	USACE ERDC/CHL Jane McKee Smith, Donald K. Stauble, Brian P. Williams, and Michael J. Wutkowski	Impact of Savannah Harbor Deep Draft Navigation Project on Tybee Island Shelf and Shoreline	April 2008

56	Arthur Freedman Associates, Inc.	Assessment of Chloride Impact from Savannah Harbor Deepening	April 2011
57	USACE ERDC/CHL	Reanalysis of Ship Forces at the Shoreline in Savannah Harbor	July 2011
58	US Geological Survey	Simulation of Specific Conductance and Chloride Concentration in Abercorn Creek, Georgia	June 2011
59	USACE Savannah District SAS-PD	Savannah Harbor Expansion Project Channel Extension Evaluation	July 2011
60	USACE Savannah District SAS-EN	Savannah Harbor Expansion Project Evaluation of Marsh/Wetland Impacts with Proposed Mitigation Plan (Addendum)	July 2011
61	Tetra Tech, Inc. & Eco Oxygen Technologies, LLC	Analysis of Oxygen Injection in the Back River	July 2011
62	Tetra Tech, Inc.	Model Comparison Report	July 2011
63	Camp Dresser & Mckee	City of Savannah Seawater Effects Study	December 2011
64	Camp Dresser & McKee	Raw Water Impoundment Geotechnical Evaluation Memorandum	December 2011

1.2 CHANNEL STATIONING

Currently, the federally authorized navigation channel extends from 60,000 feet (11.4 miles) offshore, across the ocean bar, to 112,500 feet (21.3 miles) upstream on the Savannah River. Stationing, along the navigation channel, is designated every 1,000 feet. Station 0 is located at the mouth of the river at Fort Pulaski. The standard notation for the stationing is in thousands of feet designated as +000. For example, Station 20+000 is 20,000 feet upstream of the mouth of the entrance channel. To determine the river mile, divide by 5,280 feet/mile i.e., Station 20+000 is located at river mile 3.8. Stations on the bar or entrance channel are designated by a negative number. For example, Station -40+000 is located 40,000 feet (7.6 miles) offshore from the mouth of the river.

Stationing for the SHEP 47 ft project extends from -97+680 (mouth of the entrance channel offshore) to 103+000 (upstream of GPA’s Port Wentworth Terminal on the Savannah River). The Federally authorized navigation project extends up to Station 112+500. However, the depths proposed for SHEP only extend up to Station 103+000. The total length of the SHEP navigation channel is 38.0 miles. This length is an increase of 7.1 miles from the currently authorized navigation channel due to the extension of the offshore entrance channel.

1.3 DATUMS

Several datums are referenced throughout the numerous reports and studies completed for the SHEP for various reasons (See **Table 1.3-1**). The hydrodynamic modeling references elevations in meters to the NGVD29 datum, while the bathymetry is surveyed in feet referencing mean lower low water (MLLW) based on the 1983-2001 tidal epoch.

Table 1.3-1: Referenced Datums Used in Engineering Studies

Engineering Investigations Supplemental Materials		Datum Used
2	Value Engineering Study Summary Report Savannah Harbor Expansion Project	MLW, MLLW
3	New Savannah Bluff Lock & Dam Fish Passage Facility Engineering Report	NGVD
4	Oxygen Injection Design Report	NGVD, MLLW
5	Development of the Hydrodynamic and Water Quality Models	meters NGVD
9	Simulations of the Water Levels and Salinity in the Rivers and Tidal Marshes	MLLW, NAVD
14	Impacts of the Savannah Harbor Expansion Projects	MLW, MHW, MLLW, NAVD, meters NGVD
15	Savannah Harbor Entrance Channel Nearshore Placement of Dredged Material Study	MTL, MLW
16	Vertical Ship Motion Study for Savannah, GA Entrance Channel	MLLW
17	Ship Forces on the Shoreline of the Savannah Harbor Project	MLLW
19	Channel Extension Boring Locations	MLLW
21	Hurricane Surge Modeling	meters NGVD
24	Savannah Harbor Expansion Bank Erosion Study	MLLW
25	Savannah Harbor Expansion Bank Erosion Study Update	MLLW
26	Savannah Harbor Expansion Bank Stability Report	MLW

Table 1.3-1: Referenced Datums Used in Engineering Studies (Continued)

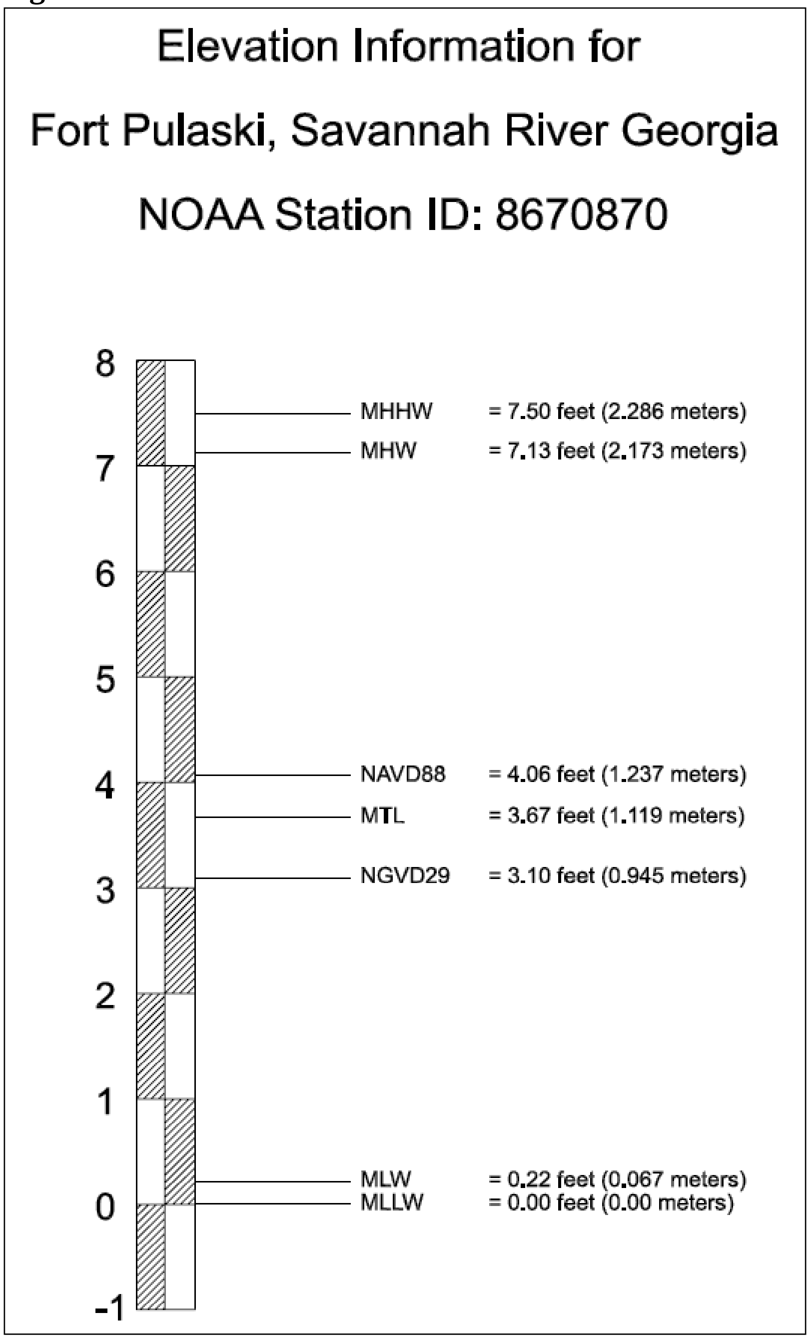
Engineering Investigations Supplemental Materials		Datum Used
27	Dredged Material Physical Analysis Report	MLW
28	Evaluation of Chloride Impacts and Proposed Mitigation Plan	NGVD
29	Evaluation of Fishery Habitat Impacts with Proposed Mitigation Plan	NGVD, MLLW
30	Evaluation of Hurricane Surge Impacts with Proposed Mitigation Plan	meters NGVD
31	Evaluation of Marsh Wetland Impacts with Proposed Mitigation Plan	meters NGVD
32	Evaluation of Water Quality Impacts with Proposed Mitigation Plan	meters NGVD
33	Mitigation Evaluation for Marsh Wetland Impacts	meters NGVD
34	Sensitivity Analysis of Proposed Navigation Meeting Areas	meters NGVD
35	Sensitivity Analysis of Proposed Sill on Middle River	NGVD, MLLW
36	Potential Ground-Water Impacts to the Upper Floridian Aquifer	MLW, NAD83, MLLW, MSL NGVD
39	Dredged Material Management Plan	MHW, MLW
40	Savannah Harbor Expansion Project Impacts to O&M	MLLW
41	Savannah Harbor Long Term Management Strategy	MLW, MHW
42	Sediment Quality Evaluation	MLW
43	Correspondence Regarding Pipeline Crossings	MLLW
44	Sedimentation Analysis	MLW, meters NGVD, MLLW
63	City of Savannah Seawater Effects Study	NAVD
64	Raw Water Storage Impoundment Geotechnical Evaluation	NAVD

The datum reference is from the NOAA tidal station at Fort Pulaski Savannah River, Georgia (Station ID 8670870). NGVD29 and NAVD88 are both fixed datums. NAVD88 supersedes NGVD29 as the national standard geodetic reference for heights. The tidal datums: MHW (mean high water), MHHW (mean higher high water), MLW (mean low water), MLLW (mean lower low water), and MTL (mean tide level) are determined over a 19 year National Tidal Datum Epoch and are referenced to a local mean sea level (MSL). The Tidal Datum Epoch for this station is from January 1983 to December 2001. To convert between datums, it is helpful to reference the station datum shown in **Figure 1.3-1**.

To convert elevations between the NGVD29 and MLLW datums use the following equation:

MLLW elevation – difference between MLLW and NGVD29 (3.1 feet or 0.945m) = NGVD29 elevation

Figure 1.3-1: Datum Reference Chart



1.4 SOFTWARE VALIDATION

Engineering software used in the SHEP study are technically sound and in accordance with accepted engineering practice. The document and models have undergone many reviews including District Quality Control (DQC), Agency Technical Review (ATR) and an Independent External Peer Review (IEPR). The Quality Control Appendix to the GRR and the SHEP Peer Review Plan discuss the review process in greater detail.

The USACE Enterprise Standard (ES)-08101 *Software Validation for the Hydrology, Hydraulics and Coastal Community of Practice* describes the process for validating engineering software for use in planning studies to satisfy the requirements of the Corps' Scientific and Engineering Technology (SET) initiative and is applicable to all USACE elements having Civil Works responsibility and are using engineering models and analytical tools for planning. The software used in this study that is documented as preferred or allowed for use is listed in **Table 1.3-1** below.

Table 1.4-1 SHEP Engineering Software and Analytical Tools Documented as USACE Preferred or Allowed for Use

Software	USACE Designation
Bentley MicroStation	Preferred
Bentley InRoads	Preferred
EFDC	Allowed for Use
WASP	Allowed for Use
HEC-RAS	Preferred
HEC-DSSVue	Allowed for Use
ADCIRC	Preferred
MCACES	Allowed for Use
ArcGIS	Preferred
CORPSCON	Preferred
TABS-MD	Preferred
SSFATE	Allowed for Use
STWAVE	Preferred
WIS	Preferred
WISWAVE	Allowed for Use
GENESIS	Preferred
UTEXAS3	Preferred
CADET	Preferred

Software validation and the SET initiative are relatively new and the lists of allowed and preferred models are living documents that are continually being revised and updated. The Engineering and Construction Bulletin outlining the interim guidance on software validation issues for engineering software (ECB No. 2007-6) was issued in April of 2007 well after the SHEP study was underway. Many of the SHEP modeling efforts and impacts determinations began prior to that date and several of the studies were even completed. The PDT has ensured that models chosen for the study are appropriate for use. None of the models listed as "Not Allowed for Use" on the software validation lists were used for SHEP.

Table 13.1-2 lists the models utilized in SHEP that are not currently included on the model approval lists developed by the Hydrology, Hydraulics and Coastal Community of Practice (HH&C CoP). Many of these models and study conclusions underwent extensive independent peer review, the results of which are documented in several locations, including the model development reports in the Engineering Investigations Supplemental Materials and the SHEP Peer Review Plan.

Table 1.4-2 Additional SHEP Engineering Software and Analytical Tools

Software	Description
Artificial Neural Networks (ANN)	ANNs are empirical based data mining and statistical analysis tools. Dr. Bernard Hsieh of USACE ERDC conducted an expert technical review of the ANN used for quantifying chloride impacts in October-December, 2010. Comments resulting from the review were incorporated into the final analysis tool. The ANN was used to confirm chloride impact projections developed through the EFDC and WASP modeling. M2M and MSM are also ANN models and were considered for use early in the study process to quantify environmental impacts to marshes adjacent to the channel due to salinity intrusion. However, use of these models was abandoned when it was discovered that the data mining and extrapolation did not hold true with the flow-altering mitigation features.
CSC Virtual Ship 2000 Models	USACE ERDC Ship Simulator is comprised of Computer Sciences Corporation's (CSC) Virtual Ship 2000 Models. CSC's ship models have been fully verified and accepted by leading USCG, US Navy SWOS, and MSC subject matter experts. USACE Engineering and Design Guidance ER 110-2-1403 titled *Studies by Coastal, Hydraulic, and Hydrologic Facilities and Others* states "studies associated with the planning, design, construction, operation, and maintenance of navigation channels will include a ship or tow simulation investigation".
General sand TRANsport Model (GTRAN)	GTRAN is a USACE ERDC developed model that calculates sand transport direction and magnitude under current-dominated and combined wave-current sediment transport regimes. GTRAN uses peer-reviewed published methods for sand transport in these regimes, and automatically determines the appropriate sediment transport formula to use based on the characteristics of the bottom boundary forcing. GTRAN also includes cohesive effects for mixed sediment beds. GTRAN is limited to application outside the surf zone and has been applied by ERDC on several recent studies completed on Savannah Harbor and Tybee Island.
D-CORMIX	D-Cormix is an EPA supported analytical tool for hydrodynamic mixing zone analysis of continuous dredge disposal sediment plumes. The D-CORMIX model was used in the July 2003 ERDC Nearshore Placement Study to predict loss of sediment during a pipeline placement operation where placement is especially a concern in the nearshore where released sediment could migrate toward the beach.

(Continued on next page)

Table 1.4-2 Additional SHEP Engineering Models and Analytical Tools (Continued)

Software	Description
DGSLOPE	DGSLOPE is a slope stability analysis program that was used, along with UTEXAS3, for the 2010 SHEP Bank Stability Report. The program was developed by USACE ERDC and has been widely used throughout USACE.
DYNCFT	DYNCFT is a coupled groundwater flow and transport code developed by Camp, Dresser and McKee (CDM) which combines DYNFLOW and DYNTRACK code to simulate the effect on groundwater flow of fluid density gradients associated with solute concentration gradients. The codes have been extensively tested and documented by CDM for over 25 years, reviewed and tested by the International Groundwater Modeling Center (IGWMC, 1985) and evaluated by the ASCE Groundwater Technical Committee (Pandit, 1997). The results of the DYNCFT simulations were reviewed by experts from the USGS, HEC, GA DNR-EPD and SCDHEC, as well as two other independent peer review experts during June-July, 2005 and February-March, 2006.

The bathymetry data was obtained from several sources because there is not one continuous bathymetry dataset that encompasses the entire system. To simply prescribe a channel design template onto the model grid does not adequately reflect the continuous sedimentation and dredging that is ongoing in the harbor, and is necessary to identify environmental impacts.

The models account for the overdredge volume in the navigation channel by assuming the overdredge is the same in the 42-foot channel (existing conditions) as it would be in any dredged channel depth.

Furthermore, since advance maintenance is proposed to be essentially the same for deepened conditions as for existing conditions, we subtracted 2 feet from the existing (42-foot channel) bathymetry for the 44-foot depth, 3 feet for the 45-foot depth, 4 feet for the 46-foot depth, 5 feet for the 47-foot depth, and 6 feet for the 48-foot depth bathymetry inputs. All depths reference Mean Lower Low Water (MLLW). The reality in the model is that the 48-foot depth is closer to 52 feet which includes the additional depths due to advance maintenance and overdredge.

Overdredging and advanced maintenance are included in all model studies addressing environmental impacts, including hydrodynamic, water quality and hurricane surge models. Non-inclusion of overdredging and advanced maintenance provides a more conservative approach for navigation channel design models (Wave and Current Modeling for Navigation Study, Ship Forces on Shoreline of the Savannah Harbor Project, SHEP Bank Erosion Study). The Impacts on Waves, Currents and Sediment Transport Study is offshore where there is no advance maintenance.

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2.0 PHYSICAL SETTING

2.1 SAVANNAH HARBOR

Savannah Harbor is a deep-draft harbor located on the South Atlantic US coast, 75 statute miles south of Charleston Harbor, South Carolina and 120 miles north of Jacksonville Harbor, Florida. The harbor comprises the lower 21.3 miles of the Savannah River (which, with certain of its tributaries, forms the boundary between Georgia and South Carolina along its entire length of 313 miles) and 11.4 miles of channel across the bar to the Atlantic Ocean.

Within the harbor limits, the Savannah River is generally divided into two channels by a series of islands. From the Atlantic Ocean to River Mile 10 (Station 53+000), where the river converges, the harbor is separated into South and North Channels. Within this area, the navigation channel is maintained in North Channel. After divergence of the river into Front and Back Rivers at River Mile 11 (Station 59+000), the navigation channel is maintained in Front River and passes by the business district of the City of Savannah. The navigation channel is maintained in Front River to the upper limits of the harbor at River Mile 21.3 (Station 112+500). The Atlantic Intracoastal Waterway (AIWW) crosses the navigation channel approximately 5.5 miles upstream of the entrance to the harbor (Station 27+000). The Savannah River Below Augusta Project, which is a shallow-draft navigation channel authorized for 9 ft deep and 90 ft wide, extends upstream from the harbor (River Mile 21.3) to River Mile 202.6 at Augusta, Georgia. See **Figures 2.1-1a** and **2.1-1b**.

2.2 TYBEE ISLAND

Tybee Island, a barrier island located downdrift of the Savannah Harbor navigation channel, loses an estimated 227,000 cubic yards of sand per year from its shelf and shoreline. A study was completed in April 2008 by ERDC titled *Impact of Savannah Harbor Deep Draft Navigation Project on Tybee Island Shelf and Shoreline* that found a significant portion of the loss, 78.5%, is attributed to the maintenance of the navigation channel. This study was funded and performed separately from the SHEP studies. However, results of the study have been considered in the impact analysis for the SHEP, and the report is included in the Engineering Investigations Supplemental Materials. This study indicates that the current entrance channel (including the entrance channel jetties) causes a pattern of ebb shoal deflation on the Tybee Shelf and appears to be nearly a complete sink for any sediment moving from north to south along the Tybee shelf. These studies estimated that the combined shelf and shoreline impact at Tybee Island to be 78.5%. This means that an estimated 78.5% of the reduction in sand volume on the Tybee Island shelf and shoreline can be attributed to the existing project with the remainder of the erosion attributed to natural processes. Any mitigation for this effect would be the responsibility of the existing Savannah Harbor Navigation Project Despite the impacts to Tybee Island, operation and maintenance of the Federal Navigation Project is expected to continue. This includes all aspects of the project, i.e. the structural features (jetties and advance maintenance areas), the deep-draft navigation channel, and the sediment placement areas. Currently, sediments removed during periodic maintenance of the entrance channel are deposited in the EPA approved Ocean Dredged Material Disposal Site (ODMDS) located approximately 3.7 nautical miles off of Tybee Island.

Renourishment of the Tybee Island shoreline is also expected to continue. The current project authorization is through the year 2024. The renourishment project authorization and funding are not related to the SHEP.

Figure 2.1-1a: Overview Map of Savannah Harbor

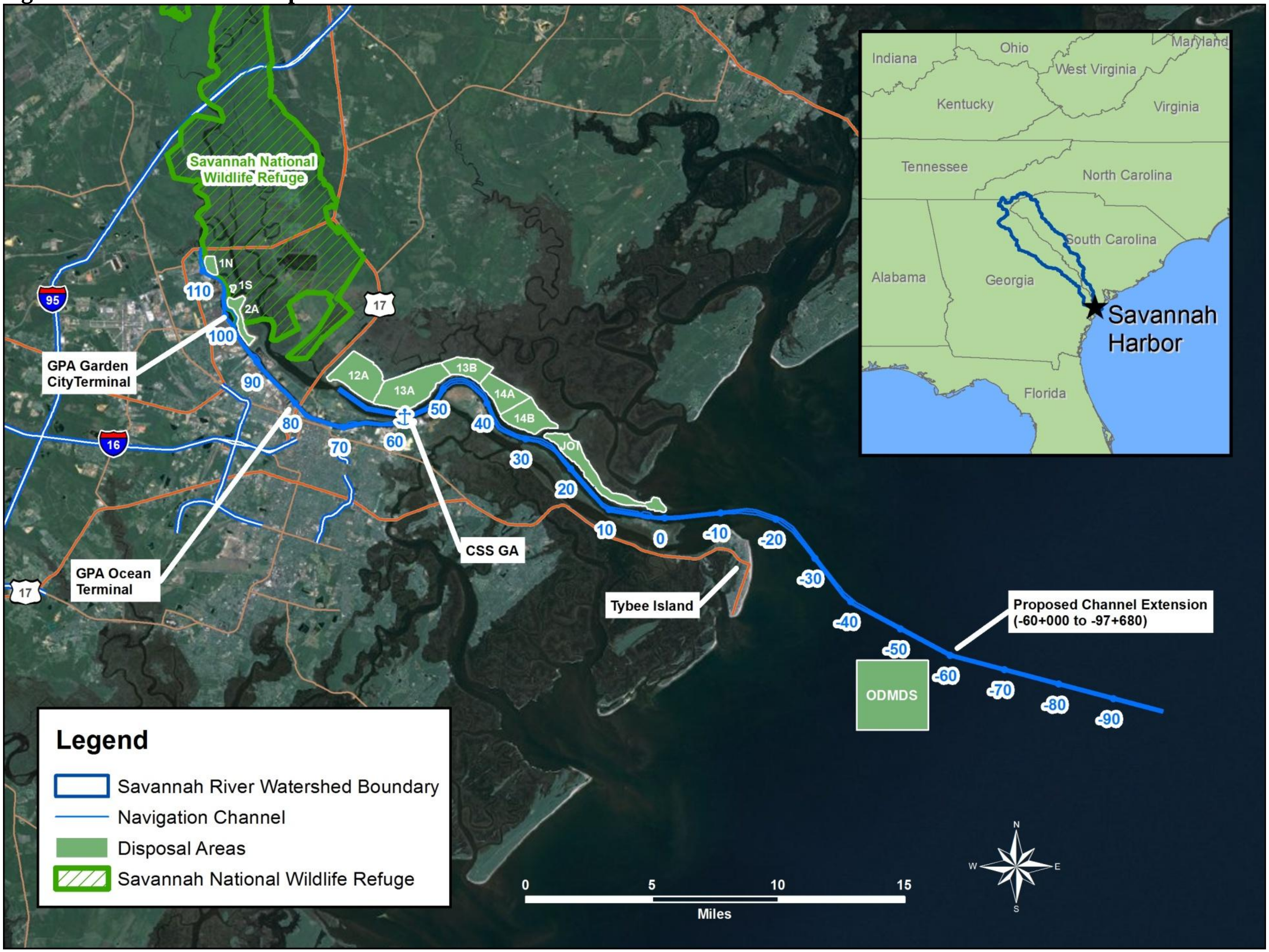
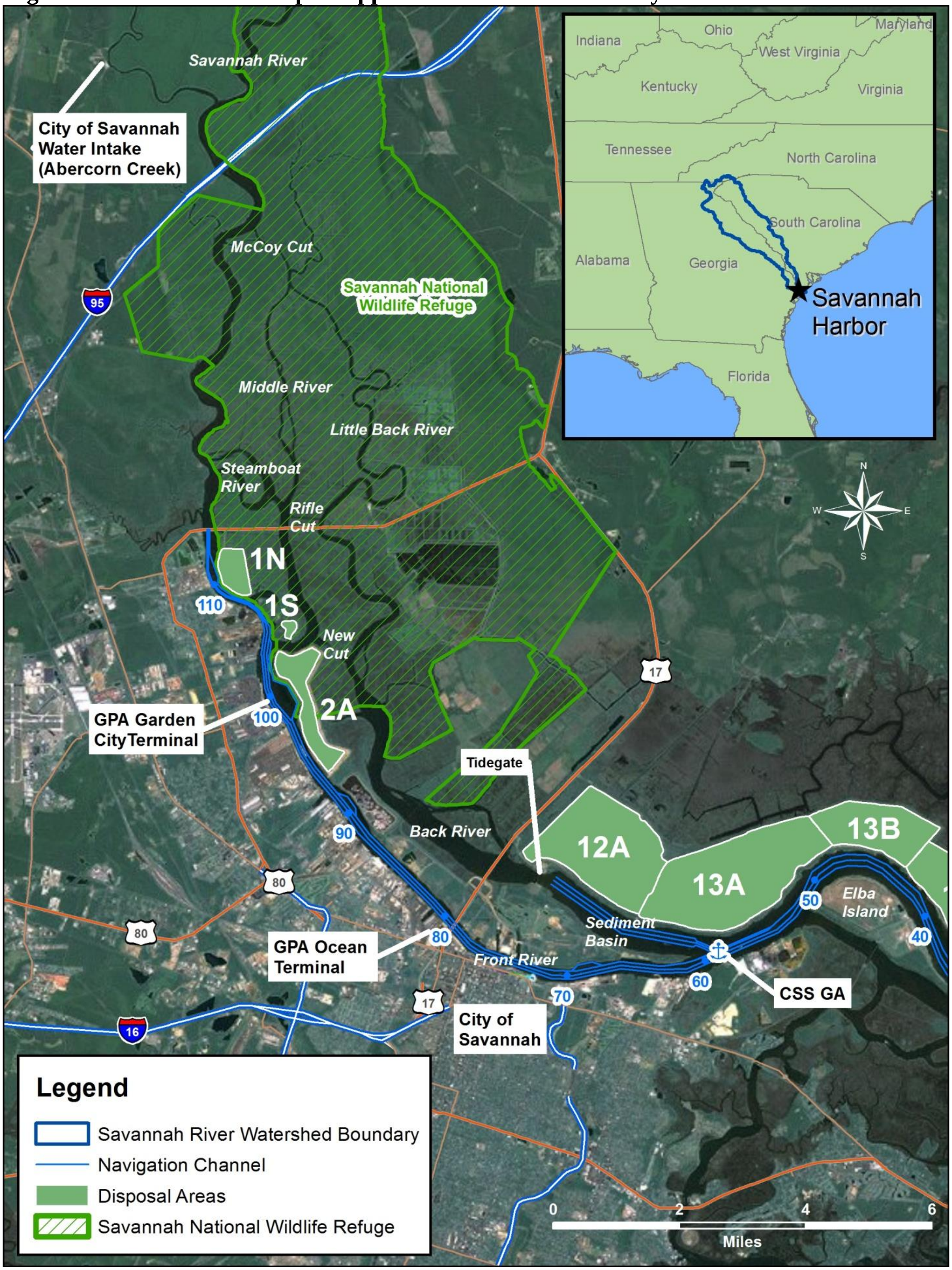


Figure 2.1-1b: Overview Map of Upper Savannah River Estuary



2.3 WILDLIFE REFUGES

2.3.1 Savannah National Wildlife Refuge

The Savannah National Wildlife Refuge, which is owned and maintained by the US Fish & Wildlife Service, lies on both the South Carolina and Georgia sides of the Savannah River just upriver from the City of Savannah. The refuge was established on April 6, 1927 and consists of 29,175 acres of freshwater marshes, tidal rivers and creeks, and bottomland hardwoods. The refuge includes approximately 6,000 acres of impounded freshwater wetlands for waterfowl habitat. Those impoundments include 3,000 acres which are actively managed by 22 water control structures. Two management schemes are primarily used for the impoundments; drawdown pools and permanently flooded pools. The drawdown pools are drained annually between March 15 and May 15 and manipulated to promote growth of emergent waterfowl food plants. These areas are flooded in the fall of each year. Permanent pools remain flooded all year to promote growth of submerged aquatic plants and to provide wood duck brood-rearing and alligator habitat. Permanently flooded pools are drained, dried, burned, and mowed when undesirable vegetation becomes a problem or productivity of desirable plants decreases. These pools may require additional water at any time to make up for transpiration and evaporation. An adequate supply of fresh water is needed for management of the impoundments. Fresh water is supplied through Lucknow Canal from Little Back River.

Salinity concentrations in the estuary, especially at Lucknow Canal, directly impact the overall health of the ecosystem within the freshwater impoundments of the refuge. High salinity concentrations in the river have the potential to intrude into the freshwater marshes and habitats, making it difficult for freshwater marsh plant species to compete with species more tolerant of brackish conditions.

2.3.2 Tybee National Wildlife Refuge

The Tybee National Wildlife Refuge, also owned and maintained by the US Fish & Wildlife Service, was established on May 9, 1938 as a breeding area for migratory birds and other wildlife. The refuge consists of 400 acres of wetlands and diked low lands located at the mouth of the Savannah River across the river from the Fort Pulaski National Monument. Much of the site is diked and is used for placement of sediments dredged from the Savannah Harbor Navigation Project. The vegetated portions of the upland areas are densely covered with red cedar, wax myrtle, and groundsel. Saltwater marsh borders much of the island. The low tide shoreline provides feeding and resting areas for shorebirds and migratory birds. The site is closed to public use.

2.4 ESTUARINE CONDITIONS

2.4.1 Hydrology

The Savannah River drainage basin is over 10,500 square miles and drains lands in North Carolina, South Carolina, and Georgia from three physiographic regions: the Blue Ridge Mountains, the Piedmont, and the Coastal Plan. The headwaters originate in the mountains with the Seneca and Tugaloo Rivers. The confluence of these two rivers forms the Savannah, which flows from Lake Hartwell. There are several reservoirs located on the river in the upper portion of the basin that regulate river flow. Three of these reservoirs are large multipurpose USACE projects, Hartwell Lake, Richard B. Russell Lake, and J. Strom Thurmond Lake. Thurmond Dam is the most downstream reservoir and is located 220 miles upstream of the mouth of the river near Augusta, Georgia. It was filled in 1954.

Water flow in the Savannah River varies considerably both seasonally and annually even though they are largely controlled by releases of the USACE owned and operated Thurmond Dam. Discharge is typically high in winter and early spring and low in the summer and fall, but regulation by upstream reservoirs has reduced natural flow variations. **Figure 2.4.1-1** below illustrates how river flows have been altered after the dam was constructed. This information is from the USGS gage on the Savannah River at Augusta, GA (Station ID 02197000). The USGS gage on the Savannah River at Clyo, Georgia (Station ID 02198500), located approximately 61 miles upstream of the mouth of the river accounts for inflows downstream of Thurmond Dam and is useful in evaluating the freshwater flow coming into the Harbor and estuary. See **Figure 2.4.1-2** for flow data collected at the Clyo gage station.

Figure 2.4.1-1: Savannah River Flow at Augusta, Georgia (1884 to 2001)

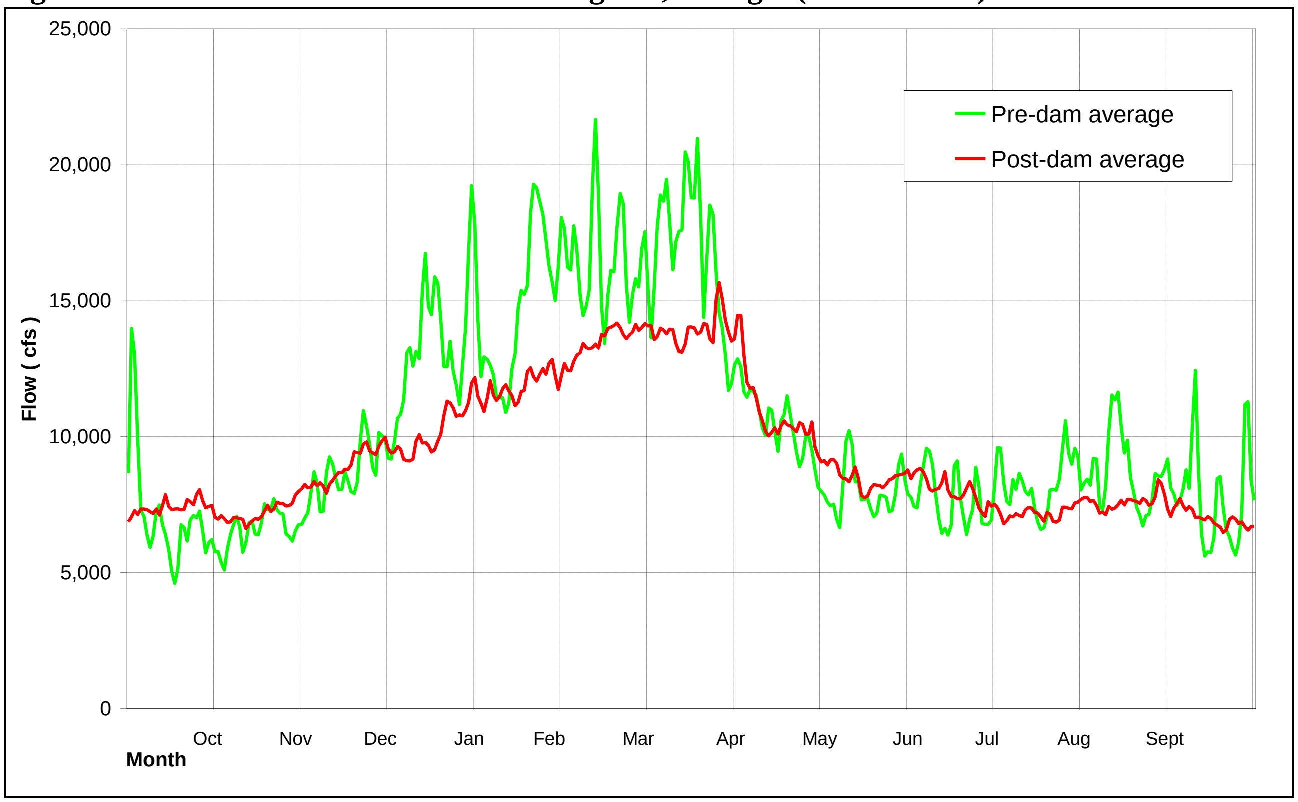
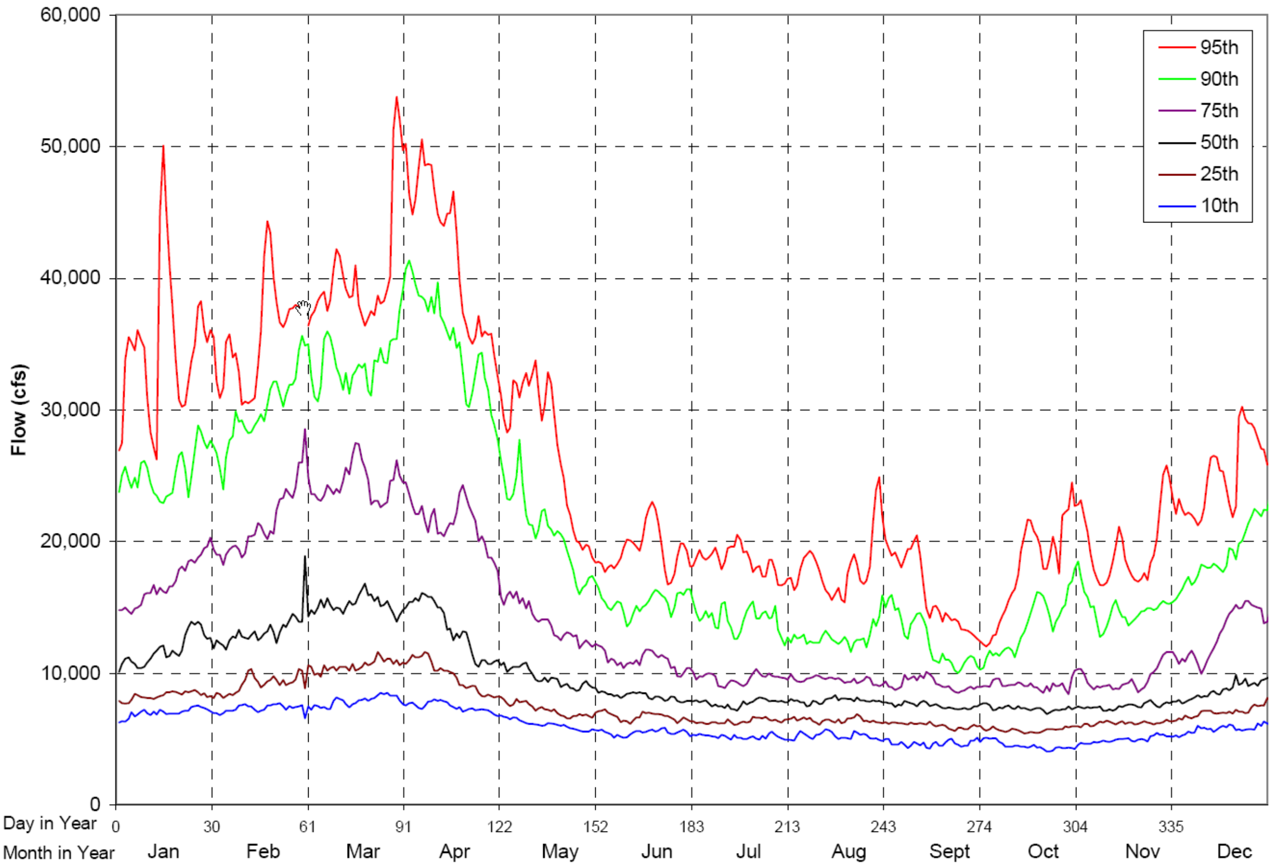


Figure 2.4.1-2: Savannah River Flow Percentiles at Clyo, Georgia for the Period of Record (1929 to Present)



2.4.2 Tides

Savannah Harbor is located in an area of semi-diurnal tides with two high waters and two low waters each lunar day. A long term tide gage established in 1935 by the National Oceanic and Atmospheric Administration (NOAA) is located at the entrance channel on Cockspur Island near Fort Pulaski (Station ID: 8670870). According to the long term data recorded at the gage, the mean range, the difference between mean high water and mean low water, is 6.92 ft. The diurnal range, the difference between mean higher high water and mean lower low water, is 7.50 ft. The variability in tidal amplitude is due to tidal range cycles between neap and spring tides. Neap tides are periods of lowest tidal amplitude, and spring tides are periods of highest tidal amplitude. Neap tides range between 5 and 6 ft, and spring tides can range upwards of 8 ft.

2.4.3 Salinity

Salinity throughout the estuary is highly variable and influenced by the tidal cycle and freshwater inflows. Freshwater flows coming downstream have a dramatic effect on salinity concentrations in the estuary. During very dry and drought conditions, salinity intrudes much further upstream that it does during wetter periods. Data collected during the summer of 1997 result in average bottom salinity concentrations at river mile 4.5 of 24.9 ppt and concentrations at river mile 20.5 of 3.2 ppt.

Salinity concentrations in the estuary have direct influence on the adjacent marshes. A portion of these tidal marshes currently supports freshwater plant and aquatic habitat. As salinity levels in the estuary intrude upstream, the high salinity concentrations in the river have the potential to intrude into the freshwater marshes and habitats, making it difficult for freshwater marsh plant species to compete with species more tolerant of brackish conditions.

The salinity concentrations also vary vertically, through the water column, due to the density of incoming ocean water and tidal mixing cycles. The more dense high concentration saline water coming in with each tide is carried in on the bottom of the water column, while the fresh water coming down the river rides on the top. Because of this stratification of salinity, the salt/fresh interface is known as a saltwater wedge. The saltwater wedge is characterized by high salinity concentrations water on the bottom and near zero salinity concentration on the surface. The prominence of the saltwater wedge is attributable to the tidal amplitudes of neap and spring tides which affects how well the water column is mixed. Greater vertical mixing occurs during spring tides where the amplitude is much higher. This causes the high saline water on the bottom to mix with the fresh water at the surface. The mixing causes less stratification and the saltwater wedge is less apparent. The neap tides, with lower tidal amplitudes, do not produce as much vertical mixing and allow for more stratified conditions with the denser, concentrated ocean water remaining on the bottom of the channel. Neap tidal cycles can cause salinity differences in the water column between surface and bottom to be as great as 17 ppt. Spring tides can reduce the salinity stratification to just 6 ppt.

2.5 GENERAL GEOLOGIC SETTING

The study area is underlain with unconsolidated and partly consolidated Atlantic Coastal Plain sediments. These sediments generally consist of unconsolidated to semi-consolidated layers of sand and clay and semi-consolidated to very dense limestone and dolomite and can achieve a thickness of about 5,500 ft. They range in age from late Cretaceous (approximately 65 million years old) to Recent (Holocene). The Atlantic Coastal Plain sediments overlie sedimentary strata and volcanic rocks of Triassic age to early Jurassic age (approximately 230 million years old to about 170 million years old, respectively). These rocks overlie crystalline basement rocks of Paleozoic age (from 680 to 230 million years old) consisting of intrusive igneous and low-grade metamorphic rocks. The rock record is not continuous, and time gaps exist where either no sediment deposition occurred or where erosional events removed the rock record. In the project area, the post-Cretaceous sediments (those deposited within the last 65 million years) are estimated to be about 1,800 to 2,500 ft thick. These strata intersect a horizontal plane in a northeast to southwest trend and dip and thicken to the southeast. For the purpose of this study, the strata will be referred to based on time-rock units (i.e., rocks deposited during the same geologic time division).

3.0 CURRENTLY AUTHORIZED PROJECT

3.1 PROJECT DIMENSIONS

The existing authorized Federal Savannah Harbor Navigation Project extends from the seaward end of the entrance channel in the Atlantic Ocean, Station -60+000 up the Savannah River to Station 112+500 (river mile 21.3). The authorized depth for the existing Savannah Harbor navigation project is 42 ft below MLLW in the inner harbor channel and 44 ft below MLLW in the entrance channel. **Table 3.1-1** lists the authorized depths and widths for the existing project located within the limits of the proposed harbor expansion.

The 42-foot project is designed for Panamax vessels drafting 38 ft. However, vessels capable of drafting in excess of 38 ft are presently using the harbor by either light-loading or taking advantage of the considerable tide ranges.

Table 3.1-1: Currently Authorized Project Dimensions

Station	Authorized Depth (-ft MLLW)	Bottom Width (ft)
Inner Harbor		
103+000 to 102+000	42	400
102+000 to 100+000	42	400
100+000 to 79+000	42	500
79+000 to 70+000	42	500
70+000 to 50+000	42	500
50+000 to 41+000	42	500
41+000 to 24+000	42	500
24+000 to 0+000	42	500
Entrance Channel		
0+000 to -14+000	42	500
-14+000 to -60+000	44	600

Note – Station Numbers are measured in ft from the harbor entrance (Sta. 0+000). Negative numbers indicate the reach is in the entrance channel.

3.2 TURNING BASINS

Five authorized turning basins, shown in **Table 3.2-1**, are located within the reaches proposed for deepening. There is also a private turning basin at Elba Island between the Oysterbed and Fig Island turning basins, which has recently been enlarged and is used by ships delivering liquefied natural gas (LNG) to the LNG facility on Elba Island.

Table 3.2-1: Currently Authorized Turning Basin Dimensions

Name	Length (ft)	Width (ft)	Authorized Depth (-ft MLLW)	Station
Kings Island	1,600	1,500	42	101+298 to 97+750
Marsh Island	900	1,000	34	91+610 to 89+485
Fig Island	1,500	1,000	34	69+740 to 67+386
Oysterbed Island	1,050	1,200	40	4+395 to 2+345
Rehandling Basin	5,000	300	40	10+175 to 4+395

3.3 SEDIMENT BASIN

The sediment basin is located adjacent to Confined Disposal Facility (CDF) 12A and 13A in the Back River and is an O&M feature that was constructed, along with the Tidegate Structure, between 1972 and 1975 to reduce high shoaling rates along the City Front Reach and provide a safer and more economically maintained navigation channel. Physical model tests were performed prior to construction to determine the effectiveness of the sediment basin. The project was designed to include a set of tidegates that opened on the incoming tide, allowing tidal flushing of the Back River but closed on the outgoing tide, forcing all of the flow through New Cut to Front River and scouring the navigation channel, thus reducing dredging requirements within the navigation channel on Front River. The project was very successful at its intended purpose, but the flow alterations resulted in increased salinity in the Back River. As part of mitigation for the previous harbor deepening, the Tidegates were removed from service in March 1991, and the New Cut, constructed as part of the Tidegate/Sediment Basin project, was blocked off and filled in 1992. However, the sediment basin continues to trap sediments that make their way up Back River.

The sediment basin consists of an entrance channel, or throat, which is 1,600 ft long and 300 ft wide with an authorized bottom elevation of -38 ft MLLW. The basin itself is approximately two miles long and 600 feet wide with an authorized bottom elevation of -40 ft MLLW. Note that these depths do not coincide with the 42 ft authorized depth below MLLW for the navigation channel. The sediment basin was initially dredged in 1972, but regular maintenance of the sediment basin did not begin until 1977. It has not been dredged since 2004 due to funding constraints. Recent surveys of the basin, completed in February 2010, found the bottom elevation to range between 21 and 26 ft below MLLW.

3.4 ANNUAL MAINTENANCE

3.4.1 Annual Maintenance Dredging

The inner harbor channel extends from Station 0+000 at the mouth of the Savannah River to Station 112+500, for a distance of 21.3 miles. Currently, the inner harbor channel above Station 28+000 captures all of the clay and silt which enters the estuary from upstream sources. The inner harbor below Station 28+000 shoals with material primarily from ocean sources deposited during slack tide. Approximately 6.2 million cubic yards (cy) of sediments are available to be removed each year from the inner harbor of the Savannah Harbor Navigation Project by USACE. Material dredged from the inner harbor is placed in upland CDFs that have been designed and constructed specifically for maintenance of the navigation channel.

The entrance channel (Stations 0+000 to -60+000) is a sediment sink, which totally interdicts the littoral transport, i.e. it is a trap for all the sediment that is transported to it. Approximately 1 million cy of material are removed from the entrance channel by hopper dredges each year. Currently, this material is placed in the EPA-approved Ocean Dredged Material Disposal Site (ODMDS). The project is also authorized to deposit maintenance sediments in Sites 2 and 3, as well as other designated nearshore sites.

The material dredged from the Savannah Harbor is a mixture of sands, silts, and clays. Sands are dredged from the lower and upper reaches of the project, while the predominant material removed from the middle harbor and sediment basin is silt. The inner harbor sediments are primarily silts and clays from Station 56+000 to Station 103+000. The reach from Station 28+000 to Station 56+000 is a transition reach that has a higher percentage of sand in its distributions than the sediment distributions of the upstream reach. A notable exception is in the vicinity of Station 36+000, which has a high percentage of silts and clays and almost no sand. This location is near the confluence of the inner harbor channel and both Elba Island and Fields Cut. The inner channel sediment distributions from Station 28+000 to the mouth of the Savannah River are primarily sand, which indicates that the source of sediment from this reach is offshore and disposal area erosion. A breakdown of sediment characteristics by dredging reach is shown in **Table 3.4.1-1**.

Table 3.4.1-1: Sediment Characterization of Annual Maintenance Material by Reach

	Stations	**Sand (%)**	**Fines (%)**
Outer Harbor	0+000 to -10+000	86%	14%
	-10+000 to -20+000	81%	19%
	-20+000 to -30+000	79%	21%
	-30+000 to -40+000	77%	23%
	-40+000 to -50+000	74%	26%
	-50+000 to -60+000	93%	7%
Inner Harbor	0+000 to 4+000	90%	10%
	4+000 to 24+000	92%	8%
	24+000 to 40+000	15%	85%
	40+000 to 50+000	30%	70%
	50+000 to 70+000	23%	77%
	70+000 to 79+000	8%	92%
	79+000 to 97+750	16%	84%
	97+750 to 102+000	54%	46%
	102+000 to 103+0000	64%	36%
	103+000 to 112+000	80%	20%

Sediment characterization is done in accordance with the Unified Soils Classification System per ASTM D2487-00, Standard Practice for Classification of Soils for Engineering Purposes.

Maintenance dredging is performed regularly in the entrance channel and the inner harbor in accordance with the practices and procedures outlined in an Annual Work Plan developed from guidelines set forth in the Dredged Management and Material Plan (DMMP) done in accordance with the Long Term Management Strategy (LTMS). Maintenance dredging in the entrance channel, also known as the bar channel, is performed by hopper dredges that generally work from December through March of each year. Dredging is restricted to this period to minimize the impact dredging has on sea turtles. Material is placed in the EPA approved offshore disposal site. The project is also authorized to deposit maintenance sediments in Sites 2 and 3, as well as other designated nearshore sites. Hopper dredges are generally used in the bar channel because they are designed to withstand open ocean conditions, are well suited for removal of sandy materials deposited in the entrance channel, and can move easily between the navigation channel and open water disposal facilities.

Pipeline dredges perform maintenance dredging in the inner harbor and are designed to remove material with higher silt content using a cutterhead and pump it to adjacent CDFs. Material dredged between Stations 0+000 and 112+500 in the inner harbor is placed in upland CDFs. Dredging upstream of Station 66+310 cannot be performed between March 15 and May 30th of each year. This restriction is imposed by the Georgia Department of Natural Resources to protect the spawning of striped bass in the upper estuary of the harbor. Maintenance dredging is generally being performed in the harbor throughout the year except during restricted times.

Monthly project condition surveys are performed in the navigation channel and turning basins to assist in planning and directing the operation of maintenance dredges. Condition surveys are four profiles run on a monthly basis. These profiles are located on the bottom of the design channel, a specific distance from the toe and centerline on each side of the channel. Thus, there is an inside and outside profile on the north and south sides of the channel. Each of these survey lines are known as “quarters”. The results of these surveys are also furnished to the harbor and docking pilots, towing companies, and other navigation and shipping interests. Generally, maintenance dredging is performed when a shoal 2 ft or more above the authorized project depth occurs in any two adjacent quarters of the channel.

Savannah District constructed and operated the tide structure and sediment basin in the Back River between 1974 and 1991. This feature shifted a significant portion of the shoaling from the navigation channel in the Front River to the sediment basin in the Back River. After construction of the sediment basin and tidegate structure, the river channel shoaling volume between Stations 40+000 and 70+000 was reduced by 2 million cubic yards. The majority of this material shoaled in the sediment basin.

In 1992, New Cut, which was the connecting channel between the Front and Back Rivers, was closed as a separately authorized Section 1135 project. The sediment basin continues to trap maintenance material and is periodically dredged. However, its efficiency has been reduced with the closure of New Cut and removal of the tidegates. The remainder of the material shoals in the navigation channel. Material is easily removed from the sediment basin and is placed in adjacent CDFs at a considerable cost savings compared to the cost of removing material from the navigation project, due to the reduced pumping distance. Due to funding constraints, the sediment basin has not been dredged since 2004.

Several reaches in the inner harbor are considered to have rapid shoaling. Advance maintenance is authorized in the harbor to reduce the frequency of dredging in areas with additional depth in these rapid shoaling areas. Advance maintenance is the additional depth beyond the authorized project dimensions which is required to be dredged for the purpose of reducing overall maintenance costs by decreasing the frequency of dredging. Without this practice, it would be difficult and more costly to provide a navigable project for deep draft vessels. The shoaling locations in the navigation channel have changed since New Cut was closed, and the tidegates were taken out of operation and the need for additional advance maintenance was evaluated. The Kings Island turning basin functions as a sediment trap in the upper reaches of the harbor and an additional 8 ft of advance maintenance was approved and dredged. The locations and depths of approved advance maintenance are shown in **Tables 3.4.1-2** and **3.4.1-3**.

Table 3.4.1-2: Currently Authorized Channel Dimensions (Including Advance Maintenance)

Station	Authorized Depth (-ft MLLW)	Bottom Width (feet)	Advance Maintenance (feet)	Maintenance Dredging Depth (-ft MLLW)
Inner Harbor				
103+000 to 102+000	42	400	0	42
102+000 to 100+000	42	400	2	44
100+000 to 79+000	42	500	2	44
79+000 to 70+000	42	500	2	44
70+000 to 50+000	42	500	4	46
50+000 to 41+000	42	500	4	46
41+000 to 37+000	42	500	4	46
37+000 to 35+000	42	500	6	48
35+000 to 24+000	42	500	4	46
24+000 to 0+000	42	500	2	44
Entrance Channel				
0+000 to -14+000	42	500	2	44
-14+000 to -60+000	44	600	0	44

Table 3.4.1-3: Currently Authorized Turning Basin Dimensions (Including Advance Maintenance)

Name	Length (feet)	Width (feet)	Authorized Project Depth (-ft MLLW)	Advance Maintenance (feet)	Maintenance Dredging Depth (-ft MLLW)
Kings Island 101+298 to 97+750	1,600	1,500	42	8	50
Marsh Island 91+610 to 89+485	900	1,000	34	0	34
Fig Island 69+740 to 67+386	1,500	1,000	34	4	38
Oysterbed Island 4+395 to 2+345	1,050	1,200	40	0	40
Rehandling Basin 10+175 to 4+395	5,000	300	40	0	40

Note: Station Numbers are measured in feet from the harbor entrance (Sta. 0+000). Negative numbers indicate the reach is in the entrance channel.

3.4.2 Annual Maintenance Disposal Facilities

3.4.2.1 Upland Confined Disposal Facilities (CDFs)

Material dredged in the inner harbor navigation channel, turning basins, the sediment basin, and berthing areas is placed in upland confined disposal facilities adjacent to the river. These disposal facilities are regularly maintained and upgraded based on disposal capacity needs projected in the Long Term Management Strategy (LTMS), the 1995 document which established the approved methods for harbor maintenance. These CDFs are located in Chatham County, Georgia and Jasper County, South Carolina. The Georgia Department of Transportation is the local sponsor responsible for providing and maintaining these areas in concert with the USACE, Savannah District.

Table 3.4.2.1-1 shows the location of each CDF relative to the navigation channel stations. Station numbers followed by "BR" are located adjacent to the Back River (sediment basin). The site acreage shown in the table describes the interior footprint of the disposal areas; it does not describe the total area including the dikes. Details of each CDF and location follow the table. See **Figure 3.4.2.1-1** for a location map.

Table 3.4.2.1-1: Upland Confined Disposal Facilities

Area	Station (feet)	Site Acreage
1N	107+500 to 112+500	130
2A	93+000 to 103+000	240
12A	6+500 BR to 10+500 BR*	1040
13A	47+800 to 6+600 BR*	1307
13B	42+000 to 47+800	540
14A	37+000 to 42+000	647
14B	28+000 to 37+000	765
Jones/Oysterbed (JOI)	0+000 to 27+000	890

*BR refers to Back River or that portion of the channel located in the Back River (sediment basin).

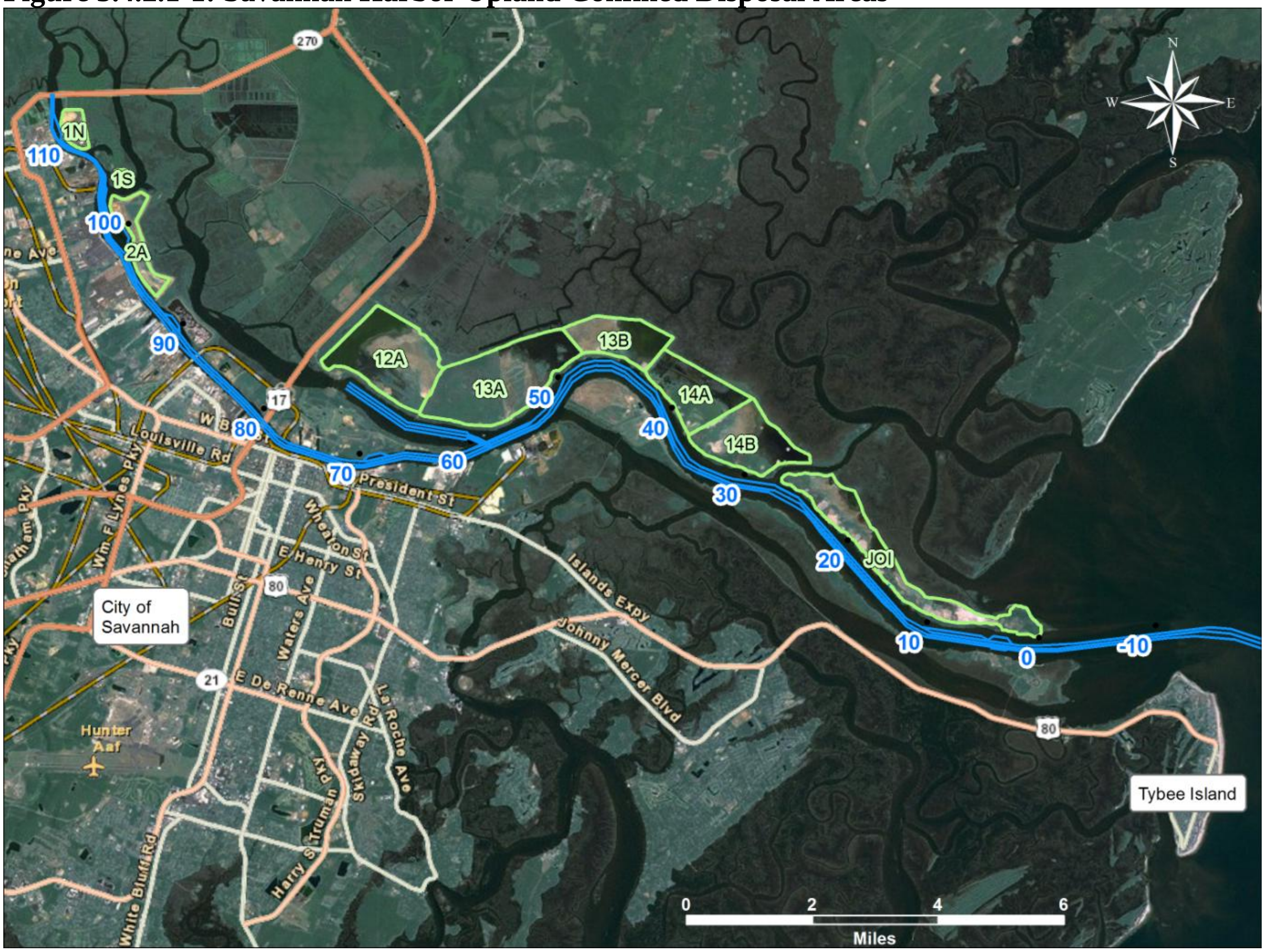
<u>Disposal Area 1N</u> – Disposal Area 1N is the uppermost disposal area in the harbor. It is owned by USF&W and currently contains medium to coarse grained sand. The USACE has limited rights for disposal to this area. All dredged material pumped to the area must be predominantly sand. It will not be utilized during the SHEP.

<u>Disposal Area 2A</u> – The dikes in Disposal Area 2A have recently been raised, and capacity is designated for annual maintenance material. Once this capacity is used, the dikes will no longer be able to be raised due to the overhead power lines which cross the center of the area. There is insufficient clearance between the top of the dike and the low-point (sag) of the power lines to safely operate equipment required to maintain and raise the dikes. In 2009, Area 2A entered a three year drying phase which coincides with construction of the deepening project. Therefore, it will not be utilized for SHEP disposal.

Disposal Areas 12A, 13A, 13B, 14A, and 14B – These five disposal areas are all diked and are contiguous. They are located from south of Highway 17 east along the Back and Front Rivers. Areas 12A and portions of 13A are adjacent to the Sediment Basin on Back River. Areas 12B and 13A were recently combined by breeching the cross dikes during a scheduled dike raising. Construction was completed in 2010, and the area is now designated as Area 13A. Area 14B receives dredged material from both the Savannah Harbor Navigation Project and the Atlantic Intracoastal Waterway (AIWW). During the SHEP construction, 13A, 14A, and 14B are planned for utilization for disposal of new work material.

Jones/Oysterbed Island – Jones/Oysterbed Island (JOI) Disposal Area is the easternmost upland CDF used for dredging. This area will be utilized for SHEP disposal of new work material.

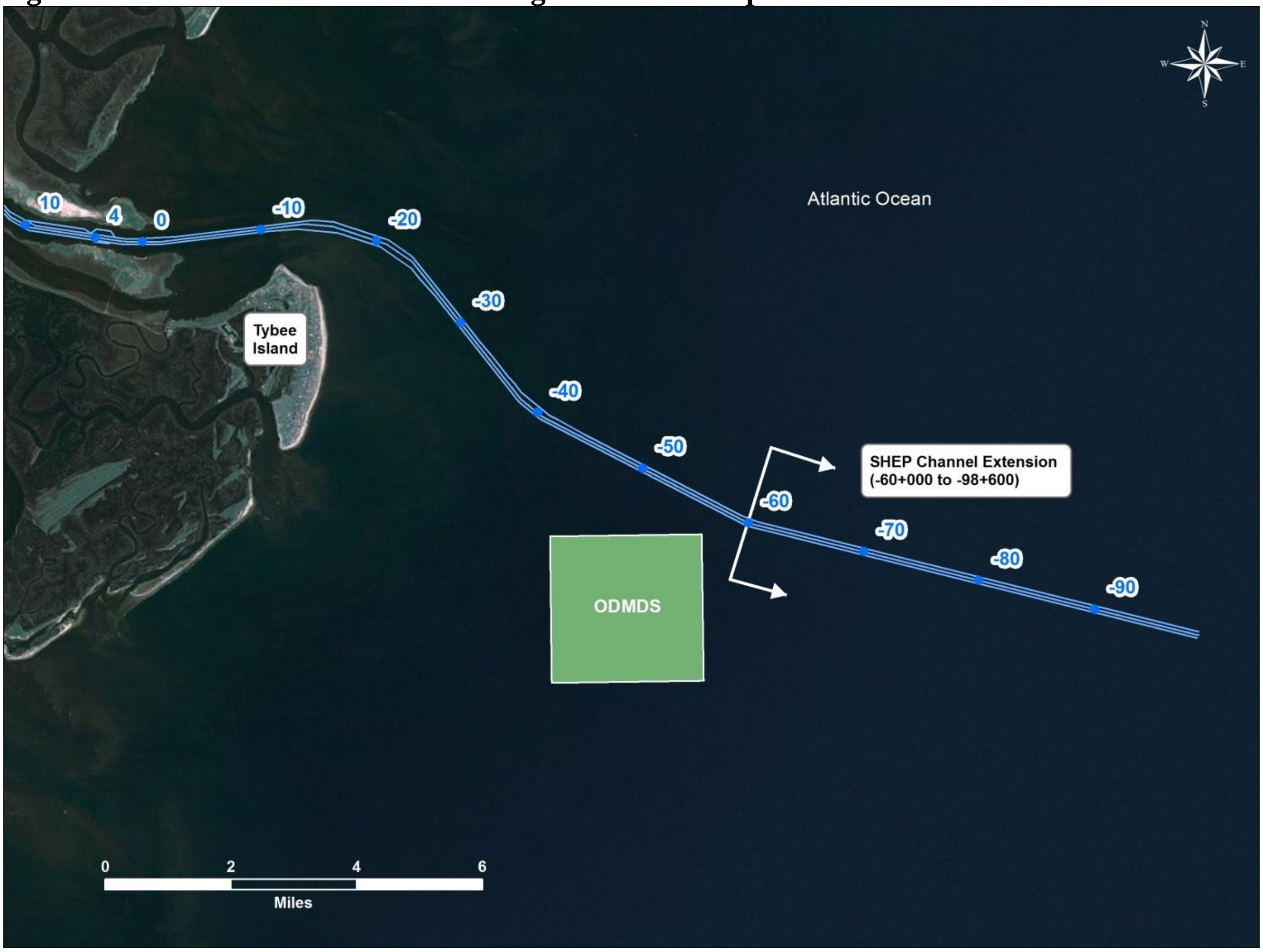
Figure 3.4.2.1-1: Savannah Harbor Upland Confined Disposal Areas



3.4.2.2 Open Water Dredged Material Disposal Areas

Annual maintenance material removed from the Entrance Channel (Stations 0+000 to -60+000) is placed in the EPA-approved Ocean Dredged Material Disposal Site (ODMDS). This 4.26 square nautical mile site is centered at 31^{o} 56' 54" N and 80^{o} 45' 34" W and is shown in **Figure 3.4.2.2-1**. The site is used for placement of the 1 million cubic yards of material that is removed by hopper dredges each year from that channel reach. The final designation of the site as an ODMDS was made by EPA on August 3, 1987.

Figure 3.4.2.2-1: Location of Ocean Dredged Material Disposal Site



3.5 OBSTRUCTIONS AND CROSSINGS

Two significant cultural resources exist adjacent to the navigation channel between Stations 59+000 and 58+000. Old Fort Jackson is a masonry civil war structure located on the south side of the river. The banks upstream and downstream of Old Fort Jackson have a history of erosion problems. On various occasions since the 1970's, the USACE has pumped dredge material around the fort to raise the ground elevation to reduce flooding, placed riprap on the riverbank adjacent to the fort property, and constructed a steel sheet pile wall at the intake structure which controls the flow of water into the moat. The moat wall sits on the riverbank and has been hit by a ship on one occasion. The most recent protection work was completed in May 2004 as an authorized Section 111 project. Work included construction of a steel sheet pile wall on the river side of a portion of the moat and placement of riprap on the downstream side of the fort.

The remains of the CSS Georgia, a civil war ironclad, are located on the north slope of the navigation channel across from Old Fort Jackson. Over the last decade, maintenance dredging operations have been modified to lessen impacts to the remains of the ironclad. Recent archaeological studies have concluded that the remains need to be removed from the channel to preserve what is left before it is totally destroyed by boring worms and corrosion, the effects of which were escalated after a maintenance dredge impacted the site of the wreck in 1983.

In addition to the cultural resources adjacent to the channel, there are submerged pipe crossings, one highway bridge, and an overhead electric power line which cross the navigation channel within the proposed expansion limits. None of these structures presently impacts performance of maintenance dredging in the project. Dredges are required to exercise extreme care, however, when dredging in the vicinity of the pipelines. Dredges are not allowed to set anchors and/or drop spuds near the identified submerged structures when they are performing maintenance dredging. Dredging inspectors constantly monitor the position of the dredge and the dredge anchors when contractors work in this area.

3.6 MILITARY RAPID DEPLOYMENT

The Port of Savannah is one of the thirteen strategic seaports designated to support deployment of selected armed forces combat units and support elements. The Port of Savannah is the designated Sea Port of Embarkation for vehicles and equipment from the 3rd Infantry Division (Mechanized) at Ft. Stewart and elements from Hunter Army Airfield and the 24th Infantry Division, Ft. Benning. The military vessels that are used for deployment at the port include large roll-on/roll-off fast sealift ships that are 946-ft long with a loaded draft of 37 to 38 ft. Since emergency deployment can occur at any time, it is vital to national security that the navigation channel, turning basins and designated container berths at Garden City Terminal are maintained at an adequate depth to accommodate these vessels. There is no alternative port of embarkation for deployment by these installations.

4.0 SITE GEOLOGY

4.1 GEOLOGIC SETTING

Eastern Chatham County is underlain by approximately 2,000 ft of sedimentary Coastal Plain sediments ranging in age from Holocene to Cretaceous. From land surface to a depth of about 500 ft, these sediments consist of unconsolidated to somewhat indurated beds of sand and clay of Recent (Holocene) and Miocene age to indurated limestones of Oligocene and Eocene age. The Oligocene and Eocene limestones comprise what is commonly referred to as the Upper Floridan aquifer.

Within the study area, the elevation of the top of the Oligocene unit, the uppermost unit of the Upper Floridan aquifer, ranges from roughly –95 ft MLW near Tybee Island to approximately –200 ft MLW near downtown Savannah. The top of the Miocene unit occurs at an average of about –45 ft MLW with generally little relief within the study area, and unit thickness ranges from less than 30 ft near Tybee Island to 160 ft near downtown Savannah. **Figure 4.1-1** shows a geologic cross section along the Savannah River navigation channel.

In this study, the Tybee high is the structural feature of most importance, namely where the tops of the Miocene confining unit and the Oligocene unit are nearest land surface. Over the crest of the Tybee high, the elevations of the top of the Oligocene unit range from –95 ft MLW beneath Tybee Island to –115 ft MLW at the channel at Fields Cut, and the Miocene unit is generally exposed in the bottom of the navigation channel. Proposed dredging operations associated with the Savannah Harbor Expansion Project could lower the channel depth to as much as –57 ft MLLW (for the 47 ft project depth) in certain locations, specifically Kings Island Turning Basin. In the harbor vicinity, this stratigraphic horizon is composed of Pleistocene-Recent and Miocene sediments.

4.2 HYDROGEOLOGIC SETTING

The Floridan aquifer system underlies parts of Alabama, Georgia, South Carolina, and Florida and supplies approximately 50 percent of the groundwater in Georgia. The aquifer system is divided into two major aquifers: the Upper Floridan and Lower Floridan. Within Chatham County and the study area, the Upper Floridan aquifer is the primary source of groundwater and supplies approximately 30 percent of the total water supply in Chatham County.

Prior to development, the flow system was considered steady state, i.e. recharge was equal to natural discharge (artesian springs, streams, etc.), and water levels showed little fluctuation from year to year. However, development within the coastal region and the associated increased groundwater withdrawal has unbalanced the recharge and discharge rates. This increased pumping has lowered water levels, induced additional recharge and reduced natural discharge, and increased total flow through the system.

The long-term pumping of the Upper Floridan aquifer in the Savannah area and surrounding coastal areas has lowered groundwater levels and reversed the seaward hydraulic gradient that existed before development. The increased withdrawal of water from the Upper Floridan aquifer has resulted in radial flow directed toward the center of pumping and a cone of depression beneath Savannah. Prior to development, heads in the Upper Floridan aquifer ranged from 20 to 150 ft above sea level in southeast Georgia and from 30 to 50 ft above sea level in Chatham County. In contrast, in May of 1998, Peck reported a maximum head of 60 ft above sea level occurring south of Brunswick and maximum drawdown occurring near the city of Savannah, where heads ranged from -10 ft to -100 ft below mean sea level (**Figure 4.2-1**).

This reversal in hydraulic gradient has resulted in lateral encroachment of seawater and downward vertical intrusion of salt water through the confining unit. Vertical leakage of water through the confining unit contributes a significant amount of water to the flow system in the study area; in fact, the leakage through the upper confining unit has been estimated to represent nearly half the water budget for the Savannah area, or about 40 million gallons per day (MGD). The aquifer effects supplemental study examines the impact of the proposed dredging on the rate of vertical intrusion, which consist of both fresh and salt water, and the resulting groundwater impacts in the Upper Floridan aquifer. The results of this study are summarized in Section 5.0 and documented in full in the report entitled *Supplemental Studies to Determine Potential Groundwater Impacts to the Upper Floridan Aquifer* which is included in the Engineering Investigations Supplemental Materials.

Figure 4.1-1: Geologic Cross Section of Study Area

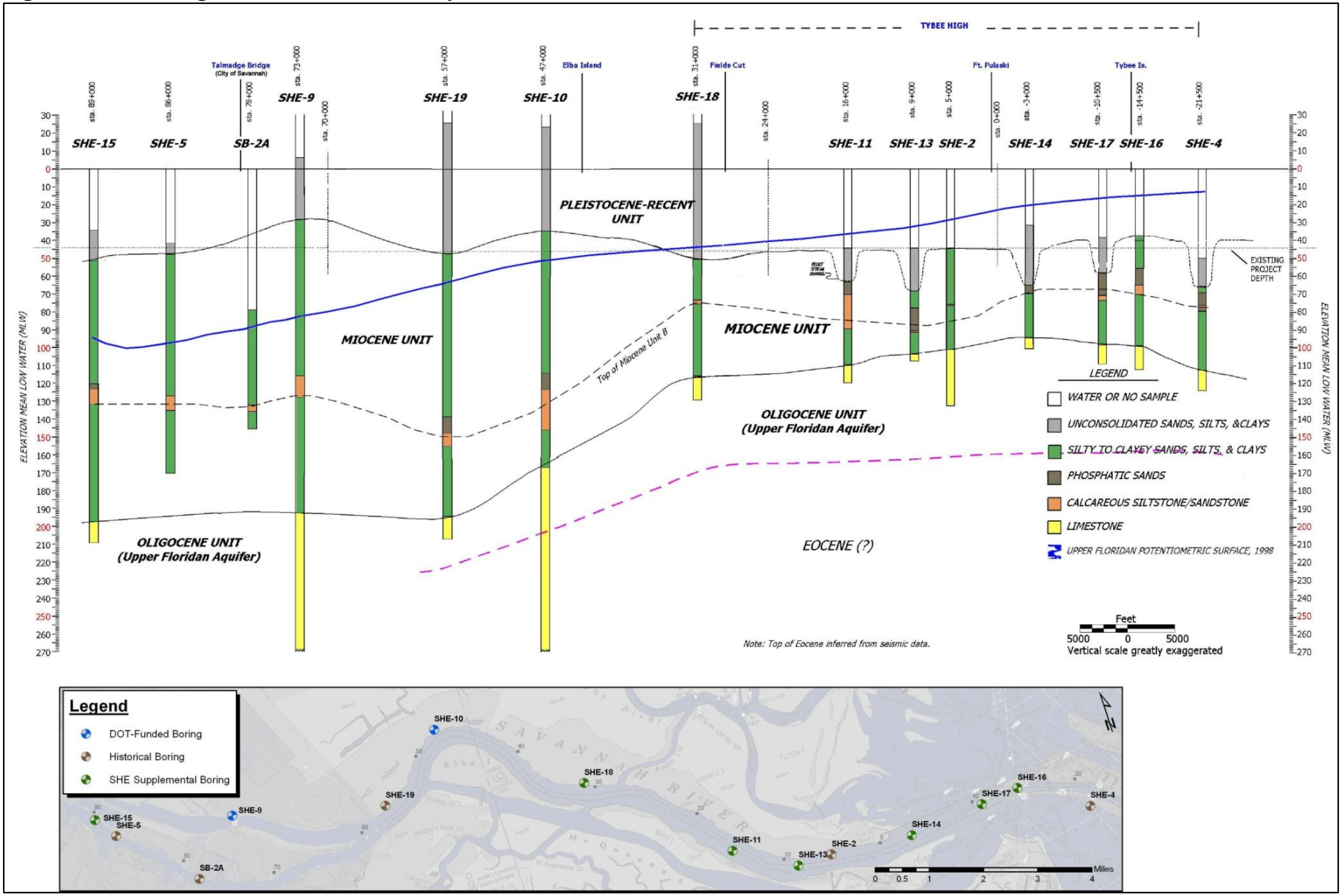
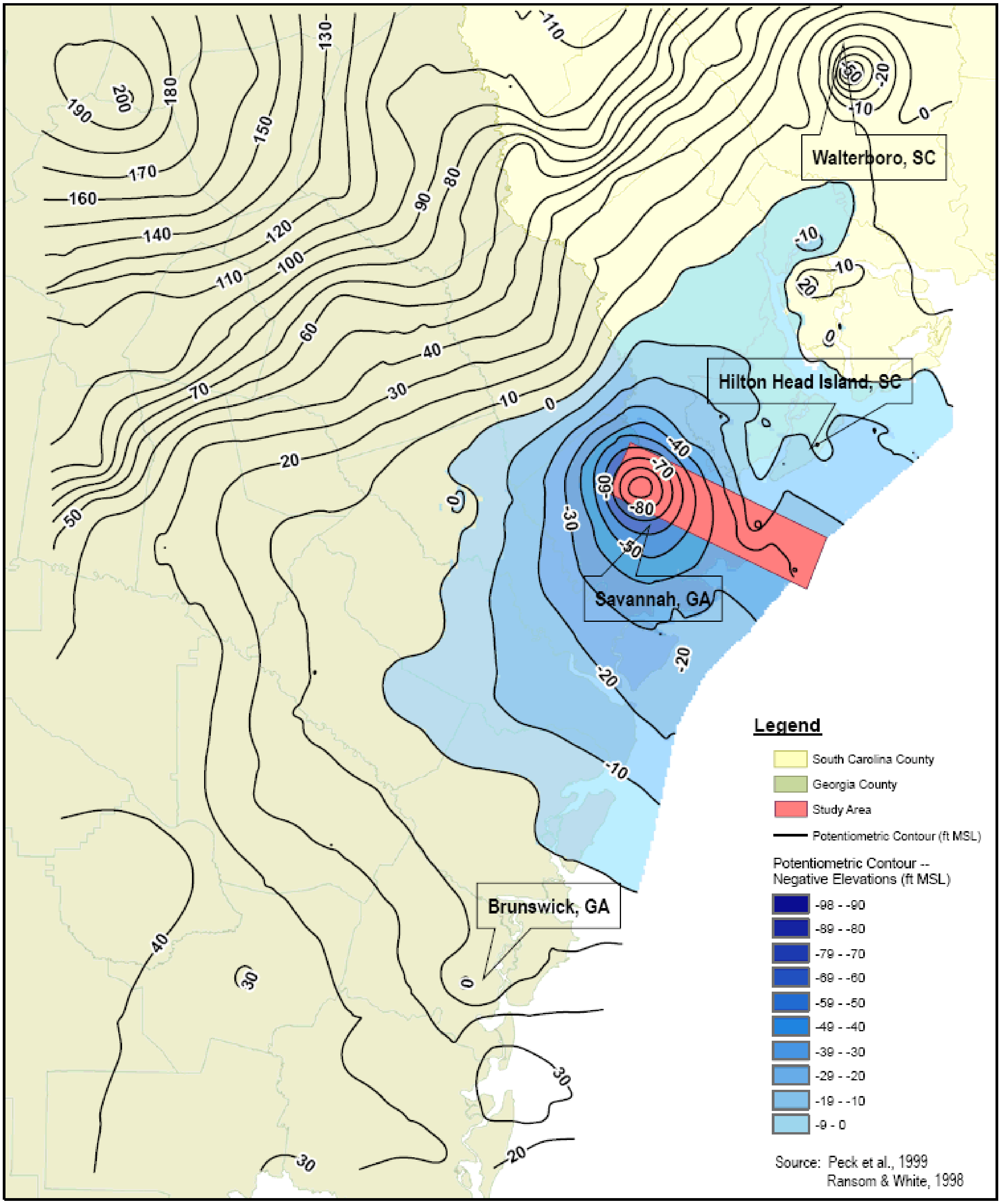


Figure 4.2-1: Potentiometric Surface of the Upper Floridan Aquifer in the Coastal Area, 1998



4.3 GEOLOGIC AND HYDROGEOLOGIC UNITS

This study focused on the hydrogeology of sediments underlying the present navigation channel, specifically the upper 150 to 200 ft, which encompasses the Oligocene, Miocene, and Pleistocene-Recent units. In the Savannah Harbor area, the geologic formations can be grouped into three broadly defined hydrogeologic units: the Upper Floridan aquifer, the Miocene confining unit, and the surficial aquifer.

4.3.1 Upper Floridan Aquifer

In the coastal area, the Upper Floridan aquifer consists of limestone of Late Eocene and Oligocene age and is characterized as vuggy and highly fossiliferous. The Late Eocene unit consists of massively bedded, fossiliferous limestone and dolomite that contains bryzoans, foraminifera, and mollusk shells. The Oligocene unit unconformably overlies the Late Eocene unit and consists of buff-colored, porous limestone with foraminifera, zones of micrite, and nonparticulate phosphate. The Oligocene unit is distinguished from the Late Eocene unit by its lack of bryozoans and its abundance of miliolid foraminifera.

The elevation of the top of the Upper Floridan aquifer (Oligocene) is approximately –200 ft MLW under the city of Savannah, and the contact gently slopes upward to the east toward Tybee Island. Over the crest of the Tybee high, the top of the Upper Floridan aquifer is closer to land surface and is typically around –100 ft MLW in elevation. In the study area, the Upper Floridan aquifer is 150 to 250 ft thick, and the uppermost two zones, zone 1 and zone 2, are the most productive. Zone 1 and zone 2, approximately 44 ft and 35 ft thick, respectively, combine to supply more than seventy percent of the water pumped from open holes tapping the entire aquifer. Pumping reached a maximum of 88 MGD in 1990 and has since slightly declined due to a reduction in industrial pumping. In the year 2000, Chatham County withdrew approximately 72 MGD from the Upper Floridan aquifer.

Transmissivity of the Upper Floridan aquifer is highly variable in the coastal area, and in the area between Port Royal Sound, South Carolina and Savannah, the transmissivity varies from 27,000 ft^2/d to 80,000 ft^2/d. The transmissivity in the Savannah area is low in comparison with other areas along the coast (27,000 ft^2/d to 33,000 ft^2/d). The low transmissivity has resulted in a substantially deeper cone of depression as compared with other major pumping centers with similar withdrawal rates.

4.3.2 Miocene Confining Unit

Strata of Miocene age in the coastal area have been differentiated into the Ebenezer Formation (upper Miocene), the Coosawhatchie Formation (middle Miocene), and the Marks Head and Parachula Formations (lower Miocene); three depositional sequences of similar lithology each bounded by unconformable contacts. Hydrogeologists refer to the units collectively as the “confining bed” or “confining unit” overlying the Upper Floridan aquifer. In the Savannah area, the Miocene sediments unconformably overlie limestone of Oligocene age, establishing a lithologically and geophysically distinctive contact.

The confining unit is a series of lithologically complex sequences of predominately clastic sediments containing low-permeability clays, silts, clayey silts and sands, and clayey or silty sands. Each sequence comprises a geologic unit that consists of a basal carbonate layer, a middle clay layer, and an upper sand layer and is bounded above and below by an unconformity. These units were each defined by persistent geophysical markers designated as A, B, and C and are basal contacts for each of the Miocene units, referred to as upper, middle, and lower Miocene, respectively.

In the project area, Miocene units A and B occur, and Miocene unit C is generally absent or eroded such that only the basal carbonate layer remains and is indistinguishable from the basal contact of unit B. Miocene units A and B consist of green-colored, silty clay and clayey or silty sands underlain by a basal dense, phosphatic limestone or dolomite. Underneath the navigation channel, the basal contacts range in thickness from less than 1 foot to 10 ft thick, and the overall thickness of the confining unit (units A, B, and C) ranges from about 30 ft thick near the Tybee high to over 150 ft thick near downtown Savannah.

4.3.3 Pleistocene-Recent Unit

The shallow sands and clays that occur from land surface to a depth of typically 60 to 75 ft, but locally as much as 130 ft, comprise the Pleistocene-Recent unit. The Pleistocene-Recent unit overlies the Miocene unit in the project area, and the contact is marked by an erosional unconformity, which is sharp in some areas but gradational in others.

Pleistocene to Recent sediments in the Savannah area consist of phosphatic, micaceous, and clayey sand of Pliocene age; arkosic sand and gravel containing discontinuous clay beds of Pleistocene age; and mud, sand, and gravel of Holocene age. Although the geometry and lithologies of these Pleistocene-Recent sediments are geologically complex, with typically lenticular bodies of sand or clay, aquifer sands near the base of the Pleistocene are laterally persistent although not necessarily continuous throughout the coastal region.

Along the present day navigation channel, the Pleistocene-Recent sediments range from 0 to 30 ft thick and are predominantly composed of clays and silts. Depositional environments of the Pleistocene-Recent unit within the Savannah River corridor include off-channel deposits of sands and clays and in-channel deposits of fluvial sands, silts, and clays (paleochannels).

Groundwater within the surficial aquifer occurs under both unconfined (water table) and confined (artesian) conditions in the coastal zone. In places, a basal Pleistocene sand, typically about 15 to as much as about 40 ft thick, is separated from an upper fine-grained sand by a low-permeability dark-gray clay. These sands are recharged by local rainfall, and groundwater moves laterally with typically very low hydraulic gradients toward local streams and tidal water bodies. In the Savannah area, daily combined withdrawals from the upper and lower water-bearing zones range from 120,000 to 855,000 gallons per day.

4.4 SUMMARY OF MATERIALS TO BE ENCOUNTERED DURING DREDGING

4.4.1 General

The sediments underlying the project area are largely a result of varying depositional environments. As such, the sediments are discontinuous both vertically and horizontally and numerous variations occur over short distances. Boring logs completed for these investigations are included in the *Dredged Material Physical Analysis Report* that is included in the Engineering Investigations Supplemental

Materials. The report also includes laboratory results of the mechanical analyses of the samples collected during these investigations and a boring plan of all borings drilled in Savannah Harbor.

4.4.2 Soil

Sediment characterization is done in accordance with the *Unified Soils Classification System* per ASTM D2487-00, *Standard Practice for Classification of Soils for Engineering Purposes*.

The uppermost recent sediments consist of varying mixtures of poorly-graded sands (SP), silty sands (SM), poorly-graded gravels (GP), organic silts (OH), low liquid-limit and high liquid-limit silts (ML and MH), clayey sands (SC), and low liquid-limit and high liquid-limit clays (CL and CH). Standard penetration tests from borings indicate the consistency of the fine-grained soils (silts and clays) range from very soft (0 to 4 blows per foot) to very dense (50 or greater blows per foot), while the coarse-grained soils (sands and gravels) range in consistency from dense (30 to 50 blows per foot) to very dense (50 or greater blows per foot). Typically, these soils vary in color from tan, gray, brown, light brown, and greenish to bluish gray. Generally, soils at the river bottom exhibit lower consistency than the deeper soils. The bottom soils are often very loose and semi-liquid and can extend from the bottom of the river channel to only a few inches to several feet deep.

The underlying Miocene-aged soils consist of silty sands (SM), clayey sands (SC), high liquid-limit silts (MH), and low liquid-limit and high liquid-limit clays (CL and CH). Standard penetration tests indicate the consistencies of the fine-grained soils range from stiff (8 to 15 blows per foot) to hard (30 or greater blows per foot), while the coarse-grained soils range in density from dense (30 to 50 blows per foot) to very dense (50 or greater blows per foot). In general, these soils are characterized by a significant increase in blow counts, uniform consistency, and cohesiveness. These soils are often described as grayish green, green, olive gray, and olive green.

4.4.3 Rock

Lenses of moderately hard to hard limestone have been encountered in borings around the project area; however, its occurrence has been below the depths of concern for this project. In addition, borings drilled in 1969 identified compaction shale in the northern end of the channel, near Kings Island turning basin. This lithology has not been identified in any of the more recent borings, and this material may be analogous to the greenish-gray to olive green, stiff to hard, fat silts and dense to very dense silty sands that have been described in later borings.

A high resolution, subbottom seismic survey was performed in the channel during the early 1990's as part of the investigation program for the previous harbor deepening to -42 ft MLW. This survey showed an area of high acoustic impedance within the middle channel (Stations 70+000 to 24+000). Borings drilled in this area and subsequent dredging indicated this material was similar to the greenish-gray to olive green, stiff to hard, fat clays and dense to very dense silty sands described above.

Thin layers of rock were identified in a 2002 boring at the interface between the post-Miocene sediments and the top of the Miocene. The layer was no more than a few centimeters thick and was a green, fine-grained, sandstone. These more recent borings also describe black gravels and oblong rock fragments within the Miocene sediments, consistent with the geological descriptions of the Miocene phosphatic marker beds provided above in Section 4.3.2. These materials were encountered both within and below the project depth.

4.5 SUBSURFACE INVESTIGATIONS

4.5.1 Aquifer Effects Evaluation Supplemental Study

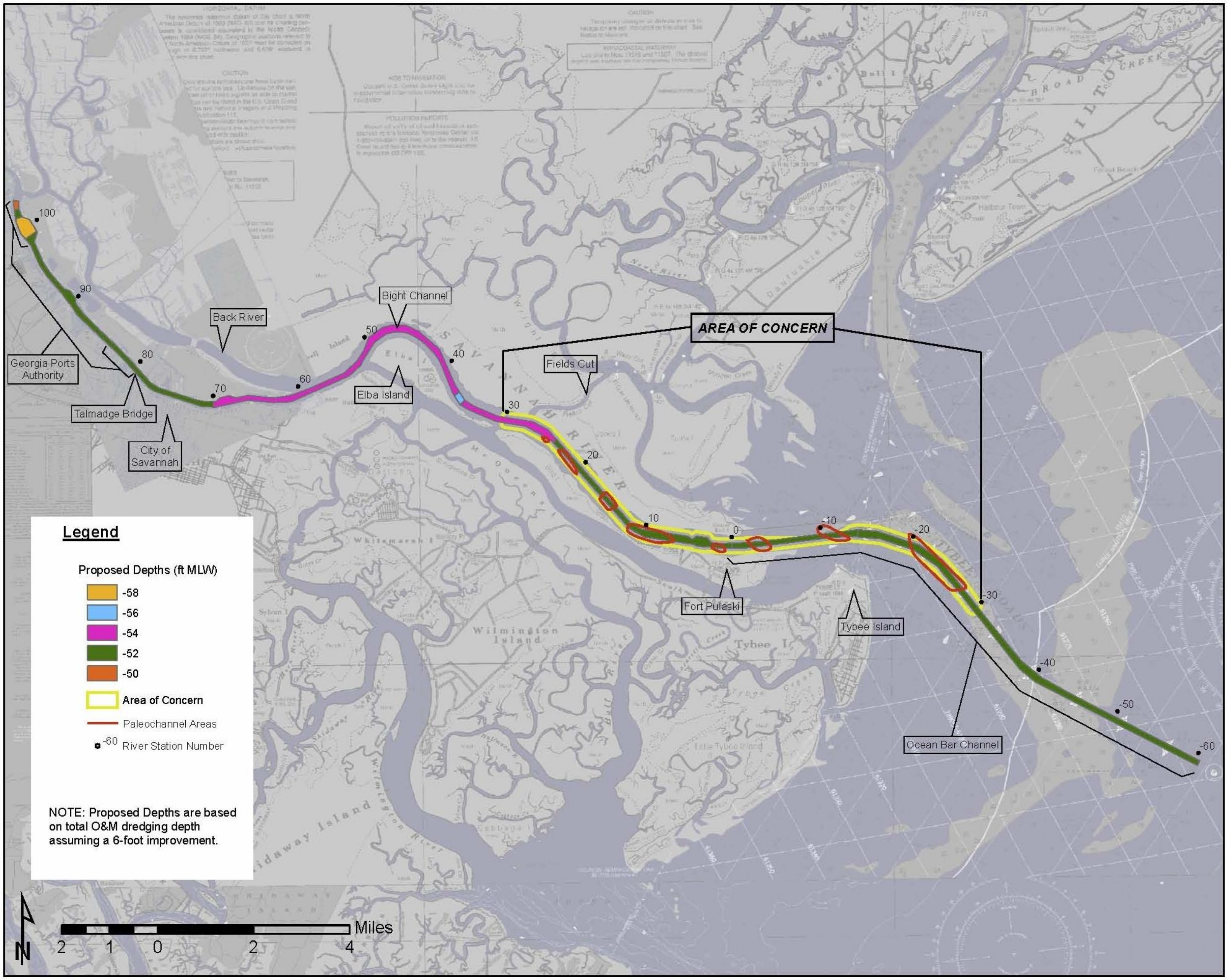
4.5.1.1 Purpose and Scope

The methods employed in the current aquifer study were intended to build and expand on the information from previous studies, particularly the 1998 *Potential Groundwater Impacts* for the Savannah Harbor Expansion Feasibility Study that was prepared as part of the Tier I EIS. Following the release of the 1998 study, the Savannah District, with input from the United States Geological Survey (USGS), Georgia Environmental Protection Division (GAEPD), South Carolina Department of Health and Environmental Control (SCDHEC), and the Stakeholders Evaluation Group (SEG), developed a conceptual plan and work outline to address comments from the 1998 report and establish new supplemental study objectives.

The principal objective of the current study was to determine how much proposed dredging activities would contribute to increased chloride levels in the Upper Floridan aquifer and evaluate the associated impacts on aquifer water quality. Based on the 48 ft depth alternative, the proposed dredging activities to deepen the navigation channel would impact materials contained between -42 ft and -58 ft MLLW, which is comprised primarily of Miocene-aged sediments. Consequently, the study focused on the Miocene-aged upper confining unit (i.e. confining layer) along the navigation channel, especially from Fields Cut to approximately two miles offshore of Tybee Island, where the confining layer naturally thins and relict channels have cut further down into the confining layer as shown in **Figure 4.5.1.1-1**.

The Savannah District Geology/Hydrogeology, and HTRW Design Section evaluated the study objectives according to six major tasks that included completing additional seismic surveying, conducting additional land and marine drilling that incorporated porewater and hydraulic testing, developing a groundwater model, determining the feasibility of conducting an aquitard test, and incorporating data, past and present, into a comprehensive Geographic Information System (GIS). The results and conclusions are discussed in Section 5.0, and the complete report entitled *Supplemental Studies to Determine Potential Groundwater Impacts to the Upper Floridan Aquifer* is included in the Engineering Investigations Supplemental Materials.

Figure 4.5.1.1-1: Overview Map of Project Area of Concern



4.5.2 Environmental Sediment Quality Evaluation

Sediment quality evaluation was conducted between geologists and biologists to determine environmental implications. Details for the full evaluation can be found in the EIS. A summary of those findings is found below.

In 1997, sediment core samples were collected and examined for sediment physical and chemical properties. The sampling area covered the entire project area proposed for harbor deepening, extending from deep water in the ocean to the Kings Island Turning Basin (Station 103+000). Parameters investigated included metals, PCBs, PAHs, petroleum hydrocarbons, phenols, pesticides, dioxin congeners, cyanide, organotins, and nutrients. The evaluation found that most of the sediments did not provide any concern for potential contaminant-related impacts associated with the proposed dredging and dredged sediment placement. However, three potential issues were identified.

One issue involved sediments near the old RACON Tower site. Subsequent sampling conducted in 2005 revealed that sediments at that location do not pose a potential for contaminant-related environmental impacts.

The second issue pertained mostly to whether the sediment chemistry data for pesticides, PAHs and phenols, especially achieved detection limits, were adequate for comparison to screening criteria. This issue was addressed during the 2005 sampling. The confirmatory sampling within the channel revealed there are no potential sediment contaminant concerns related to pesticides, PAHs, phenols, or metals other than cadmium.

The final issue involved the concentration and distribution of cadmium within the new work sediments. Sampling was conducted in 2005 to address this issue. Cadmium was found to occur naturally in unusually high levels within Miocene soils that would be excavated during the SHEP dredging. Evaluation of the laboratory results could not rule out the potential for adverse impacts from sediments with elevated cadmium levels in some reaches of the channel. Therefore, additional sampling and detailed analyses were conducted in 2007. The potential pathways by which cadmium might enter the environment were evaluated. Pathways of particular concern were identified to be exposure of cadmium-containing clays within the channel with subsequent movement of cadmium into the river ecosystem, and potential environmental impacts associated with placement of cadmium-containing sediments within the confined disposal facilities (CDFs).

The recommendations resulting from these evaluations are as follows:

New work sediments from the reaches 6+375 to 45+000, 51+000 to 57+000 and 80+125 to 90+000 should be isolated within a CDF and capped/covered with sediment from another reach. The high cadmium sediments should not be disturbed further and should not be allowed to be later excavated and placed in any exposed upland area.

Once a CDF is selected to receive the sediment, the high cadmium sediments should be pumped into the area first. Once placement of sediments from this reach is completed, markers should immediately be placed on the surface of the sediment, to allow easy determination of when the proper cap/cover depth has been attained. At least 1 foot of additional sediment from another reach should then be placed in the area as soon as practicable, but as part of the SHEP, to ensure minimal environmental impacts from birds feeding within the CDF. Due to expected variability in construction techniques, the project design will use 2 ft of capping/covering sediments. Details of the disposal plan for cadmium sediments are discussed further in Section 11.0.

4.5.3 Radiological Screening

4.5.3.1 Purpose

The project area is located downstream from the Savannah River Site, where radioactive materials are processed. Releases of radioactive materials from the site have been documented over the years, raising concern as to whether such releases could cause accumulation of these materials within the sediments that are to be removed from the project area. The likelihood of this occurring has been considered improbable; nevertheless, a decision was made to screen selected sediment samples that were collected during the 2002 investigations for gross radioactivity. The complete report is included as part of the *Dredged Material Physical Analysis Report* included in the Engineering Investigations Supplemental Materials.

4.5.3.2 Procedures

All samples collected during the 2002 subsurface investigations were placed in sample jars, labeled, inventoried, boxed, and shipped to the US Army Corps of Engineers, Savannah District Environmental and Materials Unit located in Marietta, Georgia. Prior to performing the mechanical analyses, randomly selected jar samples from each borehole were screened for gross radioactivity. This process involved using a Bicron Geiger-Mueller gauge equipped with a pancake probe. The probe was placed approximately 1 centimeter above the soil samples, and the highest reading was recorded. Readings were also collected from around the facility to determine ambient background levels.

4.5.3.3 Results and Conclusions

All results were measured in milli-Rems per hour (mRem/hr). Background values varied between 0.01 and 0.03 mRem/hr. Values from the sediment samples ranged from 0.02 to 0.04 mRem/hr. These results are consistent with the background values, with only a few samples exhibiting an increase of 0.01 mRem/hr over the background range. The slight increase in some samples was expected, as the soils encountered are known to contain phosphatic and glauconitic minerals, along with other clays, that exhibit naturally-occurring radiation levels above background. The results are two magnitudes below acceptable action levels approved by OSHA and the EPA that range between 1 mRem/hr and 5mRem/hr. Based on these results, it is believed that the levels of gross radioactivity measured in the samples is naturally occurring and poses no hazard to the environment.

5.0 INVESTIGATIONS OF IMPACTS TO THE UPPER FLORIDAN AQUIFER

5.1 PROCEDURES

The methods employed in the aquifer study were intended to build and expand on the information from previous studies, particularly the internal studies done by the Savannah District and USGS Bulletin 113 by Clarke et al. The Savannah District used input from various agencies including the USGS, GAEPD, SCDHEC, SEG, and GPA to develop a scope of work for the supplemental studies. The study was implemented according to six tasks (**Table 5.1-1**), each of which is summarized below. Detailed methods for each task are included in the report *Supplemental Studies to Determine Potential Groundwater Impacts to the Upper Floridan Aquifer*, which is included in the Engineering Investigations Supplemental Materials.

Table 5.1-1: Tasks Comprising Aquifer Effects Evaluation Supplemental Studies

Task	Subject	Description
1	Subbottom Seismic Survey	Conduct additional subbottom seismic surveying with particular emphasis to better define paleochannel geometry and Upper Floridan confining unit thickness. All seismic data will be acquired in digital format to facilitate analysis and storage in the GIS.
2	Marine Drilling	Conduct additional marine continuous core borings to further characterize in-filled sediments of paleochannels and Miocene confining unit below paleochannels.
3	Land Drilling	Conduct additional continuous core borings on land adjacent to navigation channel to top of Upper Floridan aquifer at three strategic locations where geologic or hydrogeologic data is sparse.
4	GIS	Combine existing geologic, hydrogeologic, seismic, and engineering data from previous studies into the harbor-wide GIS being constructed for Savannah Harbor. Add future supplemental data to the GIS to allow enhanced analysis and visualization.
5	3-D Numerical Hydraulic Model	Develop 3-D coupled flow and transport model of the hydrologic system focused on the navigation channel, and use model to compare before and after dredging results as related to projected chloride changes in the Upper Floridan aquifer.
6	Aquitard Test Feasibility	Conduct trial step-drawdown pumping test on two recently installed Upper Floridan wells located adjacent to river channel to determine feasibility of hydraulic testing of confining unit. If results indicate hydraulic testing of confining unit is feasible, estimate design parameters and assumptions for full aquitard testing.

Ocean Surveys, Incorporated (OSI) performed a supplemental subbottom geophysical survey to fulfill requirements outlined in Task 1. The survey served as an addition to the extensive work completed in 1997, in which relict paleochannels and the underlying stratigraphy were defined along the centerline of the navigation channel. OSI conducted the supplemental survey along the sides of the navigation channel between River Stations 30+000 to –30+000, where the majority of paleochannels cut across the navigation channel, in an effort to better determine the orientation of the paleochannels and the thicknesses of the underlying units.

Seven marine continuous core borings were drilled adjacent to the navigation channel, six of which were drilled in known paleochannels, to fulfill the requirements outlined in Task 2. The cores were drilled to the top of the Upper Floridan aquifer to further define the stratigraphy underlying Savannah harbor. Each core was drilled using fresh water and analyzed for porewater geochemistry, geophysical markers, grain size, porosity, and vertical hydraulic conductivity.

Similar to Task 2, Task 3 entailed drilling two additional land borings in an effort to complete the geologic transect along the entire length of the navigation channel. The borings were strategically drilled in areas where geologic or hydrogeologic data was sparse, and core samples were analyzed for porewater geochemistry, geophysical markers, grain size, porosity, and vertical hydraulic conductivity. In addition, the data from the borings will be used to install two sets of multi-level wells near existing Upper Floridan wells. The wells will be installed within the surficial aquifer and the Miocene confining unit and will be used to collect hydraulic head and groundwater data at discrete depth intervals over long periods of time.

The porewater geochemistry methods employed in this study were developed over the past ten years by researchers and scientists that have extensive sub-surface knowledge of the coastal region. The innovative technique allowed Savannah District scientists to evaluate dredging effects based on an unprecedented amount of actual field data, not just assumed or estimated values. For each boring, samples of porewater, water contained within the pore spaces of a geologic material, were collected at regular intervals from, at minimum, the top of the confining unit to the top of the Upper Floridan aquifer. Several sampling methods were employed to collect in-situ porewater at discrete depths throughout the confining unit. The porewater samples were then analyzed for concentration of several dissolved ions, including chloride. The resulting concentrations were then plotted according to the depth at which they were collected, yielding profiles of chloride concentration within the confining unit versus elevation for each boring location.

Task 4 concerned the development of a comprehensive harbor-wide GIS. Specifically, the task aimed to compile existing geologic, hydrogeologic, seismic, and engineering data from available historical reports published by the Savannah District, USGS, GAEPD, SCDHEC, or otherwise into a comprehensive GIS for enhanced analysis and visualization.

Task 5 entailed developing a three-dimensional (3-D) numerical hydraulic coupled flow and transport model of the hydrologic system in the immediate vicinity of the navigation channel. The Savannah District issued a contract to CDM to perform this task. The modeling software used for the study is called DYNCFT, which includes applications from DYNFLOW and DYNTRACK. The model incorporated hydraulic properties, confining unit thickness, and historic and present pumping rates to determine a range of plausible aquifer responses to deepening the navigation channel. Simulations were run according to a no dredging scenario and a worst-case dredging scenario, where “worst-case” refers to a maximum project depth of -48 ft MLW, the associated overdredging allowances, and a three foot disturbance depth. The model outputs were compared to evaluate the potential effects of dredging on water quality in the Upper Floridan aquifer.

Task 6 was intended to be a trial pumping test on two existing Upper Floridan wells in order to determine the feasibility of performing an aquitard test on the confining unit. Prior to conducting this task, several model simulations were performed to check the validity of previous results from pumping tests conducted at the Tybee Island test well cluster. Since the Tybee Island tests and the model simulations indicated a full aquitard test would likely require months to pump millions of gallons of water from the aquifer to acquire potentially little meaningful data, full aquitard testing was not felt to be warranted. Further details on the simulated pump tests are included in the complete report, *Supplemental Studies to Determine Potential Groundwater Impacts to the Upper Floridan Aquifer*, which is included in the Engineering Investigations Supplemental Materials.

5.2 RESULTS

The detailed study approach allowed for a greater understanding of the geologic and hydrogeologic framework underlying the navigation channel. Measured porewater data, hydraulic conductivity data, head data, seismic data, and confining layer thickness data were used to build upon a regional model built by USGS and refine it to address water quality issues specifically associated with dredging impacts. In order to ensure the groundwater model results were conservative, the dredging scenarios were run assuming an additional three feet of material would be removed below the proposed dredging depths. In addition, the model simulations used two values of hydraulic conductivity that provided two sets of results that bracketed true conditions, yielding best-case and worst-case scenarios for both dredging and no dredging conditions. Selected results are summarized below. Detailed results for each task are included in the report *Supplemental Studies to Determine Potential Groundwater Impacts to the Upper Floridan Aquifer*, which is included in the Engineering Investigations Supplemental Materials.

5.2.1 Porewater Profiles

The porewater data derived from this work indicate that, as expected, seawater is moving downward through the Miocene confining layer toward the Oligocene limestone (Upper Floridan aquifer), and in some locations, low concentrations of chlorides appear to have migrated entirely through the confining layer and into the limestone. The pronounced profiles show that chloride concentration decreased with depth from the top to the bottom of the confining layer, and chloride values ranged from a high of 20,000 mg/L near the top of the layer to a low of 15 mg/L near the bottom of the layer. The data also suggest somewhat enhanced leakage of salt water in areas where deep paleochannels cut across the present navigation channel that are underlain by punctuated decreases in chloride concentration below the Miocene unit A contact. **Figure 5.2.1-1** shows a geologic cross section of borings completed and their corresponding chloride porewater profile curves.

5.2.2 Geophysical Survey

The subbottom seismic survey provided a comprehensive data set of the stratigraphy underlying the navigation channel within the area of concern (River Stations 30+000 to –30+000). The seismic profiles generated from the survey were used to better understand the three dimensional relationship of the navigation channel, paleochannels, and the confining layer. A typical plan and cross section illustrating the relationship between the navigation channel and paleochannels is shown in **Figure**

5.2.2-1. In general, subbottom data indicated that the paleochannel features identified in the entrance channel are oriented oblique to the present-day course of the river. The subbottom data indicated that the minimum thickness of Miocene confining material underlying the navigation channel was about 26 ft near Station 9+000.

5.2.3 Groundwater Model

The groundwater model results indicated that the expected increase in downward volume of flow of saline water from the area underlying the Savannah River navigation channel due to dredging is small. The area affected by dredging accounted for a total downward flow between 50 to 250 gallons per minute depending on the hydraulic conductivity assigned to the Miocene confining unit. Dredging the navigation channel increased the total downward flow between 2 to 7 gallons per minute, or 3 to 4 percent. The contribution is negligible when compared to groundwater production in the Savannah area from the Upper Floridan aquifer, which is on the order of 80 million gallons per day (55,555 gallons per minute).

The concentrations presented represent only the contribution from the river and navigation channel. Other salt water sources (salt marshes, offshore) were not included as part of simulating the explicit impacts of dredging.

Figure 5.2.3-1 shows plan view simulated chloride concentrations in the Upper Floridan aquifer for the years 2000, 2050, and 2200 for both dredging and no dredging scenarios. The distributions indicated that chloride plumes tend to move parallel to the river due to the groundwater flow direction induced by heavy pumping near downtown Savannah. Thus, the concentration results discussed above are relevant only for chloride concentrations directly below the river. Simulated impacts north or south of the river dissipated over a relatively short distance. The simulated chloride concentrations in the Upper Floridan aquifer using the mid-range value of hydraulic conductivity showed negligible difference between the dredging and no dredging scenarios.

In the year 2200, the upstream chloride concentrations in the Upper Floridan aquifer beneath the river were defined as approximately 0 mg/L for low-value hydraulic conductivity simulations and up to 100 mg/L for the mid-range hydraulic conductivity simulations. Downstream, chloride concentrations directly beneath the river approached 500 mg/L after 200 years for the low-value hydraulic conductivity simulations. For the mid-range hydraulic conductivity simulations, total breakthrough (equilibrium) occurred after approximately 100 years, and the maximum chloride concentration (1,400 mg/L) in the Upper Floridan aquifer occurred in the downstream portion of the study area.

In the upstream reaches of the river, where the surface water model predicted minimum increases in chloride concentrations, the differences in chloride concentrations in the top of the Upper Floridan aquifer between the dredging and no dredging scenarios were minor. Downstream, where higher surface water chloride concentrations were predicted to occur, the corresponding differences in concentrations in the Upper Floridan aquifer directly below the river ranged from 10 to 200 mg/L and were typically observed 50 or more years into the future. These concentrations represent only a small percentage of the total concentrations expected in the aquifer, thereby yielding the contribution from dredging to the total concentration in the aquifer insignificant when compared with the combined chloride contributions from other salt water sources.

Figure 5.2.1-1: Chloride Porewater Profiles

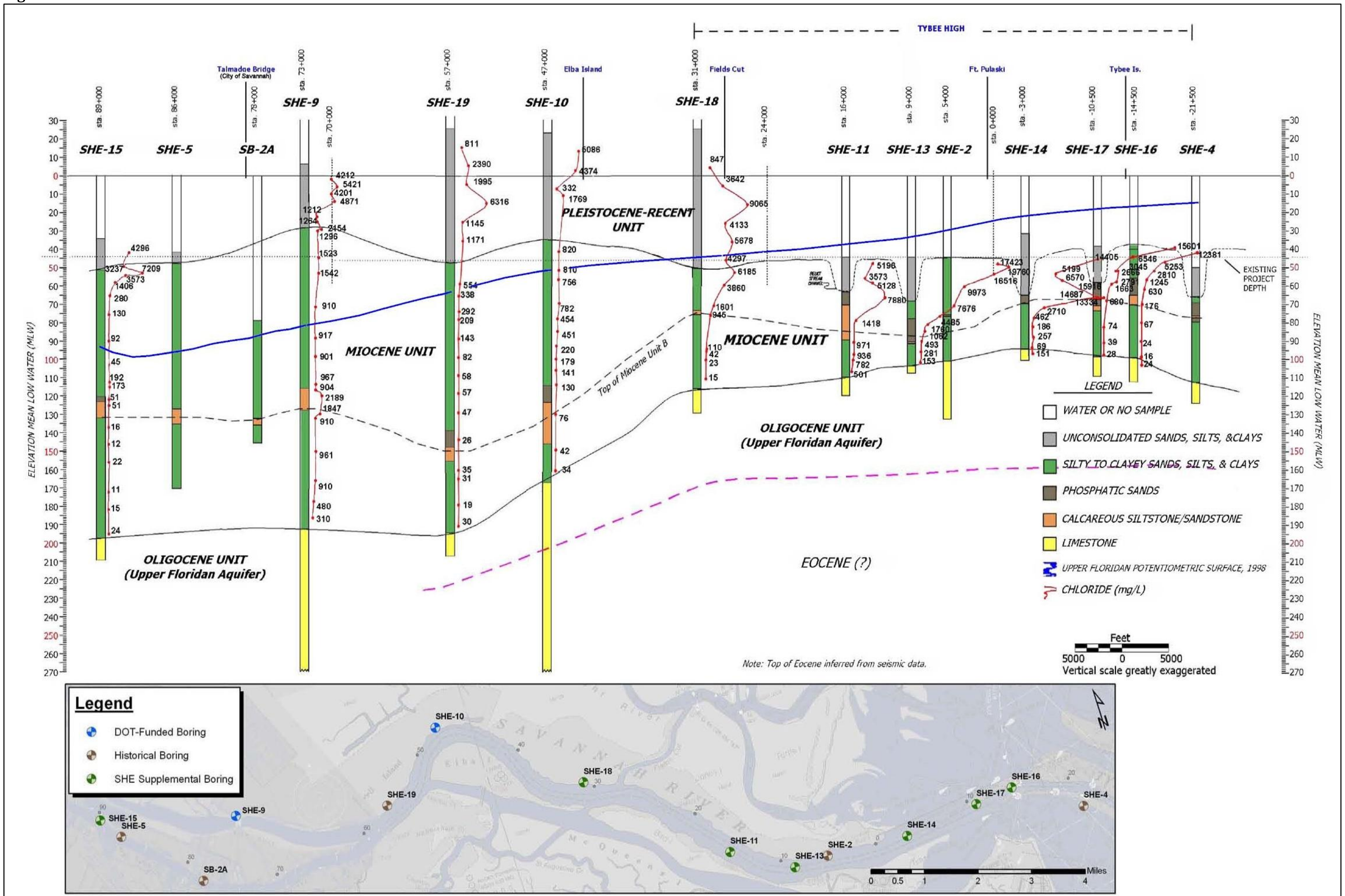


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Figure 5.2.2-1: Paleochannel Orientation and Contact Correlation with Borings

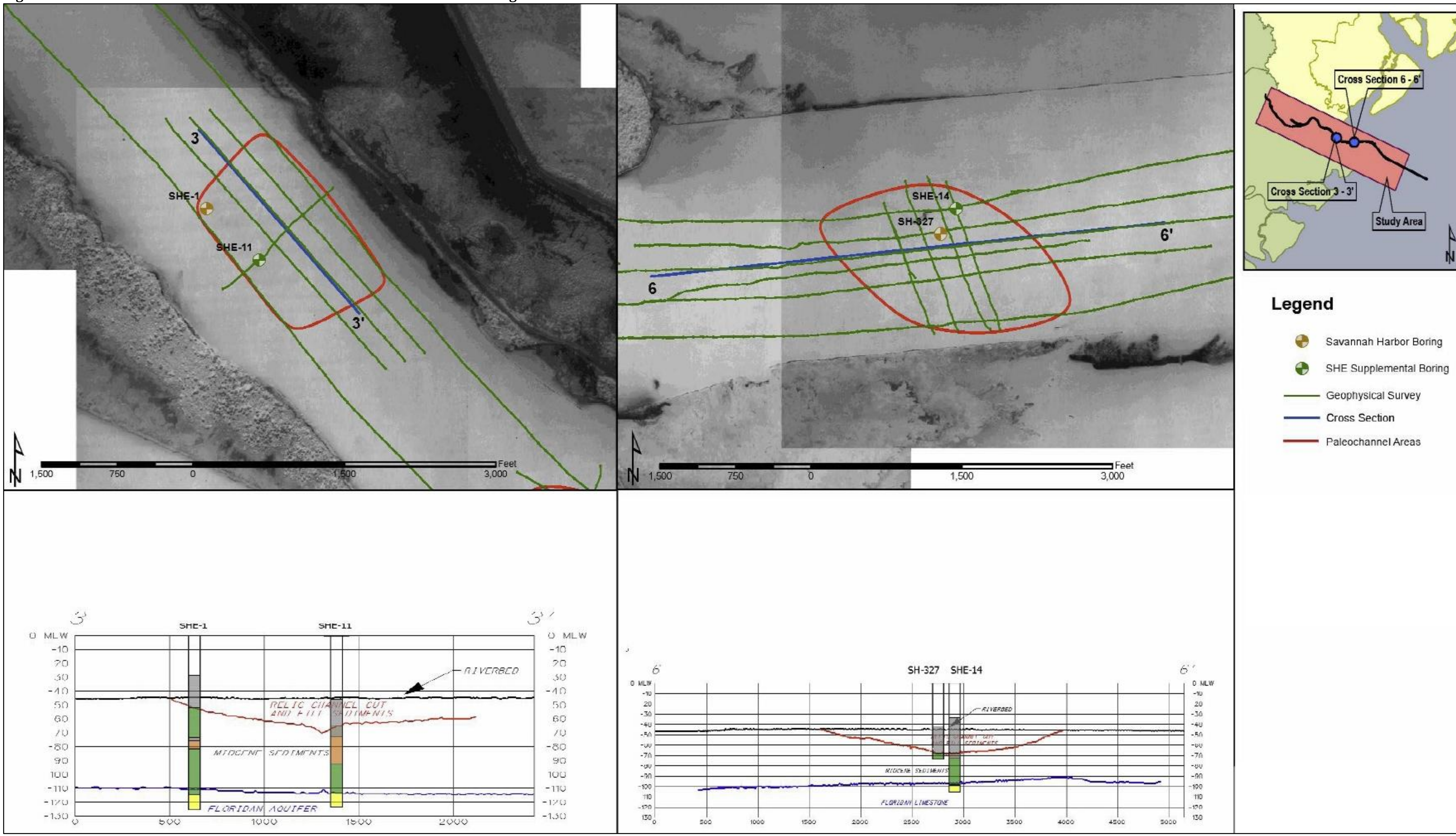

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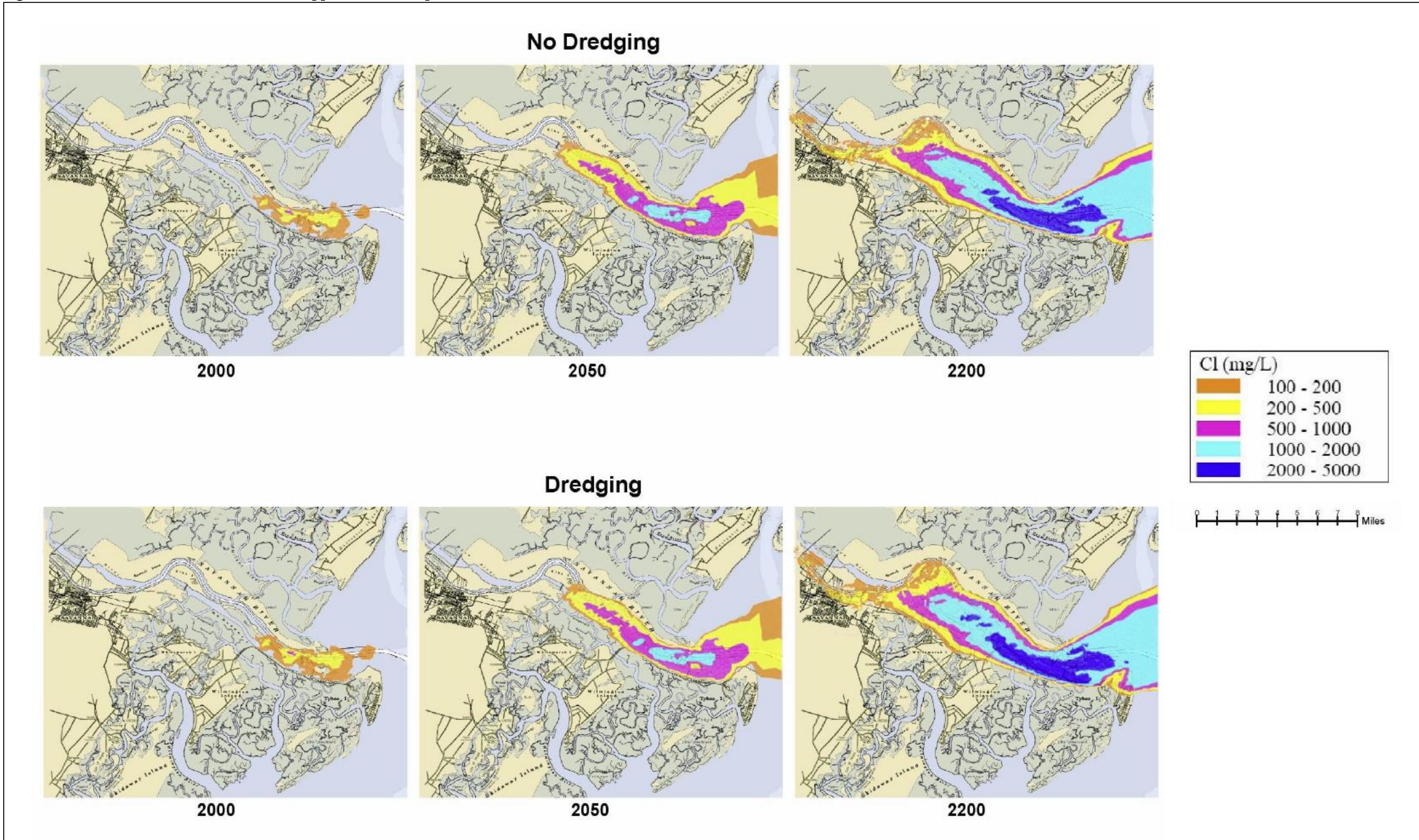
Figure 5.2.3-1: Chloride Distributions in the Upper Floridan Aquifer



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5.3 CONCLUSIONS

The results from field work, groundwater modeling, and GIS analyses conducted as part of the supplemental aquifer study provided the most comprehensive picture of the geology and hydrogeology underlying the Savannah River to date. The conclusions and recommendations listed below are based on compiled historic data as well as data collected specifically for the supplemental studies. Whenever applicable, conservative assumptions were applied in order to ensure recommendations were based on a worst-case impact.

A site-specific seismic subbottom survey was performed from River Station 30+000 to –30+000, and the results of the survey provided detailed stratigraphy and information about all major paleochannels within the area of concern. The location, attitude, and extent of all paleochannels were mapped and incorporated into the Miocene surfaces created for the GIS and the groundwater model to determine their role in potential dredging impacts. Groundwater model results indicated that any additional contribution of chloride by the paleochannels is negligible when compared to the total contribution from other adjacent salt water sources outside paleochannels along the river bottom. The impacts of dredging on the in-fill sediments of the paleochannels, which were simulated in the model to represent sand, were small when compared to the impacts of dredging elsewhere in the channel where the Miocene confining unit is impacted. GIS analyses indicated that the minimum thickness of the Miocene confining material occurs where paleochannels have incised into the top of the unit, and the proposed dredging activities would not further impact the Miocene confining layer in these areas.

All model results and concentrations reported are based on chloride concentration effects specifically associated with dredging the navigation channel. They do not account for other salt water sources including salt marshes or the Atlantic Ocean. As such, the values reported *do not* represent total concentrations or distributions expected; they represent the contribution from the river and navigation channel to the total concentration. This contribution is a small percentage when compared to the total concentration expected from other salt water sources.

The location of the maximum negative head gradient, i.e. the center of the cone of depression, poses the largest potential for enhanced salt water leakage through the confining layer. The porewater data and model results, however, showed that the thickness of the confining unit (>100 ft) and the lower salinity of the river water at this upstream location minimize this impact in the upstream reaches of the navigation channel and production wells located in and around Savannah.

The downstream areas, however, specifically near the Tybee high, showed a gradual increase in chloride concentrations in the Upper Floridan aquifer ranging from 500 to 1400 mg/L depending on hydraulic conductivity of the confining layer. The enhanced salt water intrusion in this area is attributed to a combination of factors: the induced negative head gradient from pumping in Savannah; the overlying seawater or saline water with minimal freshwater input from the Savannah River; the naturally thin confining layer (40-60 ft); and the paleochannels that have further removed Miocene material.

Although lab results indicated that zones within the paleochannel fill material have comparable hydraulic conductivities to the Miocene material, porewater profiles constructed within paleochannels suggest that they still have some influence on the rate of salt water intrusion. The porewater profiles also showed that chloride concentrations decrease rapidly below the Miocene contact. This punctuated reduction in concentration supports the notion that dredging paleochannel material would have minimal effect on the downward rate of salt water intrusion. Additionally, if the paleochannel material were not considered "confining", then dredging in these areas would not reduce the thickness of the underlying confining unit. Instead, the potential impacts on water quality due to dredging should focus on the entire thickness of material overlying the aquifer and the amount of Miocene-aged material removed.

Near the Tybee high, the aquifer is predominantly overlain by seawater, and the Miocene confining layer is thin. These two naturally occurring factors significantly contribute to the enhanced salt water intrusion in the area and locally affect water quality in the Upper Floridan aquifer. However, the model results showed that the proposed dredging would have little effect on this process. The groundwater model results showed that, in the year 2200, the concentration increase in the navigation channel due to dredging translated to only a small increase in the aquifer directly below the navigation channel (10-200 mg/L dependent on hydraulic conductivity). Production wells located in the downstream reaches of the river showed negligible differences between the dredging and no dredging scenarios, and the contribution to total chloride concentrations increased by a range of 0 to 50 mg/L after 200 years.

The groundwater model simulations were run 200 years into the future with a constant pumping rate in the Savannah area, and the results indicated that this rate of pumping would cause total breakthrough of seawater to occur *regardless of dredging* at some downstream locations in approximately 100 to 300 years depending on hydraulic conductivity of the confining layer.

In summary, the negative head gradient induced by pumping in Savannah has caused limited breakthrough of chlorides to occur in the downstream reaches of the Savannah River. The porewater profiles and model results from this study indicated that increased salinity in the Savannah River and the reduced thickness of the confining layer due to dredging will not significantly affect the timing of breakthrough of chlorides along the navigation channel in the Upper Floridan aquifer. Furthermore, the aquifer study results showed that the proposed dredging would have minimal impacts on water quality in production wells that tap the Upper Floridan aquifer in and around the city of Savannah.

6.0 CHANNEL DESIGN

The Channel Design Drawings, including typical sections, are located in Attachment 1 to the Engineering Investigations Appendix.

6.1 DESIGN VESSEL

The post-Panamax S-class containership, the *Susan Maersk*, was chosen as the design vessel for the Savannah Harbor Expansion Project in September 2001. The *Susan Maersk* is considered the best representation of the vessel of the future considering length, width, and draft. Dimensions of the *Susan* or "S" Class Maersk are: 1,138 ft long, 140.4 ft wide, 47.6 ft design draft. A photo is shown below in **Figure 6.1-1**.

Figure 6.1-1: The *Susan Maersk*

6.2 DESIGN CRITERIA & SPECIAL CONSIDERATIONS

A preliminary channel layout for a 48 ft project depth channel was developed by Savannah District based on EM 1110-2-1613, *Hydraulic Design Guidance for Deep-Draft Navigation Projects*. This guidance states that “the design channel width for navigation projects with maximum currents greater than 3.0 knots should be developed with the assistance of a ship simulator design study”. Savannah Harbor routinely experiences currents greater than 3.0 knots. Paragraph 7c of ER 1110-2-1403, *Studies by Coastal, Hydraulic, and Hydrologic Facilities and Others*, 1 January 1998 states that “Hydraulic design studies associated with the planning, design, construction, operation, and maintenance of navigation channels will include a ship or tow simulation investigation unless omission of such an investigation is approved by HQUSACE". Ship simulation was conducted by ERDC for the SHEP and details are documented in the reports titled: 1) *Navigation Study for Savannah Harbor Channel Improvements 2004* 2) *Savannah Harbor Simulations Study 2009* 3) *Savannah Harbor Entrance Channel Simulations 2010 Report* 4) V*ertical Ship Motion Study for Savannah, GA Entrance Channel 2011*. These documents are included in the Engineering Investigations Supplemental Materials and the results of these four studies are discussed in detail in the following sections.

6.2.1 Discussions with the Harbor Pilots

The existing navigational channel is not presently designed for two-way traffic for all vessels using the project. However, the harbor pilots indicated that they have instituted their own system of traffic control that allows them to have two-way traffic in certain reaches. The traffic control system generally consists of the pilots onboard any vessel underway being in constant contact with pilots on the other moving vessels. This permits the pilots to adjust the speed of the vessel and time meetings when the vessels are in reaches where the currents, channel banks, and/or other moored vessels do not affect the handling of the vessels under way.

According to the pilots, deep draft vessels avoid meeting in the City Front Channel (approximately Stations 80+000 to 70+000) and in the Bight Channel (approximately Stations 55+000 to 40+000). These are areas where ships are aligning to transit under the Talmadge Bridge or tidal currents affect ship handling. It should be noted that Kings Island Turning Basin (Stations 101+298 to 97+750) is the only place the deepest draft vessels can turn around in the harbor.

The harbor pilots also indicated that they prefer 4 ft of underkeel clearance to move a vessel. Vessels drafting more than 38 ft wait for adequate tide stage to provide the desired underkeel clearance. See Section 6.2.2 for additional discussion of underkeel clearance.

The harbor pilots indicated there are reaches in the channel where they have difficulty maneuvering deep draft vessels. One area is the bend in the vicinity of Station 36+000. They indicated that the currents on the outside of this bend affect vessels on the inbound transit and additional width would help them navigate through this reach. In addition, the reach between Stations 72+000 and 59+000 is difficult to navigate on the north side during certain stages of the tide, and additional width through this turn would be beneficial.

The docking pilots have expressed concern over the size and depths in the existing turning basins other than the Kings Island Turning Basin. However, the proposed design vessel will not be calling at terminal facilities which use these turning basins, and it was determined that they would not be included in this study. Another concern of the docking pilots is the width of the Kings island Turning Basin. They felt that the basin should be wider to accommodate turning the expansion project's design vessel when other vessels are moored across the river at Container Berths (CB) 1 and 2 which are located directly in front of the turning basin. The docking pilots also expressed concern that turning a vessel moored at CB 7 and 8 would be difficult. They are located immediately upstream of the Kings Island Turning Basin, and vessels will have to be backed one way between the basin and the berth. This comment was considered in the modification of the turning basin design that was submitted and approved as part of the ship simulation study.

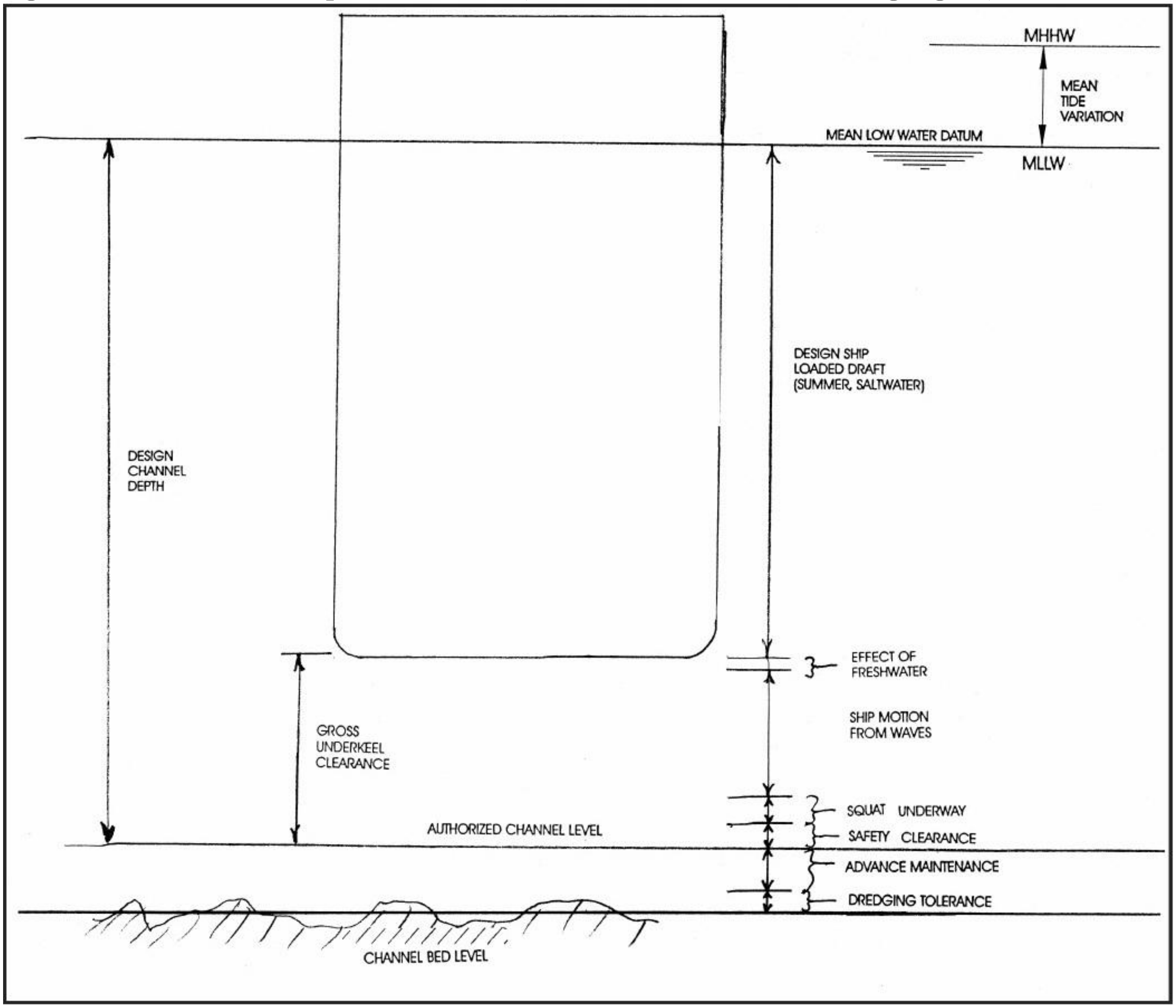
6.2.2 Underkeel Clearance

Underkeel clearance is the distance between the bottom of the vessel and the channel bottom required for safe maneuvering of the vessel during transits in the channel and harbor (See **Figure 6.2.2-1**). Gross underkeel clearance (UKC) is the distance under the ship after subtracting the design ship loaded draft from the design channel depth while the ship is in a static condition (such as moored at the dock). The UKC is composed of several ship-related factors including (a) the effect of fresh water, (b) ship motions from waves, (c) squat underway, and (d) safety clearance or net underkeel clearance. The net underkeel clearance (net UKC) is what is left after subtracting the first three components from the UKC.

The following references were used in the evaluation of Underkeel Clearance for SHEP:

Port of Savannah Minimum Underkeel Clearance Guidelines (November 1996)
Port of Savannah Industry Guidelines for Minimum Underkeel Clearances (Revised February 2009)
EM 1110-2-1100, *Coastal Engineering Manual*, Part V, Coastal Project Planning and Design
Savannah Harbor Deepening Georgia and South Carolina Design Memorandum (October 1992)
ERDC/CHL CHETN-I-72, *Ship Squat Predictions for Ship/Tow Simulator* (August 2006)
33 CFR 157.455, *Minimum Underkeel Clearance*
EM 1110-2-1613, *Hydraulic Design of Deep-Draft Navigation Projects* (May 2006)

Figure 6.2.2-1: Channel Depth Allowances (Definitions Outlined Following Figure)

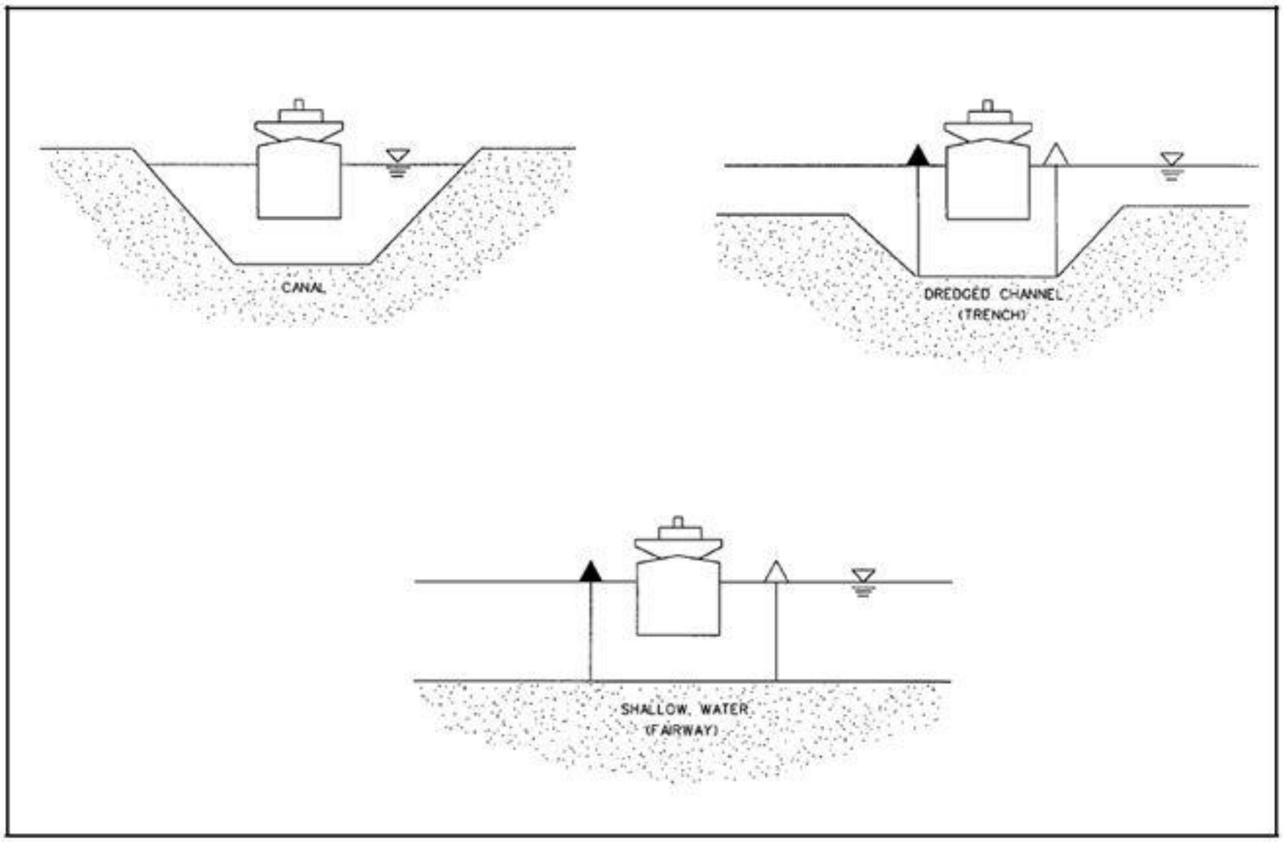


Effect of Fresh Water – Savannah Harbor is dominated by salt water; therefore, the effect of fresh water is not significant. Savannah Harbor Pilots Association determines their required underkeel clearance using formulas that are based on a freshwater interface, which yields conservative values for underkeel clearance.

Ship Motion from Waves – Vessel vertical motion in response to waves must be considered in design of channel depth at exposed locations. These vertical motions are composed of heave, pitch, and roll. They are functions of ship loading, draft, speed, and handling characteristics; channel depth and cross-section; and wave parameters including period, height, and relative wave direction. Entrance channel design depth is typically greater than interior harbor channel depth because of the need to accommodate wave-induced vertical vessel motions. The accepted practice along the Atlantic coast is to design an entrance channel deeper than an inner harbor channel. A vertical motion study was conducted for the Savannah Harbor Expansion Project which confirmed that the entrance channel depth should be 2 ft deeper that the inner harbor project depth.

Squat Underway – Squat is the reduction in underkeel clearance between a vessel at-rest and underway due to the increased flow of water past the moving body. This amount of clearance is primarily dependent on ship characteristics and channel configurations. The main ship parameters include ship draft, hull shape, and ship speed. On average, the Savannah Harbor Navigation Channel can be considered a trench channel. A trench channel is defined as a deepened passage with submerged overbanks on either side as opposed to a canal which has an enclosed cross section with exposed land adjacent to both sides of the channel or a fairway which is a passage with no lateral constraints. See **Figure 6.2.2-2**.

Figure 6.2.2-2: Navigation Channel Types



Safety Clearance or Net UKC – To protect vessel hull, propellers, and rudders from bottom irregularities and debris, a channel depth allowance for safety is included. A larger clearance (minimum 3 ft) is needed where the channel bottom is hard (such as rock, consolidated sand, or clay) compared to a lesser clearance (2 ft) where the channel bottom is soft. The safety clearance for the SHEP would be considered 2 ft because of the mostly soft channel bottom and absence of rock. Port designers have historically relied on deterministic approaches with large safety factors for channel design. Risk-based models are now recommended to define a useful lifetime with an acceptable level of risk of accidents or groundings. In a risk-based or probabilistic design such as used in this study, a net UKC does not have to be separately included since the clearance has been provided implicitly in the inclusion of predicted ship motions.

Vessel Speed – Squat and vertical ship motions are dependent upon ship speed. Ship speed, in turn, is dependent upon several factors including climate conditions, direction and strength of the tide, and the physical characteristics, including handling, of the vessel itself. Vessel speeds for container vessels can vary from as much as 14 knots in clear areas to as little as 4.5 knots in congested areas or while “meeting” another vessel. Design ship speeds are 8 kt for the Inner Bar Channel and 10 kt for the Outer Bar or Entrance Channel.

6.2.2.1 Squat & Underkeel Clearance

To determine the required underkeel clearance, values for squat were calculated for various vessel speeds for the project using the design vessel *Susan Maersk*. The Inner Channel was assumed to have a restricted trench type cross-section with a depth of 47 ft plus 3 ft tidal advantage, width of 500 ft (average), average trench height (measured from the bottom) of 20 ft, and side slopes of 3 (run/rise). The Entrance Channel was assumed to have an unrestricted or open cross-section with a depth of 49 ft plus 3 ft tidal advantage, and width of 600 ft (average). Seven different squat predictors including Ankudinov, Barrass, BNT (Beck, Newman, and Tuck), Eryuzlu, Huuska, Römisch, Yoshimura, were averaged and are listed below (See **Table 6.2.2.1-1**). The BNT squat predictor is used in ERDC’s CADET (Channel Analysis and Design Evaluation Tool).

Table 6.2.2.1-1: Vessel Speeds and Squat Along Inner Harbor and Entrance Channels

Vessel Speed (knots)	**Inner Harbor Squat(ft) Restricted****	**Outer Bar Squat(ft) Unrestricted****
6	0.9	0.6
8	1.6	1.1
10	2.5	1.7
12	3.7	2.5
14	5.4	3.5
16	8.0	4.7

***Unrestricted refers to a fairway or open channel whereas restricted refers to a trench channel.*

Using the values for squat provided by ERDC for the *Susan Maersk*, **Table 6.2.2.1-1** was used to develop underkeel clearance values for the ship speeds which are listed in **Table 6.2.2.1-2** for the inner harbor and outer entrance channels. The first four rows list the design ship draft, limiting ship speed, project depth, and depth with +3 ft tide for both channels. The next set of three lines shows the excess or additional clearance that is available in a deterministic sense based on the difference between the UKC and the required safety clearance. In this case, there is actually 0.5 ft additional clearance for both channels. The last set of lines shows the similar calculation for the net UKC in a probabilistic sense using the predicted ship squat and vertical motions from the CADET risk-based study. The resulting net UKC is sufficient from this standpoint as well.

Another consideration is the tidal window available for the transits. Both Inner and Outer Channels will require approximately 2 hr for the transit through the channel. Thus, a total of 4 hr is required for transit through both channels. The minimum required tide increase of +3 ft tide has durations of 6 hr for 360 days a year. Pilots will schedule inbound and outbound transits so that they sail with the tide so that the currents are working for them instead of counter flow. Faster ship speeds can be accommodated if the pilots have higher tide levels during the transit. The tradeoff is that the durations of these higher levels will be fewer days per year, but less transit time will be required. In summary, since the fully-loaded Susan Maersk will not be calling every day of every year, there will be more than sufficient numbers of days with 4+ hr durations to satisfy their requirements.

Table 6.2.2.1-2: Underkeel Clearance Along Inner Harbor and Entrance Channels

Description	Units	Inner Harbor Channel	Outer Entrance Channel
Design ship draft	ft	47.5	47.5
Limiting ship speed	kt	8.0	10.0
Project depth	ft	47.0	49.0
Depth with +3 ft tide	ft	50.0	52.0
Underkeel Clearance Calculations – Deterministic Method 1			
Gross Underkeel Clearance	ft	2.5	4.5
Required Safety Clearance	ft	2.0	4.0
Excess or additional UKC	ft	0.5	0.5
Underkeel Clearance Calculations – Probabilistic Method 2			
Ship squat	ft	1.6	1.7
Average vertical ship motions	ft	0.0	1.1
Sum of squat + ship motions	ft	1.6	2.8
Net Underkeel Clearance	ft	0.9	1.7

Thus, it can be concluded that vessels with a 47.5 ft draft can safely transit the inner harbor at 8 knots and the entrance channel at 10 knots. A minimum depth of 50 ft (+3 ft tide) in the Inner Harbor Channel and 52 ft (+3 ft tide) in the Entrance Channel is required for safe transits for the fully-loaded, 47.5-ft-draft *Susan Maersk.* Faster ship speeds are possible if higher tide levels are used, but the available durations are reduced such that a transit may not be possible every day of the year. This is an example of the factors that are used to determine underkeel clearance. More detailed analysis is contained in Section 6.3.4.4 which discusses the ERDC vertical motion study.

6.2.2.2 Existing Policy, Port of Savannah

Port of Savannah Minimum Underkeel Clearance Guidelines were discussed in October 1996 by a "Port Users Workgroup" composed of representatives of the Port of Savannah and Port of Brunswick and adopted November 27, 1996. Guidelines were established to proactively prevent maritime accidents and casualties as well as removing ambiguity and inconsistency in procedures necessary to reduce the likelihood of vessels over 1600 gross tons grounding during transit or while at an assigned berth within the Port of Savannah. The following guidelines were adopted for all conditions of tide and weather:

- 4 feet for transits in the navigation channel between the sea buoy, across the Entrance Channel, through Jones Island Range, Station -14+000, where the project depth of the channel decreases by 2 feet.

- 2 feet for transits between Jones Island Range and the point in the navigation channel which is adjacent to the facility of destination.

These are minimum guidelines and are not intended to be limiting for pilots, operators, or owners that choose to require a higher degree of safety for their operations to ensure a vessel's safe transit and mooring. These guidelines are to be reviewed bi-annually by the signatory body to address any necessary changes and ensure all parties are following guidelines as agreed upon. The most recent update confirming these guidelines was published in February 2009 in the *Port of Savannah Industry Guidelines for Minimum Underkeel Clearances*. Additionally, this guidance does not amend or supersede applicable underkeel clearance requirements for single bottom tank ships and barges detailed in 33 CFR 157.455.

An interview conducted with the Master Pilot revealed that current pilot regulations require a minimum underkeel clearance of 4 feet, and a pilot will usually wait to take advantage of the tide (tidal lift) if it appears 4 feet will not be available during the entire transit of a vessel. Past practice has shown that pilots move vessels when they have 4 feet of underkeel clearance. This requires that some vessels wait at the docks or the sea buoy until adequate depth can be gained from tidal fluctuation. Since Savannah Harbor experiences an average tidal range of 7 to 8 feet, it is a common practice for the pilots to use the tides to advantage in loading and transit of deep draft ships.

6.2.3 Pipeline Impacts

To avoid damaging any existing utilities which cross the navigation channel, it was necessary to determine a) the location, both horizontal and vertical, of the utilities and b) the amount of overburden required to provide an adequate factor of safety for the pipelines after deepening of the channel. Consideration was also given to future maintenance of the channel in the pipeline areas.

Pipeline industry standards for clearance guidance stated that 4 ft of cover was required in areas of new pipeline placement (Pipeline and Hazardous Materials Safety Administration, DOT Rules and Regulations, section 192.327 Cover, paragraph (e)…"**all pipe installed in a navigable river, stream, or harbor must be installed with a minimum cover of 48 inches**"). The Engineering Research and Design Center (ERDC) Dredging Operations Technical Support (DOTS) program was also contacted to request the US Army Corps of Engineers (USACE) policy regarding dredging over pipelines. ERDC confirmed that no standard exists. Documentation of the DOTS assistance titled, *Dredge Vertical Construction Accuracy*, is included in the Engineering Investigations Supplemental Materials.

6.2.3.1 Pipeline Locations

All available District permits applicable to river crossings in the project area were collected. This effort helped to determine which areas had potential conflict with the harbor deepening based on as-built drawings on file with regulatory permits. In addition, a response was solicited from industries and property owners adjacent to the navigation channel by public notice in April 2007, informing them of the USACE's intention to study the proposed deepening of the harbor and inquiring to gather information on any existing or proposed utility crossings. The results of the investigation are as follows:

- It was determined that Georgia Power crossings are all overhead lines and the former "in-channel" crossings discovered by research of district permit applications no longer exist.

- It was determined that all underground utility crossings to Hutchinson Island at approximate Stations 68+000, 72+000, and 78+000 would not conflict with the proposed deepened channel based on drawings received from Hussey, Gay, Bell & DeYoung. The depths of these lines were 31 ft, 33 ft, and 23 ft respectively below the proposed channel bottom alleviating any concern for their safety.

- Areas of potential conflict included the Southern Natural Gas (SNG) pipelines at Harbor Station 51+500 (4.5 ft clearance based on 1977 as-built drawings) and the International Paper (IP) pipeline at Harbor Station 89+250 (8 ft clearance). (Note: the amount of clearance is based on the difference between the elevation of the pipeline and the maximum dredging depth of 15 ft below the existing 42 ft project depth which includes 6 ft deepening, advance maintenance (varies from 2 ft to 4 ft based on reach of channel), 2 ft allowable over-depth, and 3 ft dredging disturbance.

The USACE met with local officials from SNG to present its concerns. In February 2008, the USACE received results from SNG of the survey of their pipeline crossing done by T. Baker Smith in October 2007. After correspondence with T. Baker Smith in which accurate proposed channel location coordinates were provided and the difference in vertical datum was resolved, it was determined that there is between 11 ft to 12.5 ft of clearance between the maximum proposed disturbed channel depth and the top of the easternmost pipeline. Survey results from the westernmost line were received in May 2009. The West line appears to be 3 feet above the East line at the closest point to the proposed channel excavation at -68 ft MLLW. SNG officials believe this will still be acceptable as long as they are able to coordinate closely with USACE and the dredging contractor.

The USACE also contacted International Paper's (IP) point of contact to advise them of plans for deepening the harbor. IP provided an electronic file showing a cross section of their pipeline. The USACE provided the design template for the deepened channel including disturbance depth on the cross section. All data received to date, indicates that the IP pipeline will retain 8 ft of cover in addition to 3 ft allowed for dredging disturbance on the Northern side of the channel after Deepening.

6.2.3.2 Dredging Methodologies in Areas of Pipeline Crossing

The Savannah District solicited input from the Mobile District as well as the dredging industry input to determine their level of accuracy in dredging over pipelines. Input from Industry and USACE Districts is summarized in the following paragraphs.

Industry Input – Conversation with Norfolk Dredging revealed that the company routinely dredges within 3 – 4 ft horizontal distance away from pipelines in Philadelphia District (Schuylkill River – 400 ft wide cut). The pipelines locations are well documented and the dredging firm takes precautions, (i.e., swing slow, cut off cutter-head over line, etc.). The dredge operator can tell when he is dredging over previously dredged material. This issue was also discussed with Great Lakes Dredge and Dock, Inc. They stated from experience, the dredge can remove material within 5 ft vertical distance of a pipeline based on the distance between the over-depth prism and top of pipe. They also stated that if the dredge operator was not absolutely sure of the pipeline location he could also use the mechanical dredge to come within 25 ft of either side of the pipeline to remove material and recommended using suction without the cutterhead turning if material is soft over the pipeline

New Orleans District Input –New Orleans District has a great deal of experience in dredging over pipelines having administered over a hundred contracts dealing with this issue in Mobile, Galveston, New Orleans, Vicksburg, and Philadelphia. They provided specifications listing necessary precautions involving dredging over pipelines and stressed the need to insure correct calibration of cutterhead positioning. They did express concern that we have to deal with greater tidal exchange than what they normally deal with on the Gulf coast. This fact complicates tidal height calculations in Savannah Harbor. New Orleans District also stated that regarding dredging over pipelines, dredging is usually limited to 10 ft below project grade or 8 ft below required depth with 2 ft for advance maintenance and allowing 1 ft for over-depth. This equates to a 7 ft dredging tolerance over pipelines. They agreed with the tolerance of 8 ft being used for Savannah which includes 3 ft dredging disturbance as part of the tolerance.

Savannah District Input – Engineering Division initially recommended dredging up to but not over the SNG pipeline and allowing river currents to remove remaining new work dredged materials. The same practice would apply to O&M dredging operations. Velocities were examined in vicinity of the SNG pipelines to determine if this approach was feasible as this is the current practice adjacent to Old Fort Jackson. Even though current velocities in the Old Fort Jackson reach are influenced by ebb tides coming out of the Back River, the current velocities near the SNG facility are similar.

Detailed investigations determined the velocities to be comparable in front of Old Fort Jackson (maximum bottom 1.4 – 1.8 fps, maximum surface 3.7 – 4.2 fps) and over the SNG pipeline (maximum bottom 1.4 – 1.8 fps, maximum surface 3.7 – 4.5 fps). Maximum velocities over the International Paper pipeline were somewhat smaller (maximum bottom 1.6 fps, maximum surface 3.4 fps). These measurements suggested that it may be possible to provide adequate depth over the SNG pipeline by dredging on either side of the pipeline and allowing the currents remove maintenance material similar to how the area in front of Old Fort Jackson is maintained.

However, additional research revealed that although the reach in front if Old Fort Jackson is not currently dredged as part of the maintenance contract, it was initially deepened to 44 ft by a cutterhead dredge. Dredge scars in the Miocene layer of clay are still visible in the Miocene material in photos taken by side scan sonar. Since these cuts are still evident and have not been "smoothed out" by currents, it is unwise to assume that currents are swift enough to remove deepening, new work materials. The clays of the Miocene layer are too stiff to be removed by anything other than mechanical equipment. The clays reside at elevation -45 ft to -47 ft MLLW in front of Old Fort Jackson and from -43 ft to -46 ft MLLW in the SNG pipeline area, and around -44 ft MLLW at the International Paper pipeline.

6.2.3.3 Mitigation During Construction

Based on the proximity to the pipelines, the USACE would specify the following constraints during construction dredging:

- Signs and ranges would indicate "no dredge" areas in the field and would be indicated on the drawings as well during construction and operations and maintenance (O&M) dredging.

- SNG would be contacted in advance of dredging to shut off flow of gas into pipeline.

- Cutterhead rotation would be suspended within 25 ft of the pipelines and an attempt would be made to remove material using only suction from the dredge pump. If material is too difficult to remove over pipeline, the contractor may be required to incorporate clamshell operations to more safely remove material.

6.2.3.4 Conclusion/Recommendation

The process of dredging in the SNG and IP pipeline areas will follow the accepted practice of approaching utility crossings from both upstream and downstream without "spudding" or dredging directly over the pipelines. The Savannah District will specify strict adherence to these procedures and will specify that its contractors stay within prescribed limits as determined by as-built drawings provided by utility owners.

Engineering Division recommends adopting the current dredging policy in areas of concern by:

- Not "spudding" over the pipelines but allowing dredging over and within the vertical limits established by construction (5 ft for gas pipeline).

- Limiting the amount of advance maintenance in the affected areas to provide necessary clearance if the pipeline location falls within 5 ft of required clearance and use precautions listed above.

- If new work material is unable to be removed by suction without turning the cutterhead, the contractor may have to employ mechanical dredge to initially remove material.

Letters detailing this information were sent to International Paper and Southern Natural Gas on May 5, 2008 and are included in the Engineering Investigations Supplemental Materials.

6.2.4 Air Draft Analysis

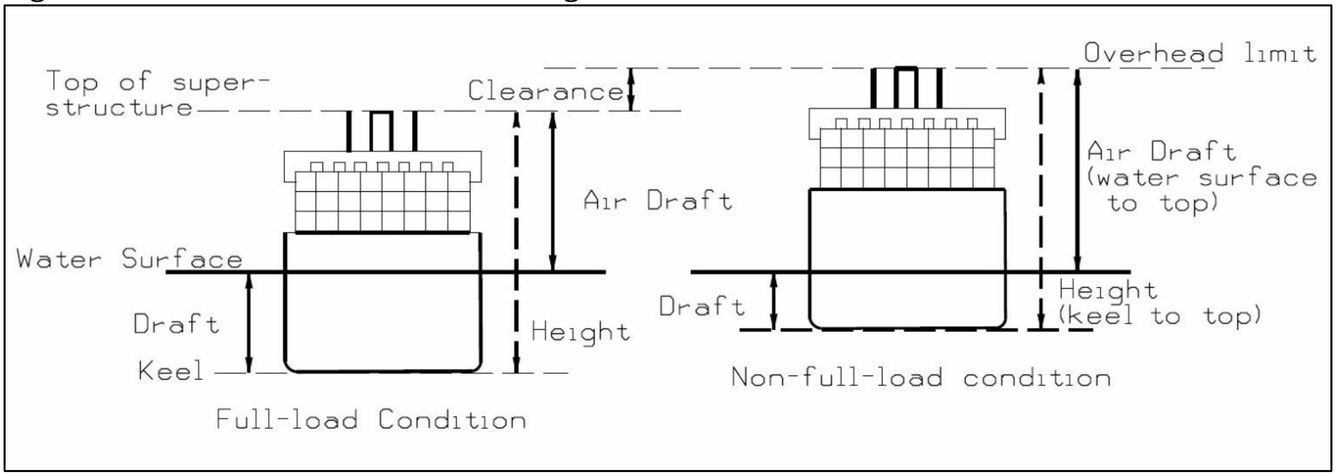
6.2.4.1 Purpose for Analysis

The purpose of the air draft analysis is to confirm that air draft of the design fleet mix for the SHEP does not violate the air draft listed for the air draft at Mean Higher High Water (MHHW) associated with the Talmadge (Savannah River) bridge.

6.2.4.2 Definitions

The term "air draft" associated with a vessel is commonly accepted as the distance between the elevation of the water line in which a vessel rests in the water and the top of the superstructure of the vessel. This distance varies based on the draft of the vessel and whether the height of the vessel includes antenna or other superstructure height. Draft can depend on how heavy a vessel is loaded, salinity, water temperature, etc. Many large vessels have antenna or superstructure that can be lowered to decrease air draft. **Figure 6.2.4.2-1** shows the relationship between height (from Keel to Top) versus height, or air draft (from surface of the sea to Top) as well as how air draft varies depending on loading condition.

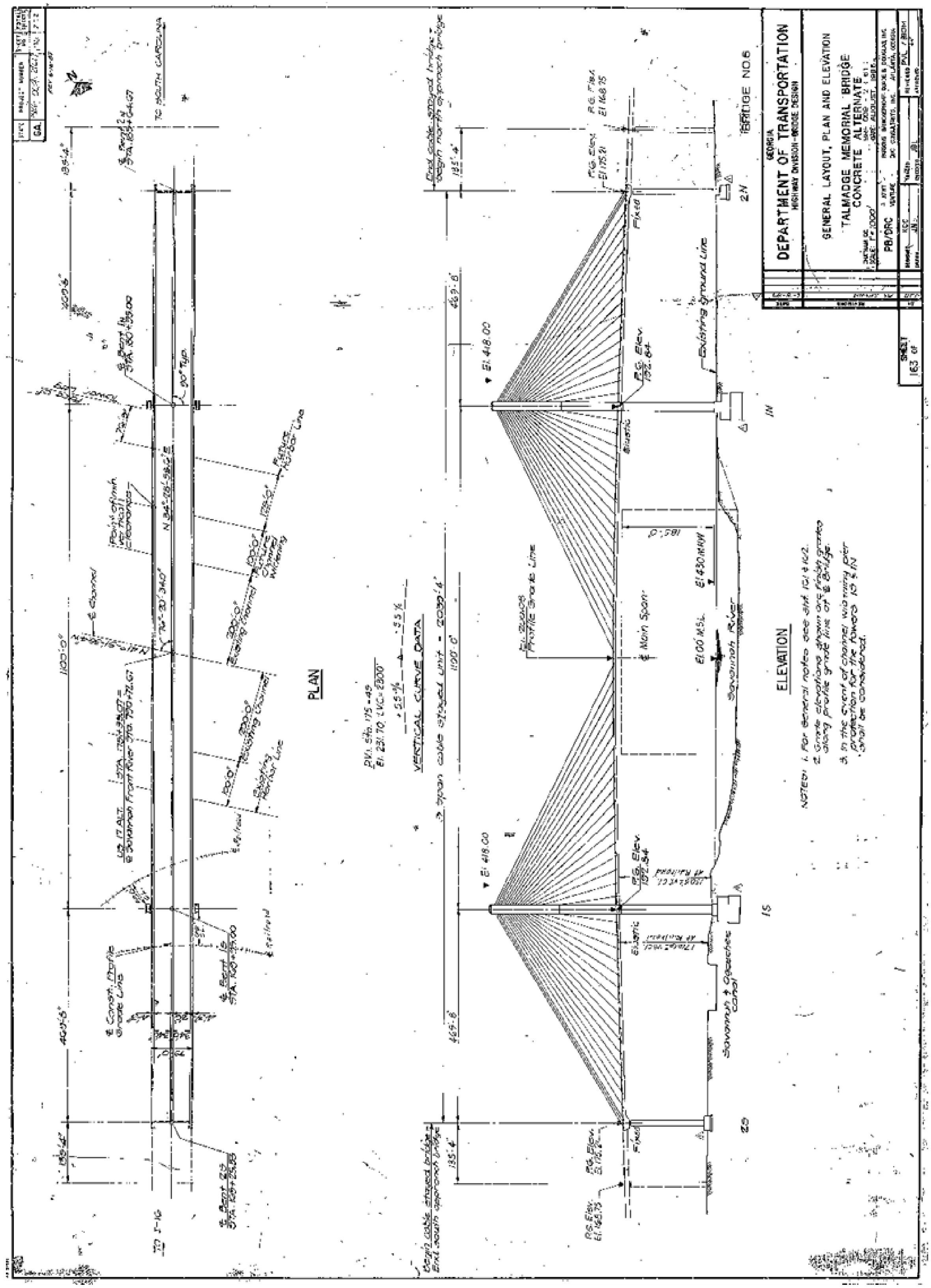
Figure 6.2.4.2-1: Air Draft Definition Diagram



6.2.4.3 Bridge Height Restriction

The Talmadge (Savannah River) bridge has an air draft height of 185 ft above MHHW, as per design drawings provided by Georgia DOT. See **Figure 6.2.4.3-1**. This height is based on the lower edges of the span above the navigation channel. Height above MHHW actually ranges from 192 ft to 200 ft in the middle of the span. The 185 ft distance is used by the Savannah Harbor pilots as the official (conservative) air draft of the bridge.

Figure 6.2.4.3-1: Talmadge Bridge Design Drawing



6.2.4.4 Tolerance

The Savannah office for the USCG deferred to the Savannah River Harbor Pilots Association for restrictions on air draft. The Savannah River Harbor Pilots Association stated that there was no official policy regarding the air draft of vessels coming into the harbor. From information gained, a vessels air draft is provided to the pilot and the Coast Guard before the vessel enters the channel. One carrier interviewed stated they use 3 ft as minimum allowance.

6.2.4.5 Design Vessel Specifications

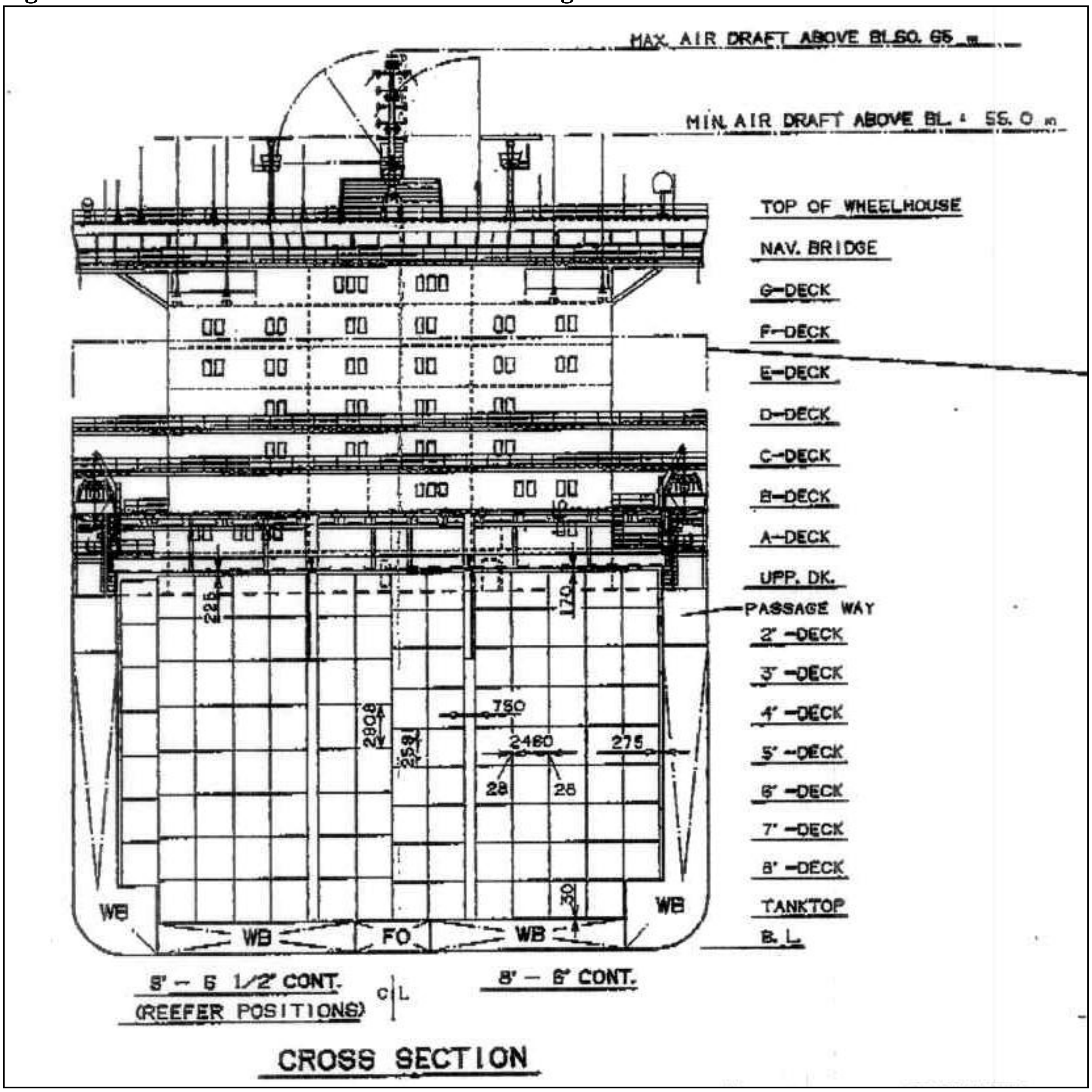
Research consultation with Maersk Lines provided information for the design vessel *Susan Maersk* regarding air draft. Apparently, the term "air draft" is also used to refer to the height of a vessel from the BL (which can refer to base line or bilge line) to the top of the superstructure. The height for the *Susan Maersk* is defined as 55 meters (180.4 ft) with antenna down, minimum height. The maximum height is 60.65 meters (199.0 ft), antenna up. The Maersk Lines representative stated that the antenna is not removed to reach the minimum height but tilted back. This is part of a routine check list but is only required in a few ports to meet air draft requirement. The draft of the *Susan Maersk* ranges from the design draft of 47.6 ft to a minimum of 29 ft, thus, the air draft would range as follows:

Max height (Antenna up)		Draft		Air Draft
199.0 ft	-	47.6 ft (design)	=	151.4 ft
199.0 ft	-	29 ft (minimum)	=	170.0 ft

Max height (Antenna back)		Draft		Air Draft
180.4 ft	-	47.6 ft (design)	=	132.8 ft
180.4 ft	-	29 ft (minimum)	=	151.4 ft

Figure 6.2.4.5-1 shows a schematic of the *Sovereign Maersk* (a "sister" vessel of *the Susan Maersk*) provided by Maersk Lines which substantiates their definition of height being from keel to top of structure.

Figure 6.2.4.5-1: Maersk Air Draft Definition Diagram



6.2.4.6 Design Fleet Mix

Discussions between the District and the USACE Institute for Water Resources (IWR) regarding the design fleet mix are outlined in the following paragraph.

IWR discussed proprietary information with the District listing vessels that were considered to make up the design fleet. The “workhorse” for the projected fleet is expected to be an 8200 (+/- 400) TEU vessel. The upper height limit for these vessels was listed at 62 m (meters) or 157 ft for the design draft of 47.6 ft. Even if the superstructure was raised 10 ft to accommodate another tier of containers and the vessel was light loaded by an additional 10 ft (any more would not be economically considered according to IWR), the air draft would only increase to 177 ft, which is still within an acceptable tolerance considered by the Savannah River Harbor pilots. IWR was comfortable in extending this range up to a 9000 TEU vessel.

6.2.4.7 Conclusion

Neither the design vessel nor the design fleet mix will violate the air draft restriction presented by the Talmadge (Savannah River) Bridge.

6.2.5 Berth Design and Dredging

To accommodate the SHEP design vessel, GPA requested that an offset of 160 ft from Container Berths (CB) 2-9 be incorporated into the channel design. To provide the extra berth width, the existing channel either had to be realigned or a portion of the channel had to be abandoned. Realigning the channel in front of the berths would have required considerable real estate acquisition on the north side of the channel. During ship simulation, pilots were able to navigate this reach of channel using a reduced width of channel to accommodate the design berth width but will be restricted to one-way traffic in this reach. Results of the ship simulation for this channel realignment are detailed in the March 31, 2009 memorandum from ERDC titled *Savannah Harbor Simulations Study 2009*, which is included in the Engineering Investigations Supplemental Materials.

GPA has already deepened CB 2-3 and 8-9 to safely dock deeper draft vessels that have come into port using the tidal advantage. CB 4-7 will be deepened at a future time. There are no current plans to deepen CB 1. GPA will be responsible for O&M dredging of the widened berth area. **Table 6.2.5-1** shows the quantities of material that were/are required to provide necessary depths in front of the berths. Plates 01 and 02 in Attachment 1 show the berth design requested by GPA.

Table 6.2.5-1: GPA Container Berth Material Quantities

Berth	Location Relative to Channel Centerline Station	42 ft Depth (cy)	43 ft Depth (cy)	44 ft Depth (cy)	45 ft Depth (cy)
CB-2	95+364 to 96+359	1,049	2,006	3,876	7,104
CB-3	96+359 to 97+687	2,046	3,585	6,137	9,537
CB-4	97+687 to 98+362	947	1,520	2,744	5,207
CB-5	98+362 to 99+352	3,985	6,059	8,664	11,652
CB-6	99+352 to 100+052	6,472	11,914	18,038	24,377
CB-7	100+052 to 101+153	19,011	26,621	35,199	44,814
CB-8	101+153 to 102+326	1,216	2,468	5,385	11,128
CB-9	102+326 to 103+192	5,313	9,268	14,288	20,526
Berth	**46 ft Depth (cy)**	**47 ft Depth (cy)**	**48 ft Depth (cy)**	**49 ft Depth (cy)**	**50 ft Depth (cy)**
CB-2	10,891	14,770	18,696*	22,652	26,610
CB-3	13,852	18,775	23,940*	29,151	34,634
CB-4	8,765	12,824	16,889	20,953	25,017
CB-5	14,801	18,022	21,244	24,466	27,689
CB-6	30,829	37,488	44,234	51,001	57,771
CB-7	54,990	65,350	75,751	86,164	96,582
CB-8	18,486	26,074	33,680*	41,287	48,894
CB-9	27,097	33,672	40,464*	47,168	53,873

* Berths already deepened.

6.3 PROPOSED CHANNEL DESIGN

6.3.1 Design Template

In order to reduce the amount of upland required to expand the channel, it was determined that the inner harbor channel would be deepened on its existing 1V/3H (1 vertical/3 horizontal) side slope. This design decision was made by the members of the project delivery team for the SHEP and was included in the Savannah Harbor Deepening Feasibility Report (Section 203) and Tier I EIS 1998 report and received no negative comments. Prior to adoption, this design was elevated through the vertical team including the South Atlantic Division office (SAD), USACE Headquarters and ASA (CW). See **Figure 6.3.1-1** and **6.3.1-2** for typical sections for the SHEP inner harbor and entrance channel. **Tables 6.3.1-1** and **6.3.1-2** outline the SHEP alternative project widths and depths.

Previous deepening projects had deepened the channel using the existing widths, requiring additional real estate. By deepening on the existing side slope, the new bottom width would be narrower than the existing width, but no additional real estate would be required except in the areas requiring bend wideners in the channel to accommodate the longer design vessel. The bottom width in the areas not requiring bend wideners is dependent on the design depth. In addition to the minimization of real estate requirements, this design minimizes the amount of dredging required which is a significant cost savings and also minimizes impacts to cultural resources and wetlands along the channel banks. This channel design was used in the ship simulation study and was determined adequate for one-way traffic. Details of the Ship Simulation Study are included in the report titled *Navigation Study for Savannah Harbor Channel Improvements*, Engineering Investigations Supplemental Materials and discussed in Section 6.3.2.

The study analysis resulting in bend wideners, meeting areas, and the channel extension required for inclusion in the project are discussed in detail in the following sections and supporting documents in the supplemental materials. **Figure 6.3.1-3** shows the locations of all of the SHEP navigation features proposed for the project.

Figure 6.3.1-1: Typical Channel Cross Section (Inner Harbor)

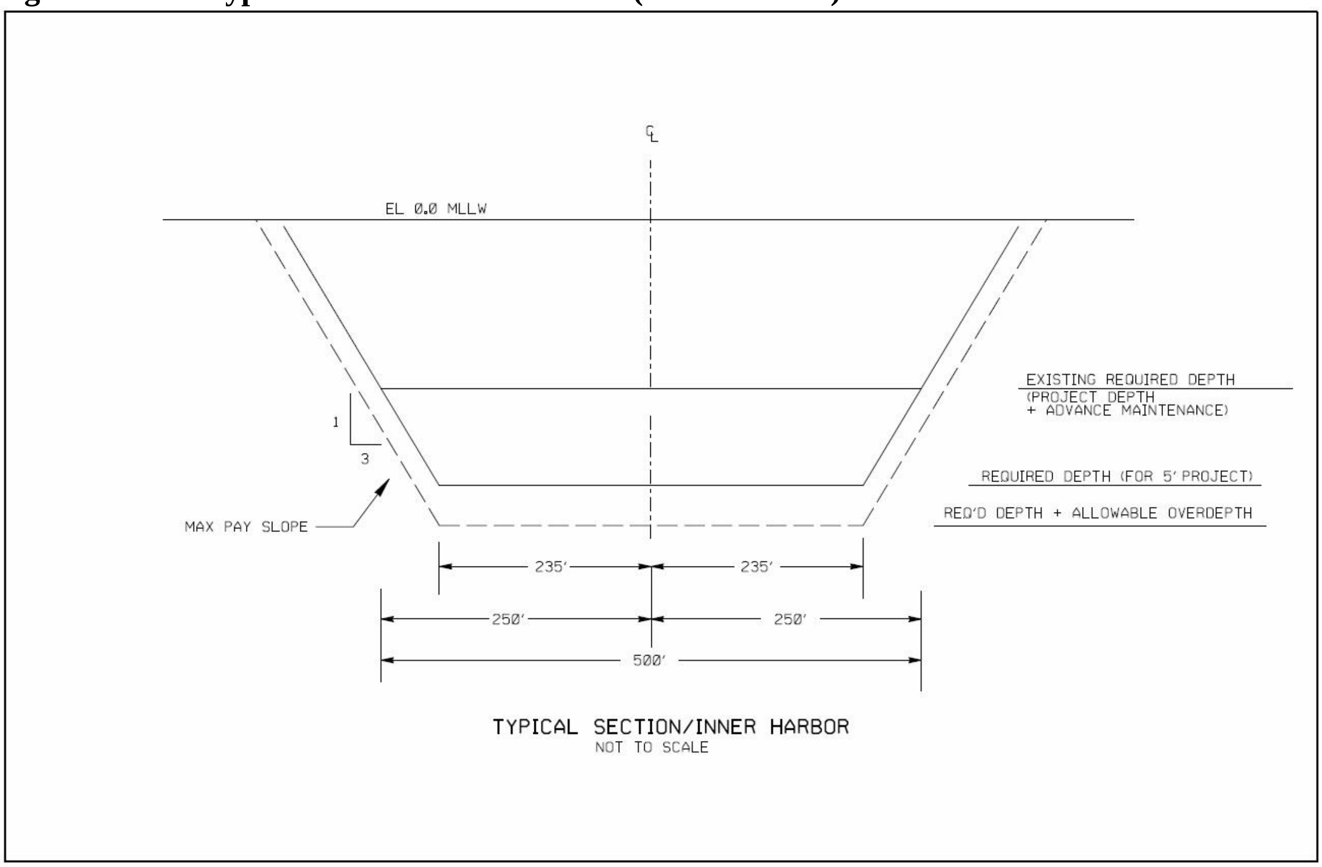


Figure 6.3.1-2: Typical Channel Cross Section (Entrance Channel)

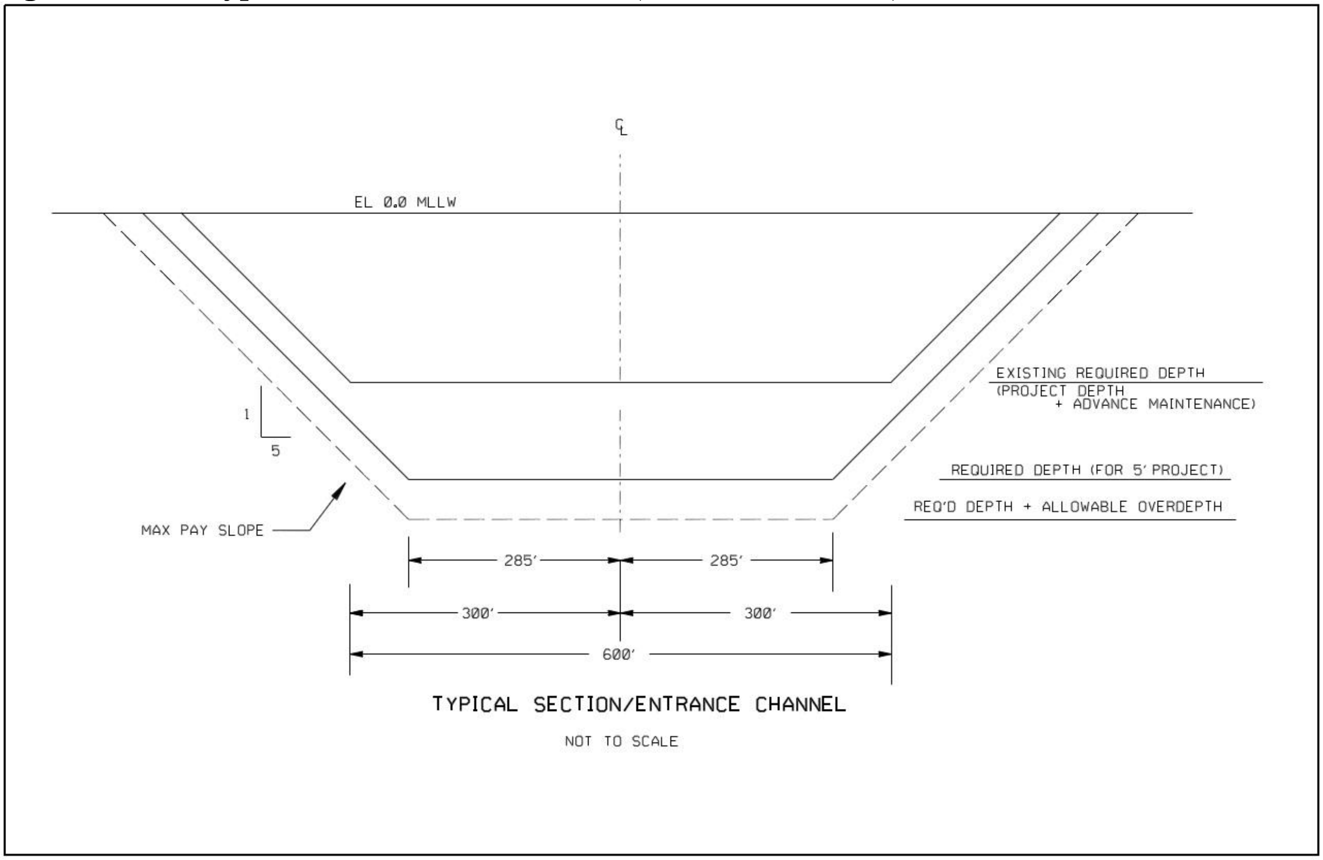


Figure 6.3.1-3: Location of SHEP Navigation Features (Bend Wideners, Meeting Areas and Channel Extension)

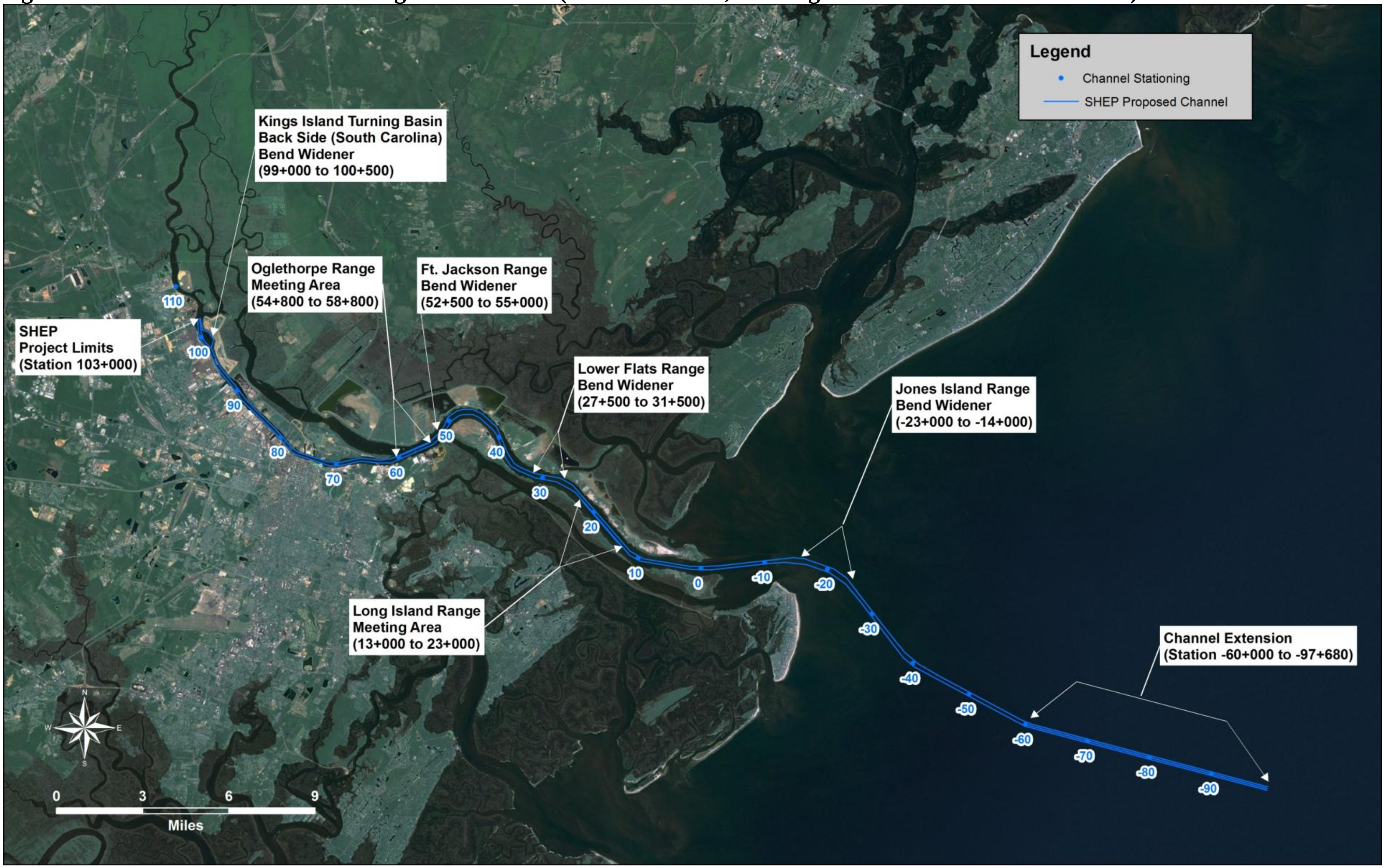


Table 6.3.1-1: Currently Authorized Channel and SHEP Channel Alternative Widths (ft)

Range Name	Lower Stationing	Upper Stationing	Currently Authorized Project	46 ft Project	47 ft Project	48 ft Project
S8	-98+600[A]	-60+000	*not applicable*	576	570	564
Tybee	-60+000[A]	-40+522	600	576	570	564
0A	-40+522	-38+186	800	776	770	764
Bloody Point	-38+186	-23+475	600	576	570	564
1A	-23+475	-20+832	800	858[B]	858[B]	858[B]
Jones Island	-20+832	-16+142	700	758 [B]	758 [B]	758 [B]
2A	-16+142	-13+771	800	864 [B]	861 [B]	858 [B]
Tybee Knoll Cut	-13+771	-1+380	500	476	470	464
4	-1+380	1+552	Varies	Varies	Varies	Varies
New Channel	1+552	9+526	500	476	470	464
6	9+526	11+385	600	600	600	600
Long Island Crossing	11+385	24+920	500	476	470	464
Long Isl. Meeting Area[1]	*13+000*	*23+000*	*not applicable*	*576*	*570*	*564*
8	24+920	27+317	800	776	770	764
Lower Flats	27+317	31+037	600	664 [B]	661 [B]	658 [B]
10 through 12	31+037	36+948	600 to 700	600 to 700	600 to 700	600 to 700
Upper Flats	36+948	40+437	550	538	535	532
14	40+437	41+693	500 to 700	488 to 688	485 to 685	482 to 682
Bight Channel	41+693	49+489	700	700	700	700
Ft. Jackson Channel	49+489	53+127	Varies	Varies [C]	Varies [C]	Varies [C]
21	53+127	54+481	600	664 [B]	661 [B]	658 [B]
Oglethorpe	54+481	61+405	500	488	485	482
Oglethorpe Meeting Area[2]	*54+800*	*60+680*	*not applicable*	*588*	*585*	*582*
23	61+405	63+277	Varies	Varies	Varies	Varies
24 through 25	63+277	69+734	500	470	467	464
26	69+734	71+128	600	588	585	582
City Front Channel	71+128	76+537	500	476	470	464
28	76+537	77+283	550	538	535	532
Marsh Island Channel	77+283	87+642	500	476	470	464
32	87+642	90+701	Varies	Varies	Varies	Varies
33	90+701	93+933	500	476	470	464
34	93+933	95+378	500	449	446	443
35	95+378	97+543	500	437	434	431
Kings Isl. Turning Basin	97+543	103+000	Varies	Varies [B]	Varies [B]	Varies [B]

Notes:
[1] Includes 2,000-foot transition, [2] Includes 1900-foot transition
[A] Existing project starts at -60+000, the 47 ft project would require 37,680 linear ft of channel extension to -97+680
[B] Width expansion on north side of channel only
[C] Width expansion on south side of channel only

Table 6.3.1-2: Existing and Alternative Project Channel Depths (ft MLLW)

	Currently Authorized Navigation Channel			**SHEP Alternative Project Depths***				
Stations	**Authorized Project Depth**	**Advance Maintenance**	**Maintenance Dredging Depth*****	**44 ft Project Depth**	**45 ft Project Depth**	**46 ft Project Depth**	**47 ft Project Depth**	**48 ft Project Depth**
-98+600 to -60+000**	n/a	n/a	n/a	48	49	50	51	52
-60+000 to -14+000	44	0	46	48	49	50	51	52
-14+000 to 0+000	42	2	46	48	49	50	51	52
0 to 24+000	42	2	46	48	49	50	51	52
24+000 to 35+000	42	4	48	50	51	52	53	54
35+000 to 37+000	42	6	50	52	53	54	55	56
37+000 to 50+500	42	4	48	50	51	52	53	54
50+500 to 52+750	42	4	48	50	51	52	53	54
52+750 to 54+000	42	4	48	50	51	52	53	54
54+000 to 60+250	42	4	48	50	51	52	53	54
60+250 to 66+750	42	4	48	50	51	52	53	54
66+750 to 70+000	42	4	48	50	51	52	53	54
70+000 to 102+000	42	2	46	48	49	50	51	52
102+000 to 103+000	42	0	44	46	47	48	49	50
Kings Island Turning Basin	42	8	52	54	55	56	57	58

**Alternative Project Depths include Currently Authorized Advance Maintenance and Over-depth.*
*** Stations -98+600 to -60+000 are not currently part of the Federal Project, the SHEP Alternative Project Depths require that the navigation channel be extended across the ocean bar.*
Station -98+600 is the extended channel station associated with the 48 ft project depth. The 47 ft project depth extends to station -97+680.
****Maintenance Dredging includes 2 ft of Over-depth.*

6.3.2 Ship Simulation

A navigation study utilizing real-time ship simulation to evaluate the proposed improvements to Savannah Harbor was conducted by ERDC for the SHEP. Details are documented in the reports titled: 1) *Navigation Study for Savannah Harbor Channel Improvements* 2) *Savannah Harbor Simulations Study 2009* 3) *Savannah Harbor Entrance Channel Simulations 2010 Report* 4) V*ertical Ship Motion Study for Savannah, GA Entrance Channel*. These documents are included in the Engineering Investigations Supplemental Materials. The results of the first two studies are discussed in detail in the following sections. The results of the third and fourth studies are discussed in the next section.

Two design ships were used for the study: one to represent existing vessel traffic and the other to represent future traffic. The design ship for the currently authorized channel was the SL Performance, a Panamax containership loaded to 33 ft. The SL Performance is 950 ft long with a beam of 106 ft. The design ship for the proposed channel improvements was the *Susan Maersk*. The *Susan Maersk* is a post-Panamax S-class containership with a length of 1,140 ft and a beam of 140 ft. For ship simulation, the *Susan Maersk* is assumed to be loaded to a draft depth of 47.5 ft.

A reconnaissance trip to Savannah Harbor was made on December 3 through 5, 2001, to observe navigation conditions in the Savannah River. The project site was photographed to create the simulation visual scene using digital video and still cameras. Information concerning problem areas and pilot practices was obtained during the inbound transit of the Ludwigshafen Express on December 3 and the Hanjin Tokyo on December 5. The vessels were tracked using a handheld global positioning unit.

The Ludwigshafen Express is a Panamax containership, 965 ft in length overall with a beam of 105 ft. The ship's draft was 35 ft. The transit was conducted at high water, estimated to be +9 ft MLLW. Typically, high water is +8 ft MLLW. The ship was turned in the Kings Island Turning Basin prior to docking at the Georgia Ports Authority (GPA) Port Wentworth Terminal. The Hanjin Tokyo, a Panamax ship, which had a draft of 34 ft. The transit commenced approximately 2 hours after low tide and was also terminated at the GPA Port Wentworth Terminal.

The following insights to the navigation conditions on the Savannah River were offered by the pilots during the transits:

- Wind-driven currents occur in the Atlantic, typically with a wind from the south/southwest and combined with flood tide. The crosscurrents occur in the two easternmost channel segments of the entrance channel.

- The currents are usually aligned with the channel in the protected portions of the river.

- Widening the Long Island Crossing Range by 100 ft on the south would provide additional room for meeting.

- Ships are restricted to minimum underkeel clearance of 4 ft in the entrance and 2 ft in the inner harbor, according to US Coast Guard guidelines.

- A new liquefied natural gas (LNG) facility and turning basin were under construction along the south side of Upper Flats Range. The turning basin is not part of the channel improvements being evaluated by this study. Ships will not meet near the facility when an LNG tanker is docked. It should be noted that the construction of the LNG facility and turning basin was completed after the ship simulator study was initiated.

- Two-way traffic occurs over most of the project at the pilots' discretion. Pilots try to time their meetings for the straight reaches and avoid meeting at City Front or the Bight Channel.

- Ships are affected by currents in an area inside the entrance channel jetties and another near Fort Jackson due to currents ebbing from Back River.

- Ebb-tidal currents work against inbound ships when making the turn from the Bight Channel to Fort Jackson Channel.

6.3.2.1 Proposed Channel Improvements

The ship simulator model investigation documented in the report titled *Navigation Study for Savannah Harbor Channel Improvements* was conducted by ERDC in Vicksburg, MS. The simulator experiments were performed during September and October 2003 by personnel of the Coastal and Hydraulics Laboratory (CHL) and river pilots from the Savannah Pilots Association. During the course of the model study, representatives of the Savannah District and other navigation interests visited ERDC to observe the simulator and discuss tests results. Both the existing and the 48 ft project channels were modeled. Two-way traffic on both maximum ebb and flood tide conditions were tested. The currents were calculated using the TABS-MD model. The ship models used in the simulator study were the *Susan Maersk* loaded to a 47.5-foot draft and the SL Performance loaded to a draft of 33 ft. The simulators were coupled together for two-way traffic conditions and tugs and bow thrusters were available to the pilots during the simulation runs. Various combinations of tides and vessels were tested for meeting situations.

Savannah District provided ERDC with a proposed channel design based on deepening the existing navigation channel on the existing side slopes and recommended bend wideners in areas that required additional width as determined by design standards in EM 1110-2-1613, *Hydraulic Design Guidance for Deep-Draft Navigation Projects*. Alternatives that were developed from this design are as follows:

<u>Plan 1</u>
- Deepening the navigation channel on existing side slopes
- Bend wideners in turns
- Shifting Ft Jackson Range eastward to avoid impact to pipeline
- Bend widener in transition area between Ft Jackson and Oglethorpe Range

<u>Plan 2</u> – Same as Plan 1 with the following exceptions:
- Long Island Range – Additional channel width for meeting area on south side of range
- Ft Jackson Range – ERDC recommended abandoning triangular portion on north side of channel

<u>Plan 3</u>
- Deepening on existing channel toes (maintaining existing width)

ERDC modeled these three plans in ship simulation and developed a recommended plan based on pilot navigation of the alternatives. A detailed description of the plans and ERDC's recommended revisions for an optimum channel design are discussed below.

The changes to the horizontal channel limits are presented, beginning with the entrance channel and proceeding inland. Unless otherwise indicated, these changes are part of Plan 1. The existing Tybee Range is 600 ft wide. Deepening to 48 ft on the existing side slopes will narrow the proposed channel to approximately 550 ft. The west side of the turn onto Bloody Point Range is deepened on the existing side slope resulting in a channel width reduction of 25 ft on the west side. To facilitate navigation, the Savannah District has proposed a 75 ft widener on the east side of the turn. The Bloody Point Range is presently 600 ft wide. The 25-ft reduction on each side caused by deepening on the existing side slopes will narrow the channel to approximately 550 ft. The Jones Island Range, which is the turn from Bloody Point Range to Tybee Knoll Cut Range, was reduced by 25 ft on the north side, but the Savannah District proposal widened the channel by 75 ft on the south. Therefore, the width of Jones Island Range was increased from 700 ft to 750 ft. Tybee Knoll Cut Range is presently 500 ft wide. Deepening to 48 ft on the existing side slopes reduced that width to 450 ft.

The New Channel Range is 500 ft wide. The proposed 48-ft channel will narrow New Channel Range to 450 ft. The west side of the turn onto Long Island Crossing Range is deepened on the existing side slope and thus the channel width is reduced by about 25 ft on the west side. However, the District has proposed a 75 ft widener on the east side of the turn. The Long Island Crossing Range is presently 500 ft wide. There are two proposed widths for Long Island Crossing Range. Plan 1, deepened upon the existing side slopes, reduces the channel width to 450 ft. Plan 2, widens the western side of the Long Island Crossing Range deepened channel by 100 ft, thus increasing the channel width to 550 ft. Plans 1 and 2 were identical for the reaches immediately east and west of Long Island Crossing Range. The existing Lower Flats Range is 600 ft wide. The proposed Plan 1 widened the north side of Lower Flats Range by 75 ft. The south side of Lower Flats Range was narrowed by 25 ft due to deepening on the existing side slope. Therefore, the Plan 1 Lower Flats Range is 650 ft wide. The District has provided an additional 150 ft on the west side of the turn between Lower Flats Range and Upper Flats Range. The east side of the turn was narrowed by 25 ft due to deepening on the existing side slope. The Upper Flats Range is 550 ft wide. The proposed Plan 1 Upper Flats Range was widened by 75 ft on the west side. The east side of the proposed Plan 1 Upper Flats Range was narrowed by 25 ft due to deepening on the existing side slope. Therefore, the Plan 1 Upper Flats Range was 600 ft wide.

The Bight Channel is currently a series of 800 ft wide segments. The north side of Plan 1 Bight Channel was narrowed by 25 ft due to deepening on the existing side slope. The south side of the Plan 1 Bight Channel was widened 75 ft. Thus, the Plan 1 Bight Channel is 850 ft wide. The channel immediately west of The Bight Channel, the Fort Jackson Reach, was realigned to avoid relocating pipelines. The first proposed alignment, Plan 1, shifted the channel to the east and provided a large triangular area for the ship's stern to swing when heading outbound. However, concern over inbound ships using the triangular area and then running aground because they were too far west led to Plan 1 being abandoned and Plan 2 being developed. Plan 2 removed the large triangular area. Plan 3 is the existing channel deepened to 48 ft. Plan 3 did not deepen on the existing side slopes, so its footprint is the same as the existing Fort Jackson Range.

There was only one proposed deepening plan each for Oglethorpe Range and the Wrecks Channel. Both plans deepened on the existing side slopes. This reduced the width of both deepened channels from 500 to 450 ft. A 75 ft widener was proposed for the turn at the western end of the Wrecks

Channel. A second plan, Plan 2, was developed without the widener. The existing turn was deepened along its side slopes, thus reducing the south side of the turn by 25 ft. Plan 1 also widened the turn east of City Front Channel by 75 ft on the north side.

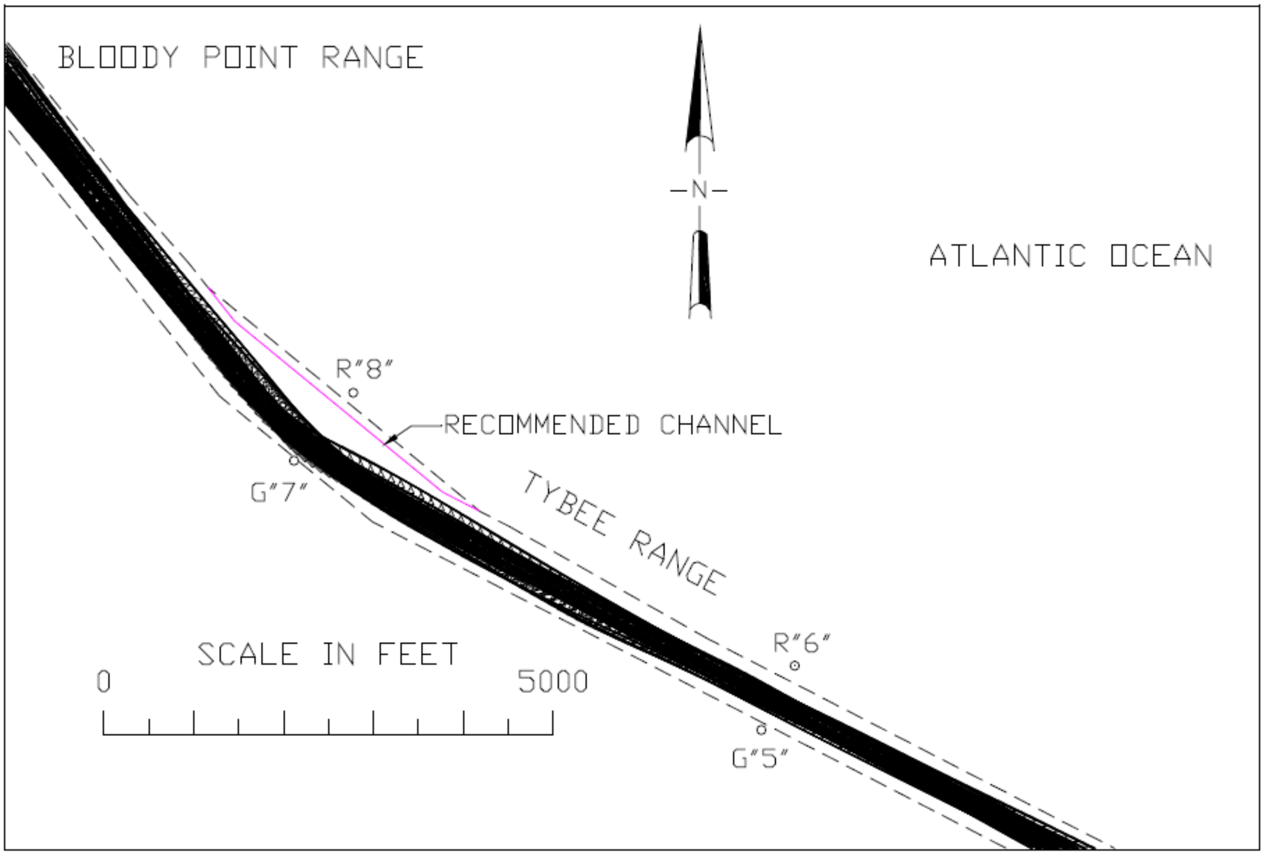
The turn between City Front Channel and Marsh Island Channel was widened by 75 ft as part of Plan 1. The remainder of Marsh Island Channel was deepened on the existing side slopes, thus reducing the channel width from 500 to 450 ft. Plan 1 widens the turn between Marsh Island Channel and Kings Island Channel by 75 ft on the north side. The remainder of Kings Island Channel was deepened on the existing side slopes, thus reducing the channel width from 500 to 450 ft. The north side of Kings Island Turning Basin was widened by 75 ft. The rest of the basin was reduced by 25 ft by deepening along the existing side slopes.

6.3.2.2 Recommendations

Recommendations from the ship simulation study are presented below in order from the Atlantic Ocean, heading inland.

ERDC recommends that the Tybee Range Channel may be improved as per the Plan 1, which specifies that the range be deepened on its existing side slopes. Deepening to 48 ft on the existing side slopes will result in a 550 ft wide channel. The widener on the north side of the turn between Tybee Range and Bloody Point Range that was included as part of Plan 1 was not used during any of the S-class simulations. Therefore, it is recommended that the channel be deepened on its existing side slopes as shown in **Figure 6.3.2.2-1**.

Figure 6.3.2.2-1: Recommended Turn with Tracks of All S-Class Containership Simulations



ERDC recommends that the Bloody Point Range be improved as per Plan 1 by deepening the range on its existing side slopes.

Plan 1 widened Jones Island Range on the south side. However, the simulated vessels showed a strong tendency to stay to the north side while making the turn between Tybee Knoll Cut Range and Bloody Point Range (**Figure 6.3.2.2-2**). Therefore, it is recommended that the widening be shifted to the north side of Jones Island Range as shown in **Figure 6.3.2.2-3**.

Figure 6.3.2.2-2: Jones Island Range, All S-Class Containership Tracks

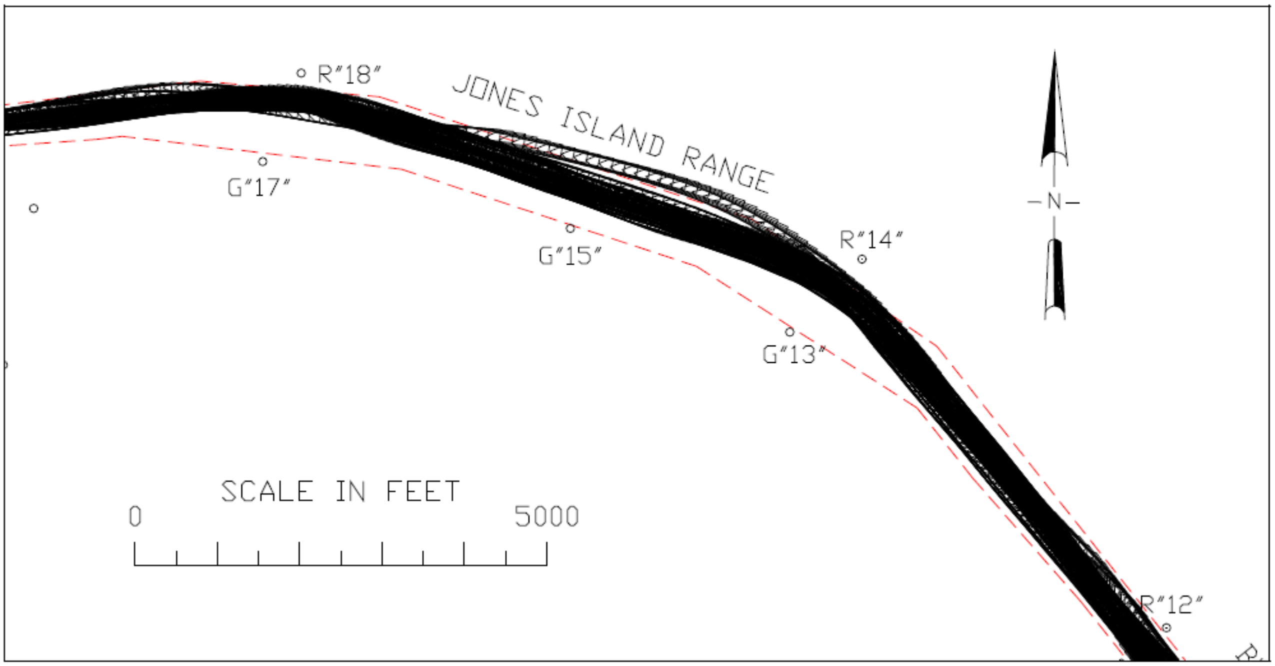
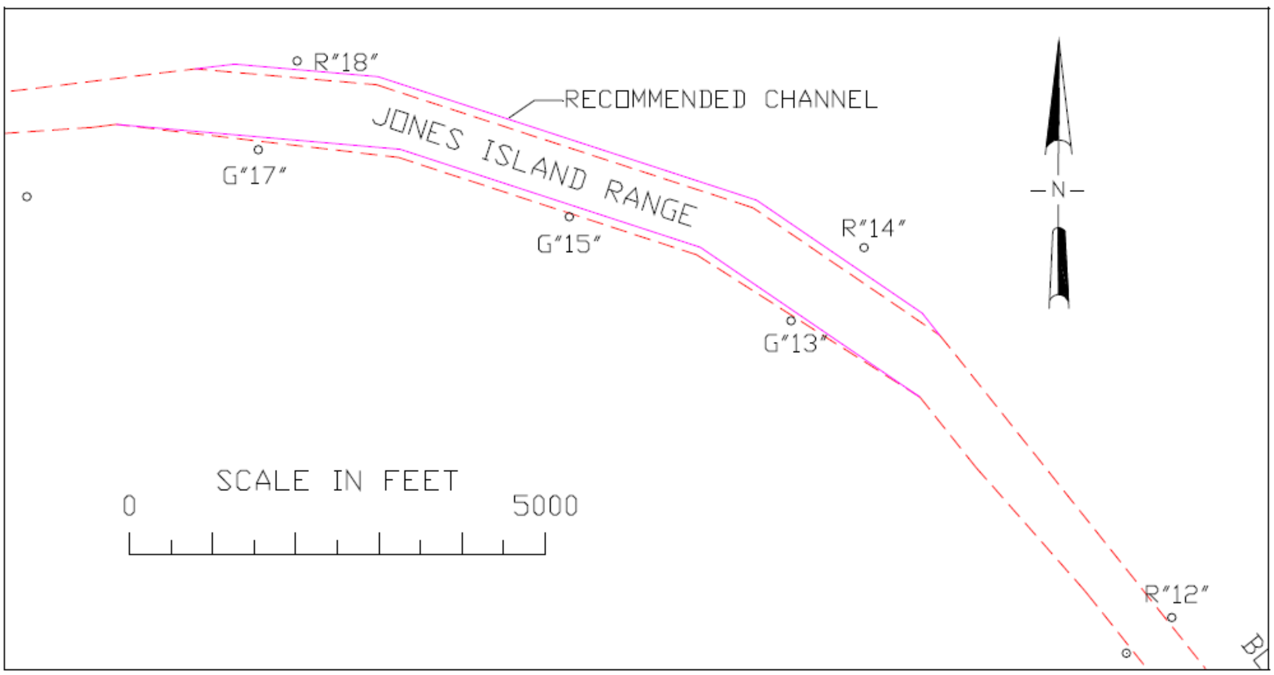


Figure 6.3.2.2-3: Recommended Widener for Jones Island Range

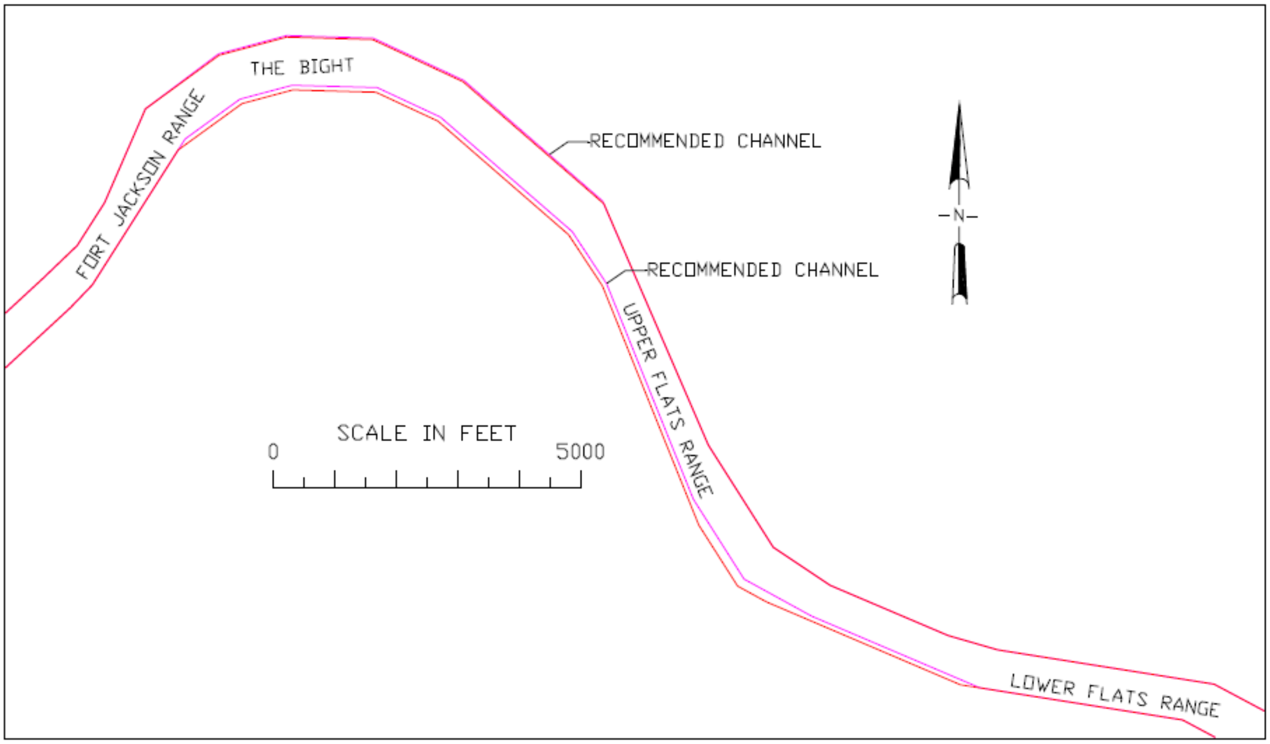


The Tybee Knoll Cut Range and the New Channel Range may be deepened to 48 ft on their existing side slopes as simulated for Plan 1 conditions.

The wider Plan 2 channel is recommended for the Long Island Range. This is a long reach, fairly centrally located in the project. Providing the extra width will provide an excellent area for meeting of extremely large ships.

The Plan 1 channel is recommended for the Lower Flats Range which widens the east side of the channel by 75 ft. The pilots did not use the widening on the west side of the turn between Lower and Upper Flats Ranges or on the west side of the Upper Flats Range or on the south side of the Bight Channel as proposed in Plan 1. Therefore, the 48 ft channel can be deepened on the existing toe limits in this area (**Figure** 6.3.2.2-4). Ships tended to stay to the north side of The Bight Channel. It is recommended that this area not be deepened on the existing side slopes but be deepened on the existing channel toe to provide adequate width in this reach.

Figure 6.3.2.2-4: Channel recommendations Lower Flats Range through The Bight Channel



The Plan 2 channel is recommended for Fort Jackson Range. The additional 100 ft will provide a safe meeting area that the pilots can use with confidence. The pilots consistently relied upon the additional width for two-way traffic during the simulations. Fort Jackson Range is long enough that the pilots can easily coordinate meeting there if necessary.

Because of the ships' tendency to leave the channel a bit on the north side, it is recommended that the north side of Oglethorpe Range and Wrecks Channel not be deepened along the existing side slope but be deepened on the existing toe limits.

None of the wideners on the Wrecks Channel/City Front Channel area specified in the Plan 1 design were used. They may be omitted from the 48-ft project.

The proposed 1,600-ft-diameter Kings Island Turning Basin is adequate for turning the *Susan Maersk*. It is recommended that the basin be dredged as proposed.

6.3.3 Meeting Areas

Meeting areas provide areas for the design vessels to be able to meet in transit to avoid delays that would otherwise be incurred if a vessel had to either wait in the entrance channel or at dock until a design vessel had exited the channel. For Savannah Harbor, all “passing” lanes are defined as meeting areas. “Passing” is typically defined as ships overtaking each other. “Passing” in this sense is not practiced in Savannah Harbor; therefore, any subsequent reference to “passing” shall be understood as “meeting”.

From discussion with the Savannah Harbor pilots, pilots “can meet all vessel classes using the harbor now including two post Panamax vessels, but that is rare and will take a significant amount of coordination”. This coordination produces delays in sailing times for other vessels as meetings can only occur at certain wider areas of the channel and cannot occur in turns or where vessels are docked at berth.

Due to changes in the channel width and the handling capability of the deeper draft post Panamax vessels, the pilots requested that meeting areas be included in the project to ensure greater flexibility in vessel movement. The pilots indicated that the lengths for meeting areas need to be at least 2,000 ft long but preferably 3,000 ft.

6.3.3.1 Locations

During the initial stages of the SHEP navigation study, the harbor pilots expressed a need for a meeting area and suggested the Long Island Range (Station 16+500 to 19+500) as a long straight reach that would be appropriate. As a result, this area was incorporated in the ship simulation study and was used successfully by the pilots in simulation runs.

During the simulation runs, pilots typically met in the Fort Jackson range using a widened portion of the design channel as a meeting area. As a result, the navigation study recommended this area to be used as a meeting area. Pilots had requested Oglethorpe Range (Station 55+000 to 58+000) be considered as a meeting area being centrally located on a long straight reach. A meeting area was added on the north side of the range.

In a subsequent meeting with the pilots, a need was expressed for a meeting area across from the CITGO dock. Initial design was a 3,000 ft meeting area that runs through the Marsh Island Turning Basin (89+934 to 92+000). Subsequent investigation limited this area to 2,200 ft to avoid impact to an International Paper effluent pipeline. This area was eventually removed from consideration as attempts to provide more adequate length for a meeting area would have produced considerable upland taking of real estate.

The first two areas identified above were analyzed in the HarborSym economic model to determine their economic justification to be included as part of the Savannah Harbor Expansion project. A discussion of this analysis is included in the Economic Appendix.

6.3.3.2 Ship Simulation Verification

A follow-up ship simulation was conducted January 14-23, 2009 at ERDC to determine the following:

- The necessary length for meeting on Long Island Range.
- If the proposed meeting area on Oglethorpe Range was adequate in length.
- If narrowing the navigation channel above Marsh Island to provide adequate berth width for the design vessel would adversely affect the pilot's ability to navigate that range or hamper their ability to turn vessels in the Kings Island Turning Basin.

Two pairs of pilots visited the facility to run the ship simulator using the revised meeting area designs. Model runs and pilot's input via questionnaires were used to determine final meeting area design. The results are as follows:

Long Island Range – Based on post processing of the model runs and pilot input, ERDC determined that a 100 ft wide and 8,000 ft length meeting area would be required for vessels to meet safely with 1,000 ft transitions back to the navigation channel width. Final location (center of range) was determined by consultation with pilots. Location was determined to be from Station 14+000 to Station 22+000 for the full 100 ft meeting area (Station 13+000 to 23+000 including transitions).

Oglethorpe Range – Adequacy of the initial proposed length of 3,000 ft had been questioned during the August 2008 Alternative Formulation Briefing (AFB). Therefore, the length of the meeting area was extended by 1000 ft for ship simulation. A width of 100 ft from Station 54+800 to Station 58+800 (Station 54+800 – Station 60+700 with transition) for the range was used in simulation runs and determined to be adequate. Track plots showed that pilots required the full length so no further restriction in length was tested and a full length of 4,000 ft is recommended.

Pilots successfully navigated restricted channel at the Marsh Island Range and approach into Kings Island Turning Basin was not affected.

6.3.4 Entrance Channel Extension

In 1997, the original hydrographic survey for the entrance channel extension, which was conducted in association with the SHEP, extended 25,000 ft beyond the existing end of the Federal project, which occurred at Station -60+000. That survey also extended approximately 3,000 ft oceanward of the 50 ft below MLLW contour, the depth necessary for the deepest entrance channel alternative that was being considered in the feasibility study. At the oceanward extent of the hydrographic survey, the water depth was consistently deeper than 54 ft below MLLW at that time and no additional surveys were conducted. Given that an adequate depth was verified beyond the 50 ft contour below MLLW, it was recommended at the time that the entrance channel be extended 25,000 ft for the 50 ft channel depth (to Station -85+000).

After review of the most current NOAA Chart#11512, it was determined that some shallower shoals exist offshore of the original 1997 survey area. The District obtained and evaluated existing NOAA surveys and conducted a bathymetric survey the week of October 19, 2009. The new survey indicates that there are additional shoals offshore, beyond the end of the entrance channel as it is currently designed based on the 1997 hydrographic surveys.

Additional entrance channel extension routes were developed with the goal of providing the most direct, safest and most cost effective route from the vicinity of the current entrance channel to the 50 ft contour below MLLW either north or south of the current and proposed entrance channel. The length of the extension is dependent upon design channel depth and alignment. Changes to the entrance channel configuration will require additional aids to navigation.

6.3.4.1 Alternative Routes

Due to the similarity of ocean sediments surrounding the entrance channel, it is expected that sediments in the new reach would be similar to those found between Stations -60+000 and -85+000 in the previously proposed alignment. Confirmatory sampling is required through additional channel borings during the PED phase of the project. Previous data suggests that the entrance or ocean bar channel sediments are primarily sand, with exceptions between the jetties and at Station -45+000, which have large silt and clay components.

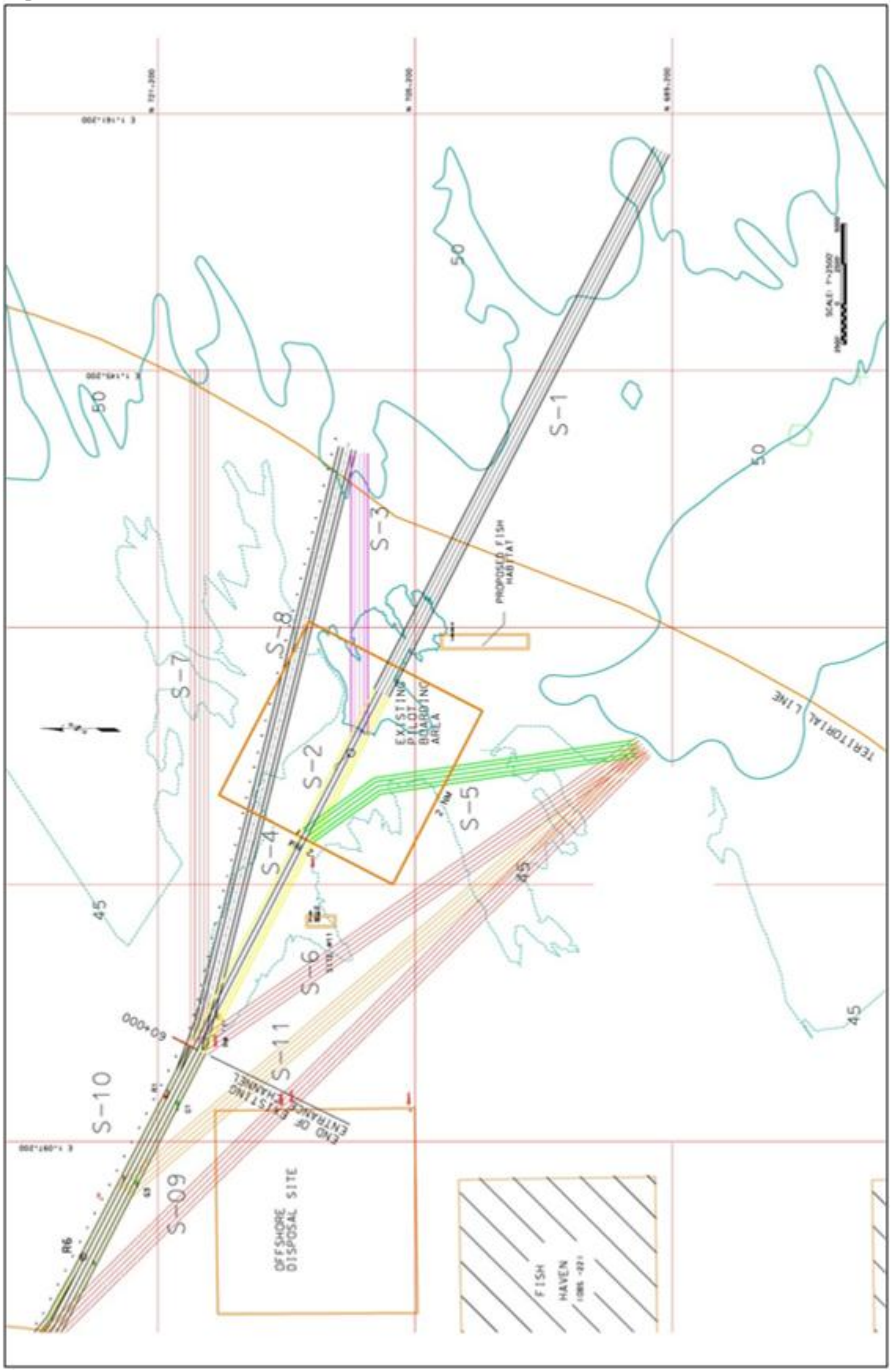
In addition to consideration of the sediment characteristics found in the new channel extension, consideration was also given to cultural resources and environmental impacts. Impacts to cultural resources are not expected this far offshore; however, surveys will be conducted during PED to confirm or refute this assumption. Environmental impacts to endangered species (Right Whales) were also considered. Details of this analysis can be found in the EIS.

Eight alternative channel routes were developed for consideration. See **Figure 6.3.4.1-1**. These routes were designed from the beginning of the current entrance channel (Station -60+000) out to the 50 ft contour below MLLW. Route S-08 was developed as a result of input from the Savannah Bar Pilots. **Table 6.3.4.1-1** shows the dredging volumes for the four least costly alternatives.

Table 6.3.4.1-1: SHEP Entrance Channel Extension Route Alternatives (Description and Quantities)

Route Description	Route	Volume cubic yards (-49 ft MLLW)	Volume w/Over-depth cubic yards (-51 ft MLLW)
Straight Line -60+000 to -123+000	S-01	1,235,481	2,988,367
-60+00 to -82+000 S-3, then east to -50 ft MLLW	S-03	957,870	2,409,643
-60+000 to -78+000 S-5, then South to -50 ft MLLW	S-05	1,861,076	3,646,245
-60+000 on tract ESE S-8 to -50 ft MLLW	S-08	2,041,954	3,401,689

Figure 6.3.4.1-1: SHEP Entrance Channel Extension Route Alternatives



6.3.4.2 Ship Simulation for Channel Extension

In November 2009, the District contacted ERDC to conduct additional simulations for two different alignments for the Entrance Channel. The alignment changes, or doglegs, were proposed so that when the Entrance Channel is deepened, it will reach naturally deep water in a shorter distance, reducing dredging costs and transit times. This study focused on alignments S-3 and S-8.

The simulation databases from the earlier study were modified to reflect alignments S-3 and S-8. Currents and waves for the proposed alignments were modeled in separate ERDC studies. These documents are being finalized and will be included in the Final SHEP documents. Maximum flood and ebb currents from the hydrodynamic model were added to the simulator database. Bend wideners were added to both alignments. Inbound ranges were also built for each alignment in the visual scene and radar.

Results from the simulation are as follows:

- Without ranges, alignment S-3 was not adequate for the design vessels. This is true for the alignment with and without the bend widener.
- The addition of inbound ranges improved both alignments.
- Alignment S-8 appeared to be adequate for one-way traffic, even without ranges. The addition of ranges and the bend widener allowed for meeting of a Panamax ship with a post-Panamax ship.
- Crabbing occurs in adverse wind conditions. Under the conditions tested, crabbing was not a serious issue. Neither alignment made crabbing more severe.
- Alignment S-8, with ranges and the bend widener, was recommended by the ship simulation study.

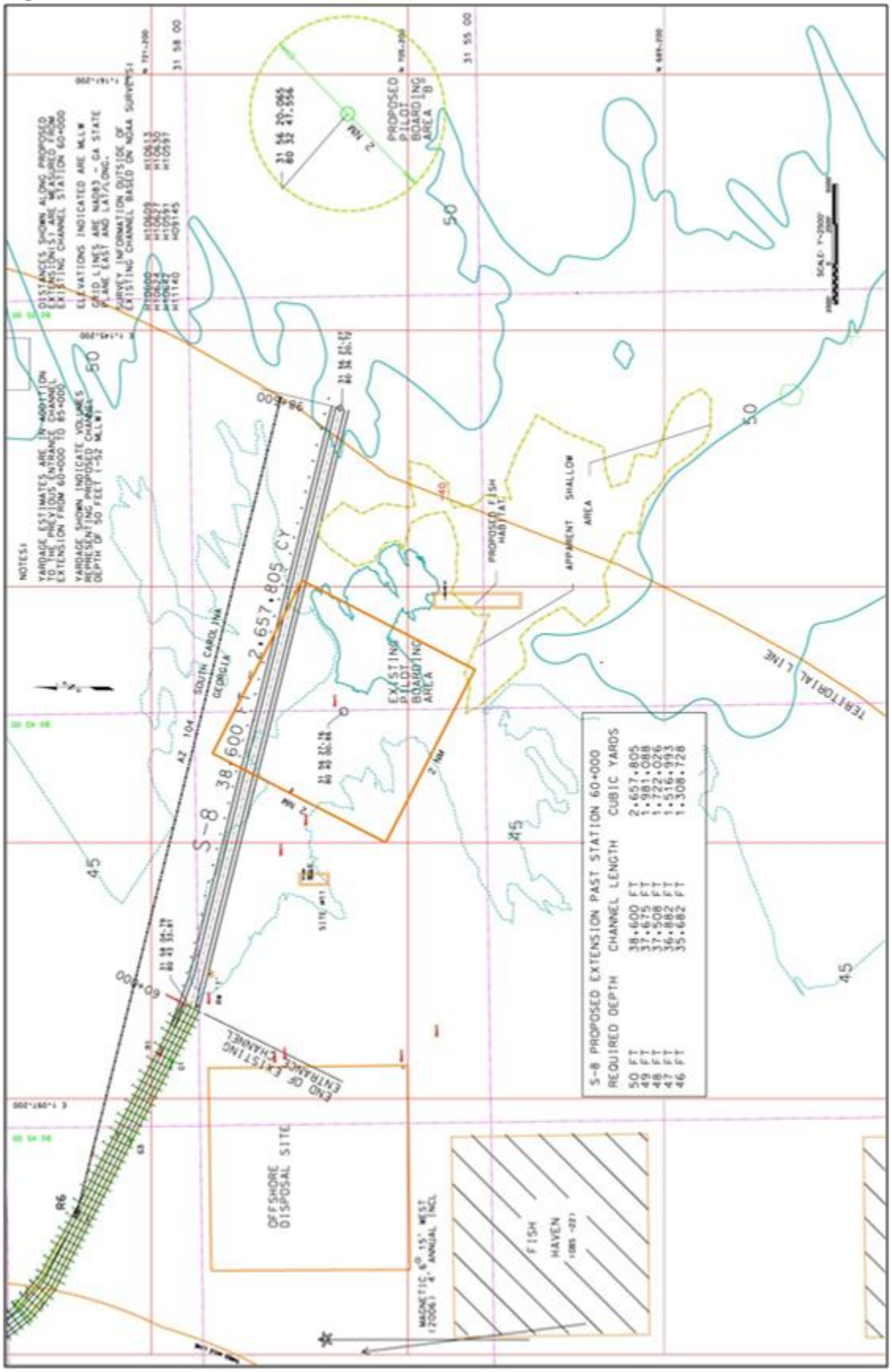
6.3.4.3 Selected Channel Extension Route

Through the planning process, the 8 entrance channel extension routes were evaluated with the goal of selecting the most direct, safest and most cost effective route from the vicinity of the current entrance channel to the 50 ft contour below MLLW either north or south of the current and proposed entrance channel. The length of the bar channel extension varies with the proposed depth alternative (**Table 6.3.4.3-1**). This process, along with ship simulation verification, resulted in the recommendation of Route S-08 for selection as the entrance channel extension. **Figure 6.3.4.3-1** illustrates the maximum 38,600 foot long extension of the ocean bar channel from Station -60+000 to -98+600 for the 48-foot depth alternative.

Table 6.3.4.3-1: Length of Channel Extension Required for Depth Alternatives

Depth Alternative	Channel Extension Stationing	Length of Extension
44	-60+000 to -95+680	35,680 ft
45	-60+000 to -96+880	36,880 ft
46	-60+000 to -97+510	37,510 ft
47	-60+000 to -97+680	37,680 ft
48	-60+000 to -98+600	38,600 ft

Figure 6.3.4.3-1: SHEP Entrance Channel Selected Route



Prior to construction, additional core borings for sediment characterization and a cultural resources impact analysis of the route and sediment placement sites will need to be completed. The additional borings are located along the new alignment (S-08) to determine material types to be dredged. Nineteen borings will be drilled at approximately 2,000-foot intervals as shown on the drawing titled, *Channel Extension Boring Locations,* which is included in the Engineering Investigations Supplemental Materials. The Standard Penetration Test (SPT) borings will be drilled to at least 60 ft below MLLW. According to borings drilled along the previous extension alignment to the South, the material to 60 ft below MLLW is expected to be predominately sand. Since the silty, clayey fine sands of the top of the Miocene confining layer are known to exist at around 55 to 60 ft below MLLW in this area, some of the borings will likely bottom out in this material. For additional detail on dredging and annual maintenance of the entrance channel extension see Section 11.0 and 12.0.

6.3.4.4 Vertical Motion Study

A vertical ship motion study was conducted by ERDC to evaluate channel depth requirements. Detailed results are documented in two reports titled 1) *Navigation Study for Savannah Harbor Channel Improvements* and 2) *Vertical Ship Motion Study for Savannah, GA Entrance Channel* which are included in the Engineering Investigations Supplemental Materials.

The first study, *Navigation Study for Savannah harbor Channel Improvements,* consisted of a series of simulations with a computer-controlled ship transiting Tybee Range. This study was conducted to evaluate extending Tybee Range from Station -60+000 to -85+000 on new channel option S-1 shown in **Figure** 6.3.4.1-1. Two drafts of the *Susan Maersk* were simulated, 46 ft and 47.5 ft. Wave conditions for this study were selected based upon a concurrent wave model study conducted at ERDC. Based upon analysis of the 35% highest waves, the following conditions were chosen for the study:

- Waves coming from a direction of 215 degrees
- Wave heights of 6, 8, and 10 ft were simulated
- Wave periods of 8 and 10 seconds were simulated

Water depths from 48 to 56 ft were simulated in 1-foot increments. Water depth is defined as the authorized channel depth plus tide. Therefore, a water depth of 54 ft could represent either a 54-ft channel at low water or a 50-ft channel with a 4-foot tide.

The vertical motion model used in the study was developed by Tracor Hydronautics and operated in fast-time. The simulated ship maneuvered with computer-controlled speed and heading based upon input conditions. Results show that for the *Susan Maersk* at a 47.5-ft draft, grounding would occur in a 52 ft channel depth under conditions at or worse than a 6 ft wave height and 8 second wave period. Also, for the *Susan Maersk* at a 46-ft draft, grounding would occur in a 51 ft channel depth under conditions at or worse than a 10 ft wave height and 10 second wave period. Harbor pilots attempting transit under these sea states would wait for additional depth from tides.

Some shallower offshore shoals were discovered that might influence the safety and efficiency of navigation if the project proceeds as originally proposed. The U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL), conducted a vertical ship motion study to evaluate three proposed channel alignments S-1, S-3, and S-8. These alignment changes (doglegs) are proposed to allow ships to reach deeper water in less distance, with reduced dredging costs. This second study, *Vertical Ship Motion Study for Savannah, GA Entrance Channel*, incorporated findings from the first study for wave height transformation and transformation factors. The Channel Analysis and Design Evaluation Tool (CADET) was used to predict vertical ship motions due to wave-induced heave, pitch and roll. PIANC and Ankudinov ship squat were calculated and compared with the CADET/BNT squat predictions. The CADET squat formula is based on the work of Beck, Newman, and Tuck. The CADET days of accessibility, vertical motion allowances, and net underkeel clearance (net UKC) were calculated based on these vertical ship motion components to provide a risk-based or probabilistic method of evaluating different channel depths.

Based on the separate economic study for the Savannah Harbor Expansion Project, a final proposed project depth of 49 ft was selected for the Outer Channel. This represents a 1 ft reduction in the original project depth of 50 ft used in this engineering study. However, since the range and duration of the tides is sufficient to increase water levels to accommodate safe transits for the ship drafts discussed in this study, results are still valid and accurate. Water depths were adjusted for the tides and discussed as necessary in this report.

A joint probability distribution of wave height and period was created in ten 22.5 degree direction bands from 11.25 degrees to 236.24 degrees. It consisted of 158,138 observations representing 90.2% of the deepwater data from the WIS 20-year hindcast buoy (WIS370). A total of 99 empirical directional wave spectra were created from this joint probability distribution. Parameters for these directional spectra were based on wave period and height for a TMA frequency spectrum and Cos^n directional spreading function. The spectra wave heights were reduced at each reach along the Savannah Channel according to the previous study results of Thompson.

The Ship Tow Simulator (STS) requires the wave period, height, and direction combination at stations along the channel to prepare wave cases for each simulation run. Since it is impractical to model multiple wave cases in the STS, the most representative wave cases were selected based on the 3-5 % highest waves in the 20-year hindcast at deepwater station WIS33. Values from an earlier study using the STWAVE transformation model were provided for Stations 0 to 87 in all channel options including the Existing channel. These data were then extrapolated seaward to Station 99 for the new channel options S-1, S-3, and S-8.

Ship squat was compared for PIANC, Ankudinov, and CADET/BNT predictions. The five PIANC empirical squat formulas included those of Barrass, Eryuzlu, Huuska, Römisch, and Yoshimura. The Ankudinov formula was originally used in the STS. Based on ship velocity, an average squat value was determined from these methods to be used for channel design.

This report summarizes remaining underkeel clearance(net UKC) for the average and maximum (or worst case) ship squat predictions. For the light-loaded *Susan Maersk* drafting 46 ft at the shallowest water depth of h=50 ft (+1 ft tide), pilots should proceed with caution if attempting ship speeds faster than 10 to 12 knots (kt), especially if any significant wave activity is present. For water depths greater than 50 ft, there should be sufficient net UKC for speeds as high as 14 to 16 kt. For the fully-loaded ship drafting 47.5 ft at this shallow 50 ft depth (+1 ft tide) depth, pilots should exercise extreme caution if attempting to move at speeds as high as 10 kt. This speed is probably not even possible unless wave heights and periods are relatively small. For the deeper depths when tides are present, the available underkeel clearance should be sufficient up to speeds of 12 to 14 kt. Of course, pilots should be vigilant at higher speeds as ship squat can always be reduced by slowing down.

In general for both light- and fully-loaded ships, days of accessibility increase for slower ship speeds, outbound transits, interior reaches, and reduced loads/lesser drafts. The minimum depths are 50 and 52 ft (+3 ft) for the light- and fully-loaded ships, respectively. Based on the CADET analysis of underkeel clearance and the corresponding days of accessibility, the S-8 alternative is the best option among the three choices S-1, S-3, and S-8, and much better than the existing channel. The S-8 option will probably have slightly less risk of grounding than S-1 and S-3.

Based on the CADET tidal analysis for the light-loaded ship, the Savannah entrance channel will have an additional water level of +1 ft above the 49 ft proposed project depth (i.e., *h*=50 ft) for durations of 8 hr for 365 days (i.e. 100 percent) of every year. To accommodate the fully-loaded ship, an additional water level of +3 ft above the 49 ft proposed project depth (i.e., *h*=52 ft) will be required. This depth will have durations of 6 hr for 365 days (i.e., 100 percent) of every year, but decreases to only 25 days per year for 8 hr durations. Water depths of 53 to 57 ft will have continually decreasing durations from 4 hr (365 days per year) to 1 hr (7 days per year).

Thus, for the recommended S-8 option in Reach 1, the light-loaded 46-ft-draft 46 ft draft *Susan Maersk* in a depth of 50 ft (+1 ft tide) will have a total of 360 days of accessibility per year during inbound transits at 10 kt. At this depth, tide levels are not a problem as the Outer Channel will have 8-hr durations for 365 days per year, more than sufficient to accommodate transit times. Increased ship speed can be accommodated with decreasing durations of 7 hr or less as the tide level increases. For instance, 338 days per year are predicted for inbound transits at 14 kt at 50 ft depth. A tide level increase of 3 ft to a depth of 52 ft will accommodate inbound transits at 14 kt for 356 days per year. However, this water level is only available for 6 hr durations every day of the year at this depth of 52 ft. Durations up to 8 hr are available but only for 25 days per year. Using the percentage of tide level fraction, this is equivalent to reduced days of accessibility of only 24 days per year (i.e., 356 days * 25 days/yr/365 days/yr).

Similarly, for the recommended S-8 option in Reach 1 for the 47.5 ft draft *Susan Maersk*, a depth of 52 ft will provide a total of 360 days of accessibility per year during inbound transits at 10 kt. At this depth, tide levels are available every day of the year for durations up to 6 hr. If necessary, a duration of 8 hr is available for 25 days a year. As before, this is equivalent to a reduced days of accessibility of only 24 days per year for this 8-hr duration. Increased ship speed can be accommodated with decreasing durations of 6 hr or less as the tide level increases above a depth of 52 ft. For instance, 357 days per year are predicted for inbound transits at 14 kt at 52 ft depth. A tide level increase of +4 ft to a depth of 53 ft will accommodate inbound transits at 14 kt for 362 days per year. This water level is only available for 4 hr durations every day of the year at this depth of 53 ft. Durations up to 6 hr are available, but only for 144 days per year. The equivalent reduced days of accessibility for these conditions is 43 days per year (i.e., 362 days/yr * 144 days/yr/365days/yr). Increasing the depth by 5 ft to 54 ft would provide 364 days per year accessibility, but only for a duration of 4 hr for 242 days per year to 3 hr for 331 days per year. In this case, the equivalent reduced days of accessibility is only 241 days per year (i.e., 364 days/yr * 242 days/yr/365days/yr).

Wave-induced vertical ship motions are composed of the combined effects of heave, pitch, and roll at the five critical points on the bottom of the ship. CADET calculates these vertical motion allowances for each ship loading condition, channel reach, and water depth. In general, outbound transits are much less of a problem than inbound transits as their motion allowances are much smaller. For the light-loaded ship, the motion allowances tend to increase as ship speed increases. The motion allowances for the fully-loaded ship, however, tended to decrease until reaching a ship speed of about

12 kt before increasing again at the highest speeds. These comparisons indicated that the S-8 option is the preferred alternative as it has smaller predicted motions than the existing S-1 or the S-3 option for all speeds, especially for 10 kt.

The net UKC is obtained by subtracting draft, squat, and ship vertical motion allowances from the water depth (i.e., net UKC = gross UKC – squat – ship vertical motion allowance). In general for the light- and fully-loaded *Susan Maersk*, net UKC increases with change in transit direction from inbound to outbound, increases in water depth, and decreases in speed. The S-8 option is as good as or better than the existing S-1 and S-3 option channel configurations.

For the light-loaded inbound ship at a speed of 10 kt, only twelve of the 99 waves indicated grounding conditions at the recommended depth of 50 ft(+1 ft) based on the accessibility results. These 12 waves represent relatively rare occurrences, as all of them only occur for a total of 4.9 days/yr. For the fully-loaded inbound ship at a speed of 10 kt, only seven waves indicated possible grounding conditions at the recommended depth of 52(+3 ft) ft from the accessibility results. These seven waves also represent relatively rare occurrences with a total of only 2.6 days/yr. These are relatively large wave periods and heights compared to the typical waves at Savannah, but pilots should be particularly aware of possible grounding conditions when they occur.

The wave-induced vertical motion allowances and corresponding net UKC support and confirm the days of accessibility results for the minimum water depths required for safe transits in both inbound and outbound directions. In summary, a depth of 50 ft (+1 ft) is the minimum acceptable depth for safe transits at 10 kt for the light-loaded, 46-ft-draft *Susan Maersk* in the Savannah Outer Channel. A minimum depth of 52 ft (+3 ft) is required for safe transits at 10 kt for the fully-loaded, 47.5-ft-draft *Susan Maersk*. Faster ship speeds up to 14 kt are possible if higher tide levels are used, but the available durations are reduced such that a transit may not be possible every day of the year.

6.3.5 Bend Wideners

6.3.5.1 Location

Based on design standards in EM 1110-2-1613, *Hydraulic Design Guidance for Deep-Draft Navigation Projects*, the District proposed bend wideners in the initial channel design to accommodate the design vessel *Susan Maersk*. Bend wideners included an additional 100’ from the bottom of the design channel toe and are listed in **Table** 6.3.5-1 under column heading “Savannah”. A total of 10 bend wideners were proposed (8 inner channel and 2 entrance channel). An additional 100’ on the backside of Kings Island Turning Basin was also included as a design feature.

6.3.5.2 Ship Simulation Verification

As a result of ship simulation, ERDC determined that only 3 of the bend wideners would be required to accommodate the design vessel. The location of one of the proposed wideners (Long Island Range) was shifted from the south to the north side of the range. Those bend wideners are listed in **Table 6.3.5-1**, column heading “ERDC”. Of the remaining 7 proposed bend wideners, ERDC determined that deepening those ranges on the existing width would provide sufficient channel width for the design vessel and are also listed under the column heading “ERDC” marked with an asterisk(*). Ship simulation also confirmed the need for the additional 100’ width on the back side of the Kings Island Turning Basin.

Table 6.3.5-1: Proposed and Recommended Bend Wideners

Range#	Range/Reach Name	Station(Approx)	Savannah	ERDC
0a	Tybee/Bloody Pt	-41+000 to -38+000	Yes/North	No*/North
1a,2,2a	**Jones Island**	**-23+000 to -14+0000**	**Yes/South**	**Yes/North**
6	New Channel/ Long Island Crossing	9+500 to 11+500	Yes /North	No*/North
9	**Lower Flats**	**27+500 to 31+500**	**Yes /North**	**Yes /North**
10 To 20	Upper Flats/Bight/ Ft Jackson	31+000 to 52+500	Yes/South	No*/South
20,21	**Ft Jackson & Transition**	**52+500 to 55+000**	**Yes /North**	**Yes /North**
24,25	Wrecks Channel	65+500 to 66+500	Yes /South	No
25,26,27	Wrecks Channel/City Front	69+000 to 71+500	Yes /North	No*/North
31	City Front/Marsh Island	76+000 to 78+000	Yes /North	No*/North
34,35	Marsh Island Turning Basin	87+500 to 90+000	Yes /North	No*/North
	Kings Island Turning Basin Back (South Carolina) Side	**99+000 to 100+500**	**Yes**	**Yes**

*Deepen on existing width

6.3.6 Channel Volumes

In summary, the channel volumes to be excavated for each SHEP depth alternative are summarized in **Table 6.3.6-1** and **6.3.6-2**. These quantities are based on the design template for each depth alternative and represent new work material only including the passing lanes and Kings Island Turning Basin.

Table 6.3.6-1: New Work Inner Harbor Dredging Volumes

Station	44 ft Project (cy)	45 ft Project (cy)	46 ft Project (cy)	47 ft Project (cy)	48 ft Project (cy)
0+000 to 4+000	101,482	166,705	235,626	305,674	375,403
4+000 to 6+375	48,128	87,346	130,559	174,073	217,263
6+375 to 30+000[A]	1,264,730	1,756,993	2,258,262	2,759,203	3,259,272
30+000 to 45+000	684,583	1,052,928	1,426,462	1,802,866	2,181,609
45+000 to 51+000	324,752	508,740	699,013	892,307	1,088,128
51+000 to 57+000[B]	652,793	801,504	951,201	1,101,114	1,251,494
57+000 to 67+000[C]	588,884	807,450	1,026,002	1,244,681	1,463,393
67+000 to 80+125	444,210	691,727	944,611	1,196,291	1,446,786
80+125 to 90+000	380,724	570,368	759,169	946,436	1,132,066
90+000 to 103+000[D]	1,438,457	1,803,823	2,169,594	2,533,434	2,895,175
Total	5,928,743	8,247,584	10,600,499	12,956,079	15,310,589

A – Includes Long Island Meeting Area, Station 14+000 to 22+000
B – Includes Oglethorpe Meeting Area, Station 55+000 to 57+000
C – Includes Oglethorpe Meeting Area, Station 57+000 to 59+000
D – Includes Kings Island Turning Basin

Table 6.3.6-2: New Work Entrance Channel Dredging Volumes

Station	44 ft Project (cy)	45 ft Project (cy)	46 ft Project (cy)	47 ft Project (cy)	48 ft Project (cy)
-98+600 to -57+000*	1,667,123	2,242,371	2,925,432	3,736,308	4,613,909
-57+000 to -53+500	156,623	235,127	313,391	391,437	469,252
-53+500 to -40+000	646,796	975,843	1,304,385	1,632,346	1,959,186
-40+000 to -30+000	505,693	771,105	1,038,620	1,305,921	1,573,800
-30+000 to -20+000	529,910	801,974	1,076,638	1,352,115	1,628,379
-20+000 to -10+000	473,047	746,614	1,028,399	1,311,222	1,594,871
-10+000 to 0+000	346,997	532,621	723,394	917,064	1,110,713
Total	4,326,189	6,305,655	8,410,259	10,646,413	12,950,110

**Station -98+600 is the extended channel stationing for the 48 ft project depth. Channel stationing for the 47 ft project depth across the ocean bar terminates at Station -97+680.*

6.4 CHANNEL SIDE SLOPE STABILITIES

6.4.1 Introduction

A report prepared by USACE titled *Savannah Harbor Expansion Bank Stability Report* is included in the Engineering Investigations Supplemental Materials. The report includes computations, sketches, analyses, and preliminary drawings regarding channel side slopes. Computations are based on drilling data, test results from soil samples taken at drilling locations, the latest survey data as of 2005, hill survey data as requested for specific locations, observation/review of channel side slopes resulting from previous harbor widening and deepening projects, and other information from previous dredging projects regarding channel side slope performance. Observations and survey measurements made from about 1816 to 2003 in Savannah Harbor establish that the ship channel or prism side slope averages 1 vertical to 3 horizontal for the harbor channel and about 1 vertical to 5 horizontal for the jetties and bar channel reaches. Variations have been noted at isolated locations that have either flatter or steeper side slopes that generally are the result of past efforts at filling older drainage features or other man-made modifications. The variations noted occur well beyond the channel sailing prism and do not appear to impact or influence vessel movements.

6.4.2 General

Addressed during analyses are the known locations or reaches of possible problem areas regarding channel side slopes, sloughing of materials, and/or real estate acquisition requirements. Each is discussed separately in the *Savannah Harbor Bank Stability Report* dated 2003. The results of this report were updated in June 2010 to incorporate changes in channel alignment, wideners and meeting areas recommended by the ship simulator study. Areas that are not specifically addressed were also reviewed in detail using the proposed channel geometries and the most recent survey/sounding information. Review of these areas indicates the proposed deepening/expansion will not have a direct effect on real estate above mean low water (MLW) nor interfere with structures located on the river's edge.

Inspections were performed as a part of obtaining riverbank and structural information within the limits of this project in 2001 and again in 2003. The inspections document the condition of structures on or along the shoreline within the limits of the project. A copy of the reports, photographs, and descriptions are included as Appendix to the *Bank Stability Report*.

Several hundred borings have been drilled within and adjacent to the Savannah Harbor. The majority of these borings were drilled along the north side of the channel for the Savannah Harbor Widening and the Savannah Harbor Deepening projects, then supplemented with additional borings for the Expansion. These borings were drilled to obtain information necessary for evaluating the in-situ materials within specific areas of the channel for harbor modification projects. The majority of the borings were water-borne. The land-based borings were completed to identify soil materials within the channel side slopes to help determine the most probable channel side slopes resulting for each proposed harbor modification. The investigations have used a variety of methods to obtain subsurface data, including Vibracore, splitspooning, coring, cone penetration tests, and other methods. Standard penetration sampling using a split-barrel sampler was the method most often used. Using this method, a 1-3/8 inch inner diameter standard split barrel sampler was driven through the material using a 140-pound hammer with a 30-inch fall. The sampler was retrieved and the material was described in accordance with the Unified Soil Classification System.

6.4.3 Analyses Overview

Slope analyses for selected locations were performed using the Modified Slope Stability Package with Kansas City Analysis (DGSLOPE) and the computer program UTEXAS3. Both programs were used for either original analysis and for the checking and verification of results. Final input data sets, illustrations, cross-sections, slip circles and/or wedge sections, and the results obtained are provided in the Bank Stability Report. The project team adopted a method of deepening that calls for maintaining the existing side slopes to the proposed deepened elevations thus negating the need to acquire real estate throughout the harbor. Exceptions occur at planned meeting areas and curve wideners.

6.4.4 Summary

The planned wideners in the area of Channel Station 102+000 and Kings Island Turning Basin expansion were analyzed with regard to most probable slopes and the resultant requirement for real estate acquisition. Each was evaluated and acreage determined assuming an acquisition need measured from the zero (0) mean low water elevation and again from the plus eight (+8) mean low water elevation. The widener proposed north of Kings Island Turning Basin requires approximately 3.9 acres above elevation zero or approximately 1.1 acres above elevation +8, described as “taking” or acquisition. Kings Island Turning Basin requires approximately 5.0 acres above elevation zero and approximately 4.4 acres above elevation +8 described as “taking” or acquisition. An additional 30-foot wide temporary right-of-way should also be obtained for use during construction. The areas are shown in **Figure 6.4.4-1**.

The area between Channel Stations 95+700 to 97+000, north side adjacent to Confinement Area 2A, is included because the existing side slopes appear to be in a constant state of failure. However, in theory, no real estate will be required. In total, there are four (4) areas identified for real estate acquisition, each addressed in detail within the *Bank Stability Report* and outlined below in **Table 6.4.4-1** and the Figures that follow, **Figure 6.4.4-1** to **Figure 6.4.4-3**. There are no other known areas of concern with respect to the channel prism and the stability thereof.

Table 6.4.4-1: Areas Identified for Real Estate Acquisition

Channel Location		44 ft Project Alternative		45, 46, 47 and 48 ft Project Alternatives		
From	To	Acquisition Above 0 ft MLLW Acres	Acquisition Above 8 ft MLLW Acres	Acquisition Above 0 ft MLLW Acres	Acquisition Above 8 ft MLLW Acres	Relating Figure
101+200	102+500	3.7	0.9	3.9	1.1	6.4.4-1
98+200	100+500	4.8	4.2	5.0	4.4	
96+000	97+000	1.2	0.5	1.2	0.5	6.4.4-2
86+000	88+500	1.9	0.4	2.1	0.6	6.4.4-3

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Figure 6.4.4-1: Areas Identified for Real Estate Acquisition (Station 98+200 – 102+500)

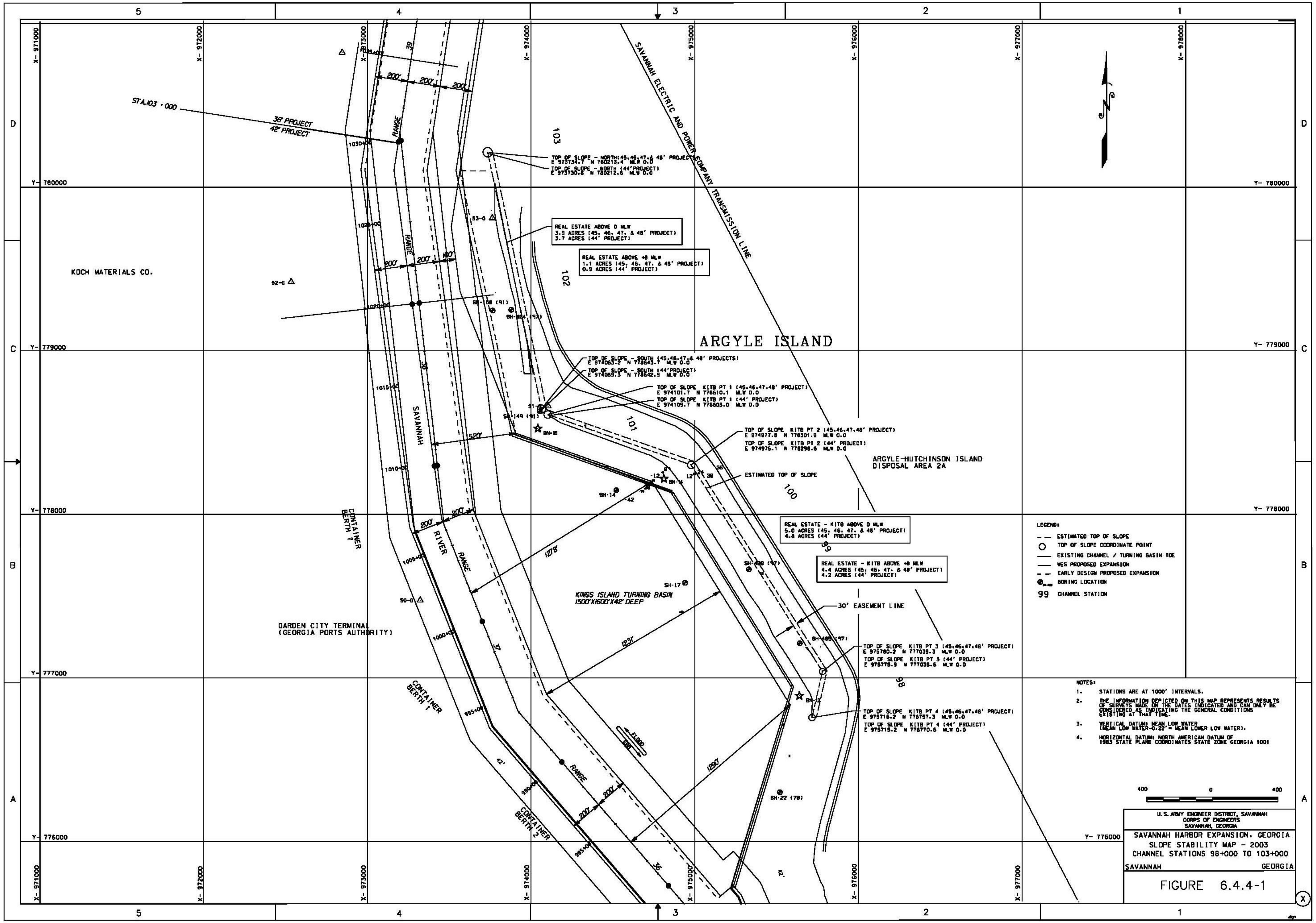

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Figure 6.4.4-2: Areas Identified for Real Estate Acquisition (Station 96+000 – 97+000)

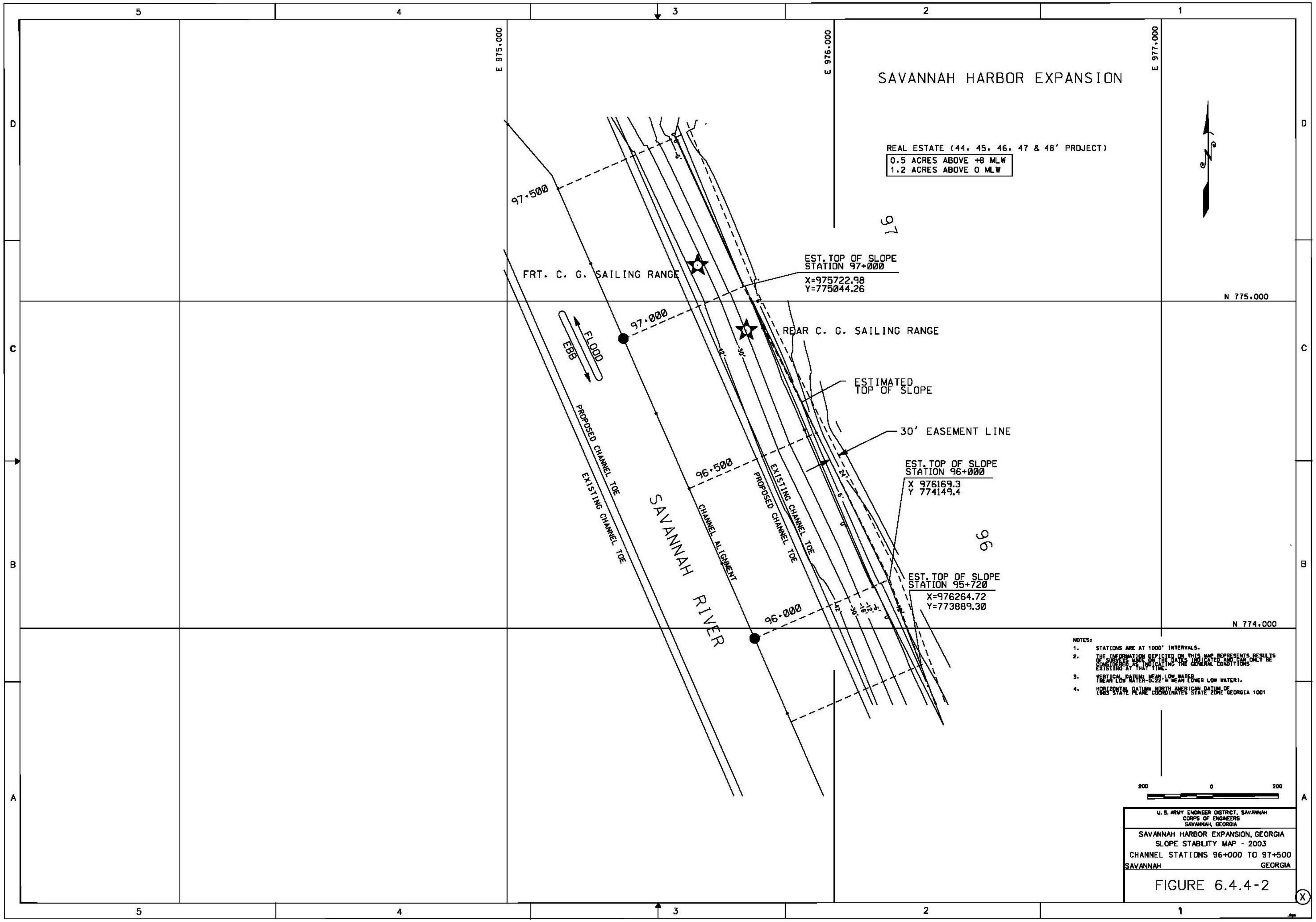

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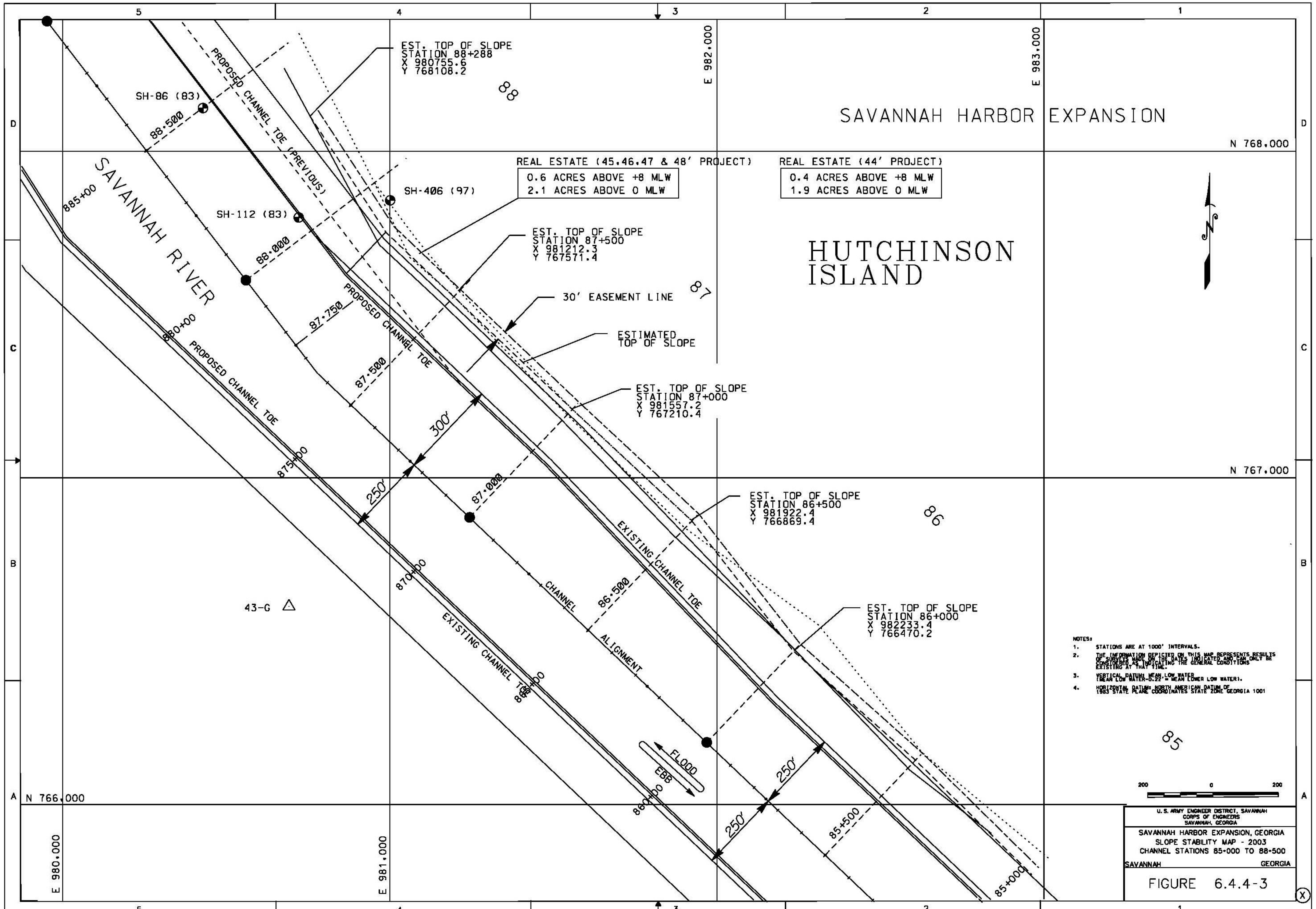
Figure 6.4.4-3: Areas Identified for Real Estate Acquisition (Station 86+000 – 88+500)



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7.0 HYDRODYNAMIC MODELING

The three-dimensional hydrodynamic model Environmental Fluid Dynamics Computer Code (EFDC) and the Water Quality Analysis Simulation Package (WASP) models were enhanced and calibrated for application to the SHEP. A summary of the model development and calibration is contained in the following sections of this Appendix, greater detail can be found in a report prepared by Tetra Tech titled *Development of the Hydrodynamic and Water Quality Models for the Savannah Harbor Expansion Project* dated January 2006 which is included in the Engineering Investigations Supplemental Materials.

For SHEP, modification and calibration of the EFDC and WASP models was necessary to have a means to (1) predict and quantify impacts to the estuary and (2) to develop mitigation features and plans that are scientifically sound and acceptable to all of the Federal Agencies involved in SHEP. The SHEP model applications have been designed to meet the expectations of the Modeling Technical Review Group (MTRG) consisting of federal and state agencies. For documentation from the agencies stating support of the models developed for SHEP, see the document titled *Correspondence between USACE & Federal/State Agencies Regarding Hydrodynamic & Water Quality Model Acceptability* which is included in the Engineering Investigations Supplemental Materials.

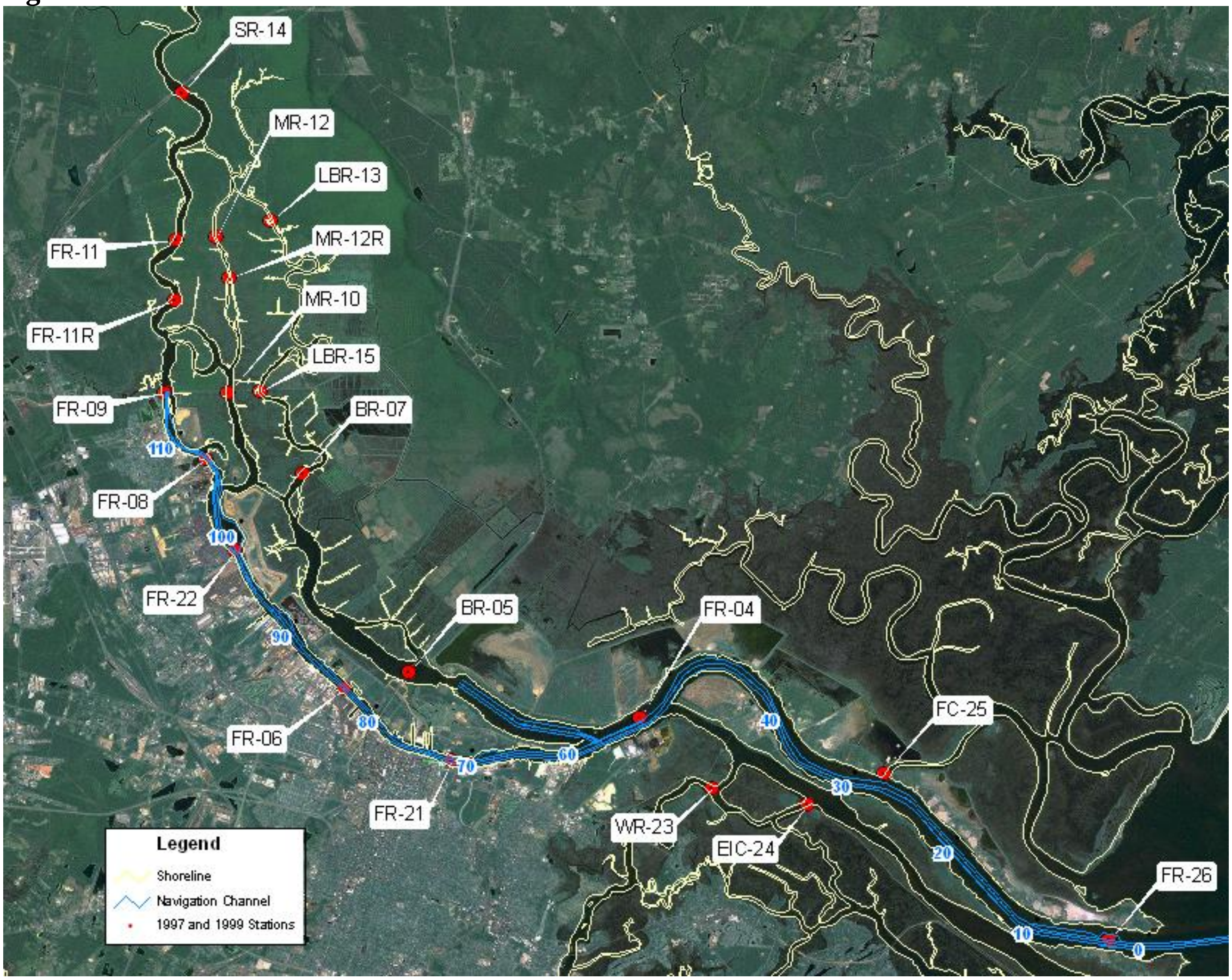
The model code, modeling results, in both time series and statistical formats, and a database, which contains model comparison data were made available for peer review in addition to federal and state agency review prior to use of the model for SHEP impact determination and mitigation design. Comments and responses to the peer review and the model team review can be found in the 2006 report prepared by Tetra Tech.

In addition to the model review, Kinetic Analysis Corporation performed an uncertainty analysis of the SHEP model. The analysis took approximately 2 months and upon review of the results, Tetra Tech agreed that indeed there is uncertainty in the model and the uncertainty predicted was reasonable. Furthermore, having an uncertainty prediction with a mechanistic model is valid because it shows a range of conditions expected. Results of the uncertainty analysis are also included in the in the 2006 report prepared by Tetra Tech.

7.1 DATA COLLECTION

In preparation for this modeling, an extensive data collection effort took place during the summers of 1997 and 1999. Funded and directed by the GPA, the data collection was carried out by ATM, Inc. See **Figure 7.1-1** for location map. Data was collected during summer months to capture the critical period for dissolved oxygen (D.O.). In addition to D.O., salinity, temperature, and water level data were also collected at these sites. The US Fish and Wildlife Service, at the same time, began a study and collection of data of porewater salinity and associated plant species growth in the wetland areas.

Figure 7.1-1: Data Collection Station Locations



7.2 MODEL SELECTION

The Modeling Technical Review Group was setup by GPA in the late 1990s to review the model for the deepening project and determine its viability for use.

The MTRG consisted of technical modelers from federal and state agencies. Federal agencies involved included the USEPA, USF&W, USGS, NMFS (National Marine Fisheries) and the Corps of Engineers. State agencies involved included Georgia EPD and South Carolina DHEC.

The MTRG determined a previous model, developed by consultants to GPA, was not defensible for salinity intrusion and water quality effects. The model had difficulties with the vertical mixing scheme and was not accepted by the Federal agencies reviewing the project. In response, the agencies prepared a Federal Expectations Document in 2003 that described:

- the resources of primary concern in the estuary,
- the locations and conditions under which project impacts should be evaluated for those resources,
- the modeling approach to be taken,

- the statistical analyses to be performed to document the model's performance,
- the evaluation criteria.

The Expectations Document stated that its listed criteria were to be viewed as performance goals to which model predictions would be compared and evaluated for strengths and weaknesses and by which an understanding of their uncertainties may be developed. The stated criteria would not be used individually (by station and parameter) for a "pass/fail" evaluation of the model calibration and/or any post-processing routine. The Federal Expectations Document titled, *Savannah Harbor Data Analysis & Modeling Expectations of Federal Agencies,* is included in the Engineering Investigations Supplemental Materials.

The models used to assess the environmental impacts of the SHEP are:

Environmental Fluid Dynamics Computer Code (EFDC) – EFDC was selected to perform the hydrodynamic simulations because it meets the agencies expectations and project study goals. Consideration of the hydrodynamic/water quality linkage was also a concern during model selection. The EFDC model is a part of the EPAs TMDL Modeling Toolbox due to its application in many TMDL-type projects. As such, the code has been peer reviewed and tested and has been freely distributed for public use. The EFDC model is nonproprietary and publicly available through USEPA Region 4 and USEPA Office of Research and Development. EFDC was developed by Dr. John Hamrick and is currently supported by Tetra Tech.

The EFDC model comprises an advanced three-dimensional surface water modeling system for hydrodynamic and reactive transport simulations of rivers, lakes, reservoirs, wetland systems, estuaries and the coastal ocean. EFDC is a multifunctional, surface water modeling system, which includes hydrodynamic, sediment-contaminant, and eutrophication components. The model's hydrodynamic component is based on the three-dimensional shallow water equations and includes dynamically coupled salinity and temperature transport. The model also includes representation of hydraulic structures for controlled flow systems.

Water Quality Analysis Simulation Program (WASP) – WASP Version 7.0 was selected for the water quality model development. WASP7 was released by EPA on April 27, 2005 and is part of the TMDL Modeling Toolbox. WASP is a dynamic compartment-modeling program for aquatic systems, including both the water column and the underlying benthos. The time-varying processes of advection, dispersion, point and diffuse mass loading and boundary exchange are represented in the program. The water quality model incorporates normal oxygen dynamics, including reaeration, sediment oxygen demand, carbonaceous Biochemical Oxygen Demand and uptake, and Nitrogenous Biochemical Oxygen Demand and uptake.

The EFDC model provides ocean flow and tidal dynamics, upstream flow, 3D model cell structure and volumes, cell volumes, and transport and salinity and temperature. The hydrodynamic modeling information is incorporated into the WASP model through the hydrodynamic linkage file. Both of these models are certified for use by the US Army Corps of Engineers, in accordance with the Enterprise Standard (ES)-08101, Softw*are Validation for the Hydrology, Hydraulics, and Coastal Community of Practice*.

7.3 MODEL DEVELOPMENT & CALIBRATION

7.3.1 Model Grid Development

The model grid for both EFDC and WASP was enhanced from modeling previously developed and performed by ATM. The enhanced models were developed in consideration of the following efforts:

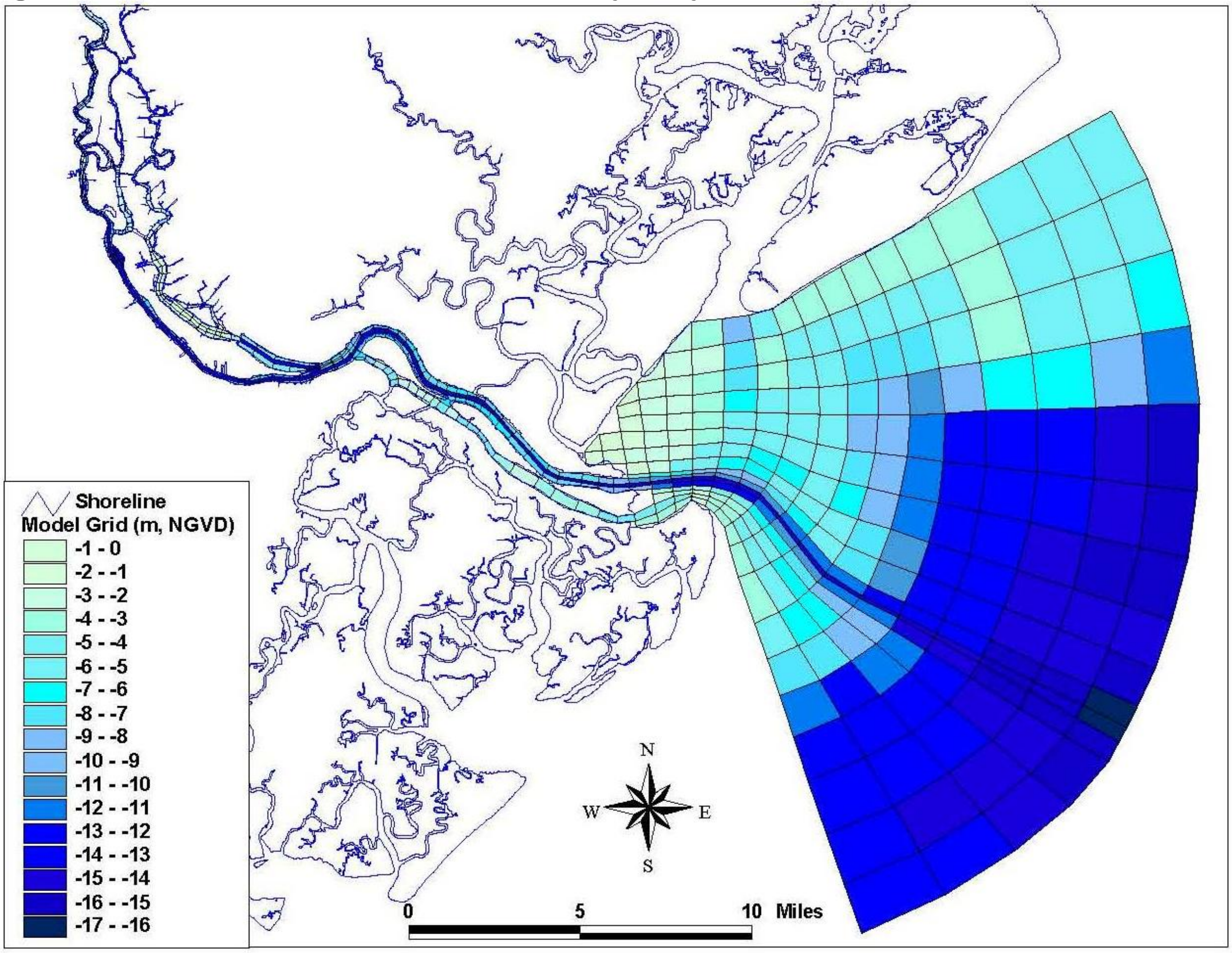
- US Army Corps of Engineers Savannah Harbor Ecosystem Restoration Project,
- Finalization of the EPA Region 4 Dissolved Oxygen TMDL,
- The States of Georgia and South Carolina issuing National Pollutant Discharge Elimination System (NPDES) permits.

EPA Region 4 issued the draft dissolved oxygen TMDL on August 30, 2004, and the public notice/comment period ended on January 31, 2005. The TMDL models were run on a coarse grid in which the EFDC hydrodynamic and the WASP water quality models were applied. The coarse grid, now referred to as the TMDL grid, met the following objectives defined by EPA:

- To represent accurately the key hydrodynamic processes of transport in the estuary,
- To utilize a model that is public domain and has been peer reviewed,
- To deliver the model to the federal agencies involved in the TMDL process,
- To run the model for multiple hydrologic periods and evaluate point and nonpoint sources, and
- To complete the effort in a timely manner in order to meet the project schedule.

The effort to develop an enhanced grid was initiated on September 29, 2004 to improve the representation of the estuary system and navigation channel. The enhanced grid is designed to allow evaluation of various scenarios such as deepening of the navigation channel and physical modifications to certain cuts and channels in the river and estuary. The major enhancements included developing a finer model grid, updating the bathymetric data used by the model, and an alternate approach for the model calculation of the river-marsh interactions. The same models, EFDC and WASP, were used on the TMDL grid and the enhanced grid. **Figure 7.3.1-1** shows the enhanced model grid and bathymetry.

Figure 7.3.1-1: Enhanced Model Grid and Bathymetry

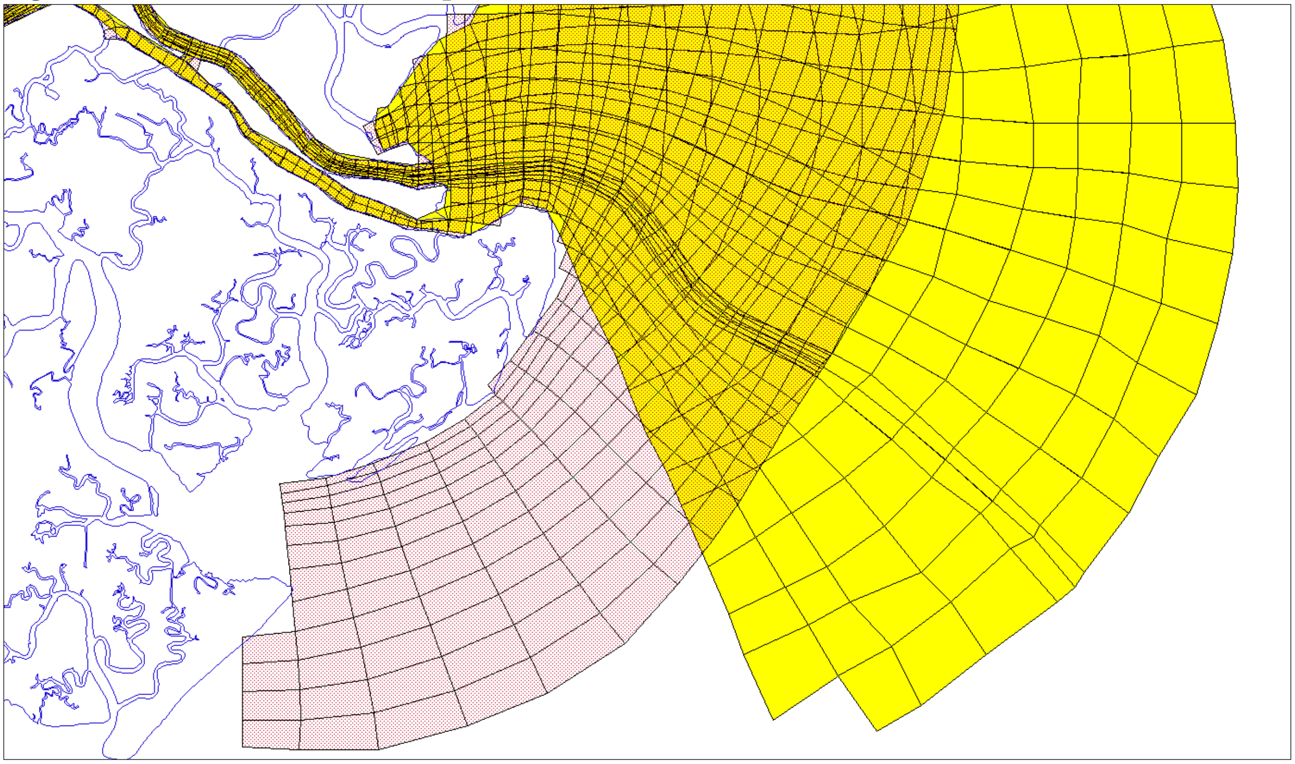


Specifically, the model grid was enhanced to include more cells in the navigation channel. There are now 931 horizontal cells that extend upstream to Clyo, Georgia (~61 miles from Fort Pulaski) and downstream to the Atlantic Ocean (~17 miles offshore from Fort Pulaski). Including the marsh cells adjacent to the river system, there are 947 total cells. The man-made connections, such as McCoys Cut, Rifle Cut, Drakies Cut, New Cut as closed, and the sill of the tidegate, were also included in the enhancement.

The original EFDC model grid developed for EPA Region 4 for the TMDL extended from the mouth of the river down the southern shoreline (See **Figure 7.3.1-2**, pink). This grid configuration had difficulty mimicking the water surface elevations in the area because the tidal forcing conditions were "bouncing" off the shoreline or back of the grid. Having tides as forcing functions on the south and east sides of the grid would create instabilities in the model.

This was overcome by reshaping the grid and allowing the reflective wave to propagate out of the model domain along the shoreline by incorporating externally specified radiation stresses. Alternatively, the shoreline boundary could have been extended further south to include the additional tidal creeks in the Ossabaw Sound, but there was no value in enlarging the model grid in this way. The purpose of the grid boundary conditions and termination 17 miles offshore was to propagate a tide and salinity condition to match the observed measurements at the mouth of the Savannah River. To be clear, the model does not have an open boundary condition on the southern end of the EFDC and WASP model grid.

Figure 7.3.1-2: Model Grid Comparison



Original model grid extending southward is shown in pink and the TMDL grid is shown in yellow. The SHEP model was based enhancements to the TMDL grid.

Additionally, there are shelf circulation patterns that are not captured or simulated by the SHEP model grid despite the tidal boundary being 17 miles offshore which affect the salinity regime. It is recognized that there are seasonal variations in continental shelf salinity due to Savannah River freshwater discharge. This was documented in research by J. O. Blanton and others, published in the Journal of Geophysical Research, 88(C8):4705-4718. This topic was presented to the MTRG and the group agreed that the tide and salinity propagation at the mouth was most important and that the seaward boundary, 17 miles offshore, is appropriate for the goals set by the MTRG for the SHEP model. The logic for this decision is that, for future with-project salinity projections, the MTRG wanted to be sure that modeled changes to salinity were due to the proposed deepening project and mitigation features, and not due to seasonal variation in the boundary condition.

Grid convergence and orthogonality were key considerations for developing a defensible model for SHEP application. Orthogonality is balanced between model grid cell size and model run time. The final grid is representative of this balance, sacrificing some orthogonality to achieve the overall goals of a representative model with workable model run times.

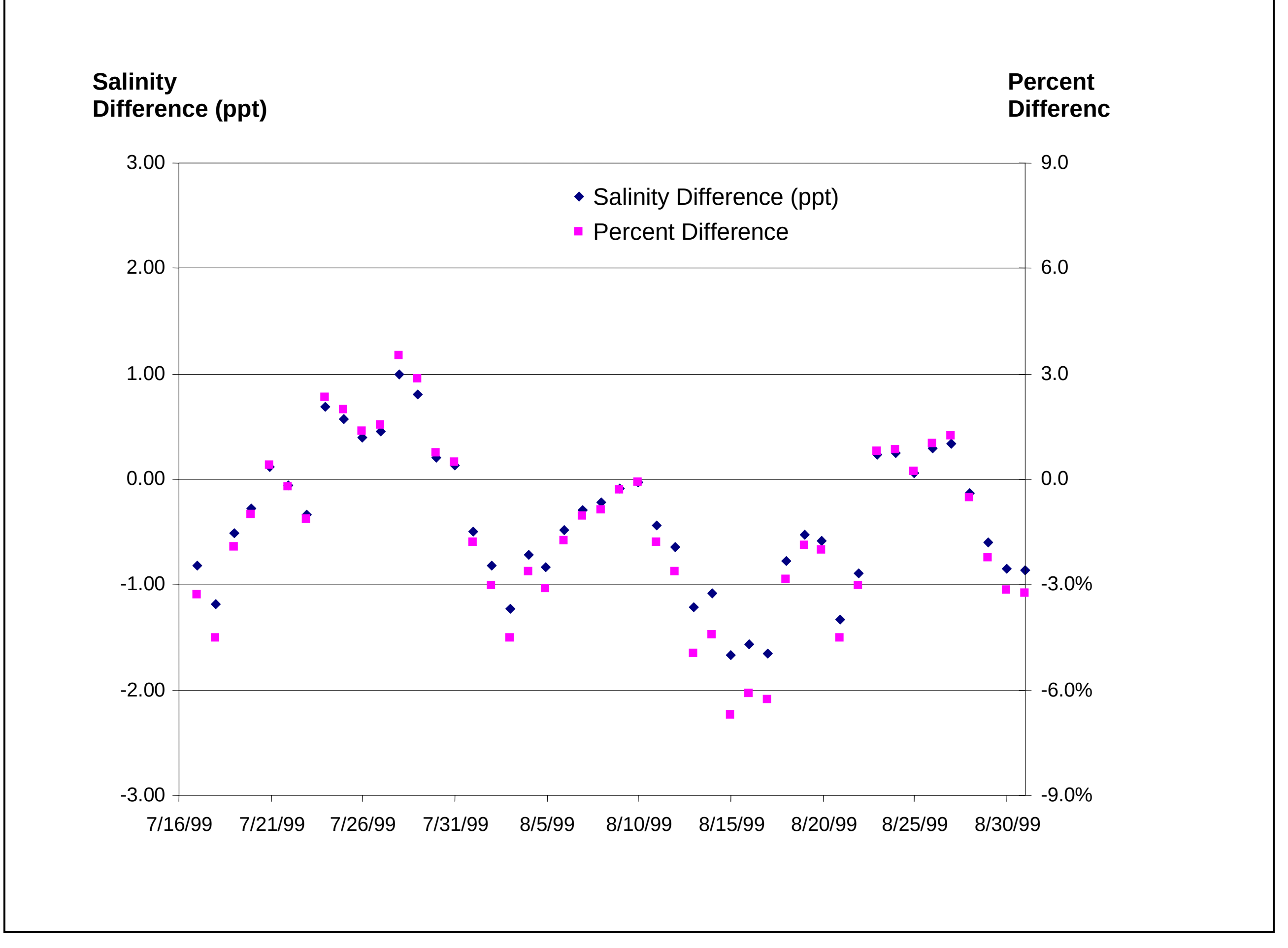
Quantification of the convergence grid test results has been performed and is presented in the **Table 7.3.1-1**. Model spin up has been excluded from these statistics.

Table 7.3.1-1: Quantification of the Convergence Grid Test Results

Site	Layer	Enhanced Grid Average Salinity (ppt)	Convergence Grid Average Salinity (ppt)	Average Difference (ppt)	Average Percent Difference
FR-09	Bottom	27.27	26.83	-0.43	-1.6%
FR-09	Surface	6.78	5.69	-1.09	-16.2%
SR-17	Bottom	0.005	0.006	0.001	21.9%

Figure 7.3.1-3 shows the daily average bottom salinity difference and percent difference at data collection Station FR-09 (see **Figure7.1-1** for FR-09 location). The figure shows no consistent trend of difference (no divergence with time). The minor difference in system response for each grid may depend more on hydrologic or tidal conditions.

Figure 7.3.1-3: Salinity and Percent Difference of Grid Convergence Results



7.3.2 Bathymetry

The bathymetry data was obtained from several sources because there is not one continuous bathymetry dataset that encompasses the entire system. To simply prescribe a channel design template onto the model grid does not adequately reflect the continuous sedimentation and dredging that is ongoing in the harbor. The sources are as follows:

- USACE Annual Surveys (1999 and 2002)
- USGS SNWR (2004) for the Back, Middle and Little Back Rivers
- USACE Upstream of I-95 (1999)
- NOAA Surveys (1980s) for the offshore non-channel and south channel areas.

By using the bathymetry datasets described above, the SHEP model includes advanced maintenance and overdredge depths for all model scenarios. The model accounts for the overdredge volume in the navigation channel by assuming the overdredge is the same in the 42-foot channel (existing conditions) as it would be in any dredged channel depth.

Furthermore, since advance maintenance is proposed to be essentially the same for deepened conditions as for existing conditions, then we subtracted 2 feet for the 44-foot depth, 3 feet for the 45-foot depth, 4 feet for the 46-foot depth, 5 feet for the 47-foot depth, and 6 feet for the 48-foot depth bathymetry inputs. All depths reference Mean Lower Low Water (MLLW). The reality in the model is that the 48-foot depth is closer to 52 feet which includes the additional depths due to advance maintenance and overdredge.

Figures 7.3.2-1 through **7.3.2-3** illustrate how the bathymetry is described in the model. **Figure 7.3.2-1** shows actual measured data from the USACE 1999 Annual Survey. **Figure 7.3.2-2** shows the cross-section in the model at the same location and **Figure 7.3.2-3** represents the modeled 6-foot deepened conditions.

Figure 7.3.2-1: Existing Depth from 1999 Annual Survey at Station 96+000

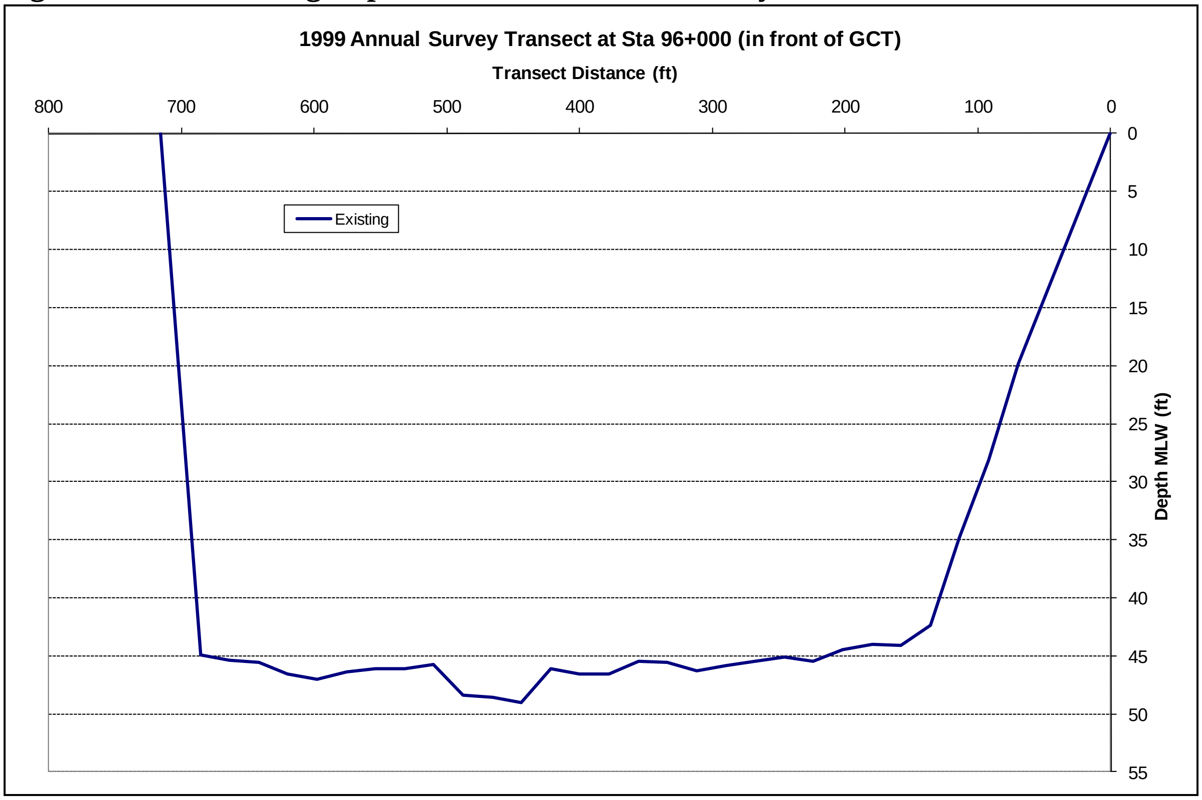


Figure 7.3.2-2: Existing Conditions Represented in the EFDC model

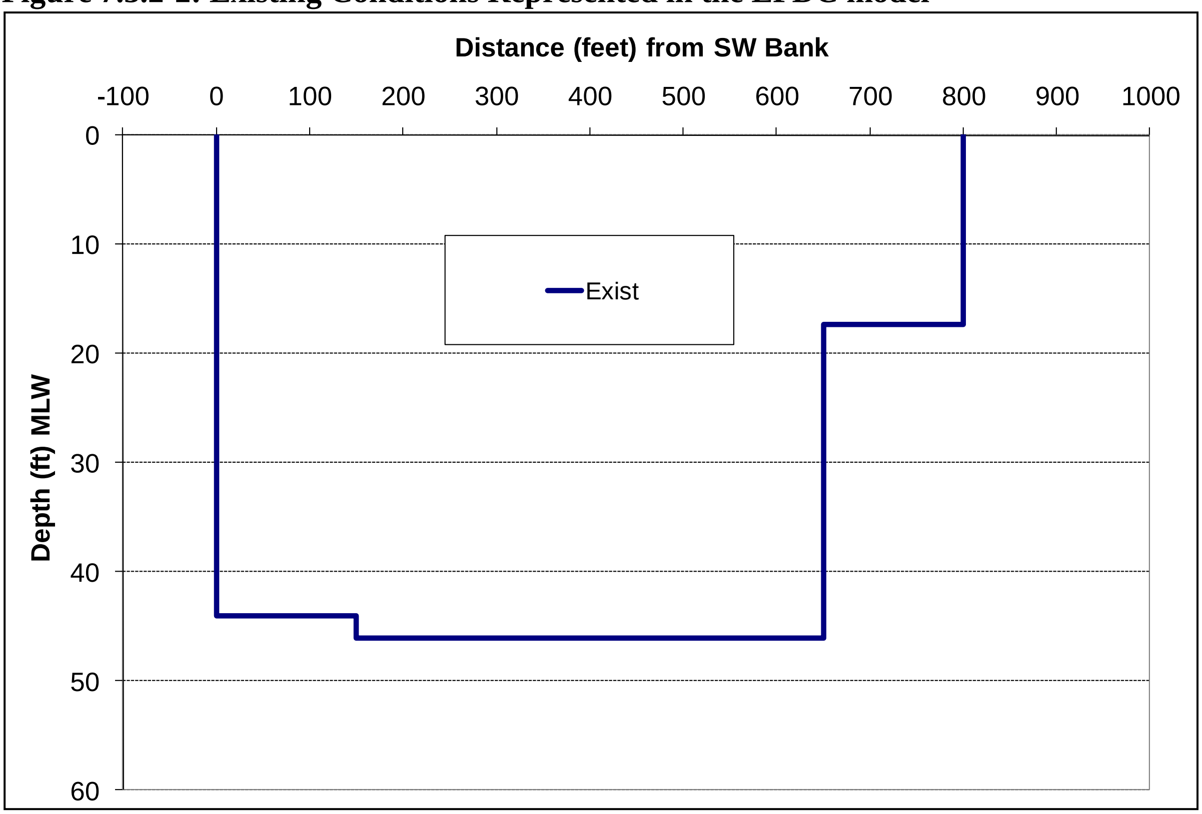
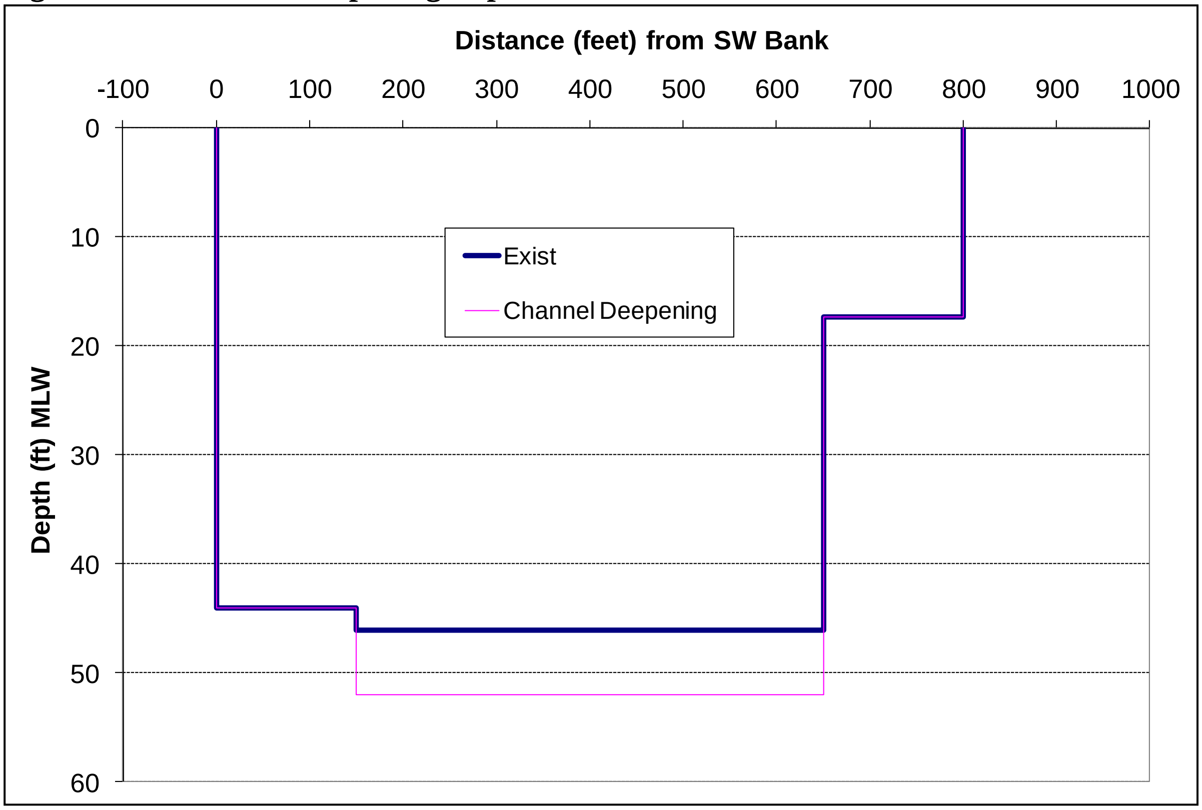


Figure 7.3.2-3: 6-foot Deepening Represented in the EFDC model



7.3.3 Water Quality Modeling Parameters

7.3.3.1 Nutrient Flux and Sediment Oxygen Demand

Ammonia fluxes were insignificant and not included in the model. Based on the sediment oxygen demand (SOD) and nutrient flux measurements collected by EPA SESD in 1999, the ammonia fluxes measured were close to zero due to the amount of scour and movement in bed materials in the navigation channel. Data sources of for SOD and nutrient fluxes include:

- *Oxygen Diffusion Study and Sediment Oxygen Demand Study*, Savannah River, Savannah, Georgia (August 2-14, 1999, EPA Science and Ecosystem Support Division, Ecological Assessment Branch, Athens, Georgia
- *Savannah River Classification Study – October 1985*, Sediment Oxygen Demand Surveys, Summary,1985, GAEPD, Atlanta Ga.
- *Application of CE-QUAL-W2 to the Savannah River Estuary*, Technical Report EL-87-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS (Hall, Ross W., 1987)

The ammonia data in the vicinity of point sources is higher than background, but everywhere else in the system, the ammonia concentrations are near detection limits at 0.02 to 0.03 mg/L. The MTRG analyzed the SOD and flux data and concluded ammonia fluxes were not significant to include in the WASP model. However, a spatial representation of SOD for the WASP model was included.

7.3.3.2 Algal Activity

The MTRG decided in 2005 that excluding algae, and therefore not using the full eutrophication model in WASP, was appropriate for SHEP. There is limited to no chlorophyll-a data measured in the harbor. Through data analysis and model simulations, it was determined that algal activity effects on dissolved oxygen are negligible.

As stated in the model calibration report prepared by Tetra Tech titled *Development of the Hydrodynamic and Water Quality Models for the Savannah Harbor Expansion Project*, "Since there is limited algal activity or primary production in the harbor, nutrients were determined not to be a significant issue by EPA Region 4 and were not included in the water quality modeling scenarios. The water quality model incorporated normal oxygen dynamics, including reaeration, sediment oxygen demand (SOD), carbonaceous Biochemical Oxygen Demand (CBOD) and uptake, and Nitrogenous Biochemical Oxygen Demand (NBOD) and uptake."

The EPA Science and Ecosystem Support Division (SESD) conducted a light-dark bottle algal study and showed no algal growth in the system due to color and fast flushing times. With the limited chlorophyll-a data, there were low levels in the surface. Temporary and sporadic elevated levels were found in bottom layers due to intrusion from the ocean.

The EPA SESD field crews had great difficulty obtaining nutrient flux measurements in the harbor, but NH_3 fluxes are not expected in the harbor because of the high velocity on the bottom. Also, the harbor is relatively insensitive to ammonia because of low decay rates due to high ocean water dilution. That said, the river model was sensitive to ammonia but again no fluxes measured or expected due to high velocities.

In addition, the dissolved oxygen signal closely matches the tidal signal. The reason being because the tidal signal is strong and the reaeration (due to turbulence and wind) cause the dissolved oxygen to rise and fall. Temperature plays an important role along with the BOD loading, low oxygen in the marshes, and SOD. Therefore, the algal growth effects due to nutrients is limited or very small compared to velocity, turbulence, wind, BOD loads, SOD, and marshes.

7.3.4 Calibration

The model calibration details can be found in the report prepared by Tetra Tech titled *Development of the Hydrodynamic and Water Quality Models for the Savannah Harbor Expansion Project* dated January 2006 which is included in the Engineering Investigations Supplemental Materials. The model development and calibration report includes both the hydrodynamic and water quality modeling results along with calibration and confirmation periods. The calibration of the models was performed to the summer of 1999 data, the period with the most comprehensive dataset. The confirmation of the model was performed to the summer of 1997 data and the USGS long-term data from January 1, 1997 through December 31, 2003.

The model calibration had two primary goals: (Task A) to modify and recalibrate the EFDC model and (Task B) to re-evaluate the calibration of the WASP water quality model, if needed, because of revisions to the EFDC hydrodynamic model. The objectives of Task A were to modify the EFDC model to improve the grid resolution, tidal-marsh interaction, and boundary conditions in response to issues raised during the SHEP federal and state technical review of the initial calibration report on the

EFDC portion of the TMDL grid. The objective of Task B was to re-evaluate the calibration of the WASP model for use in predicting dissolved oxygen in the harbor and use in future SHE Project alternatives. The USACE Savannah District along with the other Federal agencies believed that modifications to the TMDL grid of the EFDC and WASP models might also enhance the TMDL Models' capabilities.

Calibration methodology for the EFDC model included graphical time series comparisons (qualitative) and statistical calculations (quantitative). The calibration was also parameter specific including water surface elevation, currents, flow, temperature and salinity. The order in which the hydrodynamic model is calibrated is performed to address issues such as bathymetry, friction, tidal volume, cross-sectional area, and heat budget before salinity is calibrated. Salinity is the predominant signal in the model to ensure that mass is being moved horizontally and vertically with the appropriate timing and direction.

Model calibration was further improved through coordination with scientists at the Skidaway Institute of Oceanography in Savannah, GA and utilization of a time series boundary condition of salinity based on the SABSOON data (Station R2) which captured the phenomenon of high river flow events producing lower salinity at the boundary of the model**.** South Atlantic Bight Synoptic Offshore Observational Network (SABSOON) is a real-time observational network developed on the US Southeastern continental shelf providing a range of oceanographic and meteorological observations on a continuous real-time basis. It was very important to the MTRG to have a defensible boundary that would be consistent for multi-year, multi-scenario applications. The seaward boundary is appropriate for the goals set by the MTRG for the SHEP model.

The calibration salinity boundary was determined to be a best-fit linear function from 32.5 ppt (surface) to 35 ppt (bottom). Datasets available from NOAA for the "World Ocean Atlas" annual means, suggest that regional annual mean value of surface salinity may be in the range 34-36 ppt. For comparison, data from SABSOON site R2 which is located approximately 50 miles offshore from the mouth of the Savannah River indicate mean surface salinity of 36.0 ppt (range 31.5 - 36.5 ppt for the period 1999-2002).

To assess model sensitivity and the possibility of improving the calibration, the EFDC model was run for 35 ppt (surface to bottom) and 36 ppt constant boundary conditions. Results were increased salinity in the lower Front River both at the surface and the bottom. As expected, predicted salinity was increased more at Ft. Pulaski (FR-26) than upstream at sites such as FR-08, for example. Results are shown in **Figures 7.3.4-1** through **7.3.4-3** for FR-26 and FR-08. Increasing the offshore boundary condition for salinity does not improve the calibration.

Figure 7.3.4-1: Salinity Comparisons at FR-26 at the Surface

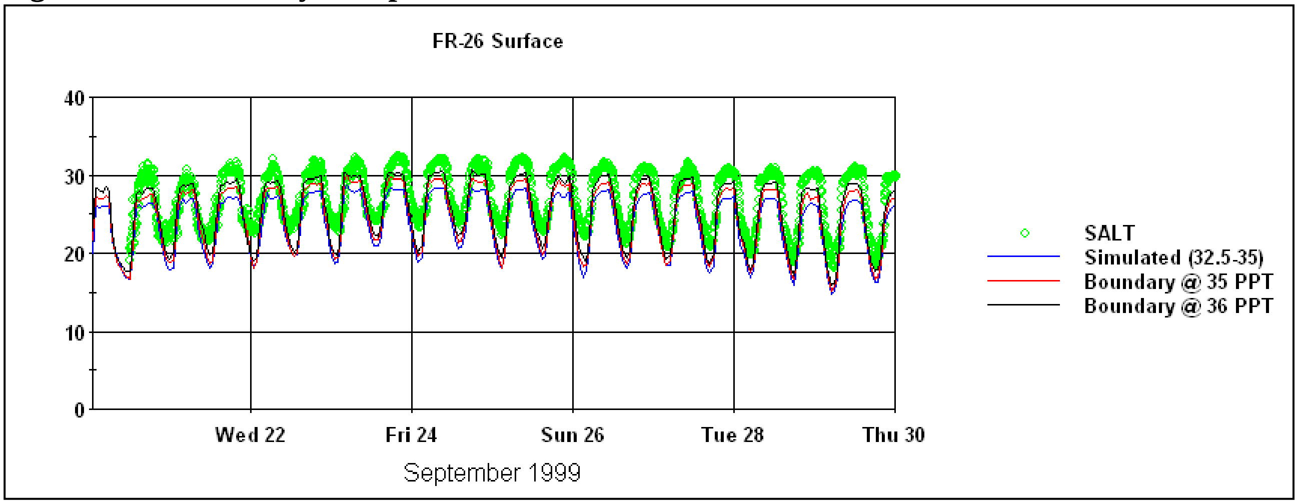


Figure 7.3.4-2: Salinity Comparisons at FR-26 at the Bottom

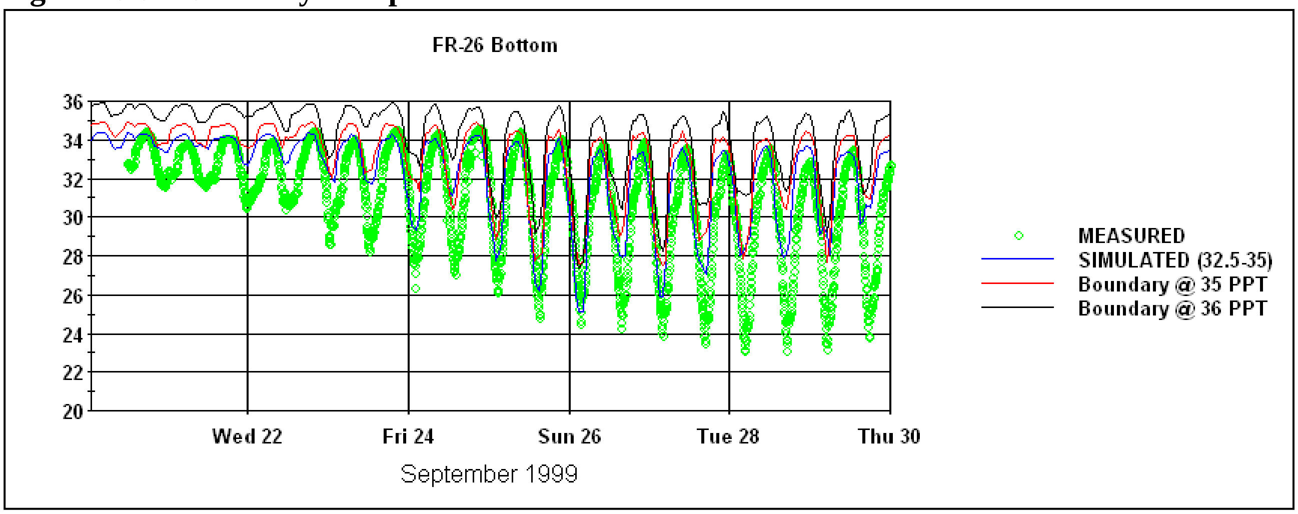
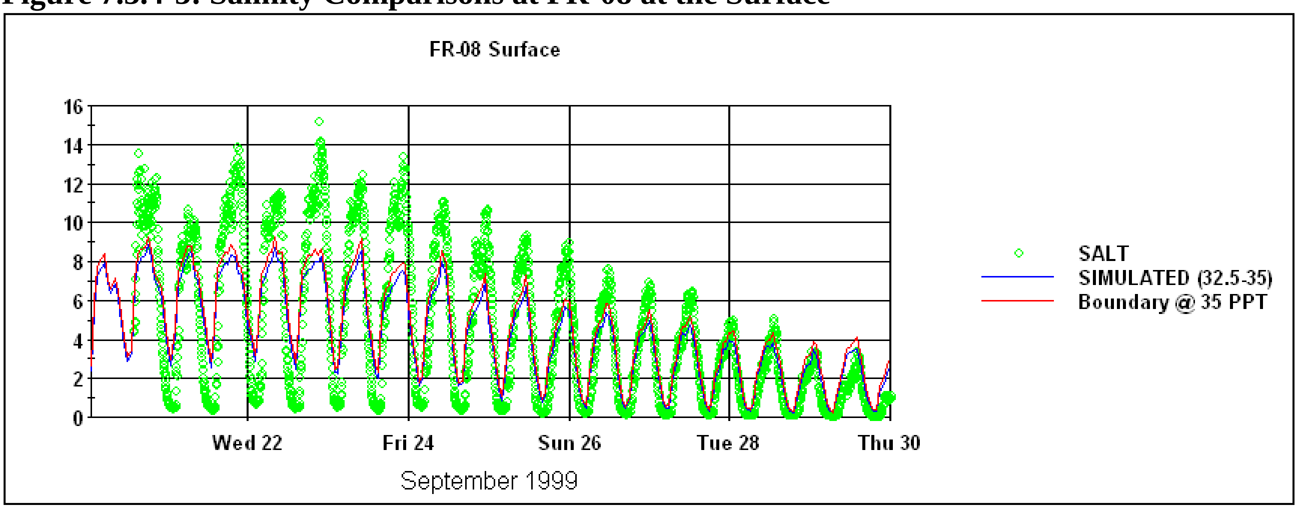


Figure 7.3.4-3: Salinity Comparisons at FR-08 at the Surface



The WASP model calibration was performed using the summer 1999 dataset. The EFDC model was used to provide the model grids, depths, volumes, velocities, and diffusion parameters along with the predicted temperatures and salinities. The main calibration parameters were minor adjustments to SOD and the reaeration scaling factor. The measured values from the data collected during the 1999 summer survey were used for calibration of the WASP model. The data consisted of dissolved oxygen, BOD, ammonia, other nutrients, and chlorophyll-a concentrations. Specifically, for the WASP model calibration, dissolved oxygen, BOD and ammonia were used.

7.3.5 Model Grid Comparison

The hydrodynamic and water quality models as developed for use in SHEP impact determination and mitigation development design were determined to be acceptable for this purpose in March 2006 by the MTRG and the state and federal agencies involved in the group including, USEPA, USFWS, GAEPD, and SCDHEC. This set of models is described as enhanced from the previously developed models in the late 1990s by ATM at the direction of GPA.

The enhanced grid is designed to allow evaluation of various scenarios such as deepening of the navigation channel and physical modifications to certain cuts and channels in the river and estuary. The major enhancements included developing a finer model grid, updating the bathymetric data used by the model, and an alternate approach for the model calculation of the river-marsh interactions.

Over the course of time, the enhanced grid came to be known as the Sigma model which is a terrain following vertical grid which allows the layers to compress and expand (stretch) with changing water surface elevation. During 2007, EPA Regoin 4 determined a need to convert the Sigma model grid (ie, the enhanced model used for SHEP purposes) to a Z-Grid. The Z-Grid model allows for variations in the number of vertical layers throughout the model domain. The Sigma Grid has six vertical layers with widely varying layer depths and the Z-Grid model has five vertical layers in the navigation channel and one vertical layer in the Middle, Back, Little Back and Upper Savannah Rivers. A benefit of the Z-Grid is that there are fewer computations over the same model domain which reduces run times significantly. When numerous runs are required, the Z-Grid can offer substantial time savings. Additionally, the Z-Grid was later updated with new Middle River and Back River bathymetry and recalibrated to 2009 USGS velocity and flow data. This newer version of the model, known now as the Z-Grid model, was further enhanced in 2010 to be used for SHEP impacts to chlorides on Abercorn Creek.

The versions of the model were compared and results from the analysis can be found in the report titled *Model Comparison Report in support of the Savannah Harbor Expansion Project* dated July 2011 and included in the Engineering Investigations Supplemental Materials. The report, prepared by Tetra Tech, documents the differences between the two models and ultimately recommends continuing to use the Sigma model for the impacts determination and mitigation development for harbor deepening.

7.4 IMPACTS DETERMINATION

In support of the Environmental Impact Statement (EIS), the hydrodynamic and water quality models were used to evaluate the impacts from deepening the navigation channel to the environment and natural resources within the Savannah River Estuary. The Interagency Coordination Team developed parameters and run scenarios for use in the models as a way to characterize and determine impacts. The Interagency Coordination Team is comprised of biologists, physical scientists, resource specialists, and engineers from both federal and state agencies that are charged with reviewing the details of the SHEP and giving environmental clearances. The team developed four basic environment/resource categories that were evaluated:

- Fishery Habitat
- Water Quality and Dissolved Oxygen
- Wetlands
- Chlorides

Each of the four categories has several run scenarios and parameter evaluation requirements used to determine impacts for that specific category. Rarely did the run scenarios overlap between categories. The result is numerous data plots, charts, figures and tables that show the results of the model run and quantitatively describe the impacts. The output is cumbersome and highly detailed and was purposefully done in an attempt to identify any and all impacts on habitat and natural resources under a wide variety of conditions in support of the needs of the Interagency Coordination Team.

7.4.1 Post Processor

To aid in quantifying the impacts, a program was developed by Tetra Tech that reads the complex model output from either the hydrodynamic (EFDC) or water quality (WASP) model, which are in BMD file formats, then analyzes and performs statistics and produces results that are more easily interpreted. The Post Processor has several modules that can calculate averages, percentiles, maximums, and minimums for a variety of parameters with speed and accuracy. The same observations and calculations could have been made manually, but with the size of the model grid and the vast amount of output generated by the model, that task would have been daunting. The Post Processor has proven to be an invaluable tool in analyzing such large datasets.

Outside of the Corps' acceptance and use of the Post Processor for the Expansion Project, the program has also been used by Tetra Tech and EPA Region 4 to determine TMDL scenarios for meeting water quality standards. Both Georgia EPD and South Carolina DHEC are also using the tool to compare and develop water quality standards for the harbor. In addition to Tetra Tech's internal peer review of the Post Processor, there have been several other modelers/engineers with federal and state agencies that have peer reviewed the tool through their use and application.

7.4.2 Fishery Habitat Modeling

The fishery habitat modeling required model runs to be developed around critical habitat times for various fish species and life cycle stages. Specifically, the species and life stages identified by the Interagency Coordination Team are:

- American Shad
- Striped Bass (eggs, larvae, and spawning)
- Shortnose sturgeon (juveniles and adults)
- Southern Flounder

Each model run for fishery habitat was one month in duration, plus 30 days for model-spin up, during the outlined critical habitat periods. Freshwater flow conditions at the upstream boundary for each run were based on long term percentiles at the USGS stream gage near Clyo, GA. Many species were evaluated for low, average, and high water flow conditions. Parameters evaluated included, salinity, DO, and velocity at various depths over the water column. All parameters, model run scenarios and habitat suitability criteria were developed by the Interagency Coordination Team. See **Table 7.4.2-1** for details.

7.4.2.1 Findings

Impacts determination for fishery habitat required both the EFDC model and WASP model runs; EFDC to predict hydrodynamics and salinity and WASP to predict dissolved oxygen. To aid in evaluation of the output and determination of habitat suitability, the Post Processor Habitat Analysis Module was utilized. This module takes the model output from the EFDC and WASP models and determines habitat suitability based on criteria outlined for each species and life cycle stage. The results include numerous maps and figures showing areas of suitable and unsuitable habitat for each species and life stage. Details of the fishery habitat impact analysis can be found in the report titled *Habitat Impacts of the Savannah Harbor Expansion Project* which is included in the Engineering Investigations Supplemental Materials. A detailed analysis of the results can be found in the EIS. Tables showing the relative acreages were also developed and a summary can be found in **Table**s **7.4.2.1-1** though **7.4.2.1-4**.

Table 7.4.2-1: Fishery Habitat Suitability Criteria

Species & Life Stage	Freshwater Flow Conditions	Simulation Period	Habitat Criteria
Striped Bass (spawning)	20%-tile of Long Term	April	Suitable habitat when (1) 90th percentile salinity <= 1 ppt, and (2) Mean velocity >= 30 cm/s
	50%-tile of Long Term	April	
	80%-tile of Long Term	April	
Striped Bass (eggs)	20%-tile of Long Term	April	Suitable habitat when (1) Mean 50th percentile salinity <= 9 ppt, (2) Mean velocity >= 30 cm/s, and (3) 10th percentile D.O. >= 4.5 mg/l
	50%-tile of Long Term	April	
	80%-tile of Long Term	April	
Striped Bass (larvae)	20%-tile of Long Term	May	Suitable habitat when (1) Mean 50th percentile salinity between 3 and 9 ppt, and (2) Mean 10th percentile D.O. >= 4.5 mg/l
	50%-tile of Long Term	May	
	80%-tile of Long Term	May	
Southern Flounder	50%-tile of Long Term	August	Suitable habitat when DO >= 4.0 mg/l at 90% exceedance (10th percentile)
American Shad	50%-tile of Long Term	January	Suitable habitat when D.O. >= 4.0 mg/l at 90% exceedance (10th percentile)
	50%-tile of Long Term	May	
	50%-tile of Long Term	August	
Shortnose Sturgeon (adult)	50%-tile of Long Term	January	Suitable habitat when DO >= 3.5 mg/l at 90% exceedance (10th percentile), >=3.0 at 95% (5th percentile), and >=2.0 at 99% (1 percentile) Suitable habitat when Max Salinity <= 25 ppt
Shortnose Sturgeon (adult)	50%-tile of Long Term	August	Suitable habitat when DO >= 4.0 mg/l at 90% exceedance (10th percentile), >=3.0 at 95% (5th percentile), and >=2.0 at 99% (1 percentile) Suitable habitat when Max Salinity <= 10 ppt
Shortnose Sturgeon (juvenile)	50%-tile of Long Term	January	Suitable habitat when DO >= 3.5 mg/l at 90% exceedance (10th percentile), >=3.0 at 95% (5th percentile), and >=2.0 at 99% (1 percentile) Suitable habitat when 50% exceedance of the Max Salinity is <= 14.9 ppt

Table 7.4.2.1-1: Fishery Impacts – American Shad (Deepening Only)

	May20%flows		May50%flows		May80%flows	
	IMPACTS (%)	**IMPACTS (acres)**	**IMPACTS (%)**	**IMPACTS (acres)**	**IMPACTS (%)**	**IMPACTS (acres)**
44 ft depth	*0.0%*	***0***	*0.0%*	***0***	*0.0%*	***0***
45 ft depth	*0.0%*	***0***	*0.0%*	***0***	*0.0%*	***0***
46 ft depth	*0.0%*	***0***	*0.0%*	***0***	*0.0%*	***0***
47 ft depth	*0.0%*	***0***	*0.0%*	***0***	*0.0%*	***0***
48 ft depth	*0.0%*	***0***	*0.0%*	***0***	*0.0%*	***0***
	January50%flows		**August Avg flows**			
	IMPACTS (%)	**IMPACTS (acres)**	**IMPACTS (%)**	**IMPACTS (acres)**		
44 ft depth	*0.0%*	***0***	*0.0%*	***0***		
45 ft depth	*0.0%*	***0***	*0.0%*	***0***		
46 ft depth	*0.0%*	***0***	*0.0%*	***0***		
47 ft depth	*0.0%*	***0***	*0.0%*	***0***		
48 ft depth	*0.0%*	***0***	*0.0%*	***0***		

Table 7.4.2.1-2a: Fishery Impacts – Striped Bass Eggs (Deepening Only)

	April20%flows		April50%flows		April80%flows	
	IMPACTS (%)	**IMPACTS (acres)**	**IMPACTS (%)**	**IMPACTS (acres)**	**IMPACTS (%)**	**IMPACTS (acres)**
44 ft depth	*-10.4%*	***-100***	*-9.7%*	***-163***	*-2.2%*	***-50***
45 ft depth	*-12.3%*	***-118***	*-11.2%*	***-188***	*-4.9%*	***-111***
46 ft depth	*-14.0%*	***-135***	*-15.9%*	***-266***	*-4.8%*	***-108***
47 ft depth	*-17.8%*	***-171***	*-20.5%*	***-344***	*-6.4%*	***-144***
48 ft depth	*-19.4%*	***-187***	*-24.5%*	***-411***	*-7.2%*	***-162***

Table 7.4.2.1-2b: Fishery Impacts – Striped Bass Larvae (Deepening Only)

	May20%flows		May50%flows		May80%flows	
	IMPACTS (%)	**IMPACTS (acres)**	**IMPACTS (%)**	**IMPACTS (acres)**	**IMPACTS (%)**	**IMPACTS (acres)**
44 ft depth	*38.0%*	***76***	*-13.5%*	***-76***	*-1.1%*	***-11***
45 ft depth	*56.4%*	***113***	*-18.6%*	***-105***	*-7.1%*	***-71***
46 ft depth	*99.5%*	***199***	*-21.0%*	***-119***	*-4.8%*	***-48***
47 ft depth	*105.8%*	***211***	*-13.8%*	***-78***	*6.6%*	***66***
48 ft depth	*104.6%*	***209***	*-13.8%*	***-78***	*6.0%*	***60***

Table 7.4.2.1-2c: Fishery Impacts – Striped Bass Spawning (Deepening Only)

	April20%flows		April50%flows		April80%flows	
	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*-7.6%*	***-49***	*-8.0%*	***-83***	*-6.2%*	***-114***
45 ft depth	*-10.9%*	***-70***	*-12.2%*	***-127***	*-6.6%*	***-121***
46 ft depth	*-12.7%*	***-81***	*-13.0%*	***-135***	*-12.8%*	***-236***
47 ft depth	*-14.3%*	***-91***	*-18.1%*	***-188***	*14.7%*	***271***
48 ft depth	*-16.9%*	***-108***	*-19.7%*	***-205***	*-17.3%*	***-318***

Table 7.4.2.1-3: Fishery Impacts – Shortnose sturgeon (Deepening Only)

	JUVENILES		ADULTS			
	January50%flows		January50%flows		August Avg flows	
	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*-5.0%*	***-86***	*-0.5%*	***-20***	-3.2%	***-45***
45 ft depth	*-10.4%*	***-179***	*-0.5%*	***-20***	-6.4%	***-89***
46 ft depth	*-15.9%*	***-274***	*-0.8%*	***-32***	-9.5%	***-132***
47 ft depth	*-19.0%*	***-328***	*-0.8%*	***-32***	*-13.3%*	***-185***
48 ft depth	-21.6%	***-373***	-1.1%	***-44***	-15.8%	***-220***

Table 7.4.2.1-4: Fishery Impacts – Southern Flounder (Deepening Only)

	August Avg flows	
	IMPACTS (%)	IMPACTS (acres)
44 ft depth	-0.3%	***-6***
45 ft depth	-2.4%	***-45***
46 ft depth	-2.4%	***-45***
47 ft depth	-7.8%	***-146***
48 ft depth	0.0%	***0***

7.4.3 Water Quality & Dissolved Oxygen Modeling

To determine water quality impacts from SHEP, specifically changes in dissolved oxygen, both EFDC and WASP models were run; EFDC for hydrodynamic and salinity predictions and WASP for dissolved oxygen predictions. The Interagency Coordination Team specified run scenarios to evaluate water quality impacts. See **Table 7.4.3-1**.

Table 7.4.3-1: Model Run Framework for Water Quality Impact Evaluation

Run Scenario	**Freshwater River Flow**	**Loading Conditions**	**Evaluation Period**
Basic Evaluation	Low Flow/Dry	2004 harbor point sources' BOD loads	1-May to 1-Nov
Sensitivity Analysis #1	Average/Typical	2004 harbor point sources' BOD loads	1-May to 1-Nov
Sensitivity Analysis #2	Low Flow/Dry	1999 harbor point sources' BOD loads	1-May to 1-Nov
Sensitivity Analysis #3	Low Flow/Dry	Permitted Harbor Point sources' BOD loads	1-May to 1-Nov

*See **Table 7.4.3-2** for the point source loads referenced in the previous table.*

Table 7.4.3-2: CBODu Point Sources Loads in Savannah Harbor

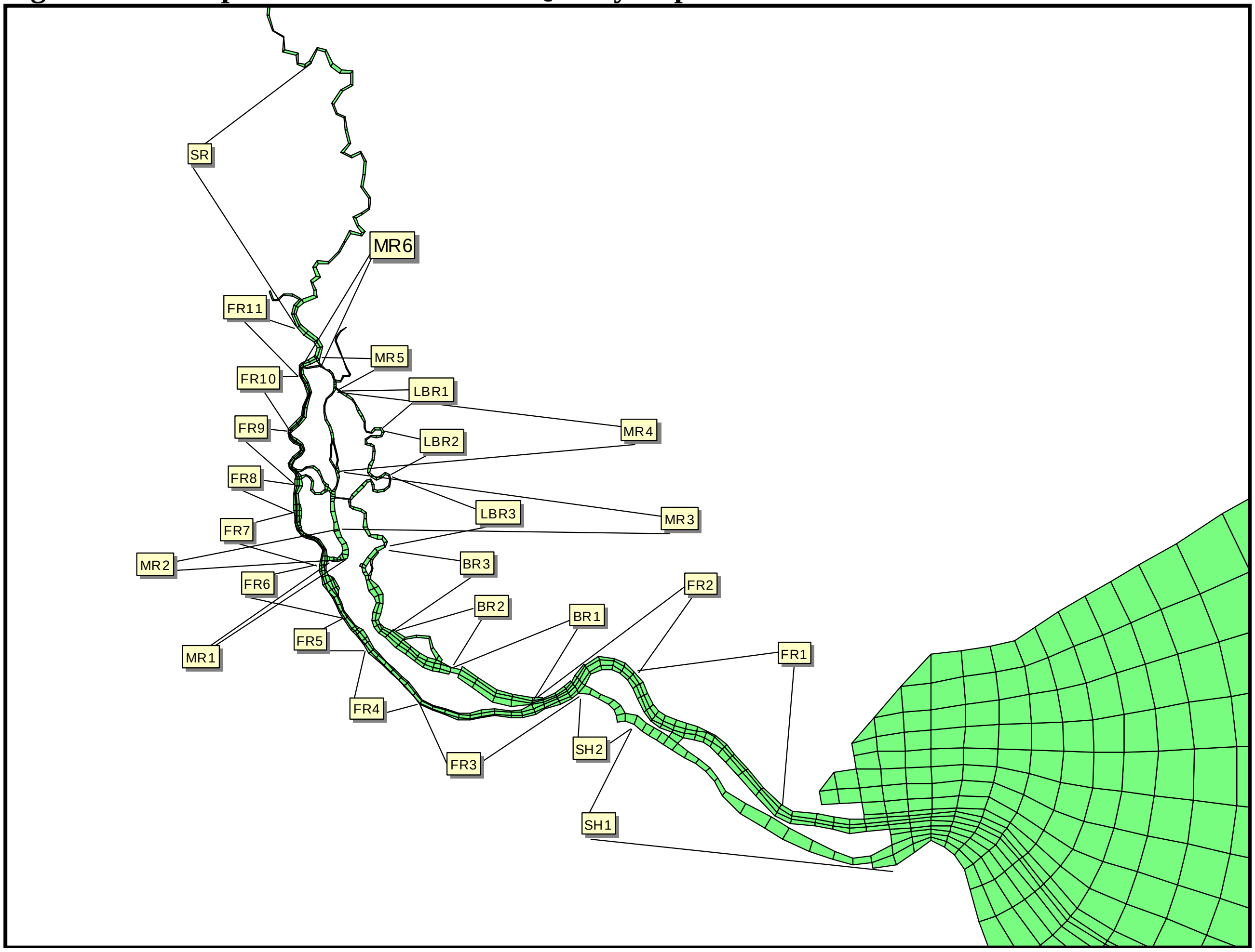
Facility name	**Location Cell (I_J)**	**Loads (lbs/day)**		
		2004	**1999**	**Permitted**
Hardeeville	14_148	13.0	25	505.55
Fort James	14_171	5,873.0	3,810.46	54,249.46
Weyerhaeuser	13_95	6,797.0	809.86	30,150
Garden City	13_77	32.0	122	2,700.7
Wilshire	13_74	0.0	737.31	2,814.79
Travis Field	13_74	27.0	129	576.35
President Street	13_54	1,489.0	4,398.99	16,246.15
IP	15_70	143,448.0	86,669.75	269,328
Engelhard	13_52	0	0.38	0

The Post Processor was also used to evaluate the WASP model output for ease in determining dissolved oxygen impacts by determining the following spatial objects:

- Critical Cell – the cell with lowest D.O. concentrations during specified simulation period
- Critical Segment – an assemblage of cross section cells located at the critical cell's j-coordinate
- Zone – an assemblage of cells that is limited by specified horizontal and vertical boundaries

Twenty-six spatial zones were delineated that cover the full estuary. See **Figure 7.4.3-1**.

Figure 7.4.3-1: Spatial Zones for Water Quality Impact Evaluation



Once the critical cells and segments were determined for each zone they were compared in the following ways:

- Comparing critical cells' D.O. concentrations for project scenarios and existing conditions with Georgia and South Carolina existing and proposed standards for D.O.
- Comparing zones' volume-weighted D.O. concentrations for existing and project scenarios, and D.O. standards.
- Comparing the percentage of water volume with D.O. concentrations that violate the D.O. standards for each zone during the selected simulation periods.
- Comparing the percentage of water volumes with specified salinity and D.O. percentiles for major parts and stations of the estuary.
- Comparing the percentage of water volumes in Upper Harbor in increments of 1^{o} C of water temperature and 0.1 mg D.O.

- Analyzing values and their changes in longitudinal profiles of D.O. distributions along critical cells of Front, Back, Little Back, and Middle Rivers.

- Analyzing values and their changes in minimum, 5th, 50th, and 95th percentiles D.O. and salinity distributions in bottom and surface areas of the estuary.

- Analyzing dynamics of 1-, 7-, and 30-day averaged D.O. and salinity and their changes in longitudinal - vertical plane of Front River.

7.4.3.1 Findings

For greater detail on the water quality analysis and results for dissolved oxygen, see the report titled *Water Quality Impacts of the Savannah Harbor Expansion Project* included in the Engineering Investigations Supplemental Materials. Some short conclusions from the analysis of the report's results are listed below. A detailed analysis of the results can be found in the EIS.

<u>Effect of the Harbor's Point Sources Loads</u>

2004 and 1999 loads scenarios comparisons:

- The zones most affected by the harbor's point sources' impact are FR2-FR9, and BR1-BR3. The scenario of 2004 point sources loads serves as a benchmark for comparisons with other loads scenarios.

- The 1999 loads provide 6-8% (0.16 – 0.2 mg/l) improvement for the 1st percentile of D.O.; 2-5% (0.1-0.2 mg/l) improvement for the 50th percentile of D.O.; and 1-3% (0.02-0.15 mg/l) improvement for the 95th percentile of D.O. for critical cells of zones FR2-FR9. The D.O. deterioration is observed only for zone FR8 50 – 99 percentiles. The 1999 loads provide 8-11% (0.14 – 0.16 mg/l) improvement for the 1st percentile of D.O.; 4-12% (0.14-0.32 mg/l) improvement for the 50th percentile of D.O.; and 2-9% (0.11-0.35 mg/l) improvement for the 95th percentile of D.O. for critical cells of zones BR1-BR3.

- These tendencies persist for D.O. values averaged over the volumes of zones also. But deterioration of the D.O. regime for zone FR8 is not observed. Results indicate an increase in percentage of volumes with violations of existing and proposed D.O. standards for the 2004 loads scenario.

- Results show insignificant differences in D.O. distributions along the vertical-longitudinal plane of Upper Harbor for scenarios A and C.

2004 and permitted loads scenarios comparisons:

- Results show that 2004 loads provide 12-25 % (0.4 – 0.8 mg/l) improvement for the 1st percentile of D.O.; 6-14% (0.2-0.6 mg/l) improvement for the 50th percentile of D.O.; and 2-9% (0.1-0.6 mg/l) improvement for the 95th percentile of D.O. for critical cells of zones FR2-FR9. They also show that 2004 loads provide 21-32% (0.41 – 0.43 mg/l) improvement for the 1st percentile of D.O.; 12-14% (0.37-0.38 mg/l) improvement for the 50th percentile of D.O.; and 5-6% (0.24-0.25 mg/l) improvement for the 95th percentile of D.O. for critical cells of zones BR1-BR3.

- These tendencies persist for D.O. values averaged over the zones' volumes also. Results indicate an increase in percentage of volume with violations of existing and proposed D.O. standards for the permitted loads scenario.

- Results show significant differences in D.O. distributions along vertical plane of Upper Harbor for scenarios A and D.

<u>Effect of 1999 (Drought) and 1997 (Average) Years Hydrological and Meteorological Conditions</u>

- Results show that the increasing of river flow strongly effects the D.O. concentrations in critical cells particularly in zones of Back, Little Back and Middle Rivers, as well as Savannah River. The 1997 flow provide 10-50% increasing of the 1st D.O. percentile, 4-14% increasing of the 50th D.O. percentile, and 10-26% increasing of the 95th percentile for zones of the estuary.

- Results indicate that increases in D.O. concentrations averaged over volume of zones are up to 29 % for the 1st percentile, up to 10% for the 50th percentile, and up to 27% for the 95th percentile.

- The D.O. and salinity distributions along vertical plane of Upper Harbor for scenarios of 1997 and 1999 flows differ significantly.

<u>Effect of the Harbor Deepening</u>

- Results indicate the D.O. regime deterioration under the impact of the ship channel deepening mostly for critical cells of Front River zones F7, F8, and F9. For the drought year 1999 the D.O. decreases are up to 16.3% (1st and 50th percentiles, zone FR7) and 18.2% (99th percentile, zone FR7) for 6 ft deepening; and between 5.1% (1st percentile, zone FR7) and 1% (99th percentile, zone FR7) for 2 ft deepening. For the average year 1997 the D.O. decrease are 22.8% (1st percentile, zone FR9), 11.5% (50th percentile, zone FR9), and 5% (99th percentile, zone FR4) for 6 ft deepening; and between 8.3% (1st percentile, FR9), 6.6% (50th percentile, BR2), and 9.0% (99th percentile, BR2) for 2 ft deepening.

- Results indicate the D.O. regime deterioration under the impact of the ship channel deepening for D.O. values averaged over volume of zones. For the drought year 1999 the D.O. decrease are up to 11.1% (1st percentile, FR9), 8.2% (50th percentile, zone FR8), and 4.9% (99th percentile, zone FR7) for 6 ft deepening; and between 4.7% (1st percentile, zone FR7) and 1.7% (99th percentile, zone FR6) for 2 ft deepening. For the average year 1997 the D.O. decrease are 9.5% (1st percentile, zone FR9), 9.3% (50th percentile, zone FR7), and 10.5% (99th percentile, zone FR4) for 6 ft deepening; and 4.0% (1st percentile, FR9), 3.2% (50th percentile, FR7), and 4.2% (99th percentile, FR3) for 2 ft deepening.

- Results show that the deepening insignificantly (1-2%) increases the percentage of volume of the harbor's waters with violations of the existing D.O. standards.

- Results show the deteriorations of lowest D.O. values along critical cells of major parts of the estuary increase proportionally to projected deepening of the ship channel.

- Resulting figures visualize upstream shifts of lower D.O. zones in bottom and surface layers of the estuary with increasing of the harbor deepening.

- Resulting figures visualize an increase in salinity intrusions in bottom and surface layers of the estuary with increasing of the harbor deepening.

- Snapshots of animations of 1-, 7-, and 30-day averaged D.O. and salinity dynamics in vertical-longitudinal plane are presented in the results along the ship channel. The snapshots conclude higher channel deepening provides increasing of salinity and D.O. stratifications particularly for zones FR7, FR8, and FR9

7.4.4 Wetland Modeling

The Interagency Coordination Team developed model run scenarios and outlined parameters to use in determining impacts to marshes and wetlands due to SHEP. Of particular interest are impacts to the freshwater tidal marshes adjacent to the estuary. The model scenarios outlined for the analysis vary by freshwater flow conditions at the upstream boundary and by sea level conditions at the ocean boundary. See **Table 7.4.4-1** for details on the model input parameters.

Table 7.4.4-1: Wetland Evaluation Model Input Conditions

Run Scenario	Freshwater River Flow	Sea Level Rise	Evaluation Period
Basic Evaluation	Average/Typical	Existing Sea Level	1-Mar to 1-Nov, 1997
Sensitivity Analysis #1	Low Flow/Dry	Existing Sea level	1-Mar to 1-Nov, 2001
Sensitivity Analysis #2A	Average/Typical	25 cm Sea Level Rise	1-Mar to 1-Nov, 1997
Sensitivity Analysis #2B	Average/Typical	50 cm Sea Level Rise	1-Mar to 1-Nov, 1997

Standard practice for lake and reservoir models would be to calculate actual volume of storage using a high resolution digital elevation model. We do not think that approach is appropriate for estuaries. A significant amount of the volume in an estuary is due to tidal prism which is dependent upon tide range, depths in the estuary, and areas of inundation. Our measure of success was based on propagating the tides (water surface elevation comparisons) and volume of the water over a tidal period. We use measured flow data to calibrate volume. Since we could not account for all of the intricacies of the marsh areas in Savannah Harbor, we developed marsh grids to "size" the volumes needed to meet the tides. So when the chloride model was extended to account for the areas in and near Abercorn Creek, it was important to check the tidal flows and volumes. Modeled flow rates at Abercorn Creek were compared against measured flow rates for the same period, and their consistency demonstrated the ability of the modeled wetland storage areas to simulate the actual areas.

Freshwater river flows specified for the evaluations were determined using USGS gage data for Savannah River near Clyo, Georgia. The EFDC model has continuous input boundary conditions for a 7 year period (1997-2003) available for simulation. Flow conditions during 1997 were found to be representative of the long term average and flow conditions during 2001 were considered to be a low flow (dry) period.

Sea Level Rise estimates were derived from the EPA published report titled *The Probability of Sea Level Rise* authored by J. G. Titus and V. K. Narayanan published October 1995 and specified for use in wetland impact evaluation by the Interagency Coordination Team. Impacts to wetlands regarding sea level change over the 50-year project life are discussed further in Section 7.5.2.2 of this report.

7.4.4.1 Findings

Impacts to marshes and wetlands during each run scenario were determined using riverine salinities predicted by the EFDC model. The EFDC model does not directly predict marsh salinity, despite the fact that the grid does include 17 marsh cells. These cells act primarily as storage areas and were found to be a critical model component for capturing the salinity trends in the upper part of the estuary, specifically, in Middle and Back River. Initially, these cells were equated with marsh acreages and used to predict marsh salinity and determine impacts. However, this proved to be an impractical way to determine changes in marsh salinities, especially changes in the freshwater/brackish marsh boundary line, and the idea was abandoned. The EFDC model grid was developed to predict changes in riverine salinity. The marsh cells were added to the model grid to enhance the riverine salinity calibration. The intended purpose of these 17 cells was not to predict and categorize the salinity changes within the marshes. Prior to abandonment, the results were documented in the report titled *Wetland/Marsh Impact Evaluation* which is included in the Engineering Investigations Supplemental Materials. See **Figure 7.4.4.1-1** for an example plot.

As an alternative, the wetland impacts were evaluated using a method where marsh salinity contour lines were extrapolated from the river into the marsh areas. The riverine salinities were predicted using the EFDC model results. This method creates contours that divide the marsh into 5 salinity categories: 0 - 0.5 ppt, which is considered freshwater, 0.6 - 1.0 ppt, 1.1 - 2.0 ppt, 2.1 - 4.0 ppt, and > 4.0 ppt. Acreages are then calculated based on these contours and impacts are assessed with comparisons. See **Figure 7.4.4.1-2** for an example. See **Table 7.4.4.1-1** for tabular results.

Table 7.4.4.1-1: Estimated Marsh Salinity Acreages with the Basic Evaluation (Deepening Only)

Estimated Marsh Salinity (ppt)	Existing (acres)	44 ft Depth (acres)	45 ft Depth (acres)	46 ft Depth (acres)	47 ft Depth (acres)	48 ft Depth (acres)
0.0 – 0.5	4072	3521	3105	3015	2895	2860
0.6 – 1.0	864	1186	1319	1050	1009	830
1.1 – 2.0	555	397	630	921	1355	1215
2.1 – 4.0	834	863	906	789	1365	739
> 4.0	2506	2865	2873	3057	2208	3188

The development of the marsh salinity contour lines is highly subjective because they are extrapolated from the riverine salinities by hand. To ensure the validity of the results, verification and repeatability tests were done and concluded that the results are valid within +/- 50 acres.

In addition to the marsh salinity predictions and extrapolations, many maps and figures were developed to show changes in riverine salinity throughout the estuary. Most of the mapping focuses on surface salinities because it is within this layer of the water column that would have the most interaction with the adjacent marsh systems. The analysis details can be found in the *Wetland/Marsh Impact Evaluation* included in the Engineering Investigations Supplemental Materials and in the EIS.

Figure 7.4.4.1-1: Changes in Freshwater Threshold Wetland Impact Evaluation

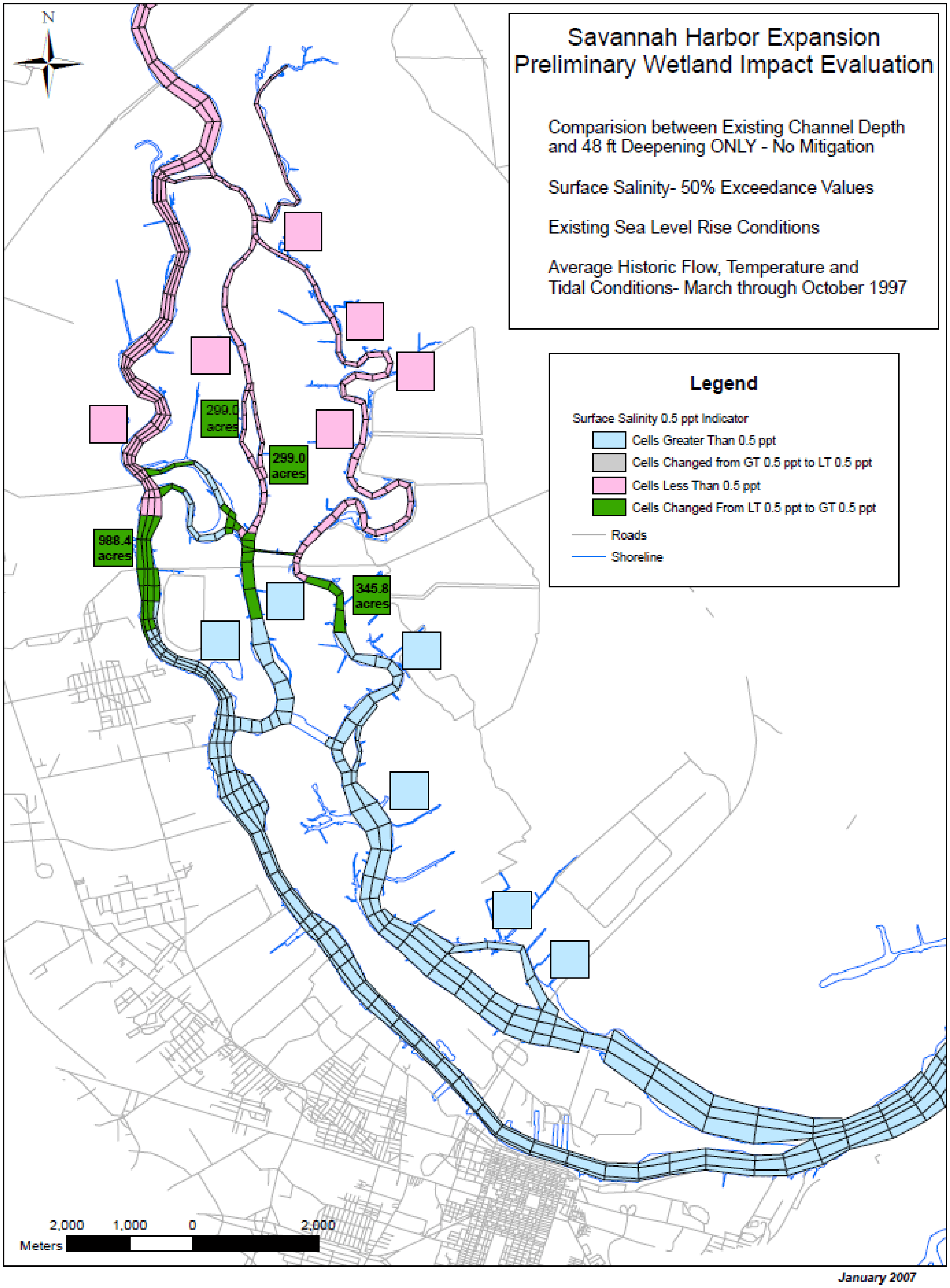
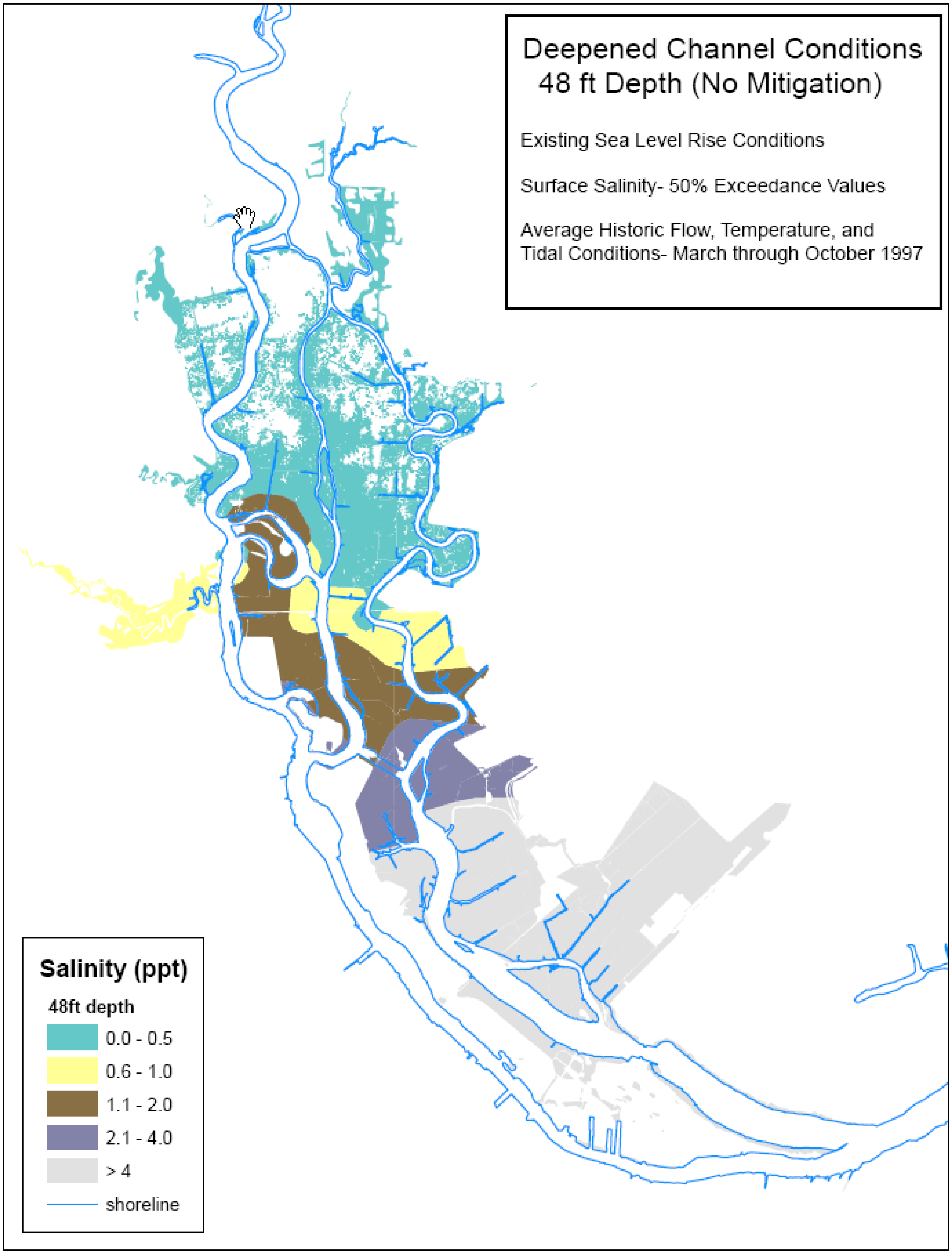


Figure 7.4.4.1-2: Wetland Salinity Projections for Wetland Impact Analysis



7.4.4.2 M2M/MSM

To aid in determinations of marsh salinity impacts for the EIS, the USGS entered into a cooperative agreement with the GPA to develop empirical models to simulate the water level salinity of the small creeks and tidal marshes adjacent to the estuary. This model is called Model-to-Marsh (M2M), an artificial neural network model, which provides a link from the hydrodynamic (EFDC) model salinity predictions to the marsh-succession model (MSM). The MSM simulates changes in plant distribution in the tidal marshes in response to changes in the water-level and salinity conditions due to geometry changes in the navigation channel. Details of M2M can be found in the USGS Scientific Investigations Report 2006-5187 titled *Simulation of Water Levels and Salinity in the Rivers and Tidal Marshes in the Vicinity of the Savannah National Wildlife Refuge, Coastal South Carolina and Georgia* included in the Engineering Investigations Supplemental Materials. Details of the MSM modeling can be found in the report titled *Savannah Harbor Deepening Project ATM Marsh Succession Model Marsh/Wetland Impact Evaluation* dated May 2007 and the report titled *Savannah Harbor Deepening Project USGS/USFWS Marsh Succession Model Marsh/Wetland Impact Evaluation* dated June 2007. Both of these reports are included in the Engineering Investigations Supplemental Materials.

The EIS includes a comparison of wetland impacts predictions using the method described in the previous section and impacts predicted by the MSM.

7.4.5 Hurricane Surge Analysis

The purpose of the hurricane surge modeling is to determine the effect deepening the inner harbor navigation channel will have on the propagation of a hurricane storm surge traveling upstream through the estuary and river system. Simulations were run for the time period of August 18 – 23, 1997 using the EFDC model grid developed by Tetra Tech for the hydrodynamic modeling. The tidal boundary was modified to incorporate a synthetic storm surge. The specified time period was chosen for the following reasons:

- 1997 closely represents historic average flow conditions at the upstream flow boundary (USGS gage near Clyo, GA)
- August is close to peak hurricane season when the likelihood of a large storm making landfall near the Savannah coastline would be more probable
- Spring tidal conditions occurred on August 19, 1997

The hurricane surge data set was developed by ATM and is based on measured water surface elevations collected at the USGS Customs House gage located in Charleston, SC during Hurricane Hugo, which made landfall on September 21, 1989. The hurricane storm surge component was separated from the harmonic tidal component for adaptation to the SHEP model. The maximum increase in water surface elevation for the hurricane storm surge component collected at the USGS Customs House gage was 7.69 ft. However, the storm surge as Hurricane Hugo made landfall varied with some places receiving a near 20-foot storm surge. Due to the difficulty in directly applying the storm surge gage data from Charleston to Savannah, the data set was ratioed to create three synthetic storm surges with peaks of 5, 10 and 15 ft. These storm surge data sets were then added to the harmonic tide in Savannah to create a synthetic storm surge scenario. Six storm event scenarios were developed from the data set. Three had the 5-, 10- and 15-foot peak surges occurring on top of the peak spring tidal condition. The fluctuation in water surface elevation in Savannah is so large that this peak-on-peak condition allowed evaluation of a worst case scenario. Three additional scenarios were also evaluated that had the 5-, 10- and 15-foot peak storm surges occurring on the falling limb of the spring peak tide. The off-peak scenario would create a tidal surge that had a lower peak than the surge during the peak-on-peak scenario but would have a longer duration. There are likely some areas where a longer duration storm would be a worst case scenario rather than a shorter peak surge.

7.4.5.1 Model Limitations

The EFDC grid was not created with emphasis on hurricane surge modeling. The shipping channel and smaller side channels including marsh areas are described in the grid. However, the higher river banks and adjacent beaches are not accounted for in the model. These areas would likely be impacted during a hurricane and would have a direct effect on the propagation of a storm surge through the river system and navigation channel. The model is useful as a comparison tool to evaluate different deepening scenarios; however, it should only be used for relative comparative purposes and not to describe flooding depths and inundation limits during a hurricane event.

7.4.5.2 Findings

The results from the hurricane surge modeling show that the change in water surface elevation due to the deepening the inner harbor is not significant. **Table 7.4.5.2-1** shows the difference in the water surface elevation between the existing project depth and the maximum 48-foot deepening for three storm events simulated at two different times in the tide cycle. The maximum difference in the water surface elevations is 0.9 ft which occurs at the I-95 Bridge during the 15-foot surge at the peak of high tide.

Table 7.4.5.2-1: Increase in Water Surface Elevation for 48-Ft Project Depth

Storm Surge Height	Increase in Water Surface Elevations	
	Ft. Jackson	I-95 Bridge
5 Ft		
Peak on peak	0.3 ft	0.7 ft
Offset peaks	0.3 ft	0.7 ft
10 Ft		
Peak on peak	0.3 ft	0.8 ft
Offset peaks	0.3 ft	0.8 ft
15 Ft		
Peak on peak	0.3 ft	0.9 ft
Offset peaks	0.3 ft	0.8 ft
Maximum Difference	0.3 ft	0.9 ft

In addition to modeling the existing project and the 48-foot deepening scenarios, deepening the navigation channel elevations 44, 45, 46, and 47 ft below MLLW were also modeled for the 15-foot surge during a peak-on-peak condition. This surge event was chosen because it has the greatest difference in water surface elevation between the existing and maximum deepening scenario. **Table 7.4.5.2-2** shows the resultant differences in water surface elevations for each modeled scenario.

Table 7.4.5.2-2: Increase in Water Surface Elevation for 15-Foot Surge (Peak on Peak Condition)

Project Depth Alternatives	Increases in Water Surface Elevation	
	Ft. Jackson	I-95 Bridge
44 ft MLLW	0.1 ft	0.3 ft
45 ft MLLW	0.2 ft	0.5 ft
46 ft MLLW	0.2 ft	0.6 ft
47 ft MLLW	0.3 ft	0.8 ft
48 ft MLLW	0.3 ft	0.9 ft

While there are increases in the water surface elevations, the difference is not significant and is due to a larger volume of water being transported through the system during the tidal cycle and storm surge. This larger volume causes a slight increase in peaks on the flood tide and surge and a slight decrease in elevations on the ebb tide. In conclusion, the hurricane surge modeling shows that deepening the inner harbor has no significant adverse impact to a propagated storm surge as it travels upstream through the river system and navigation channel. Details of the hurricane surge analysis can be found in the report titled *Hurricane Surge Modeling* included in the Engineering Investigations Supplemental Materials.

7.5 MITIGATION FEATURE MODELING AND PLAN DEVELOPMENT

All mitigation feature modeling was evaluated using the same EFDC and WASP models discussed previously. Details on analysis of specific mitigation features and mitigation plan development can be found in the following sections and in the EIS.

7.5.1 Feature/Plan Development

The mitigation feature and plan development focuses on reducing adverse impacts associated with SHEP. Many ideas of how to accomplish this goal were already under discussion before modeling of the impacts was completed. Most of the ideas that required modeling focus on isolating portions of the estuary, specifically Middle and Back River to reduce impacts of deepening on Front River or rerouting flow paths by closing and opening various cuts. A conceptual list had been developed and portions pertaining to hydrodynamic and water quality modeling are shown in **Table 7.5.1-1**. A full list of conceptual mitigation ideas can be found in the EIS.

Table 7.5.1-1: Conceptual Mitigation Actions

Wetlands
Restore sites where tidal freshwater wetlands previously existed. Isolate Front River from Middle and Back Rivers to reduce salinity levels in MR and BR. Close lower entrance of Middle River. Install a flexible curtain as a barrier to block tidal flood flows up Middle River. Construct berms along Front River to restrict high water flows across the marsh. Block Rifle Cut to eliminate higher salinity water from MR entering Back River. Block Drakies Cut and restore flows through Steamboat River to lengthen the passage of saline tidal waters up the Savannah River. Cease operation of the Sediment Basin to reduce movement of salinity up Back River (fill in the Sediment Basin). Begin operation of the tidegate with gates installed to only allow downstream flow, with no upriver movement of salt water. Remove the tidegate.
Fishery Resources
Increase dissolved oxygen levels in Front River. Increase freshwater flows down Back River to improve striped bass habitat (velocities and salinity) during spawning.
Water Quality
Add air or oxygen to low Dissolved Oxygen waters. Install air injection system on bottom of river. Install floating aerators. Install D.O. injection system on bottom of river. Construct D.O. injection system on Hutchinson Island. Mix low Dissolved Oxygen waters on the bottom with higher D.O. surface waters. Inflatable weir. Pumps. Block Drakies Cut and restore flows through Steamboat River to lengthen the passage of saline tidal waters up Front River (decrease chloride levels at City of Savannah's I&D water intake).

7.5.2 Salinity & Freshwater Marsh Mitigation

Since tidal freshwater marshes were identified by the USFWS as the single most critical natural resource in the harbor, the Corps focused on reducing project impacts to that resource. Salinity is the primary determining factor in the conversion of tidal freshwater marshes, so that parameter was identified as the focus of the mitigation modeling efforts. The hydrodynamic model (EFDC) salinity concentration predictions were used to evaluate the mitigation features. A summary of how each mitigation plan was finalized and selected is outlined below. Details of this process can be found in the EIS. Results for all plans evaluated can be found in the report titled *Mitigation Evaluation for Marsh/Wetland Impacts* included in the Engineering Investigations Supplemental Materials.

In the summer of 2006, several meetings were held to screen the mitigation options based on the EFDC model output. The first was held on July 12, 2006 and included the hydrodynamic modelers from the Corps and our consultants and scientists from various resources agencies. Model results were presented on maps showing salinity changes in the upper portion of the estuary adjacent to the freshwater tidal marshes. After lengthy discussion and review, features were either eliminated or put forward for further development. A summary of these model scenarios and notes on their viability as a mitigation feature are shown in **Tables 7.5.2-1** and **7.5.2-2**.

From this screening process, the mitigation features and the most effective design details of each feature (lengths, widths and depths) having the greatest potential to mitigate for SHEP impacts were identified. See **Figure 7.5.2-1** for a location map of these mitigation features. Details of these features, how they were modeled and how they mitigate for impacts follow the figure.

Table 7.5.2-1: Mitigation Options Model Scenarios and Results

Savannah Harbor Expansion Project - EFDC Modeling Mitigation Options Update: 27-Jul-2006

Model Run Name	Model Scenario	EFDC Model Modifications - Summary	Simulation Time Period	Parameter Examined	Results	Notes
Base Runs						
Existing_1	Existing Channel Depths	Use revised dxdy.inp file from TT dated April 4, 2006, increased roughness coeff upper reaches	Jun1-Sep1, 1997	50% Surface Salinity		All runs use this as base for Salinity Difference maps.
48ftDeepen	48-ft Harbor Deepening, No Mitigation	Copy of Existing_1 run with 6-ft deepening dxdy.inp, no mitigation options	Jun1-Sep1, 1997	50% Surface Salinity	Increase of salinity throughout system; marsh cells on Little Back increased 0.01 to 1.00 ppt.	Use the delta map as the base run comparison to all runs below
Tide Gate Modifications						
Tide_Gate_1	1.0m Sill at Gate Invert	Increase cell 31_63 invert from -3.85 to -2.85m	Jun1-Sep1, 1997	50% Surface Salinity	No impact on upper Little Back or Middle River, 0.1ppt decrease of cells near Rifle Cut; increase in salinity on Back River upstream of tidegate, salinity trapped upstream of gate.	Not a viable option
Tide_Gate_2	Restrict Gate Opening Width by 1/3	Change cell 31_63 width from 197m to 131m	Jun1-Sep1, 1997	50% Surface Salinity	No impact on upper Little Back or Middle River, 0.1ppt decrease of cells near Rifle Cut; had a higher increase in salinity on Back River upstream of tidegate than the 1.0m sill.	Not a viable option
~~Tide_Gate_3~~	Remove Tide Gates and Abutments	Connect cells 30_62 to 30_64, widen cell 31_63 33m from 196.98 to 230m	Jun1-Sep1, 1997	50% Surface Salinity	~~No impact on upper Little Back; salinity increases of over 1.0ppt Front, Middle and Back Rivers; Increasing gate opening allows more high saline water to enter the refuge area.~~	Delete output files, incorrect connection order in Mappgns.inp
~~Tide_Gate_4~~	Remove Tide Gates and Abutments - wider	Connect cells 30_62 to 30_64, widen cell 31_63 80m from 196.98 to 277m	Jun1-Sep1, 1997	50% Surface Salinity	~~Same results as Tide Gate Run 3, there were some decreases in salinity compared to Run 3~~	Delete output files, incorrect connection order in Mappgns.inp
Tide_Gate_5	2.0m Sill at Gate Invert	Increase cell 31_63 invert from -3.85 to -1.85m	Jun1-Sep1, 1997	50% Surface Salinity	Results similar to run with 1.0m sill; there were 0.1ppt increases and decreases of cells on Middle River compared 1.0m sill	Not a viable option
Tide_Gate_6	Restrict Gate Opening Width by 1/2	Change cell 31_63 width from 197m to 98.5m	Jun1-Sep1, 1997	50% Surface Salinity	Results similar to 1/3 restriction but with salinity increasing immediately upstream of gates.	Not a viable option
Tide_Gate_7	0.5m Sill at Gate Invert	Increase cell 31_63 invert from -3.85 to -3.35m	Jun1-Sep1, 1997	50% Surface Salinity	Results similar to 1.0m sill but with less cells near Rifle Cut decreasing 0.1ppt, still has increase upstream of gate structure.	Not a viable option
Tide_Gate_8	Remove Tide Gates and Abutments	connecting 30_64 to 30_62 and widen cell 31_63 from 196.98 to 230.0m	Jun1-Sep1, 1997	50% Surface Salinity	Decreased salinity along Back River from Sediment Basin to just below Rifle Cut; grid cells adjacent to Rifle Cut increased; cells along Front River near Drakies Cut increased.	Viable Option
Sediment Basin Modifications						
SedBasin_1	Fill Entire Sediment Basin to -4.5m	Increase invert to -4.5 m for cells 31_62, 31_61, 31_60, 31_59 & 17_50; -7.0m for cell 16_50	Jun1-Sep1, 1997	50% Surface Salinity	Salinity decreases of over 3.0ppt in Back River compared to existing conditions; Decrease of deepening impacts on Little Back near Rifle Cut; No impact on upper Middle and Little Back;	Viable Option
SedBasin_2	Fill Entire Sediment Basin to -9.0m	Increase invert to -9.0 m for cells 31_62, 31_61, 31_60, 31_59, 17_50 & 16_50	Jun1-Sep1, 1997	50% Surface Salinity	Decrease of deepening impacts on Back River up to Rifle Cut but not below existing salinity level as with fill to -4.5m; No impacts to upper Middle and Little Back Rivers	Viable Option
SedBasin_3	Deepen Sediment Basin to 48-ft Harbor Depth		Jun1-Sep1, 1997	50% Surface Salinity	Increase salinity level further up Back River compared to 48-ft deepening alone.	
SedBasin_4	Fill Upper 1/2 of Basin to Tide Gate Invert, -3.85m	Increase invert to -3.85m for cells 30_62, 31_62, 32_62, 30_61, 31_61 and 32_61	Jun1-Sep1, 1997	50% Surface Salinity	Salinity decreases of over 1.0ppt in Back River compared to existing conditions; Decreased impacts on Little Back near Rifle Cut; No impact on upper Middle and Little Back;	Viable Option
SedBasin_5	Sill at Sediment Basin Entrance at Front River, invert = -6.0m	Increase inverts to -6.0m for cells 16_50 & 17_50	Jun1-Sep1, 1997	50% Surface Salinity	Increased salinity impacts in Back River up to Rifle Cut; No impact on Upper Middle and Little Back	Not a viable option
SedBasin_6	Fill in Upper 1/4 of Basin to Tide Gate Invert = -3.85m	Increase invert to -3.85m for cells 30_62, 31_62, and 32_62	Jun1-Sep1, 1997	50% Surface Salinity	Lessens impact of 48-ft deepening by 1.0ppt on Back River near tide gate and by 0.2ppt upstream of New Cut; No impacts on upper Middle and Little Back	Viable Option
SedBasin_7	Sill at Sediment Basin Entrance at Front River, invert = -4.5m	Increase inverts to -4.5m for cells 16_50 & 17_50	Jun1-Sep1, 1997	50% Surface Salinity	Lessens deepening impacts on Back River up to Rifle Cut more than with -6.0m sill; No impacts to upper Middle and Little Back Rivers	Viable Option
SedBasin_8	Sill at Sediment Basin Entrance at Front River, invert = -9.0m	Increase inverts to -9.0m for cells 16_50 & 17_50	Jun1-Sep1, 1997	50% Surface Salinity	Lessens deepening impacts only on the lower Back River; no beneficial impacts to mid or upper Middle and Little Back Rivers	??
SedBasin_9	Fill Entire Sediment Basin to -10.5m	Increase invert to -10.5 m for cells 31_62, 31_61, 31_60, 31_59 & 17_50; -7.0m for cell 16_50	Jun1-Sep1, 1997	50% Surface Salinity	Lessens deepening impacts by 0.5ppt on Back River upstream of New Cut; No impacts on mid or upper Middle and Little Back Rivers	Viable Option
SedBasin_10	Fill Entire Sediment Basin to -3.85m	Increase invert to -3.85 m for cells 31_62, 31_61, 31_60, 31_59 & 17_50; -7.0m for cell 16_50	Jun1-Sep1, 1997	50% Surface Salinity	Results similar to SedBasin_1; Large decreases in salinity on Back River but with diminishing impact on Little Back River with additional filling of basin, a few cells increasing near Rifle Cut;	Viable Option
New Cut Modifications						
~~New_Cut_1~~	Open New Cut - No Other Modifications	Connect cells 29_82 and 21_82 within mappgns.inp; cell 29_82 change length to 790m, invert -4.0m	Jun1-Sep1, 1997	50% Surface Salinity	~~Large increases in salinity along Front River from Middle River to Tybee; Odd results, consult with Tetra Tech~~	Delete output files, incorrect connection order in Mappgns.inp
~~New_Cut_2~~	Open New Cut - No Other Modifications - *Copy of New_Cut_1 but did not lengthen cell 29_82*	Connect cells 29_82 and 21_82 within mappgns.inp; cell 29_82 deepen to invert -4.0m, *did not lengthen cell*	Jun1-Sep1, 1997	50% Surface Salinity	~~Same results are New_Cut_1~~	Delete output files, incorrect connection order in Mappgns.inp
Rifle Cut Modifications						
Rifle_Cut_1	Close off Rifle Cut	mask.inp thin barrier MTYPE = 3 at cell 28_98	Jun1-Sep1, 1997	50% Surface Salinity	Large increase in salinity along Back River up to New Cut; by closing Rifle Cut less ebb flow is available to flush out the Back River; Small increases of cells on Middle River, no impact on upper Little Back.	Not a viable option as a single option

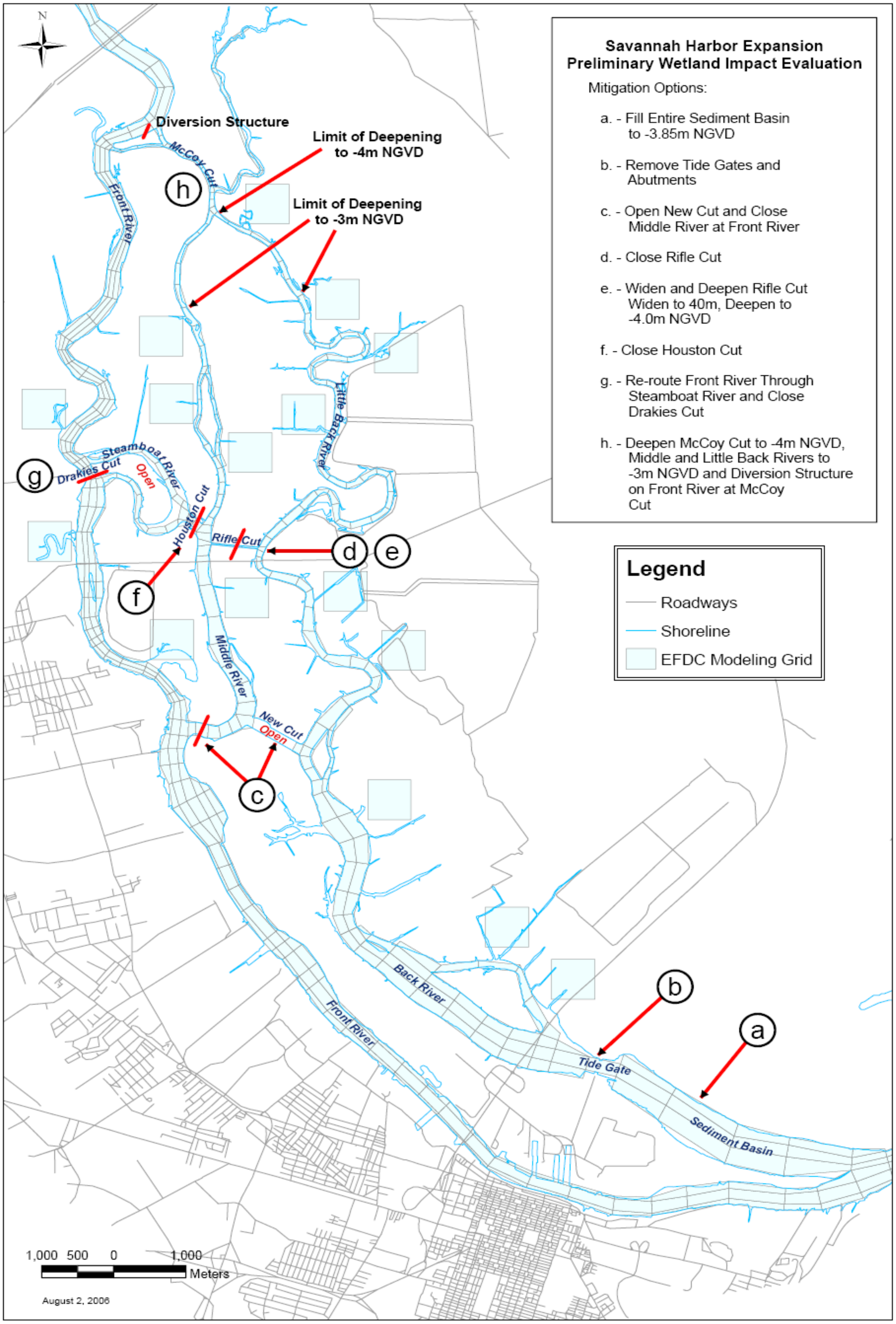
Table 7.5.2-2: Mitigation Options Model Scenarios and Results (Continued)

Savannah Harbor Expansion Project - EFDC Modeling Mitigation Options Update: 27-Jul-2006

Model Run Name	Model Scenario	EFDC Model Modifications - Summary	Simulation Time Period	Parameter Examined	Results	Notes
Rifle_Cut_2	Deepen to -4.0m, widen to 60m	Increase invert to -4.0m and widen to 60m width for cells 27_98, 28_98, 29_98	Jun1-Sep1, 1997	50% Surface Salinity	Significant decrease of 48-ft deepening impacts in Back River, upper Middle and Little Back; Increased impacts in mid Little Back. Opening Rifle cut allows additional ebb flows to flush the system.	Viable Option
Rifle_Cut_3	Deepen to -4.0m, no widening	Increase invert to -4.0m for cells 27_98, 28_98, 29_98, existing width	Jun1-Sep1, 1997	50% Surface Salinity	Impacts similar but not as great as with the 60m widening and deepening of Rifle Cut;	Viable Option
McCoy Cut Modifications						
McCoy_1	Deepen -4.0m from Sav River to Junction, -2.5m 1600m Little Back and Middle Rivers	Deepen to -4m 15_123 to 26_123; -2.5m 26_122 to 26_117 and cells 27_123 to 32_123 on Little Back	Jun1-Sep1, 1997	50% Surface Salinity	Significant decrease of 48-ft deepening impacts in Back River, upper Middle and Little Back Rivers; Deepening Mcoys allows higher ebb flows to flush out the system.	Viable Option
McCoy_2	Deepen -5.0m from Sav River to Junction	Deepen to -5m 15_123 to 26_123	Jun1-Sep1, 1997	50% Surface Salinity	Deepening McCoy only to junction has almost as great of benefit as deepening upper Little Back and Middle Rivers and McCoys; There may be a limit as to beneficial deepening of Middle River, starts to allow flood tide to reach further upstream	Viable Option
McCoy_3	Diversion Structure in Sav River @ McCoy	mask.inp thin barrier MTYPE=3 at cell 14_122	Jun1-Sep1, 1997	50% Surface Salinity	A diversion structure was able to decrease impacts on the Middle and Little Back Rivers down to New Cut; No impacts on Back River; This model decreased the Sav River by half at McCoys, not a true diversion structure.	Viable Option but more analysis required to accurately determine amount of diverted flow required.
McCoy_4	Diversion Structure; Deepen -5.0m from Sav River to Junction, -3.0m 1600m Little Back and Middle Rivers	mask.inp thin barrier MTYPE=3 at cell 14_122; Deepen to -5m 15_123 to 26_123; -3.0m 26_122 to 26_117 and cells 27_123 to 32_123 on Little Back	Jun1-Sep1, 1997	50% Surface Salinity	This combination had greater benefit than just the diversion structure or McCoys deepening alone; Harbor deepening impacts significantly decreased along Middle, Back and Little Back Rivers.	Viable Option
McCoy_5	Diversion Structure in Sav River @ McCoy	Narrow width of 14_122 from 111.7m to 55.85m	Jun1-Sep1, 1997	50% Surface Salinity	Diversion structure modeled differently, narrowed cell width by 1/2. Only a few cells changed salinity value.	??
McCoy_6	Deepen -5.0m from Sav River to Junction,-3.0m 1600m Little Back and Middle Rivers	Deepen to -5.0m 15_123 to 26_123; -3.0m 26_122 to 26_117 and cells 27_123 to 32_123 on Little Back	Jun1-Sep1, 1997	50% Surface Salinity	Compare to McCoy_1; Additional deepening of McCoy was able to decrease salinity levels on Middle and Back Rivers by 0.1ppt compared to Run McCoy_1	Viable Option
McCoy_7	Deepen -6.0m from Sav River to Junction,-4.0m 1600m Little Back and Middle Rivers	Deepen to -6.0m 15_123 to 26_123; -4.0m 26_122 to 26_117 and cells 27_123 to 32_123 on Little Back	Jun1-Sep1, 1997	50% Surface Salinity	Compare to McCoy_6; Additional deepening of McCoy was able to decrease salinity levels on Middle and Back Rivers by 0.1ppt compared to Run McCoy_6	Viable Option
McCoy_8	Diversion Structure; Deepen -5.0m from Sav River to Junction	Narrow width of 14_122 from 111.7m to 55.85m; Deepen to -5m 15_123 to 26_123	Jun1-Sep1, 1997	50% Surface Salinity	Compare to McCoy_4; Additional deepening along upper Middle and Little Back was able to decrease salinity further downstream.	Viable Option
McCoy_9	Deepen -7.0m from Sav River to Junction, -5m 1600m Little Back and Middle Rivers	Deepen to -7.0m 15_123 to 26_123; -5.0m 26_122 to 26_117 and cells 27_123 to 32_123 on Little Back	Jun1-Sep1, 1997	50% Surface Salinity	Compare to McCoy_7; Additional deepening of McCoy was able to decrease salinity levels on Middle and Back Rivers by 0.1ppt compared to Run McCoy_1; salinity increasing on Front River	Viable Option but more analysis required determine reasonable deepening and Diversion Structure.
Middle River Modifications						
~~MiddleRiver_1~~	~~Close Middle Cut at Front River~~	~~mask.inp thin barrier mtype=3 at cell 17_82~~	~~Jun1-Sep1, 1997~~	~~50% Surface Salinity~~	~~Decreased salinity levels in Middle and Upper Little back Rivers; Salinity levels did increase on Front River and Back River from Rifle Cut downstream.~~	~~Viable Option with possible filling of Sediment Basin.~~
Steamboat River Modifications						
Steamboat_1	Steamboat Widen 215m & Deepen -10.5m, Drakies Closed	mask.inp thin barrier at cells 13_101, 14_101 & 15_101 mtype=2; deepen to -10.5m widen to 215m cells 16_101 to 16_99, deepen 15_101 & 15_99	Jun1-Sep1, 1997	50% Surface Salinity	Decreased salinity in mid and lower reaches of Middle and Little Back Rivers, Back River levels decreased; No impacts on upper Middle and Little Back Rivers	Viable for lowering salinity in mid reaches
Steamboat_2	Close Houston Cut - No Other Modifications	mask.inp thin barrier at cell 24_100, mtype=3	Jun1-Sep1, 1997	50% Surface Salinity	Increased salinity in Middle River and Back River from Rifle Cut to tide gates; Closing Houston stopped freshwater inflows to Middle and Back Rivers	Not a viable stand alone option
Steamboat_3	Close Houston and Drakies Cut, Steamboat Widen 215m and Deepen -10.5m		Jun1-Sep1, 1997	50% Surface Salinity	Increased salinity in Middle River and Back River from Rifle Cut to tide gates; Closing Houston stopped freshwater inflows to Middle and Back Rivers	Not a viable option with Houston Closed
Steamboat_4	Deepen north half of Steamboat to -4.0m, close lower entrance of Steamboat at Front River	Deepen to -4.0m cells 16_101 to 25_100, mask.inp thin barrier at cell 16_99 mtype=3	Jun1-Sep1, 1997	50% Surface Salinity	Increased salinity in upper Middle River and did not greatly reduce levels in area near Rifle Cut	Not a viable stand alone option
Steamboat_5	Close Drakies, Steamboat Deepen -10.5m but No Widening	mask.inp thin barrier at cells 13_101, 14_101 & 15_101 mtype=2; deepen to -10.5m cells 16_101 to 16_99, deepen 15_101 & 15_99	Jun1-Sep1, 1997	50% Surface Salinity	Compare to Steamboat_1; Increase levels on Middle and Little Back Rivers downstream of Rifle Cut; Increase levels on mid Middle River; No impact on upper Little Back	Not a viable option

This spreadsheet was updated on July 27, 2006. Modifications and additional runs were made based on conversations at the July 12,2006 meeting to screen mitigation options.

Figure 7.5.2-1: Potential Alterations to Water Flow for Mitigation



a) Fill Entire Sediment Basin to -3.85m NGVD (-9.5 ft MLLW) The sediment basin is an O&M feature, located at the mouth of Back River that was constructed to allow for cost effective dredging maintenance of the channel. The sediment basin is approximately two miles long and 600 ft wide and has an authorized maintenance depth of -40 ft MLLW (-13.1m NGVD), near the bottom of the navigation channel. The sediment basin currently acts as a trap for sediments but also traps salinity which, due to the depths in the basin, can more readily move upstream through the tidegate into the Back River system. This is especially true during dry times when freshwater inflows coming downstream are low.

   Upstream of the sediment basin, the elevation of Back River is controlled by the sill on the tidegate structure at -9.5 ft MLLW (-3.85 m NGVD). By discontinuing the use of the sediment basin as a maintenance feature and allowing it to fill naturally, the current upstream elevation on Back River could be extended through the sediment trap to the confluence with Front River. The modeling results show that filling the basin reduces salinity concentrations upstream in Back River by allowing more mixing and flushing of the area on each tidal cycle. Also, salinity concentrations are reduced by limiting the interaction between the Back River and the lowest portions of the water column on Front River that have the highest concentrations of salinity. Due to the fact that the sediment basin does provide a cost effective means of maintaining the channel, efforts were made to preserve some of its function. Several different filling scenarios were evaluated to determine if effectiveness of reducing salinity upstream on Back River could be achieved by limiting the function of the basin without eliminating it entirely. However, those scenarios were not as effective at reducing impacts from SHEP as letting the basin fill to the same elevation of the tidegate at -9.5 ft MLLW (-3.85 m NGVD).

b) Remove Tidegates and Abutments Upstream of the sediment basin on Back River is another O&M feature of the channel known as the tidegates. The tidegates were constructed during the same time as the sediment basin, but have since been taken out of use. The structure still stands with a sill elevation at -9.5 ft MLLW (-3.85 m NGVD) but the gates themselves have been removed. The gates were taken out of operation and removed because of environmental impact concerns with trapping high concentrations of salinity in the Back River system. Though the gates themselves have been removed, the structure still restricts flow moving in and out of the Back River system. Eliminating this constriction requires removing the tidegate abutments that support the structure.

   Due to the complexity of the river system, this constriction prevents water from flowing both upstream on the incoming tide and downstream on the outgoing tide. Removing the abutments allows more freshwater flow and mixing to occur while not allowing more salinity to intrude upstream on Back River.

c) Open New Cut and Close Middle River at Front River New Cut was constructed the same time as the tidegate and sediment basin and was an integral part of the design plan to reduce O&M costs in the harbor. However, it was closed in 1992. New Cut once provided a link between Middle River and Back River; however, concerns about salinity intrusion from Front River through the cut to Middle and Back River was part of the reason it was closed. Re-opening New Cut requires that the Middle River/Front River connection be closed. This would essentially re-route Middle River through Back River and eliminate its connection with Front River. By eliminating the connection between Middle and Front River, the high saline concentrations moving up the deep navigation channel cannot enter into the Back River system.

d) Close Rifle Cut Rifle Cut is a very small cut between Middle River and Little Back River. Despite its small size, it transports large volumes of water between the two rivers on each tidal cycle. Eliminating the cut has the same rational as mentioned above in feature c) Open New Cut and Close Middle River at Front River. The closure would allow isolation of Little Back River from the higher salinity water in Middle River.

e) Widen and Deepen Rifle Cut- Widen to 40m and Deepen to -4.0 m NGVD Due to the high transport capacity through Rifle Cut, widening and deepening the cut was proposed in hopes of increasing the amount of fresh water exchange between Middle River and Little Back River. The flow regime through the cut is complex, and initially it was not understood if closing or enlarging would provide the greatest benefits. Also, due to the complexity of the system on a whole, closing or enlarging Rifle Cut may be more or less effective based on the other features that are coupled with this one.

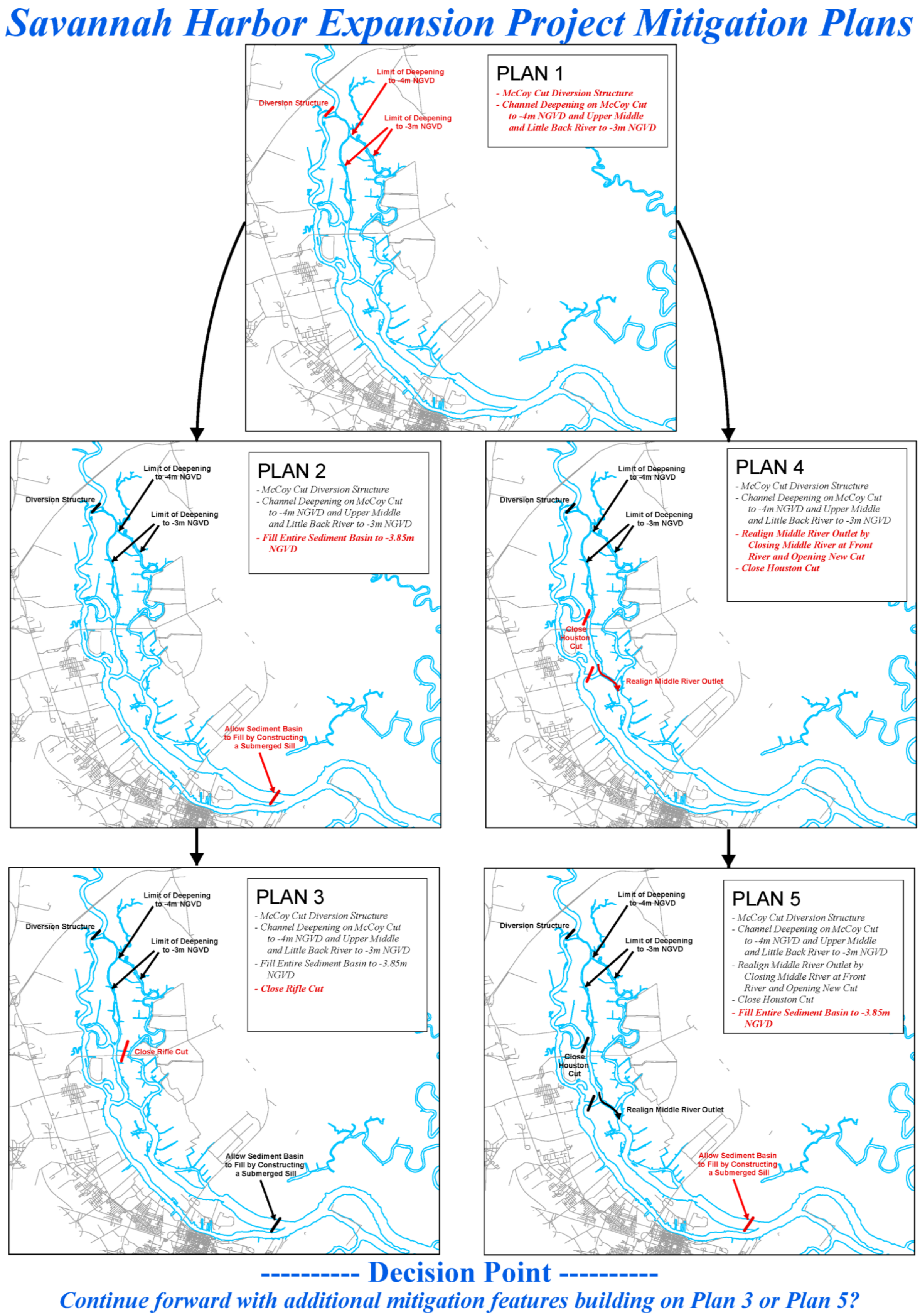
f) Close Houston Cut Closing Houston Cut has the potential to isolate Middle River from Front River and Steamboat River. Isolation would aid in eliminating the transmission of high salinity water from Front River through the cut.

g) Re-route Front River Through Steamboat River and Close Drakies Cut Re-routing Front River through Steamboat River by closing Drakies Cut restores a historic bend in the river. The bend is just over 2 miles long. It is believed that re-routing tidal waters through this bend would increase travel times on Front River during each incoming tide that would reduce the distance that salinity could travel upstream on any given tide, thereby reducing salinity intrusion and impacts upstream of the cut. However, the proximity of Steamboat River to Houston Cut, almost requires that the two features be evaluated in combination. Re-routing Steamboat without closing Houston Cut, increases the transport potential between Middle River and the high salinity concentrations on Front River. While benefits may be gained on Front River upstream of Drakies Cut, greater impacts may be seen on Middle and Back River through the Houston Cut access.

h) Deepen McCoy Cut to -4m NGVD, Middle and Little Back Rivers to -3m NGVD and Diversion Structure on Front River at McCoy Cut Deepening these areas and constructing a diversion structure would allow greater amounts of fresh water flowing downstream on the Savannah to enter into the Back River system via McCoy Cut. By providing larger volumes of fresh water, the Back River system has greater flushing capacity, especially on the outgoing tide, to wash more saline water downstream and out of the system. Also, this feature would provide a barrier to higher concentrations of saline water coming up through the system from the access points on Middle River and through the sediment basin.

Due to the complexity of the system, mitigation features were also evaluated in combination. An initial screening of the features to be evaluated in combination was completed by the Corps and the Interagency Coordination Team reviewed the results. A plan to evaluate these features in combination was established. See **Figures 7.5.2-2** and **7.5.2-3**.

Figure 7.5.2-2: Mitigation Plans 1 – 5



Once mitigation Plans 1 – 5 were evaluated a collaborative decision was made to continue with plan development following Plan 3 or Plan 5. Upon review of Plan 5, it was discovered that the flow-altering features required to re-route Middle River have two negative impacts:

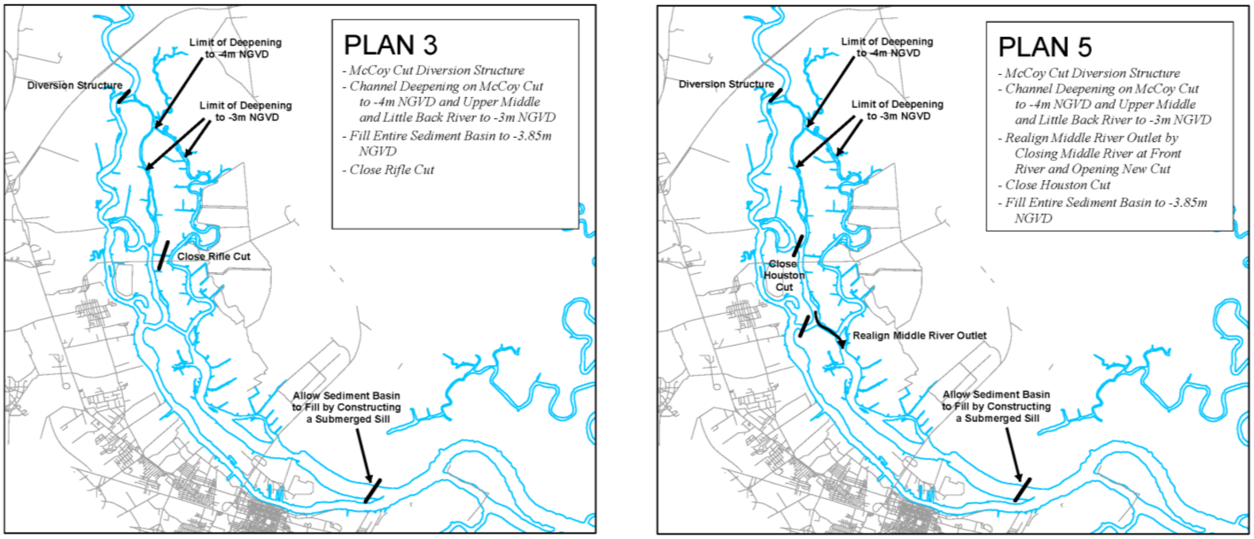
- The tidal range is decreased through Middle, Back and Little Back Rivers in a way that could negatively affect wetlands dependent on receiving freshwater tidal flows.

- Fish migrating through the estuary would no longer have the Middle River/Front River connection or pathway through Houston Cut to use for migration.

Based on these potential problems with mitigation Plan 5, it was eliminated as an alternative. Mitigation Plan 3 was then further developed with inclusion of features outlined in **Figure 7.5.2-3**.

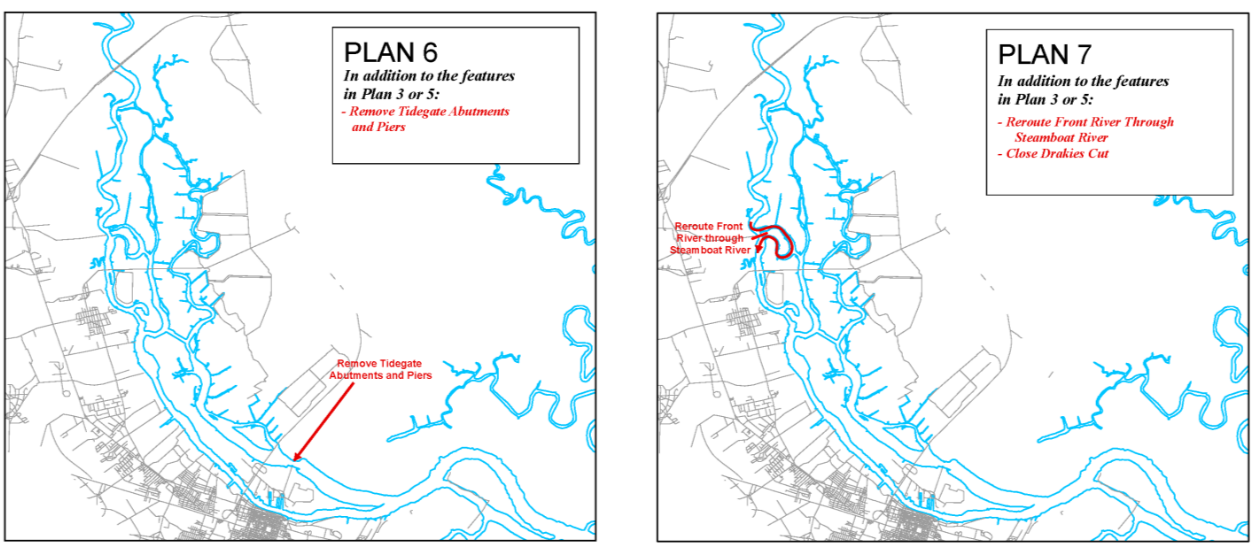
Figure 7.5.2-3: Mitigation Plans 6 & 7

Savannah Harbor Expansion Project Mitigation Plans



---------- Decision Point ----------

Continue forward with additional mitigation features building on Plan 3 or Plan 5?



At this point, Plan 6 and Plan 7 were compared and evaluated for effectiveness both from an impacts perspective and a cost perspective. Plan 7 is a mitigation concept that is particularly interesting involving re-routing Front River through Steamboat River. Steamboat River is historically the route of Front River prior to the construction of Drakies Cut. The adjustments made to the model to predict the results of plan 7 were complex and required close scrutiny to ensure the additional mitigation feature was modeled accurately. See **Figure 7.5.2-4** for an aerial view of Steamboat River.

The model predictions for Plan 7 show that the additional mitigation feature, routing Front River through Steamboat, does provide additional mitigation benefits. For the 48 ft deepening, the mapping shows that the freshwater zone ends just below Steamboat as compared to Plan 3 where the freshwater zone ends above Steamboat. It may be counterintuitive to think that the freshwater zone around Steamboat would remain fresh with the Front River routed through it. However, what the modeling shows is that when Steamboat is widened and deepened, the salinity becomes more stratified and the velocities increase. This change in geometry allows the less dense fresh water to stay on top of the water column and flush through the bend while providing fresh water to the marshes. In Plan 3 and under existing conditions, Steamboat is smaller and shallower. Velocities are low and the water column is less stratified. This allows more saline water to be available in the surface layer of the water column, for a longer period of time, to interact with the marsh; thus causing the freshwater zone to move above Steamboat. Despite the shallow depth of the sill at Houston Cut (-2.2 m NGVD), saline water moves from Steamboat through Houston Cut to Middle River and causes increases in surface salinity on Middle River. However, surface salinities on Front River above Steamboat are greatly reduced by lengthening the path that the saline water has to travel during each tidal cycle. The predominate path for surface salinity to move through the upper portion of the river system is Middle River. Near McCoy’s Cut, the surface salinities on Middle River are much higher than that of Front River. The salinity movement on Middle River appears to supply salinity to the upper portion of Little Back River.

Plan 7 does appear to be more effective in mitigating the impacts due to deepening than Plan 3 when viewing the acreage impacts based on the 50th percentile salinity contours. For the 48 ft depth, there is approximately 180 more freshwater marsh/wetland acres available. However, at the 48 ft depth, Plan 7 does not fully mitigate the impacts to freshwater marsh/wetlands and additional mitigation measures would likely be required. Despite the benefits shown by computing the 50th percentile surface salinities and subsequent marsh/wetland acreages impacted, Plan 7 does appear to increase surface salinity on Middle River. As shown in **Table 7.5.2-3**, surface salinity on Middle River increases significantly for percentiles greater than 50. The increases on Middle River are more pronounced with drought flows. During those high tide periods, Plan 7 has the potential to provide more saline water to wetlands than that of Plan 3.

Figure 7.5.2-4: Steamboat River

Table 7.5.2-3: Surface Salinity Percentile Value Comparisons

Percentile	Existing Conditions	Plan 3 48 ft Depth	Plan 7 48 ft Depth
10	0.00	0.00	0.00
20	0.01	0.02	0.01
30	0.05	0.06	0.04
40	0.11	0.14	0.11
50	0.20	0.27	0.24
60	0.33	0.45	0.48
70	0.48	0.68	0.82
80	0.68	1.06	1.42
90	1.00	1.63	2.54

**All salinity values are from Middle River – Grid Cell 26_102*

For these reasons, along with high construction costs, Plan 7 was eliminated from further review. Plan 6, however, did show value over Plan 3 and was further evaluated.

Through the mitigation development process, additional features had been identified as potentially effective at reducing salinity and were also evaluated. See **Table 7.5.2-4** for a matrix of these additional plans. Model results for these plans along with all other plans evaluated can be found in the report titled *Mitigation Evaluation for Marsh/Wetland Impacts* included in the Engineering Investigations Supplemental Materials.

Table 7.5.2-4: Additional Mitigation Plan Alternatives

	McCoy Cut Diversion Structure	**Channel Deepening on McCoy, Upper Middle and Little Back River**	**Fill Entire Sediment Basin**	**Close Rifle Cut**	**Close Lower (western) Arm at McCoy Cut**	**Remove Tidegate Abutments and Piers**
Plan 3	x	x	x	x		
Plan 3a	x	x	x	x	x	
Plan 3b	x		x	x		
Plan 3c	x		x	x	x	
Plan 6	x	x	x	x		x
Plan 6a	x	x	x	x	x	x
Plan 6b	x		x	x	x	x

The following table, **Table 7.5.2-5**, shows a summary of the estimated marsh salinity impacts for only the freshwater marsh zone (0.0 to 0.5 ppt) for each of the mitigation plans described above for the full 48 ft proposed channel depth (a greatest impact condition). These calculations, along with cost considerations were factors in selecting the final mitigation plan.

Table 7.5.2-5: Estimated Marsh Acreages for Additional Mitigation Plans

Modeled Scenario	**Estimated Freshwater Marsh Acreages (0.0 – 0.5 ppt)**
Existing Conditions	4072
48 ft Depth ONLY	2860
Plan 3	3584
Plan 3a	3531
Plan 3b	3406
Plan 3c	3383
Plan 6	3715
Plan 6a	3735
Plan 6b	3610

All mitigation plans were evaluated with the 48 ft channel depth.

The process of developing mitigation features and evaluating their effectiveness has been an ongoing, collaborative effort between the PDT and the Interagency Coordination Team. All of the mitigation scenarios were evaluated, alone and in combination, based on the effectiveness at reducing salinity impacts, observed in the initial modeling, and preliminary estimates of construction costs. Details of how each mitigation plan was finalized and selected can be found in the EIS. Model results for all plans evaluated can be found in the report titled *Mitigation Evaluation for Marsh/Wetland Impacts* included in the Engineering Investigations Supplemental Materials.

7.5.2.1 Proposed Mitigation Features

Based on analysis of the model output, the flow-altering mitigation plans that were found to be the most effective at reducing salinity impacts and protecting freshwater tidal marshes are Plan 6a for the 48 ft, 47 ft, 46 ft, and 45 ft channel depths and 6b for the 44 ft channel depth. Although the plans do not fully mitigate for all impacts to the estuary, they are expected to provide substantial benefits to the freshwater marsh ecosystems adjacent to the Back and Little Back Rivers. For details on the model output for these two selected flow-altering scenarios see the EIS and report titled *Evaluation of Marsh/Wetland Impacts with Proposed Mitigation Plan* dated November 2007 and the *Addendum* dated July 2011which are both included in the Engineering Investigations Supplemental Materials.

Plan 6b is the proposed flow-altering mitigation plan for the 44 ft channel depth. The features of this plan include a diversion structure on Front River, closure of the lower (western) arm at McCoy Cut, closure of Rifle Cut, filling of the Sediment Basin and removal of the tidegate abutments and piers. This plan provides potential for additional freshwater flows to enter the Back River System at McCoy Cut, without exiting through the lower (western) arm, and flow downstream through Middle, Back and Little Back Rivers. It also has features that will limit salt water intrusion to the Back River area through the sediment basin and Rifle Cut. See **Figure 7.5.2.1-1**.

The modeling results, regarding wetland impacts, for implementing mitigation Plan 6b with the 44 ft depth are shown in **Table 7.5.2.1-1**. A plot of the estimated wetland impacts for the 44 ft depth with mitigation Plan 6b is shown in **Figure 7.5.2.1-2**. The estimated freshwater marsh salinity impacts (range 0.0 – 0.5) are fully mitigated with the 44 ft channel depth with mitigation Plan 6b, i.e. there is no net loss. The additional created freshwater wetlands are adjacent to Middle and Back River where the mitigation plan has the greatest effect.

Plan 6a is the proposed mitigation plan for the 45, 46, 47, and 48 ft channel depths. This plan includes all the features of Plan 6b and one additional feature, channel deepening on McCoy Cut, upper Middle and Little Back Rivers. This additional feature in combination with the features in Plan 6b maximizes the potential for additional freshwater flows to enter the Back River System at McCoy Cut and flow downstream through Middle, Back and Little Back Rivers. See **Figure 7.5.2.1-3**.

The modeling results, regarding wetland impacts, for implementing mitigation Plan 6a with the 45, 46, 47 and 48 ft depth are shown in **Table 7.5.2.1-2**. A plot of the estimated wetland impacts for the 48 ft depth with mitigation Plan 6a is shown in **Figure 7.5.2.1-4**. The estimated freshwater marsh salinity impacts (range 0.0 – 0.5) are not fully mitigated with the 48 ft channel depth with mitigation Plan 6a, there is a net loss under the average flow conditions. The losses are largely adjacent to Front River where the deepening is occurring, which makes mitigation in these areas very difficult. The marsh salinity estimates show that wetlands are protected and created adjacent to Middle and Back River where the mitigation plan has the greatest effect. However, the created freshwater wetlands are not large enough to fully mitigate for the losses of freshwater wetlands on Front River.

Figure 7.5.2.1-1: Flow-altering Mitigation Plan 6b

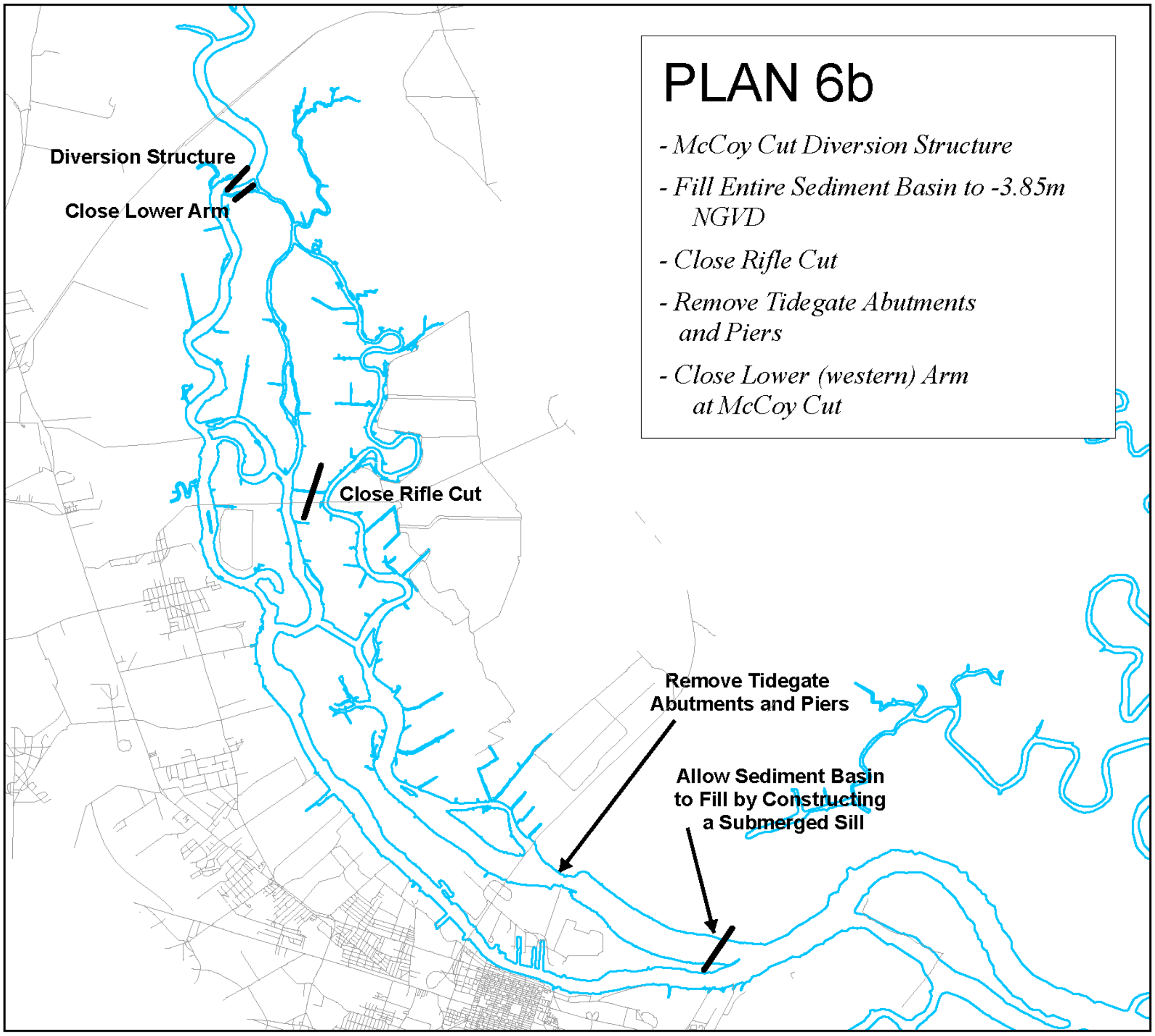


Table 7.5.2.1-1: Wetland Acreages with Mitigation Plan 6b for Basic Evaluation

Salinity Range (ppt)	Existing No Deepening No Mitigation	44 ft Depth Deepening Only No Mitigation	44 ft Depth & Mitigation Plan 6b
0.0 – 0.5	4072	3521	4394
0.6 – 1.0	864	1186	1137
1.1 – 2.0	555	397	749
2.1 – 4.0	834	863	855
> 4.0	2506	2865	1698

Figure 7.5.2.1-2: Estimated Wetland Impacts with 44 ft Depth and Mitigation Plan 6b

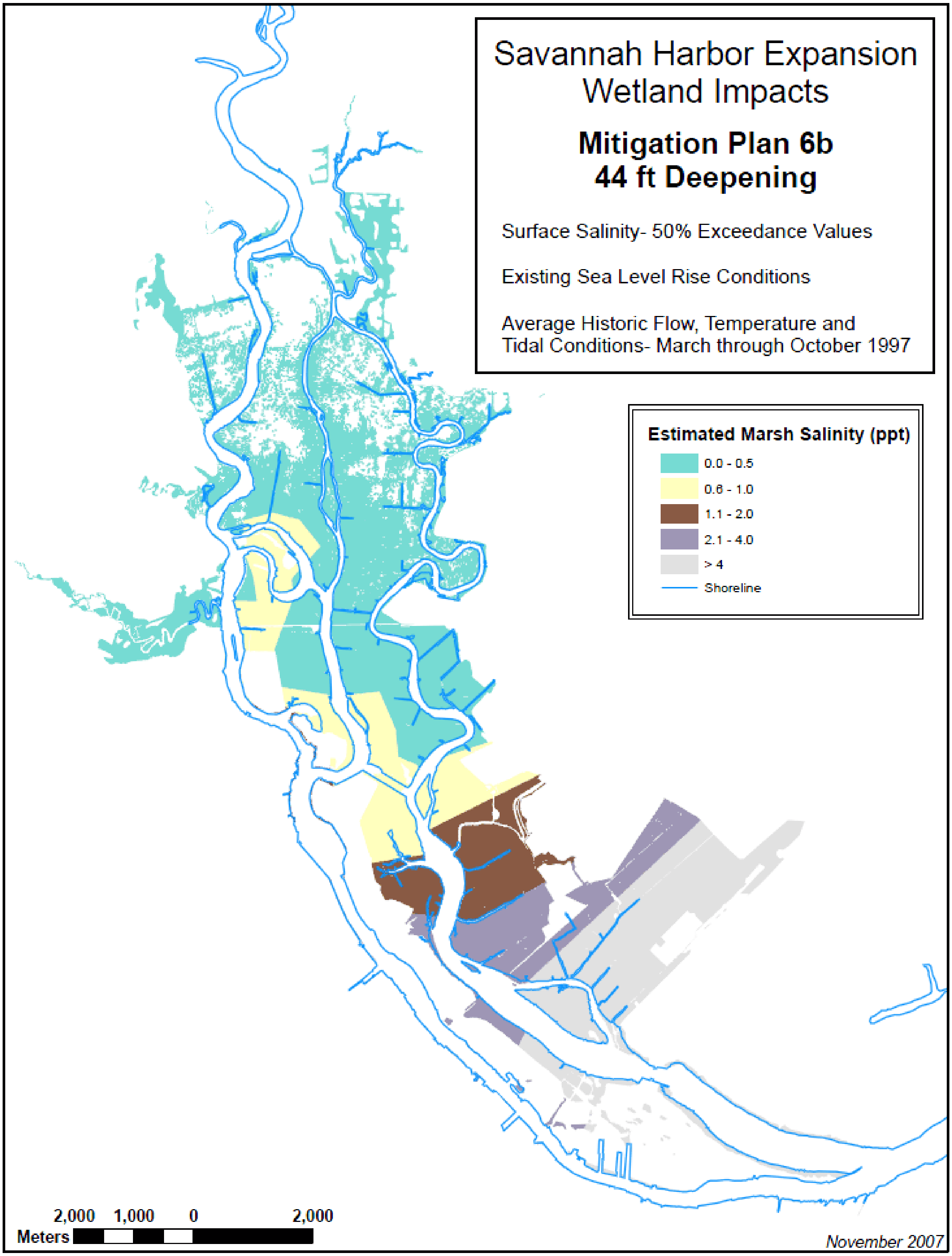


Figure 7.5.2.1-3: Flow-altering Mitigation Plan 6a

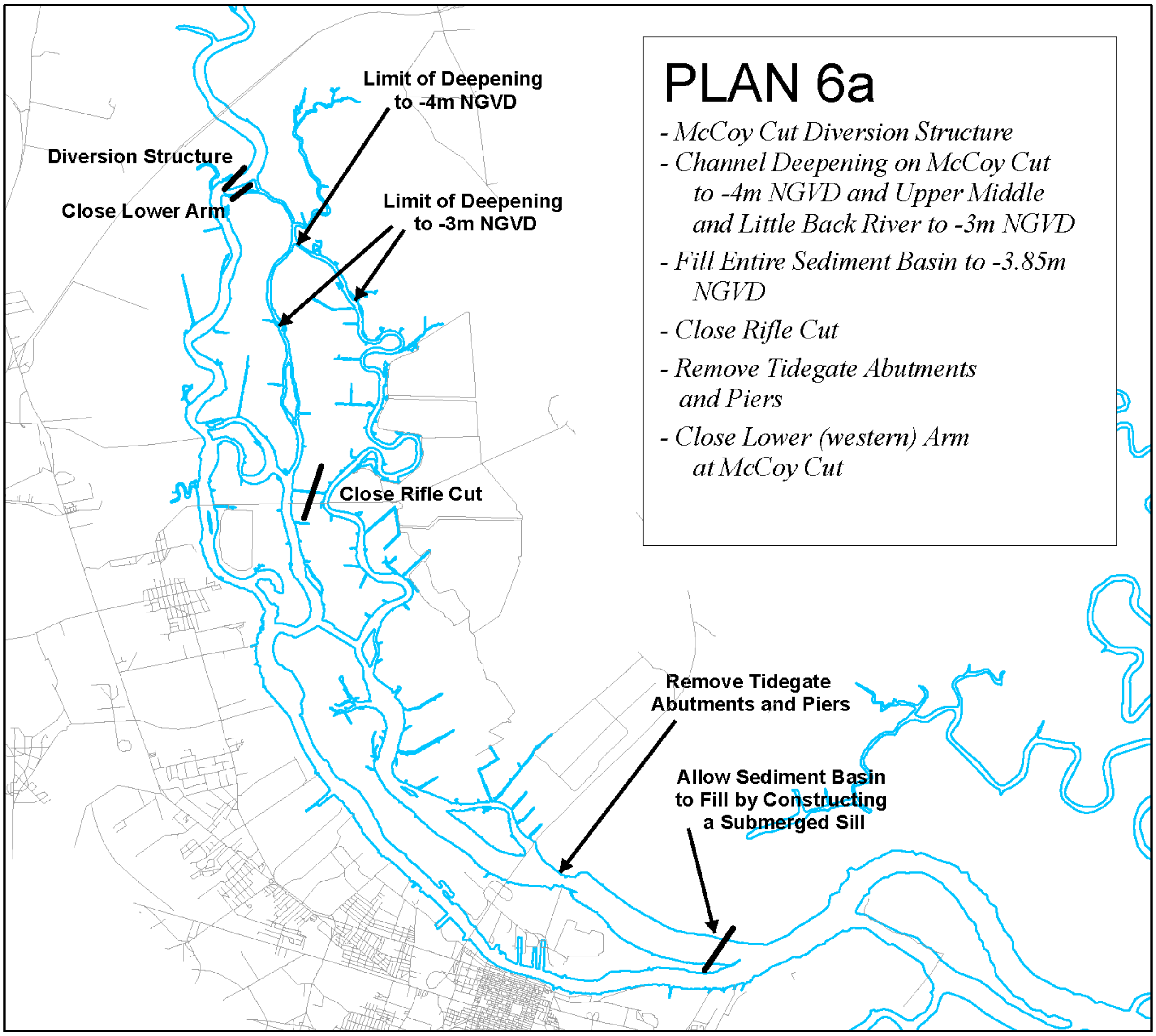
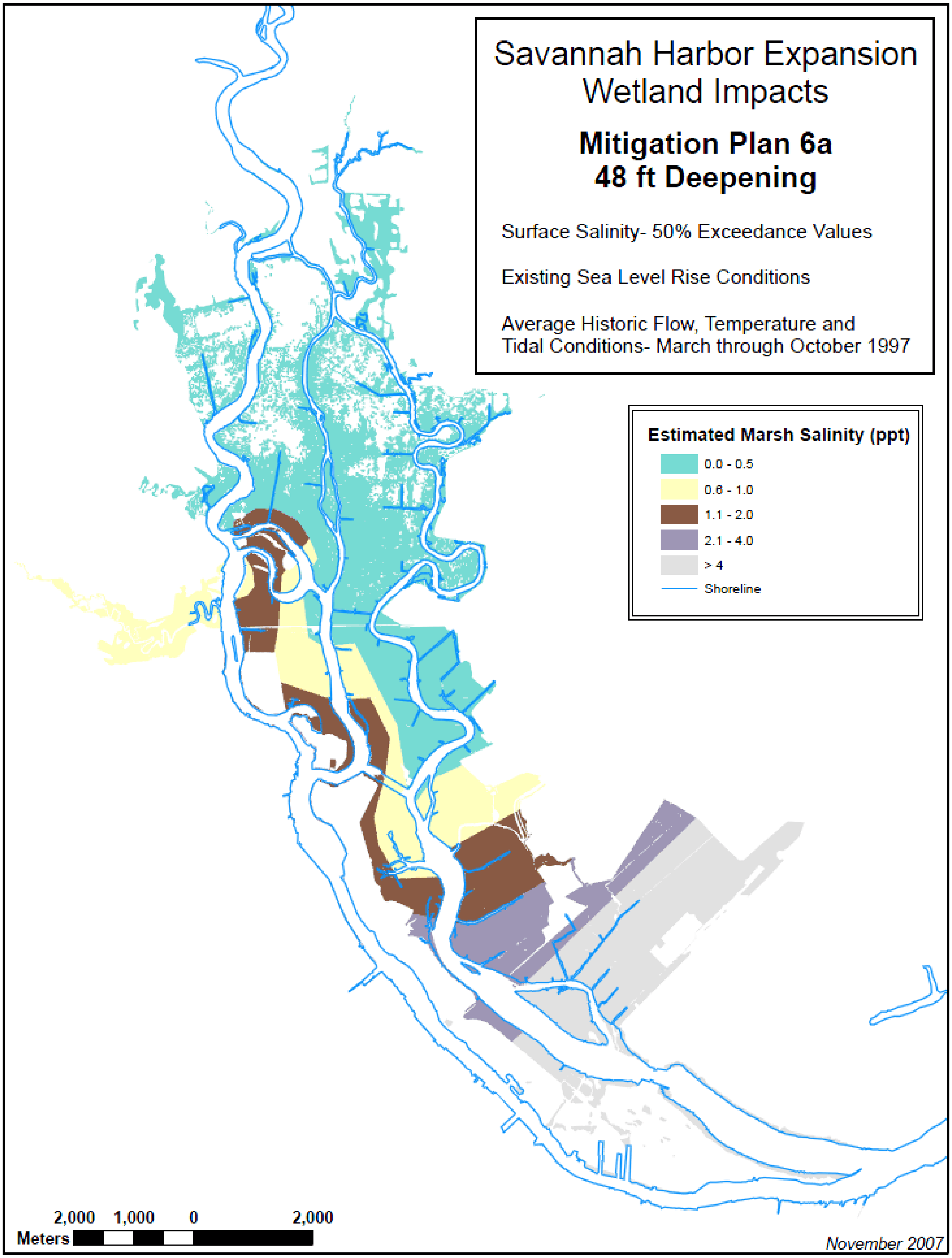


Table 7.5.2.1-2: Wetland Acreages with Mitigation Plan 6a for Basic Evaluation

Salinity Range (ppt)	Existing No Deepening No Mitigation	45 ft Depth Deepening Only No Mitigation	45 ft Depth & Mitigation Plan 6a
0.0 – 0.5	4072	3105	4040
0.6 – 1.0	864	1319	1781
1.1 – 2.0	555	630	588
2.1 – 4.0	834	906	745
> 4.0	2506	2873	1678
Salinity Range (ppt)	**Existing No Deepening No Mitigation**	**46 ft Depth Deepening Only No Mitigation**	**46 ft Depth & Mitigation Plan 6a**
0.0 – 0.5	4072	3015	3871
0.6 – 1.0	864	1050	1650
1.1 – 2.0	555	921	862
2.1 – 4.0	834	789	700
> 4.0	2506	3057	1749
Salinity Range (ppt)	**Existing No Deepening No Mitigation**	**47 ft Depth Deepening Only No Mitigation**	**47 ft Depth & Mitigation Plan 6a**
0.0 – 0.5	4072	2895	3849
0.6 – 1.0	864	1009	1641
1.1 – 2.0	555	1355	889
2.1 – 4.0	834	1365	687
> 4.0	2506	2208	1766
Salinity Range (ppt)	**Existing No Deepening No Mitigation**	**48 ft Depth Deepening Only No Mitigation**	**48 ft Depth & Mitigation Plan 6a**
0.0 – 0.5	4072	2860	3735
0.6 – 1.0	864	830	1340
1.1 – 2.0	555	1215	1191
2.1 – 4.0	834	739	790
> 4.0	2506	3188	1776

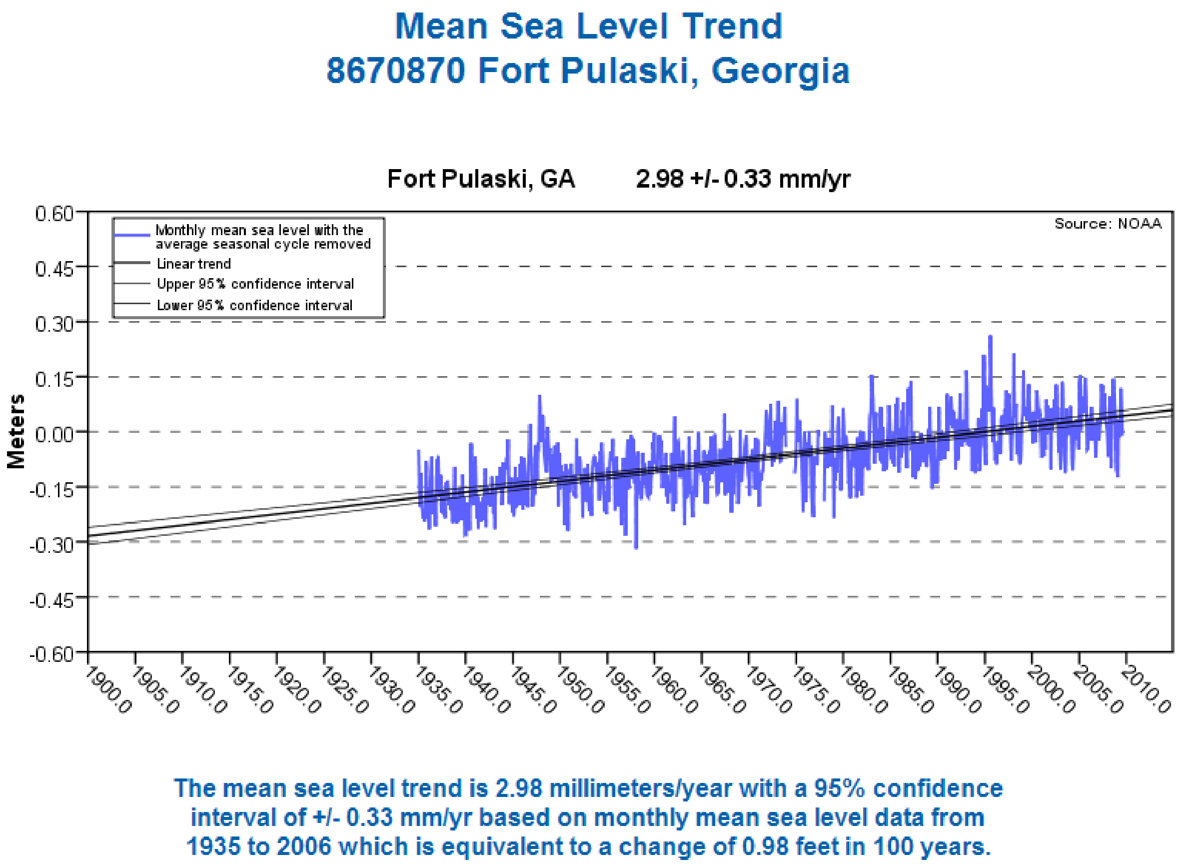
Figure 7.5.2.1-4: Estimated Wetland Impacts with 48 ft Depth and Mitigation Plan 6a



7.5.2.2 Relative Sea Level Change

According to NOAA, the historic sea level change trend at the Fort Pulaski gage based on 70 plus years of data collection is a rise of 2.98 mm/year (see **Figure 7.5.2.2-1**), which is a combination of the global sea-level rise and local vertical land movement. Scientific opinions vary on how this trend will continue in the coming years with the effect of greenhouse gases, changing climate, and other variables influencing sea level change. However, there is little debate that sea level change could become a major cause of future wetland loss throughout the coastal zone of the United States.

Figure 7.5.2.2-1: Historic Sea Level Change Trend at Fort Pulaski, Georgia



Data obtained from: http://co-ops.nos.noaa.gov/sltrends

USACE guidance, EC 1165-2-212, titled *Sea-Level Change Considerations for Civil Works Programs*; dated October 1, 2011 outlines a process for estimating local sea level changes for analysis and incorporation into Civil Works Projects. This guidance supersedes the sea level change analysis outlined in the *Planning Guidance Notebook*, ER 1105-2-100 and EC 1165-211. **Figure 7.5.2.2-2** shows the calculations to determine the relative sea level change for Savannah Harbor over the 50 year project life. As shown in **Figure 7.5.2.2-2** and **Figure 7.5.2.2-3**, which are based on the latest USACE guidance: after 50 years the sea level change estimates are 0.5 ft, 0.9 ft, and 2.3 ft, for the low, intermediate and high scenarios, respectively.

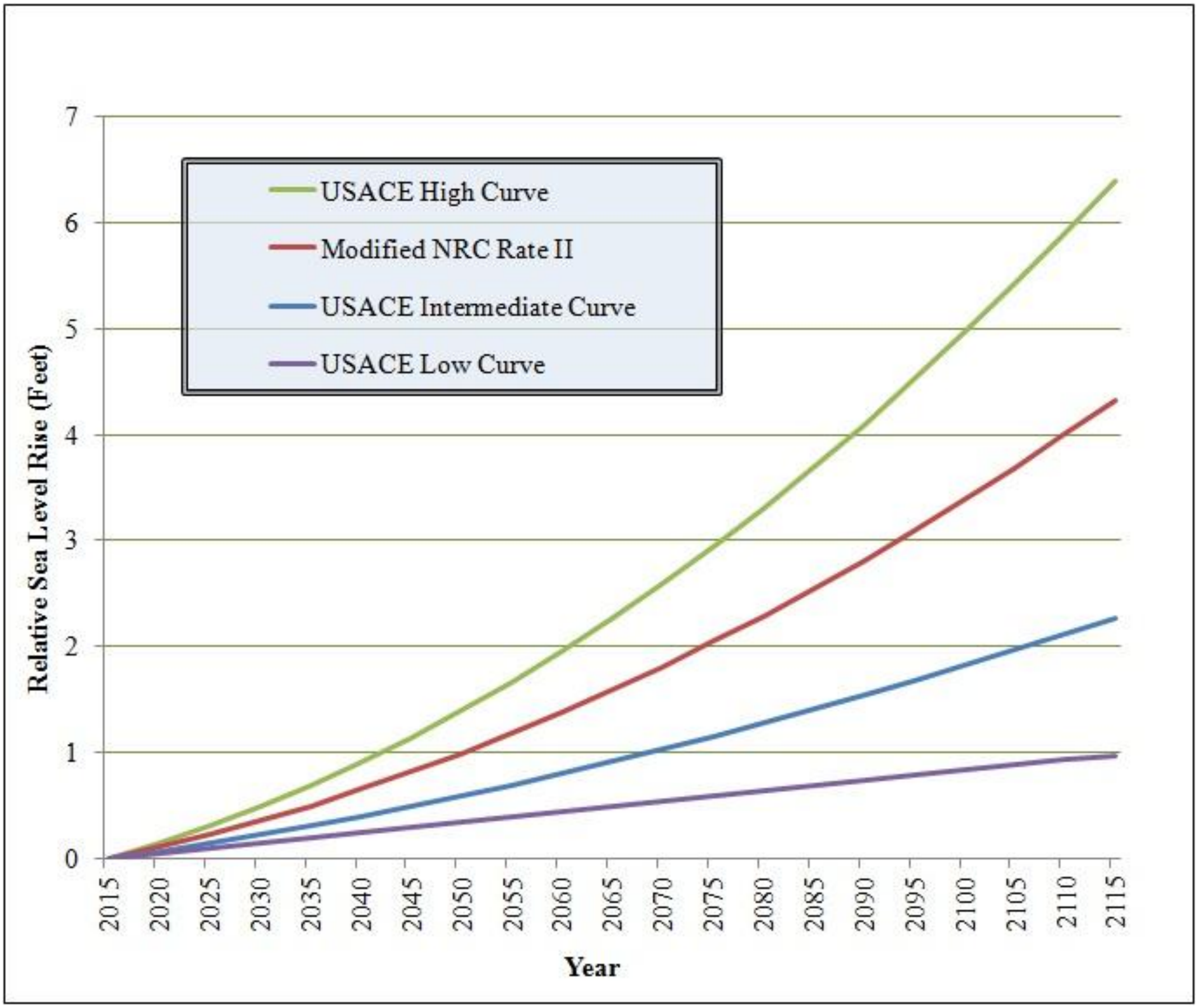
Figure 7.5.2.2-2: Relative Sea Level Change Estimates for SHEP, Based on EC 1165-2-212

Eustatic Sea Level Rise	Start	Local Land Movement (mm)	b
mm/yr	Year	mm/yr	coefficients
1.7	1992	1.28	0.0000271
8670870 - Fort Pulaski, GA: 2.98 (mm/yr)			0.0000700
			0.0001130

Project			NTDE
Start	Life	End	83-01
2015	50	2065	1992

	Sea Level Rise (feet)			
Year	Low	Int	NRC II	High
2015	0.00	0.00	0.00	0.00
2020	0.05	0.07	0.11	0.14
2025	0.10	0.15	0.23	0.31
2030	0.15	0.23	0.36	0.49
2035	0.20	0.31	0.50	0.68
2040	0.24	0.40	0.65	0.90
2045	0.29	0.50	0.82	1.14
2050	0.34	0.59	0.99	1.39
2055	0.39	0.70	1.18	1.67
2060	0.44	0.80	1.38	1.96
2065	**0.49**	**0.92**	**1.59**	**2.27**
2070	0.54	1.03	1.81	2.60
2075	0.59	1.15	2.05	2.94
2080	0.64	1.28	2.29	3.31
2085	0.68	1.41	2.55	3.69
2090	0.73	1.54	2.82	4.10
2095	0.78	1.68	3.10	4.52
2100	0.83	1.82	3.39	4.96
2105	0.88	1.97	3.69	5.42
2110	0.93	2.12	4.00	5.89
2115	0.98	2.28	4.33	6.39

Figure 7.5.2.2-3: Relative Sea Level Change Projection Over Time for SHEP, Based on EC 1165-2-212



In coordination with our Cooperating Agencies and State natural resource agencies, sea level change estimates for alternative analysis with SHEP were determined from the EPA published report titled *The Probability of Sea Level Rise* authored by J. G. Titus and V. K. Narayanan published October 1995. The recommended procedure outlined in the EPA report uses the local historic sea level change trend of a rise of approximately 3 mm/year plus a normalized projection estimate to account for contributions from greenhouse gases, a changing climate, and other factors. The normalized projection estimates the extent and probability to which future sea level rise will exceed what would have happened if the current trends simply continued. For Savannah, over the 50 year life of the project, these estimates ultimately result in a rise in sea level of 25 cm (0.8 ft) and 50 cm (1.6 ft), for the median and 1% high normalized projections. In addition to the two projections, alternatives were evaluated with a sea level rise of 0 ft, which would be the conditions experienced on day-one after project construction. **Table 7.5.2.2-1** shows a comparison of sea level rise estimates using the three methods outlined above.

Table 7.5.2.2-1: Comparison of Sea Level Rise Estimates

Scenario	Estimate per ER 1105-2-100	Estimate per EC 1165-2-212	Estimate per Cooperating Agencies
Low	0.5 ft	0.5 ft	n/a
Intermediate	n/a	0.9 ft	0.8 ft
High	2.0 ft	2.3 ft	1.6 ft

Estimates based on year-50 of a 50 year project life.

7.5.2.2.1 Impacts to Freshwater Tidal Wetlands Due to Relative Sea Level Change

Impacts to freshwater tidal wetlands/marshes, with various sea level rise estimates, were determined using model simulations and interpolation. **Figure 7.5.2.2.1-1** shows the relationship between sea level rise and the resulting acreage impacts to freshwater tidal wetlands/marshes expected to occur after construction of the deepening project and the flow-altering mitigation plans. This figure is for the 48 ft project depth; however, similar figures were developed to aid analysis for each project depth alternative. The curve shown on this figure enables predictions of the freshwater tidal wetland/marsh impacts for any sea level rise scenario considered in the analysis.

Table 7.5.2.2.1-1 and **Table 7.5.2.2.1-2** show projected wetland impact acreages using the rates of sea level rise specified in the EC as well as the rates recommended by the SHEP Cooperating Agencies. Projected estimates of wetland impact acreages presented in the tables show continued impact to wetlands, even with flow-altering mitigation, under the historic rate of sea level change. For the 48 ft project depth, these wetland impacts would decline from 337 acres in the base year to 130 acres at the end of the 50-year period of analysis. For all additional estimates of sea level change, ranging from 0.8 to 2.3 ft, at the end of the 50-year project life, there is a net gain in freshwater tidal wetlands/marshes due to implementation of the mitigation plan.

The effects of sea level rise on freshwater marsh impacts over the 50-year project life using the historic rate of sea level rise were calculated using an average annual equivalent calculation methodology and are shown in **Table 7.5.2.2.1-2**. For the 48 ft project depth, the average annual equivalent tidal wetland/marsh impact is -284 acres. When compared to the base year prediction of -337 acres, the difference is approximately 50 acres, which is within the degree of accuracy of the impact predictions. The proposed project would mitigate for the impacts that would occur from the base year. This would ensure the project fully mitigates for impacts it would produce over the entire period of analysis.

The alternative projections of 50-year future sea level rise are presented for information and for comparison. Mitigation requirements are significant at the end of construction but would likely decrease substantially by the end of the 50-year life of the project.

Although the guidance directs design for 50 years in the future, the intention is for the design to be robust and reliable throughout the entire 50-year period of analysis. The marsh mitigation acreage was computed based on conditions expected to occur in the base year when construction is complete. The historic (and most likely future conditions) sea level rise projection is within the limits of error of the modeling and impact determination.

Risk and uncertainties associated with estimates of sea level rise are outlined in greater detail in Section 15.0 of the Engineering Investigations.

Figure 7.5.2.2.1-1: Projected Freshwater Tidal Wetland/Marsh Acreage Impacts (48 ft Project Depth With Mitigation Plan 6a)

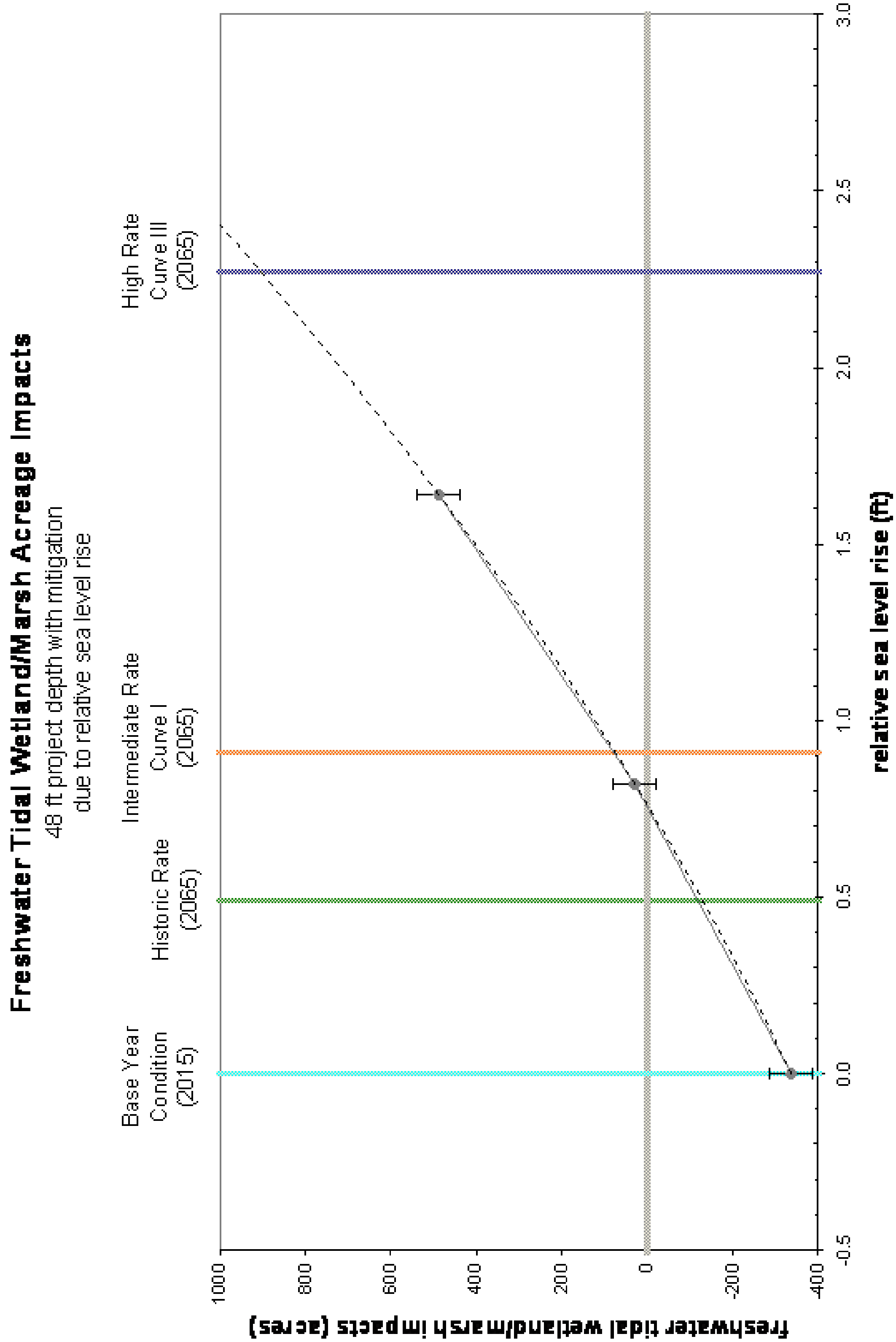


Table 7.5.2.2.1-1: Freshwater Tidal Wetland Acreage Impacts Due to Relative Sea Level Change (Base Year and 50-Year Project Life Projections)

Project Depth*	Relative Sea Level Rise (ft)** per EC 1165-2-212					
	Base Year Condition (2015)	Historic Rate (2065)	SHEP Cooperating Agencies Low Rate (2065)	Intermediate Rate EC 1165-2-212 (2065)	SHEP Cooperating Agencies High Rate (2065)	High Rate EC 1165-2-212 (2065)
	0 ft	0.5 ft	0.8 ft	0.9 ft	1.6 ft	2.3 ft
48 ft	-337	-130	29	73	488	903
47 ft	-223	-86	58	103	595	1181
46 ft	-201	-69	87	137	703	1398
45 ft	-32	96	233	275	745	1305
44 ft	322	140	152	173	653	1492

**All depth alternatives include mitigation features (depths are below MLLW).*
***Positive numbers indicate a projected net gain in freshwater wetlands after construction of deepening and flow-altering mitigation. Negative numbers indicate a projected net loss in freshwater wetlands after construction of deepening and flow-altering mitigation.*

Table 7.5.2.2.1-2: Freshwater Tidal Wetland Acreage Impacts Due to Relative Sea Level Change (Project Life Projections Historic Rate of Sea Level Rise)

Project Depth*	Historic Rate**						Average Acreage Impact
	2015	2025	2035	2045	2055	2065	
	Base Year	10 yr	20 yr	30 yr	40 yr	50 yr	
	0.0 ft	0.1 ft	0.2 ft	0.3 ft	0.4 ft	0.5 ft	
	0 cm	3 cm	6 cm	9 cm	12 cm	15 cm	
48 ft	-337	-297	-256	-214	-170	-130	-234
47 ft	-223	-202	-178	-150	-118	-86	-161
46 ft	-201	-183	-161	-134	-102	-69	-143
45 ft	-32	-13	10	36	66	96	26
44 ft	322	265	219	182	155	140	210

**All depth alternatives include mitigation features (depths are below MLLW).*
***Positive numbers indicate a projected net gain in freshwater wetlands after construction of deepening and flow-altering mitigation. Negative numbers indicate a projected net loss in freshwater wetlands after construction of deepening and flow-altering mitigation.*

7.5.3 Water Quality & Dissolved Oxygen Mitigation

The previously described, flow-altering mitigation plans do little to mitigate for SHEP impacts to dissolved oxygen (D.O.) levels in the harbor, especially on Front River in the deepened navigation channel. A study prepared for the District titled *Identification and Screening Level Evaluation of Measures to Improve Dissolved Oxygen in the Savannah River Estuary* completed by MACTEC in 2005 identified a D.O. injection system as being the most cost effective method to improve D.O. levels in the harbor. This study is included in the Engineering Investigations Supplemental Materials.

The oxygen injection technology was developed by Dr. Richard Speece. Dr. Speece invented the Speece Cone, a device originally used to add oxygen to the bottom of lakes to enhance downstream fisheries. The Speece Cones are now designed and produced by Eco-Oxygen Technologies, LLC. The Speece Cone technology is a simple process based upon the scientific principle of Henry's Law. No chemicals and no moving parts other than standard municipal wastewater pumps are used. The result is a robust, reliable, economically competitive, and environmentally friendly method. The technology pulls a small sidestream from the river, superoxygenates the water (using pure oxygen), and dilutes it back in the river to satisfy dissolved oxygen deficiencies without treating the entire river. See **Figure 7.5.3-1** for a photo of a Speece Cone.

Figure 7.5.3-1: Speece Cone Demonstration Project

Photo from the Georgia Ports Authority demonstration project managed by MACTEC in the summer of 2007. Results of the study are documented in the 2008 MACTEC report titled *Savannah Harbor Reoxygenation Demonstration Project, Savannah, Georgia* which was prepared for the GPA, which is included in the Engineering Investigations Supplemental Materials.

To mitigate for D.O. impacts resulting from SHEP, a plan has been developed to inject superoxygenated water into the estuary using the Speece Cone technology. This mitigation plan was developed by Tetra Tech and results are documented in the report titled *Oxygen Injection Design Report Savannah Harbor Expansion Project* dated October 15, 2010, which is included in the Engineering Investigations Supplemental Materials. After this report was finalized, the D.O. loading amounts to mitigate for SHEP impacts were revised to incorporate effects from flow-altering mitigation features and point source loadings in the harbor. The injection locations identified in the 2010 report did not change, only the required D.O. loadings and number of Speece Cones. Previous analysis excluded the point source loadings to capture D.O. impacts solely due to harbor deepening. However, it was determined that the point source loadings needed to be included in the modeling to capture expected future conditions and ensure that the D.O. mitigation goals were met with the injection system design. **Table 7.5.3-1** shows the mitigation success of the D.O. injection system design for each depth alternative. This summary includes the point source loadings and mitigation features in the model simulations.

Table 7.5.3-1: Mitigation Success for D.O. Injection System Design (%)

	Vertical Layer	44 ft depth	45 ft depth	46 ft depth	47 ft depth	48 ft depth
5th percentile	Surface	99.9	99.7	99.9	99.9	99.9
	Mid-Depth	94.4	98.3	98.1	98.7	98.5
	Bottom	97.2	97.4	97.8	98.1	97.2
	Water Column	98.3	99.9	99.9	99.9	99.9
10th percentile	Surface	99.9	99.9	99.8	99.9	99.9
	Mid-Depth	95.3	99.2	99.1	99	99.1
	Bottom	97.5	97.5	97.9	98.4	97.1
	Water Column	98.4	99.9	99.9	99.9	99.9
25th percentile	Surface	99.9	99.9	99.9	99.9	99.9
	Mid-Depth	95.5	99.4	99.3	99.1	99.2
	Bottom	97.9	97.7	98	98.1	97.7
	Water Column	98.7	99.9	99.9	99.9	99.9
50th percentile	Surface	99.9	99.9	99.9	99.9	99.9
	Mid-Depth	96.3	97.7	97.7	98.1	97.8
	Bottom	98	98.4	97.8	97.2	97.1
	Water Column	99.1	99.9	99.8	99.8	99.9

In developing the D.O. mitigation plan, the same EFDC and WASP hydrodynamic and water quality models approved for use with the project were used. Tetra Tech used these models to determine the amount of oxygen required to mitigate the harbor deepening effects. The results of the modeling study prescribe locations and D.O. loading amounts to mitigate for SHEP impacts for each depth alternative. See **Table 7.5.3-2.**

To calculate the number of Speece Cones required, 4,000 pounds per day was used based on the 80% efficiency (5,000 pounds per day per cone by design). This efficiency percentage was determined from lessons learned after the 2007 demonstration project.

Table 7.5.3-2: Summary of Dissolved Oxygen Loads for Mitigation

Project Depth Alternative	Load (lbs/day)	Total Number of Speece Cones Required*	Location		
			Near Plant McIntosh	Near IP (Front River)	Near IP (Back River)
44 ft Depth	36,000	9	6	1	2
45 ft Depth	32,000	8	6	1	1
46 ft Depth	36,000	9	7	1	1
47 ft Depth	40,000	10	7	2	1
48 ft Depth	44,000	11	6	4	1

** One additional Speece Cone will be installed at each site location as a spare for use when maintenance or repairs are required.*

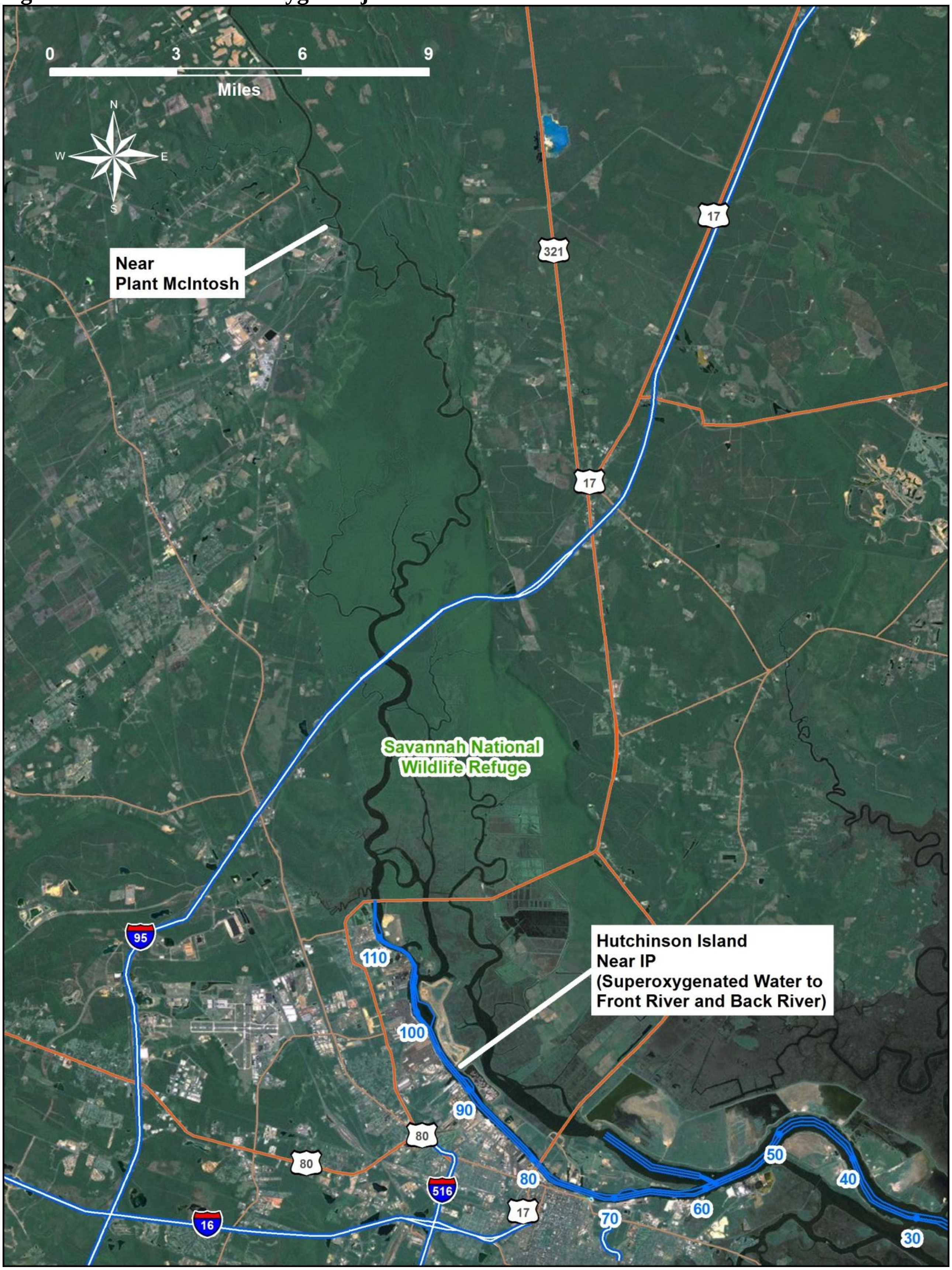
Figure 7.5.3-2 shows the locations for the proposed D.O. injection systems. For SHEP mitigation two Speece Cone injection locations are required to inject oxygen into the river: 1) Near Plant McIntosh which is in Effingham County, GA upstream of I-95 and 2) Hutchinson Island near International Paper (IP). The Hutchinson Island site has several Speece Cones which are divided, as prescribed by the modeling, to inject superoxygenated water into both Front River and Back River.

In response to questions raised by the South Carolina Department of Health and Environmental Control (DHEC) additional analyses of the oxygen injection systems were conducted. Results of the additional analyses can be found in the report titled *Analysis of Oxygen Injection in the Back River in Support of the Savannah Harbor Expansion Project* prepared by Tetra Tech and Eco Oxygen Technologies dated July 26, 2011 which is included in the Engineering Investigations Supplemental Materials.

Questions raised by DHEC included D.O. system performance concerns in shallow depths on Back River. Hydrographic surveys of the Back River were completed by Savannah District in June 2011 between the tidegate structure and New Cut. The bathymetry showed the depth near the injection location on Back River to be approximately 15 feet below MLLW.

The additional Speece cone research, consultation with the manufacturer, updated bathymetry, revised diffuser design calculations and mixing zone modeling runs conclude that the Speece cones will function as intended on Back River. The analysis concluded that the oxygen plume is readily mixed due to advection and dispersion in the Back River and does not have the potential for effervescent loss.

Figure 7.5.3-2: Dissolved Oxygen Injection Locations



Tables 7.5.3-3 through **7.5.3-14** show the D.O. output for the zone averages and critical cells by percentile for existing conditions and each project depth with mitigation (Plan 6a/6b and D.O. injection). For more information on the critical cells and zones see discussion in the previous Section 7.4.3.

Table 7.5.3-3: Dissolved Oxygen Percentiles for Zones (Existing Conditions)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%
FR1	4.09	4.16	4.21	4.32	4.44	4.55	4.68	4.76	4.82
FR2	3.86	3.91	3.95	4.15	4.28	4.41	4.51	4.58	4.62
FR3	3.58	3.66	3.72	3.88	4.06	4.25	4.52	4.60	4.81
FR4	3.52	3.57	3.61	3.82	3.96	4.18	4.75	5.15	5.41
FR5	3.53	3.62	3.69	3.91	4.08	4.58	5.26	5.53	5.69
FR6	3.79	3.83	3.92	4.13	4.36	5.04	5.69	5.86	5.97
FR7	4.25	4.36	4.52	4.92	5.78	6.15	6.38	6.53	6.68
FR8	4.71	4.92	5.13	5.57	6.13	6.42	6.67	6.79	6.96
FR9	5.60	5.87	5.99	6.24	6.53	6.80	7.05	7.21	7.33
FR10	5.71	5.85	6.01	6.30	6.57	6.81	7.16	7.23	7.32
FR11	4.88	5.10	5.28	5.59	5.88	6.18	6.45	6.55	6.68
MR1	4.29	4.41	4.55	4.79	5.06	5.47	5.77	5.89	5.99
MR2	4.17	4.29	4.47	4.73	5.05	5.40	5.73	5.84	5.98
MR3	3.84	4.02	4.09	4.36	4.71	5.19	5.55	5.67	5.79
MR4	4.38	4.50	4.60	4.77	5.04	5.23	5.43	5.53	5.69
MR5	2.31	2.55	2.96	3.46	5.33	6.16	6.53	6.82	7.01
MR6	2.15	2.53	3.05	3.58	5.69	6.32	6.80	6.94	7.27
LBR1	4.29	4.49	4.58	4.79	4.98	5.18	5.29	5.44	5.56
LBR2	3.69	3.80	3.95	4.13	4.35	4.55	4.70	4.76	4.89
LBR3	3.52	3.56	3.63	3.77	3.93	4.08	4.22	4.31	4.42
BR1	3.42	3.47	3.52	3.77	3.90	4.06	4.24	4.32	4.42
BR2	3.17	3.25	3.34	3.47	3.65	3.83	3.96	4.11	4.19
BR3	3.36	3.41	3.46	3.52	3.63	3.74	3.84	3.87	3.90
SCh1	3.40	3.46	3.53	3.61	3.72	3.87	3.95	4.02	4.08
SCh2	3.84	3.94	3.99	4.11	4.26	4.38	4.48	4.53	4.63
SR	4.90	4.95	5.18	5.52	5.84	6.17	6.35	6.41	6.48
StbR	4.73	4.91	5.07	5.39	5.75	6.06	6.25	6.38	6.54

Table 7.5.3-4: Dissolved Oxygen Percentiles for Zones (44 ft Project with Mitigation Plan 6b and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%
FR1	4.30	4.36	4.43	4.54	4.65	4.75	4.84	4.91	4.95
FR2	4.23	4.27	4.30	4.56	4.68	4.84	4.95	5.02	5.11
FR3	4.08	4.15	4.21	4.46	4.59	4.77	5.11	5.23	5.44
FR4	4.16	4.22	4.34	4.55	4.71	5.04	5.47	5.84	6.06
FR5	4.29	4.39	4.51	4.70	4.91	5.33	5.84	6.11	6.24
FR6	4.40	4.49	4.65	4.84	5.13	5.62	6.18	6.33	6.46
FR7	4.78	4.93	5.08	5.39	6.08	6.54	6.80	6.92	7.06
FR8	5.22	5.44	5.56	5.91	6.50	6.83	7.06	7.23	7.39
FR9	5.89	6.15	6.33	6.61	6.90	7.17	7.46	7.65	7.80
FR10	6.44	6.52	6.66	6.96	7.19	7.47	7.85	7.98	8.16
FR11	6.41	6.49	6.61	6.97	7.17	7.43	7.88	8.02	8.15
MR1	5.05	5.17	5.25	5.49	5.79	6.08	6.26	6.36	6.44
MR2	5.16	5.29	5.40	5.61	5.88	6.10	6.26	6.35	6.46
MR3	5.26	5.38	5.49	5.72	5.96	6.22	6.36	6.44	6.50
MR4	5.62	5.76	5.87	6.04	6.30	6.58	6.83	6.99	7.15
MR5	4.38	4.68	4.99	5.40	6.60	7.27	7.71	7.93	8.05
MR6	6.10	6.29	6.39	6.59	6.78	6.94	7.16	7.28	7.39
LBR1	5.46	5.72	5.79	6.00	6.19	6.39	6.70	6.82	6.90
LBR2	5.19	5.29	5.37	5.55	5.75	5.94	6.14	6.29	6.40
LBR3	3.95	4.09	4.21	4.45	4.78	5.10	5.32	5.41	5.57
BR1	4.32	4.40	4.47	4.59	4.77	5.01	5.11	5.18	5.27
BR2	4.25	4.35	4.43	4.57	4.73	4.89	5.03	5.11	5.20
BR3	3.53	3.62	3.69	3.84	4.00	4.19	4.32	4.41	4.53
SCh1	4.01	4.09	4.14	4.24	4.32	4.43	4.53	4.58	4.66
SCh2	4.37	4.44	4.51	4.67	4.81	4.92	5.02	5.08	5.18
SR	7.26	7.38	7.49	7.85	8.11	8.30	9.07	9.17	9.29
StbR	5.44	5.62	5.74	6.04	6.34	6.58	6.76	6.86	7.02

Table 7.5.3-5: Dissolved Oxygen Percentiles for Zones (45 ft Project with Mitigation Plan 6a and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%
FR1	4.30	4.36	4.42	4.52	4.64	4.73	4.82	4.86	4.92
FR2	4.23	4.27	4.30	4.52	4.64	4.81	4.91	4.97	5.04
FR3	4.07	4.14	4.20	4.42	4.54	4.73	5.03	5.13	5.30
FR4	4.15	4.20	4.31	4.51	4.67	4.97	5.36	5.69	5.94
FR5	4.26	4.36	4.48	4.66	4.84	5.25	5.68	5.99	6.13
FR6	4.37	4.46	4.61	4.78	5.03	5.50	6.05	6.25	6.39
FR7	4.71	4.86	5.01	5.29	5.97	6.47	6.73	6.86	6.97
FR8	5.12	5.34	5.44	5.79	6.41	6.80	7.01	7.17	7.37
FR9	5.77	6.05	6.20	6.50	6.85	7.14	7.42	7.64	7.80
FR10	6.44	6.53	6.65	6.95	7.18	7.46	7.84	7.96	8.14
FR11	6.41	6.49	6.61	6.96	7.17	7.42	7.88	8.02	8.15
MR1	4.98	5.12	5.22	5.44	5.78	6.04	6.22	6.30	6.39
MR2	5.18	5.30	5.42	5.63	5.86	6.10	6.24	6.29	6.42
MR3	5.39	5.49	5.57	5.82	6.07	6.30	6.43	6.50	6.63
MR4	5.74	5.92	6.01	6.20	6.47	6.76	7.02	7.19	7.33
MR5	5.04	5.40	5.64	5.94	6.74	7.28	7.74	7.92	8.05
MR6	4.14	4.38	4.52	4.74	5.13	5.48	5.65	5.79	5.97
LBR1	5.70	5.93	6.04	6.24	6.46	6.71	7.00	7.15	7.25
LBR2	5.49	5.57	5.67	5.88	6.09	6.29	6.51	6.62	6.75
LBR3	4.57	4.74	4.84	5.03	5.28	5.55	5.74	5.85	6.03
BR1	3.85	4.09	4.21	4.46	4.68	4.92	5.06	5.11	5.22
BR2	3.75	3.86	3.91	4.04	4.24	4.53	4.76	4.84	4.96
BR3	3.74	3.82	3.85	3.93	4.05	4.20	4.35	4.45	4.55
SCh1	3.97	4.06	4.12	4.22	4.30	4.41	4.49	4.55	4.63
SCh2	4.36	4.42	4.49	4.64	4.78	4.89	4.98	5.03	5.13
SR	7.26	7.38	7.49	7.85	8.11	8.30	9.08	9.17	9.29
StbR	5.47	5.63	5.72	6.00	6.30	6.54	6.70	6.79	6.98

Table 7.5.3-6: Dissolved Oxygen Percentiles for Zones (46 ft Project with Mitigation Plan 6a and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%
FR1	4.31	4.36	4.42	4.52	4.65	4.74	4.82	4.87	4.92
FR2	4.23	4.26	4.29	4.53	4.64	4.81	4.91	4.96	5.04
FR3	4.09	4.14	4.19	4.42	4.55	4.72	5.01	5.09	5.21
FR4	4.15	4.20	4.32	4.51	4.66	4.94	5.27	5.57	5.84
FR5	4.26	4.35	4.48	4.65	4.83	5.19	5.56	5.89	6.08
FR6	4.36	4.45	4.60	4.76	5.01	5.41	5.95	6.20	6.37
FR7	4.70	4.85	4.98	5.25	5.90	6.44	6.73	6.86	6.98
FR8	5.12	5.32	5.43	5.74	6.39	6.81	7.06	7.18	7.39
FR9	5.75	6.02	6.17	6.51	6.88	7.17	7.42	7.69	7.85
FR10	6.53	6.63	6.75	7.04	7.28	7.57	7.93	8.12	8.27
FR11	6.57	6.67	6.79	7.12	7.35	7.58	8.08	8.23	8.36
MR1	4.98	5.11	5.23	5.44	5.81	6.07	6.25	6.32	6.41
MR2	5.18	5.31	5.44	5.66	5.89	6.15	6.27	6.33	6.43
MR3	5.45	5.58	5.65	5.90	6.15	6.35	6.50	6.58	6.73
MR4	5.83	6.03	6.12	6.31	6.59	6.87	7.15	7.32	7.48
MR5	5.15	5.53	5.76	6.08	6.91	7.43	7.88	8.08	8.25
MR6	4.15	4.40	4.53	4.76	5.14	5.50	5.68	5.82	6.01
LBR1	5.85	6.05	6.16	6.37	6.59	6.85	7.14	7.28	7.38
LBR2	5.61	5.69	5.79	5.99	6.20	6.41	6.65	6.76	6.87
LBR3	4.67	4.84	4.95	5.13	5.39	5.65	5.86	5.97	6.13
BR1	3.91	4.14	4.25	4.50	4.70	4.93	5.08	5.12	5.22
BR2	3.83	3.93	3.99	4.10	4.30	4.56	4.79	4.88	5.00
BR3	3.82	3.89	3.94	4.00	4.13	4.28	4.43	4.55	4.63
SCh1	4.02	4.09	4.15	4.24	4.33	4.42	4.50	4.55	4.62
SCh2	4.36	4.42	4.49	4.65	4.79	4.90	4.99	5.04	5.14
SR	7.51	7.63	7.74	8.09	8.36	8.57	9.36	9.47	9.60
StbR	5.51	5.69	5.77	6.04	6.34	6.56	6.72	6.79	6.97

Table 7.5.3-7: Dissolved Oxygen Percentiles for Zones (47 ft Project with Mitigation Plan 6a and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%
FR1	4.34	4.39	4.45	4.56	4.64	4.74	4.83	4.88	4.94
FR2	4.27	4.30	4.33	4.55	4.65	4.82	4.92	4.97	5.04
FR3	4.13	4.19	4.23	4.44	4.55	4.74	5.01	5.08	5.20
FR4	4.21	4.28	4.39	4.57	4.76	5.04	5.34	5.67	5.90
FR5	4.37	4.47	4.59	4.73	4.95	5.28	5.61	5.87	6.05
FR6	4.50	4.59	4.73	4.86	5.09	5.44	5.90	6.15	6.33
FR7	4.80	4.93	5.05	5.31	5.86	6.38	6.70	6.85	6.95
FR8	5.19	5.37	5.45	5.76	6.34	6.79	7.04	7.16	7.34
FR9	5.73	5.99	6.13	6.49	6.85	7.15	7.41	7.64	7.82
FR10	6.54	6.63	6.74	7.05	7.28	7.57	7.94	8.11	8.28
FR11	6.57	6.67	6.79	7.12	7.35	7.58	8.08	8.23	8.36
MR1	5.05	5.20	5.28	5.47	5.81	6.04	6.22	6.29	6.38
MR2	5.22	5.33	5.45	5.67	5.88	6.12	6.25	6.34	6.40
MR3	5.44	5.58	5.65	5.89	6.14	6.35	6.50	6.57	6.74
MR4	5.85	6.03	6.12	6.31	6.59	6.87	7.16	7.34	7.47
MR5	5.16	5.53	5.76	6.09	6.90	7.42	7.89	8.08	8.25
MR6	4.14	4.40	4.54	4.75	5.14	5.50	5.68	5.82	6.01
LBR1	5.85	6.05	6.17	6.37	6.60	6.85	7.14	7.28	7.38
LBR2	5.61	5.69	5.79	5.99	6.19	6.40	6.64	6.74	6.91
LBR3	4.66	4.83	4.94	5.13	5.39	5.64	5.85	5.97	6.14
BR1	3.97	4.21	4.34	4.54	4.73	4.96	5.08	5.14	5.25
BR2	3.89	4.00	4.03	4.16	4.36	4.64	4.85	4.94	5.08
BR3	3.85	3.93	3.96	4.04	4.14	4.30	4.43	4.54	4.63
SCh1	4.02	4.09	4.14	4.25	4.33	4.41	4.50	4.56	4.61
SCh2	4.39	4.45	4.51	4.66	4.79	4.89	5.00	5.04	5.13
SR	7.51	7.63	7.75	8.09	8.36	8.57	9.36	9.47	9.60
StbR	5.49	5.68	5.76	6.03	6.31	6.54	6.72	6.78	6.95

Table 7.5.3-8: Dissolved Oxygen Percentiles for Zones (48 ft Project with Mitigation Plan 6a and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%
FR1	4.32	4.38	4.43	4.58	4.67	4.75	4.83	4.90	4.96
FR2	4.26	4.29	4.32	4.57	4.67	4.83	4.94	5.00	5.07
FR3	4.15	4.20	4.23	4.48	4.61	4.76	5.03	5.12	5.26
FR4	4.24	4.36	4.49	4.68	4.94	5.28	5.62	5.85	6.09
FR5	4.48	4.60	4.76	4.90	5.17	5.50	5.82	5.94	6.25
FR6	4.74	4.83	4.94	5.07	5.29	5.56	5.97	6.14	6.30
FR7	4.96	5.08	5.21	5.45	5.89	6.34	6.65	6.78	6.89
FR8	5.30	5.45	5.53	5.80	6.28	6.73	6.96	7.10	7.27
FR9	5.73	5.95	6.11	6.41	6.77	7.07	7.32	7.55	7.72
FR10	6.44	6.54	6.65	6.95	7.18	7.45	7.82	7.97	8.14
FR11	6.42	6.50	6.61	6.96	7.17	7.42	7.88	8.02	8.15
MR1	5.18	5.28	5.37	5.54	5.81	6.01	6.18	6.25	6.35
MR2	5.23	5.32	5.43	5.67	5.87	6.07	6.19	6.30	6.37
MR3	5.38	5.49	5.57	5.82	6.06	6.28	6.41	6.48	6.65
MR4	5.74	5.93	6.03	6.22	6.48	6.76	7.04	7.21	7.34
MR5	5.03	5.40	5.64	5.97	6.76	7.28	7.74	7.93	8.06
MR6	4.12	4.38	4.52	4.74	5.13	5.48	5.65	5.79	5.96
LBR1	5.73	5.92	6.04	6.25	6.48	6.72	7.01	7.16	7.26
LBR2	5.50	5.57	5.67	5.88	6.08	6.30	6.52	6.62	6.70
LBR3	4.54	4.71	4.82	5.01	5.27	5.54	5.74	5.82	6.02
BR1	4.03	4.26	4.39	4.58	4.78	5.01	5.14	5.18	5.27
BR2	3.92	4.01	4.05	4.18	4.40	4.67	4.91	4.98	5.11
BR3	3.78	3.89	3.92	3.98	4.10	4.24	4.35	4.45	4.54
SCh1	4.02	4.08	4.14	4.27	4.37	4.46	4.53	4.57	4.65
SCh2	4.38	4.45	4.50	4.68	4.81	4.92	5.01	5.05	5.14
SR	7.26	7.38	7.49	7.85	8.11	8.30	9.07	9.17	9.29
StbR	5.39	5.62	5.73	5.99	6.27	6.48	6.64	6.72	6.86

Table 7.5.3-9: Dissolved Oxygen Percentiles for Critical Cells (Existing Conditions)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%	Critical Cell*
FR1	3.86	3.93	3.99	4.13	4.30	4.43	4.56	4.62	4.69	I= 13 J= 40 K= 6
FR2	3.56	3.66	3.74	3.91	4.10	4.28	4.46	4.52	4.91	I= 15 J= 52 K= 6
FR3	3.36	3.47	3.50	3.71	3.90	4.14	4.59	4.90	5.48	I= 15 J= 57 K= 6
FR4	3.34	3.43	3.49	3.74	3.91	4.23	4.78	5.14	5.36	I= 15 J= 60 K= 6
FR5	3.45	3.55	3.68	3.88	4.10	4.70	5.26	5.51	5.68	I= 15 J= 67 K= 6
FR6	3.55	3.66	3.78	3.95	4.19	4.80	5.55	5.78	5.95	I= 13 J= 73 K= 6
FR7	3.98	4.06	4.14	4.40	4.86	5.93	6.22	6.34	6.48	I= 14 J= 81 K= 6
FR8	4.48	4.62	4.90	5.41	6.09	6.43	6.71	6.83	7.11	I= 14 J= 94 K= 6
FR9	4.72	4.87	5.22	5.62	6.24	6.57	6.83	6.99	7.18	I= 14 J= 98 K= 6
FR10	4.31	4.78	4.95	5.32	5.91	6.43	6.68	7.01	7.21	I= 15 J=120 K= 6
FR11	4.17	4.70	4.93	5.24	5.66	6.14	6.49	6.64	7.13	I= 14 J=123 K= 6
MR1	4.22	4.34	4.47	4.72	5.05	5.51	5.81	5.93	6.19	I= 17 J= 82 K= 6
MR2	4.01	4.13	4.30	4.60	5.02	5.47	5.73	5.84	5.98	I= 21 J= 86 K= 6
MR3	3.68	3.88	3.94	4.16	4.47	4.95	5.66	5.93	6.28	I= 26 J= 98 K= 6
MR4	3.87	4.02	4.11	4.37	4.59	4.87	5.04	5.15	5.39	I= 26 J=105 K= 6
MR5	1.49	2.04	2.41	3.05	4.97	6.23	6.56	6.89	7.11	I= 22 J=123 K= 6
MR6	2.11	2.49	3.01	3.51	5.61	6.35	6.80	7.06	7.32	I= 20 J=119 K= 6
LBR1	3.57	4.35	4.74	5.12	5.42	5.64	5.97	6.15	6.47	I= 27 J=123 K= 6
LBR2	3.68	3.86	3.97	4.15	4.38	4.59	4.77	4.86	5.24	I= 39 J=113 K= 6
LBR3	2.88	3.28	3.46	3.67	3.92	4.31	4.70	4.95	5.18	I= 30 J= 92 K= 6
BR1	3.15	3.28	3.44	3.59	3.82	4.05	4.26	4.34	4.45	I= 31 J= 63 K= 6
BR2	2.43	2.72	2.86	3.11	3.30	3.54	3.67	3.74	3.82	I= 33 J= 70 K= 6
BR3	2.87	3.12	3.32	3.48	3.65	3.80	3.93	4.00	4.13	I= 32 J= 72 K= 6
SCh1	2.25	2.41	2.53	2.68	2.88	3.30	3.69	3.80	4.08	I= 10 J= 24 K= 6
SCh2	3.62	3.78	3.88	4.02	4.19	4.35	4.48	4.56	4.70	I= 7 J= 46 K= 6
SR	4.69	4.74	4.97	5.31	5.62	5.97	6.11	6.16	6.23	I= 14 J=140 K= 6
StbR	3.83	4.19	4.53	5.01	5.66	6.16	6.47	6.62	6.81	I= 23 J=101 K= 6

** The critical cell is the cell with lowest D.O. concentration within a Zone. Surface Layer K=1; Bottom Layer K=6.*

Table 7.5.3-10: Dissolved Oxygen Percentiles for Critical Cells (44 ft Project with Mitigation Plan 6b and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%	Critical Cell*
FR1	3.96	4.05	4.11	4.30	4.43	4.60	4.70	4.73	4.77	I= 13 J= 38 K= 6
FR2	3.82	3.92	4.00	4.25	4.41	4.59	4.86	4.95	5.03	I= 15 J= 52 K= 6
FR3	3.70	3.77	3.87	4.13	4.31	4.60	5.16	5.64	6.02	I= 15 J= 59 K= 6
FR4	3.72	3.80	3.90	4.18	4.33	4.67	5.22	5.61	5.94	I= 15 J= 60 K= 6
FR5	4.01	4.12	4.30	4.46	4.66	5.04	5.75	5.98	6.18	I= 14 J= 70 K= 6
FR6	4.07	4.15	4.26	4.47	4.63	5.01	5.74	6.03	6.15	I= 14 J= 73 K= 6
FR7	4.43	4.51	4.58	4.81	5.10	6.08	6.49	6.65	6.78	I= 14 J= 81 K= 6
FR8	4.83	4.93	5.10	5.50	6.35	6.69	6.92	7.05	7.27	I= 14 J= 94 K= 6
FR9	5.03	5.14	5.37	5.74	6.52	6.93	7.19	7.31	7.51	I= 14 J= 99 K= 6
FR10	6.18	6.39	6.53	6.86	7.08	7.33	7.78	7.87	7.95	I= 13 J=120 K= 4
FR11	2.71	2.71	2.72	2.74	2.78	2.89	3.03	3.11	3.18	I= 14 J=122 K= 5
MR1	4.78	4.90	4.99	5.17	5.43	5.77	6.14	6.31	6.57	I= 17 J= 82 K= 6
MR2	4.70	4.84	4.94	5.18	5.47	5.80	6.07	6.18	6.29	I= 21 J= 86 K= 6
MR3	4.64	4.82	4.94	5.13	5.44	5.71	5.99	6.12	6.28	I= 26 J= 94 K= 6
MR4	4.80	5.49	5.66	5.97	6.26	6.67	7.03	7.28	7.80	I= 26 J=122 K= 6
MR5	2.45	2.66	2.93	3.64	6.55	7.36	7.81	7.97	8.08	I= 22 J=123 K= 6
MR6	5.82	6.17	6.35	6.62	6.88	7.10	7.35	7.41	7.67	I= 20 J=119 K= 6
LBR1	4.09	5.07	5.76	6.10	6.39	6.76	7.10	7.30	7.81	I= 27 J=123 K= 6
LBR2	4.09	4.87	4.95	5.18	5.38	5.61	5.82	6.03	6.47	I= 39 J=113 K= 6
LBR3	2.64	2.75	2.84	3.07	3.35	3.73	3.97	4.11	4.21	I= 30 J= 86 K= 6
BR1	3.72	3.98	4.16	4.30	4.49	4.68	4.82	4.89	4.98	I= 32 J= 62 K= 6
BR2	2.94	3.20	3.45	3.79	4.11	4.37	4.50	4.58	4.73	I= 33 J= 65 K= 6
BR3	2.59	2.72	2.80	3.01	3.90	5.11	5.54	5.71	5.86	I= 30 J= 76 K= 6
SCh1	2.38	2.68	2.81	2.97	3.30	4.14	4.41	4.50	4.72	I= 10 J= 23 K= 6
SCh2	3.88	4.01	4.11	4.23	4.43	4.58	4.69	4.79	4.93	I= 7 J= 46 K= 6
SR	6.14	6.25	6.46	6.83	7.06	7.34	7.81	7.91	8.01	I= 13 J=128 K= 4
StbR	4.06	4.80	5.02	5.57	6.14	6.56	6.81	7.03	7.18	I= 21 J=101 K= 6

** The critical cell is the cell with lowest D.O. concentration within a Zone. Surface Layer K=1; Bottom Layer K=6.*

Table 7.5.3-11: Dissolved Oxygen Percentiles for Critical Cells (45 ft Project with Mitigation Plan 6a and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%	Critical Cell*
FR1	3.97	4.03	4.08	4.26	4.40	4.57	4.68	4.72	4.78	I= 13 J= 39 K= 6
FR2	3.81	3.89	3.97	4.21	4.36	4.54	4.77	4.88	4.93	I= 15 J= 52 K= 6
FR3	3.70	3.77	3.86	4.11	4.26	4.54	5.06	5.62	5.95	I= 15 J= 59 K= 6
FR4	3.73	3.78	3.89	4.14	4.29	4.61	5.14	5.56	5.99	I= 15 J= 60 K= 6
FR5	4.03	4.26	4.42	4.60	4.94	5.39	5.73	6.01	6.26	I= 13 J= 67 K= 5
FR6	4.07	4.11	4.23	4.43	4.57	4.91	5.59	5.90	6.05	I= 14 J= 73 K= 6
FR7	4.79	4.89	5.04	5.32	5.88	6.40	6.69	6.85	7.50	I= 15 J= 86 K= 6
FR8	4.79	4.91	5.15	5.45	6.39	6.82	7.07	7.18	7.39	I= 14 J= 94 K= 5
FR9	4.93	5.05	5.23	5.58	6.32	6.82	7.07	7.17	7.47	I= 14 J= 98 K= 6
FR10	6.18	6.36	6.52	6.84	7.07	7.34	7.78	7.85	7.97	I= 13 J=119 K= 4
FR11	2.71	2.71	2.72	2.74	2.78	2.89	3.03	3.11	3.18	I= 14 J=122 K= 5
MR1	4.73	4.85	4.93	5.12	5.42	5.71	6.03	6.19	6.50	I= 17 J= 82 K= 6
MR2	4.77	4.91	5.03	5.26	5.55	5.79	6.01	6.11	6.22	I= 21 J= 86 K= 6
MR3	4.76	4.89	5.00	5.23	5.54	5.77	5.97	6.07	6.22	I= 26 J= 94 K= 6
MR4	4.90	5.04	5.15	5.41	5.70	6.11	6.38	6.56	6.82	I= 26 J=105 K= 6
MR5	3.17	3.78	4.08	4.77	6.61	7.36	7.81	7.93	8.11	I= 23 J=123 K= 6
MR6	2.64	2.65	2.66	2.70	2.81	3.02	3.26	3.39	3.49	I= 20 J=118 K= 5
LBR1	4.85	5.61	6.02	6.32	6.59	6.97	7.25	7.43	7.78	I= 27 J=123 K= 6
LBR2	5.16	5.43	5.56	5.73	5.95	6.19	6.41	6.56	6.76	I= 39 J=122 K= 6
LBR3	3.27	3.37	3.52	3.70	4.04	4.33	4.52	4.62	4.85	I= 30 J= 86 K= 6
BR1	3.52	3.64	3.72	3.90	4.28	4.54	4.72	4.77	4.88	I= 32 J= 62 K= 6
BR2	2.59	2.84	2.98	3.31	3.70	4.05	4.26	4.34	4.49	I= 33 J= 65 K= 6
BR3	2.99	3.08	3.14	3.30	3.57	4.02	4.22	4.31	4.41	I= 30 J= 76 K= 6
SCh1	2.33	2.65	2.75	2.93	3.30	4.05	4.35	4.45	4.62	I= 10 J= 23 K= 6
SCh2	3.87	3.97	4.09	4.22	4.39	4.55	4.66	4.74	4.85	I= 7 J= 46 K= 6
SR	6.15	6.25	6.46	6.84	7.06	7.34	7.81	7.91	8.01	I= 13 J=128 K= 4
StbR	4.15	4.70	4.99	5.49	6.07	6.53	6.78	6.94	7.14	I= 21 J=101 K= 6

** The critical cell is the cell with lowest D.O. concentration within a Zone. Surface Layer K=1; Bottom Layer K=6.*

Table 7.5.3-12: Dissolved Oxygen Percentiles for Critical Cells (46 ft Project with Mitigation Plan 6a and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%	Critical Cell*
FR1	3.96	4.02	4.07	4.25	4.40	4.57	4.67	4.72	4.77	I= 13 J= 39 K= 6
FR2	3.81	3.90	3.97	4.21	4.37	4.54	4.77	4.86	4.91	I= 15 J= 52 K= 6
FR3	3.70	3.77	3.84	4.10	4.25	4.51	5.00	5.53	5.97	I= 15 J= 59 K= 6
FR4	3.71	3.76	3.86	4.12	4.28	4.55	5.03	5.49	6.05	I= 15 J= 60 K= 6
FR5	3.99	4.10	4.21	4.39	4.56	4.87	5.40	5.64	5.93	I= 14 J= 72 K= 6
FR6	4.05	4.11	4.25	4.41	4.55	4.86	5.43	5.77	5.96	I= 14 J= 73 K= 6
FR7	4.81	4.90	5.06	5.30	5.87	6.39	6.72	6.95	7.44	I= 15 J= 86 K= 6
FR8	4.74	4.88	5.09	5.40	6.37	6.84	7.08	7.20	7.35	I= 14 J= 94 K= 5
FR9	4.89	5.01	5.22	5.53	6.35	6.88	7.12	7.31	7.70	I= 14 J= 99 K= 6
FR10	6.20	6.44	6.57	6.90	7.12	7.39	7.83	7.93	8.00	I= 13 J=120 K= 4
FR11	2.71	2.71	2.72	2.74	2.78	2.89	3.03	3.11	3.18	I= 14 J=122 K= 5
MR1	4.72	4.85	4.93	5.12	5.45	5.72	6.00	6.14	6.52	I= 17 J= 82 K= 6
MR2	4.85	5.00	5.12	5.34	5.61	5.80	6.03	6.14	6.23	I= 21 J= 86 K= 6
MR3	4.86	4.98	5.07	5.32	5.60	5.83	5.98	6.11	6.22	I= 26 J= 94 K= 6
MR4	4.99	5.12	5.22	5.51	5.82	6.25	6.53	6.75	6.97	I= 26 J=105 K= 6
MR5	3.33	3.94	4.25	4.91	6.75	7.58	8.01	8.14	8.38	I= 23 J=123 K= 6
MR6	2.64	2.65	2.66	2.70	2.81	3.02	3.26	3.39	3.49	I= 20 J=118 K= 5
LBR1	5.04	5.71	6.14	6.46	6.74	7.14	7.42	7.61	7.94	I= 27 J=123 K= 6
LBR2	5.51	5.59	5.70	5.86	6.09	6.34	6.57	6.71	7.01	I= 39 J=122 K= 6
LBR3	3.34	3.47	3.62	3.80	4.14	4.47	4.62	4.78	4.97	I= 30 J= 86 K= 6
BR1	3.59	3.68	3.78	3.95	4.32	4.56	4.72	4.78	4.92	I= 32 J= 62 K= 6
BR2	2.80	2.91	3.05	3.36	3.76	4.08	4.29	4.36	4.51	I= 33 J= 65 K= 6
BR3	3.05	3.20	3.26	3.39	3.84	4.14	4.36	4.44	4.56	I= 30 J= 75 K= 6
SCh1	2.55	2.67	2.80	2.99	3.16	3.41	3.74	3.90	4.04	I= 10 J= 24 K= 6
SCh2	3.84	3.93	4.06	4.24	4.43	4.58	4.70	4.78	4.90	I= 7 J= 45 K= 6
SR	6.15	6.26	6.47	6.84	7.07	7.34	7.82	7.92	8.02	I= 13 J=128 K= 4
StbR	4.18	4.74	5.04	5.55	6.12	6.57	6.82	6.93	7.10	I= 21 J=101 K= 6

** The critical cell is the cell with lowest D.O. concentration within a Zone. Surface Layer K=1; Bottom Layer K=6.*

Table 7.5.3-13: Dissolved Oxygen Percentiles for Critical Cells (47 ft Project with Mitigation Plan 6a and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%	Critical Cell*
FR1	3.99	4.05	4.10	4.27	4.39	4.56	4.68	4.72	4.81	I= 13 J= 40 K= 6
FR2	3.86	3.93	4.00	4.19	4.38	4.55	4.73	4.84	4.90	I= 15 J= 52 K= 6
FR3	3.74	3.82	3.89	4.14	4.29	4.59	5.01	5.64	5.97	I= 15 J= 59 K= 6
FR4	3.77	3.84	3.93	4.17	4.34	4.67	5.10	5.55	6.00	I= 15 J= 60 K= 6
FR5	4.14	4.39	4.52	4.68	5.03	5.39	5.66	5.96	6.18	I= 13 J= 67 K= 5
FR6	4.23	4.38	4.47	4.64	4.81	5.16	5.57	5.87	6.04	I= 14 J= 73 K= 5
FR7	4.88	4.97	5.12	5.37	5.91	6.41	6.78	7.09	7.87	I= 15 J= 86 K= 6
FR8	4.83	4.91	5.04	5.32	6.09	6.56	6.88	7.03	7.18	I= 14 J= 94 K= 6
FR9	4.97	5.07	5.24	5.53	6.28	6.86	7.12	7.24	7.35	I= 14 J= 99 K= 6
FR10	6.21	6.44	6.57	6.91	7.12	7.39	7.83	7.94	8.00	I= 13 J=120 K= 4
FR11	2.71	2.71	2.72	2.74	2.78	2.89	3.03	3.11	3.18	I= 14 J=122 K= 5
MR1	4.82	4.93	5.03	5.19	5.48	5.74	5.94	6.09	6.47	I= 17 J= 82 K= 6
MR2	4.86	4.98	5.10	5.26	5.47	5.72	5.95	6.06	6.27	I= 21 J= 83 K= 6
MR3	4.84	4.98	5.05	5.28	5.59	5.82	5.98	6.07	6.22	I= 26 J= 94 K= 6
MR4	4.99	5.11	5.23	5.53	5.82	6.26	6.54	6.72	6.99	I= 26 J=105 K= 6
MR5	3.34	3.94	4.27	4.92	6.75	7.58	8.02	8.14	8.38	I= 23 J=123 K= 6
MR6	2.64	2.65	2.66	2.70	2.81	3.02	3.26	3.39	3.49	I= 20 J=118 K= 5
LBR1	5.07	5.70	6.14	6.47	6.74	7.14	7.43	7.62	7.94	I= 27 J=123 K= 6
LBR2	5.53	5.60	5.73	5.89	6.11	6.36	6.61	6.80	7.12	I= 39 J=122 K= 6
LBR3	3.34	3.46	3.62	3.79	4.12	4.45	4.63	4.73	4.94	I= 30 J= 86 K= 6
BR1	3.69	3.78	3.86	4.04	4.38	4.61	4.77	4.82	4.94	I= 32 J= 62 K= 6
BR2	2.81	3.01	3.19	3.43	3.85	4.15	4.36	4.42	4.57	I= 33 J= 65 K= 6
BR3	3.12	3.20	3.26	3.43	3.72	4.15	4.32	4.47	4.58	I= 30 J= 76 K= 6
SCh1	2.39	2.66	2.80	2.97	3.21	3.99	4.33	4.44	4.60	I= 10 J= 23 K= 6
SCh2	3.84	3.93	4.07	4.26	4.43	4.57	4.69	4.74	4.87	I= 7 J= 45 K= 6
SR	6.15	6.26	6.47	6.84	7.07	7.34	7.82	7.92	8.02	I= 13 J=128 K= 4
StbR	4.15	4.75	5.01	5.54	6.11	6.54	6.80	6.92	7.11	I= 21 J=101 K= 6

** The critical cell is the cell with lowest D.O. concentration within a Zone. Surface Layer K=1; Bottom Layer K=6.*

Table 7.5.3-14: Dissolved Oxygen Percentiles for Critical Cells (48 ft Project with Mitigation Plan 6a and DO Injection)

Zone	1%	5%	10%	25%	50%	75%	90%	95%	99%	Critical Cell*
FR1	4.01	4.06	4.11	4.34	4.44	4.56	4.64	4.68	4.72	I= 13 J= 37 K= 6
FR2	3.84	3.93	3.99	4.20	4.40	4.56	4.77	4.84	4.90	I= 15 J= 52 K= 6
FR3	3.76	3.82	3.93	4.16	4.35	4.75	5.11	5.65	6.03	I= 15 J= 59 K= 6
FR4	3.80	3.86	4.00	4.22	4.42	4.85	5.26	5.65	6.05	I= 15 J= 60 K= 6
FR5	4.24	4.48	4.68	4.87	5.20	5.55	5.82	6.02	6.19	I= 13 J= 67 K= 5
FR6	4.31	4.43	4.64	4.78	5.03	5.34	5.69	5.91	6.09	I= 14 J= 73 K= 5
FR7	5.06	5.13	5.24	5.48	5.94	6.38	6.69	6.94	7.84	I= 15 J= 86 K= 6
FR8	4.97	5.06	5.18	5.43	6.03	6.49	6.82	6.93	7.10	I= 14 J= 94 K= 6
FR9	5.08	5.16	5.35	5.61	6.16	6.76	7.03	7.14	7.28	I= 14 J= 99 K= 6
FR10	6.17	6.37	6.52	6.84	7.07	7.33	7.78	7.85	7.97	I= 13 J=119 K= 4
FR11	2.71	2.71	2.72	2.74	2.78	2.89	3.03	3.11	3.18	I= 14 J=122 K= 5
MR1	4.89	5.06	5.15	5.32	5.52	5.76	5.94	6.02	6.18	I= 21 J= 82 K= 6
MR2	4.79	5.03	5.13	5.30	5.48	5.71	5.93	6.03	6.18	I= 21 J= 83 K= 6
MR3	4.74	4.92	4.98	5.23	5.52	5.76	5.91	6.01	6.20	I= 26 J= 94 K= 6
MR4	4.90	5.02	5.14	5.43	5.72	6.12	6.39	6.58	6.84	I= 26 J=105 K= 6
MR5	3.18	3.76	4.11	4.78	6.62	7.36	7.80	7.94	8.11	I= 23 J=123 K= 6
MR6	2.64	2.65	2.66	2.70	2.81	3.02	3.26	3.39	3.49	I= 20 J=118 K= 5
LBR1	4.94	5.53	6.00	6.33	6.60	7.00	7.27	7.44	7.80	I= 27 J=123 K= 6
LBR2	5.37	5.44	5.59	5.75	5.97	6.23	6.45	6.57	6.74	I= 39 J=122 K= 6
LBR3	3.24	3.34	3.49	3.68	4.01	4.32	4.53	4.61	4.80	I= 30 J= 86 K= 6
BR1	3.74	3.84	3.91	4.08	4.42	4.64	4.80	4.86	4.98	I= 32 J= 62 K= 6
BR2	2.84	3.05	3.20	3.49	3.89	4.20	4.40	4.46	4.59	I= 33 J= 65 K= 6
BR3	3.05	3.14	3.20	3.38	3.67	4.16	4.34	4.48	4.60	I= 30 J= 76 K= 6
SCh1	2.39	2.66	2.82	3.01	3.28	4.06	4.31	4.45	4.58	I= 10 J= 23 K= 6
SCh2	3.84	3.94	4.07	4.24	4.44	4.59	4.70	4.76	4.89	I= 7 J= 45 K= 6
SR	6.14	6.25	6.46	6.84	7.07	7.34	7.81	7.91	8.01	I= 13 J=128 K= 4
StbR	4.04	4.66	4.97	5.47	6.05	6.45	6.71	6.83	7.02	I= 21 J=101 K= 6

** The critical cell is the cell with lowest D.O. concentration within a Zone. Surface Layer K=1; Bottom Layer K=6.*

7.5.4 Proposed Mitigation Features and Fishery Habitat

The proposed mitigation features were developed based on salinity impacts to freshwater wetlands, partially due to the tidal freshwater marshes being identified by the USFWS as the single most critical natural resource in the harbor. However, the selected mitigation plans were also evaluated to determine impacts to fishery habitat, chloride concentrations, and hurricane surges.

Impacts to fishery habitat were evaluated with the proposed mitigation plans. Results are documented in several reports, all of which are included in the Engineering Investigations Supplemental Materials. A list of these reports is shown below:

- *Evaluation of Fishery Habitat Impacts with Proposed Mitigation Plan, January 2010*
- *Evaluation of Adult Shortnose Sturgeon (Summer) Habitat Impacts with Proposed Mitigation Plan, March 11, 2011*
- *Evaluation of Adult Shortnose Sturgeon (Winter) Habitat Impacts with Proposed Mitigation Plan, March 9, 2011*
- *Evaluation of Juvenile Shortnose Sturgeon (Winter) Habitat Impacts with Proposed Mitigation Plan, March 9, 2011*

Resulting fishery habitat impacts by acreage and percentage lost or gained as reported in these documents are shown in **Tables 7.5.4-1** through **7.5.4-4**.

The mitigation features were designed to reduce salinity intrusion in the estuary along with mitigating for impacts to water quality, specifically D.O. The features do have some benefits to fish habitat; however, the mitigation plans do not fully mitigate for impacts to fish within the estuary. Several other strategies and features have been evaluated with the model to reduce impacts to fish. However, many have not been successful. Details of the previously proposed feature intended to benefit Juvenile Shortnose sturgeon (SNS) habitat which involves construction of a sill on lower Middle River can be found in the report titled *Sensitivity Analysis of Proposed Sill on Middle River* dated September 2009 which is included in the Engineering Investigations Supplemental Materials. Results of the Middle River sill analysis do show some promise in reducing the expected salinity increases with harbor deepening in a deep hole on Middle River where Juvenile SNS have been observed during cool months of the year; however, the benefits to fish habitat from the sill are shown to be marginal.

Table 7.5.4-1: Fishery Impacts – American Shad (Deepening & Mitigation)

	May20%flows		May50%flows		May80%flows	
	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*-0.2%*	***-12***	*-0.2%*	***-12***	*-0.2%*	***-12***
45 ft depth	*-0.2%*	***-11***	*-0.2%*	***-11***	*-0.2%*	***-11***
46 ft depth	*-0.2%*	***-11***	*-0.2%*	***-11***	*-0.2%*	***-11***
47 ft depth	*-0.2%*	***-11***	*-0.2%*	***-11***	*-0.2%*	***-11***
48 ft depth	*-0.2%*	***-11***	*-0.2%*	***-11***	*-0.2%*	***-11***

	January50%flows		August Avg flows	
	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*-0.2%*	***-9***	*-0.3%*	***-16***
45 ft depth	*-0.2%*	***-9***	*-0.3%*	***-15***
46 ft depth	*-0.2%*	***-9***	*-0.2%*	***-11***
47 ft depth	*-0.2%*	***-9***	*-0.2%*	***-11***
48 ft depth	*-0.2%*	***-9***	*-0.2%*	***-11***

Table 7.5.4-2a: Fishery Impacts – Striped Bass Eggs (Deepening & Mitigation)

	April20%flows		April50%flows		April80%flows	
	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*-22.3%*	***-215***	*-9.4%*	***-157***	*7.6%*	***171***
45 ft depth	*-17.6%*	***-169***	*5.2%*	***87***	*8.0%*	***181***
46 ft depth	*-7.8%*	***-75***	*0.0%*	***0***	*6.9%*	***155***
47 ft depth	*-12.1%*	***-117***	*-11.1%*	***-186***	*5.0%*	***113***
48 ft depth	*-4.6%*	***-44***	*-10.8%*	***-181***	*3.4%*	***76***

Table 7.5.4-2b: Fishery Impacts – Striped Bass Larvae (Deepening & Mitigation)

	May20%flows		May50%flows		May80%flows	
	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*174.6%*	***348***	*-5.6%*	***-32***	*-13.7%*	***-136***
45 ft depth	*187.4%*	***374***	*1.7%*	***9***	*7.6%*	***75***
46 ft depth	*191.4%*	***382***	*5.6%*	***32***	*10.1%*	***100***
47 ft depth	*178.7%*	***356***	*-5.0%*	***-28***	*26.4%*	***262***
48 ft depth	*154.6%*	***308***	*-3.5%*	***-20***	*30.0%*	***298***

Table 7.5.4-2c: Fishery Impacts – Striped Bass Spawning (Deepening & Mitigation)

	April20%flows		April50%flows		April80%flows	
	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*-15.2%*	***-97***	*-2.9%*	***-30***	*-2.9%*	***-53***
45 ft depth	*-13.9%*	***-89***	*-9.2%*	***-96***	*-8.5%*	***-156***
46 ft depth	*-18.7%*	***-120***	*-10.0%*	***-104***	*-10.8%*	***-199***
47 ft depth	*-21.1%*	***-135***	*-13.5%*	***-140***	*-12.7%*	***-234***
48 ft depth	*-23.9%*	***-153***	*-16.1%*	***-167***	*-13.2%*	***-242***

Table 7.5.4-3: Fishery Impacts – Shortnose sturgeon (Deepening & Mitigation)

	JUVENILES		ADULTS			
	January		January		August *	
	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*-6.7%*	***-220***	*-3.9%*	***-153***	*19.0%*	***260***
45 ft depth	*-7.0%*	***-231***	*-4.6%*	***-179***	*9.8%*	***134***
46 ft depth	*-7.3%*	***-238***	*-6.2%*	***-240***	*7.3%*	***100***
47 ft depth	*-7.6%*	***-251***	*-6.9%*	***-266***	*6.5%*	***89***
48 ft depth	-11.5%	***-376***	-8.4%	***-326***	2.8%	***39***

**Habitat gains shown during August for Adult SNS are largely due to implementation of the D.O. injection system.*

Table 7.5.4-4: Fishery Impacts – Southern Flounder (Deepening & Mitigation)

	August Avg flows	
	IMPACTS (%)	IMPACTS (acres)
44 ft depth	*74.1%*	***1387***
45 ft depth	*54.2%*	***1014***
46 ft depth	*57.3%*	***1072***
47 ft depth	*57.3%*	***1072***
48 ft depth	52.9%	***989***

7.5.5 Proposed Mitigation Features and Hurricane Surge

Hurricane Surge analysis was also completed with the proposed mitigation feature design. The model has the same limitations noted in the impact analysis but is useful for comparative purposes. The results of the analysis show that there is little impact expected on hurricane surge at Fort Jackson or further upstream at the I-95 Bridge (the two areas identified for result measurements). The largest increase predicted by the model is 0.8 ft at the I-95 Bridge. Details of the hurricane surge analysis can be found in the report titled, *Evaluation of Hurricane Surge Impacts with Proposed Mitigation Plan*, which is included in the Engineering Investigations Supplemental Materials.

7.5.6 Proposed Mitigation Features and 1 Percent Annual Chance Storm (100-Year)

The 1% annual chance storm, i.e. 100-year flood event, in the vicinity of SHEP is a hurricane surge event. According to the most recent FEMA Flood Insurance Study (FIS) which was published September 26, 2008, base flood elevations for the Savannah River in Chatham County range from 12 ft NAVD (16 ft MLLW) and above along the river through the SHEP to the ocean front at Tybee Island. Based on the FEMA designated base flood elevations throughout the Savannah River estuary, all of the flow-altering mitigation features would be significantly submerged during the 1% annual chance storm and would therefore be expected to have a minimal effect on base flood elevations.

Furthermore, the FEMA FIS models used in the coastal zone do not have the detail necessary to evaluate channel deepening and mitigation features. The coastal zone models concentrate on simulating overland flow and wave run-up associated with very large wind-driven storm events. The SHEP hydrodynamic and water quality models do not fall into the same category and the resulting output is not appropriate for comparison to the published FEMA datasets.

7.6 MODEL SENSITIVITY WITH PROPOSED NAVIGATION CHANNEL FEATURES (MEETING AREAS & BEND WIDENERS)

Meeting areas and bend widener features were not originally included in the model grid when it was developed and enhanced for impact evaluation and mitigation feature development. An evaluation of the possible impacts due to incorporation of these features in the SHEP to freshwater wetlands/marshes and water quality within the estuary was performed as sensitivity analysis. The results are reported below.

7.6.1 Meeting Areas

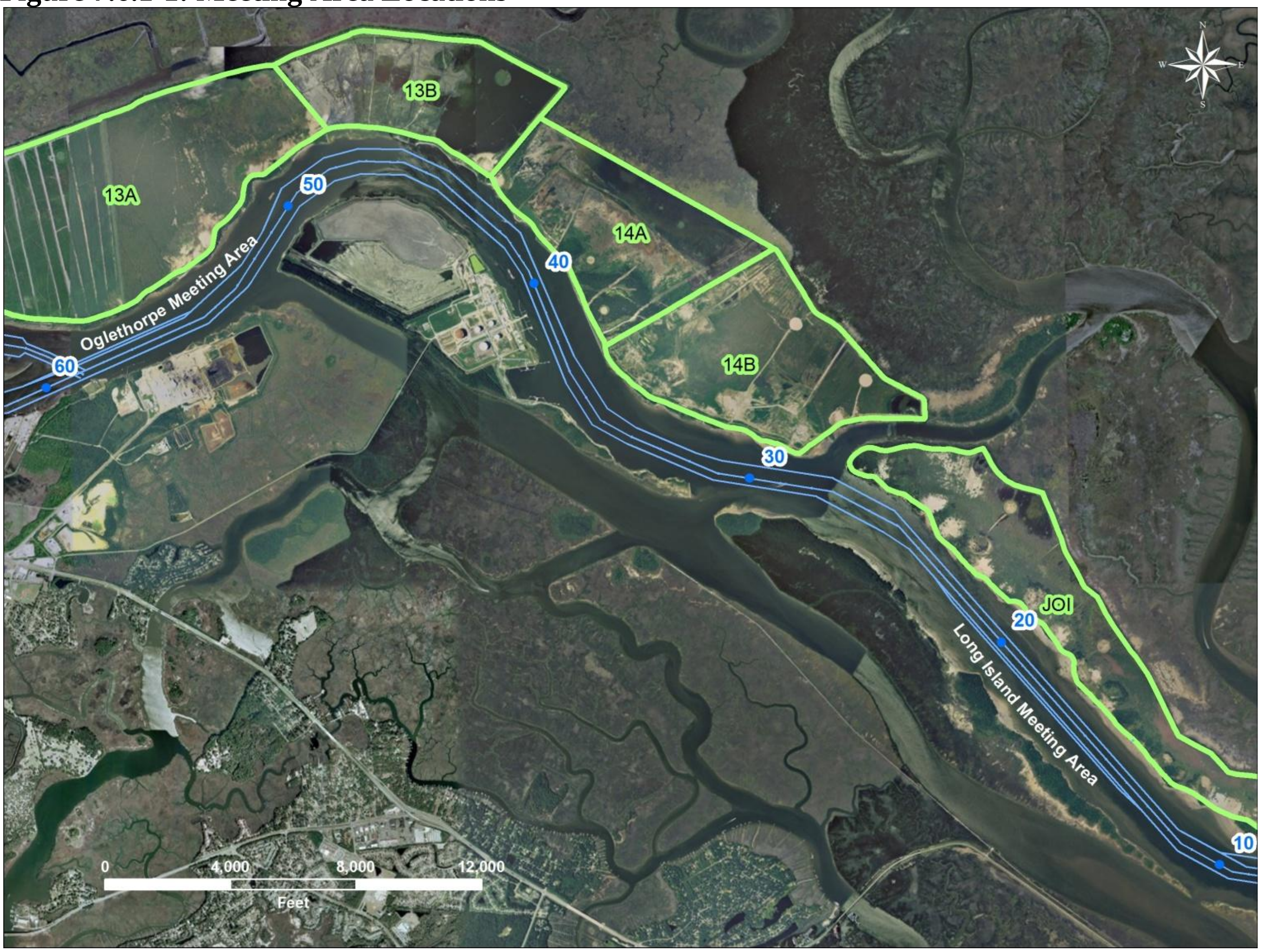
Meeting areas proposed for the final channel design were not originally incorporated into the model grid because they had not yet been developed. However, after the meeting areas were proposed, a hydrodynamic modeling sensitivity analysis was performed with the final mitigation plan and proposed meeting areas to determine the impacts to freshwater marshes/wetlands and to water quality (dissolved oxygen, D.O.) within the Savannah River estuary. Details of this analysis can be found in the report titled *Sensitivity Analysis of Proposed Navigation Meeting Areas*, which is included in the Engineering Investigations Supplemental Materials. **Table 7.6.1-1** and **Figure 7.6.1-1** show the locations of the two meeting areas (Long Island and Oglethorpe) included in the final channel design along with their stationing, length, and width.

Table 7.6.1-1: Meeting Area Dimensions and Locations

Meeting Area	Length (ft)	Width* (ft)	Stationing
Long Island	8,000	100	14+000 to 22+000
Oglethorpe	4,000	100	54+800 to 58+800

** Width is in addition to the width of the navigation channel at the toe of the channel sideslope.*

Figure 7.6.1-1: Meeting Area Locations



7.6.1.1 Hydrodynamic & Water Quality Model Input

The two meeting areas shown in **Figure 7.6.1-1** were incorporated into the EFDC and WASP model grids by adjusting the widths of several grid cells along the navigation channel. The cells that have been widened are representative channel cells; cells that have been reduced in width represent adjacent bank and sideslope transition cells. See **Table 7.6.1.1-1**.

Table 7.6.1.1-1: Meeting Area Model Parameters

Meeting Area	EFDC Model Grid Adjustments*
Long Island	widen cell **14_28** by 100 feet (30.5 m) -- from 153.75m to 184.25m reduce width of cell **13_28** by 100 feet (30.5m) -- from 265.81m to 235.31m widen cell **14_27** by 100 feet (30.5 m) -- from 154.4m to 184.9m reduce width of cell **13_27** by 100 feet (30.5m) -- from 245.15m to 214.65m widen cell **14_26** by 25 feet (7.62m) -- from 175.28m to 182.9m reduce width of cell **13_26** by 25 feet (7.62m) -- from 208.99m to 201.37m
Oglethorpe	widen cell **14_50** by 85 feet (25.9 m) -- from 163.43m to 189.33m reduce width of cell **15_50** by 85 feet (25.9 m) -- from 143.61m to 117.71m widen cell **14_49** by 100 ft (30.5 m) -- from 150.85m to 181.35m reduce width of cell **15_49** by 100 ft (30.5 m) -- from 94.36m to 63.86m widen cell **14_48** by 100 ft (30.5 m) -- from 165.93m to 196.43m reduce width of cell **15_48** by 100 ft (30.5 m) -- from 88.2m to 57.7m

**Adjustments made by changing the DX value in the dxdy.inp file.*

Several model run scenarios were analyzed to determine if there are any additional impacts as a result of the two proposed meeting areas. Two scenarios for freshwater marsh/wetland impacts were evaluated, 1) an average freshwater flow year, considered the "Basic Evaluation", and 2) a low freshwater flow year, considered "Sensitivity Analysis #1". See **Table 7.6.1.1-2**. For more information on how these run scenarios were developed see the report titled *Evaluation of Marsh/Wetland Impacts with Proposed Mitigation Plan*, which is included in the Engineering Investigations Supplemental Materials.

Table 7.6.1.1-2: Model Input Conditions for Freshwater Marsh/Wetland Impacts

Run Scenario	River Flow	Evaluation Period	Parameters Evaluated
Basic Evaluation	Average/Typical	1-March to 1-November	Surface & Bottom Salinity
Sensitivity Analysis #1	Low Flow/Dry	1-March to 1-November	Surface Salinity Only

In addition to the analysis performed for marsh/wetland impacts, an analysis was also done to determine water quality (i.e., D.O.) impacts. One time-period scenario was analyzed for this purpose. The run period is August of 1997, an average flow summer-month period.

All run scenarios incorporate the 48 ft project depth (considered a "worst-case" scenario for the sensitivity analysis) along with the proposed mitigation plan for that depth (Plan 6a and D.O. injection).

7.6.1.2 Findings

Impacts to tidal fluctuations within the estuary due to inclusion of the meeting areas were examined at several grid cell locations on Back and Middle Rivers. Details of tidal variability by water depth were evaluated. Comparisons are made between the 48 ft depth alternative with Mitigation Plan 6a in place with and without the meeting areas modeled. None of the locations along Middle and Back River show an impact to tidal depth or timing as a result of inclusion of the meeting areas within the model geometry.

Impacts to the salinity regime within the estuary due to inclusion of the meeting areas in the model grid geometry were also examined. Again, comparisons are made between the 48 ft depth alternative with Mitigation Plan 6a in place with and without the meeting areas modeled. The changes in surface and bottom salinity predictions were evaluated under conditions experienced within the estuary on an average basis (the Basic Evaluation run period). The maximum change in the surface salinity prediction for the Basic Evaluation is 0.14 ppt. This change occurs just upstream of the tidegate on Back River and causes an increase in the salinity prediction from 5.88 ppt to 6.02 ppt. Surface salinity changes under the Basic Evaluation conditions extend up to New Cut on Back River. However, the increases in prediction values in that area are minor, 0.01 ppt. Additionally, the change in salinity regime causes a decrease in predicted surface salinity values on Front River up to 0.44 ppt.

Comparatively, the maximum change in the bottom salinity prediction during Basic Evaluation conditions is 0.33 ppt and occurs on Front River just upstream of Elba Island. The addition of the meeting areas within the model cause the bottom salinity to increase from 19.25 ppt to 19.58 ppt. Changes in bottom salinity for the Basic Evaluation, although very small (less than 0.1 ppt), are seen as far upstream as New Cut on Back River. However, this change or shift in the salinity regime causes a decrease in predicted bottom salinity values on Front River up to almost 0.5 ppt.

Surface salinity changes under the Sensitivity Analysis #1 (low flow/drought conditions). Increases are seen throughout Front, Middle and Back Rivers. However, the increases are minor (under 0.17 ppt).

Impacts to D.O. within the estuary due to inclusion of the meeting areas in the model grid were also evaluated and results were analyzed for the changes in D.O. percentiles for critical cells and zones. A critical cell is the cell with the lowest D.O. concentrations during the simulation. The critical cell is found within each zone. A zone is an assemblage of cells that is limited by specified horizontal and vertical boundaries. The maximum decrease in the critical cells for D.O. as a result of modification to the model grid occurs on Back River, near New Cut. The change is 0.26 mg/L, which is a 7% decrease in the predicted D.O. value. Conversely, there are some increases in D.O. as a result of the model grid modification as well, which occur on Front River. The change is 1.47 mg/L, which is a 16.2% increase. The maximum decrease in the zones for D.O. as a result of modification to the model grid occurs on Front River at the 95th percentile. The relative difference for this change in prediction is 1.2%. Comparatively, the maximum increase in D.O. as a result of modification to the grid is 0.13 mg/L for the 1st percentile and occurs on Back River and has a relative difference of 3%.

For both critical cell and zone comparisons the 50th percentile differences are very minor with D.O. changes less than 0.1 mg/L.

In summary, the changes due to model grid modification with addition of the meeting area geometries are minor. The impacts to tidal fluctuations and the salinity regime as shown by the model results are minor. The largest changes in salinity occur on the bottom of the river and are not expected to pose an impact to adjacent wetlands.

The largest impacts to D.O. as a result of the model grid occur at the extreme percentiles, 1st and 95th while the 50th percentile salinities changes are very minor.

For completeness, the meeting areas could be incorporated into the project conditions model grid. However, as shown by this sensitivity analysis, the changes are likely to have little to no impact on the previous estimates to freshwater marsh/wetland or water quality impacts and therefore no bearing on the mitigation plan or project cost estimates.

7.6.2 Bend Wideners

Despite the fact that the model grid was refined from the coarse grid originally developed by ATM, it doesn't have the precision scale required to evaluate bend wideners. The wideners were not included in the model grid during evaluation of impacts or during mitigation feature development; however, it is not expected that they would have appreciable impacts to salinity and D.O. regimes in the estuary.

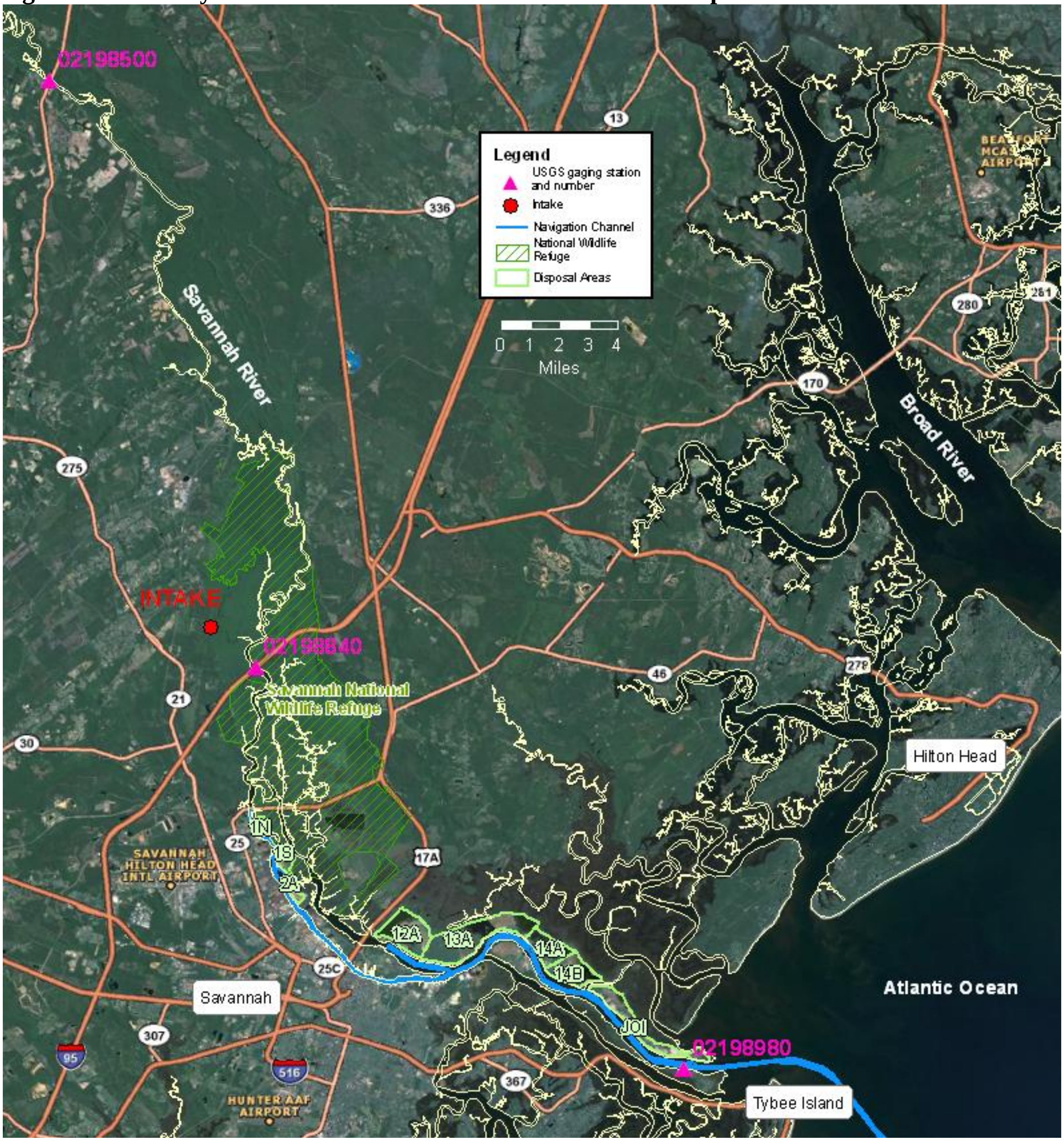
7.7 CHLORIDE MODELING AND ANALYSIS

7.7.1 Background

The City of Savannah operates a municipal and industrial water supply intake located on Abercorn Creek approximately two river miles from the confluence with the Savannah River in Effingham County, GA (See **Figure 7.7.1-1**), The water supply intake is approximately 11 river miles upstream of the proposed Savannah Harbor Expansion Project's deepening upstream limits and approximately 31 miles from the Atlantic Ocean. From the intake, raw water is piped 7.25 miles to a treatment facility in Port Wentworth, GA. Plant capacity is 62.5 million gallons per day (MGD); however, current withdrawal rates are around 30 MGD. Water supply from this source is utilized primarily by industrial users for specific plant processes; however, it also supplies residences in west Savannah, Pooler, and south Effingham County. In addition to this surface water supply, the City of Savannah operates a number of groundwater intakes at other locations and is under directive from the State of Georgia to decrease groundwater usage, which may increase demand for surface water from the Abercorn Creek intake.

Based on EFDC hydrodynamic modeling, the proposed deepening of Savannah Harbor will increase salinity and chloride concentrations in the upper reaches of the Savannah River Estuary, including Abercorn Creek. The EPA drinking water standard for chlorides has been established at 250 milligrams per liter (mg/l) as a Secondary Maximum Contaminant Level. This level is specified as a threshold of taste and odor detection and not as a health hazard. Distribution pipeline corrosion, including lead and copper in residential plumbing, and certain industrial processes are sensitive to chloride concentrations much lower than the drinking water standard.

Figure 7.7.1-1: City of Savannah Raw Water Intake Location Map



7.7.2 Location

Abercorn Creek is a tributary to the Savannah River located in the Coastal Plain of Georgia (See **Figure 7.7.2-1**). The creek is part of a braided river system in that it receives inflow from Bear and Mill Creeks, which branch off from the Savannah River as distributary streams about 10 miles upriver. The confluence of Abercorn Creek with the Savannah River is located approximately one mile upstream from the I-95 Bridge, at river mile 28.5.

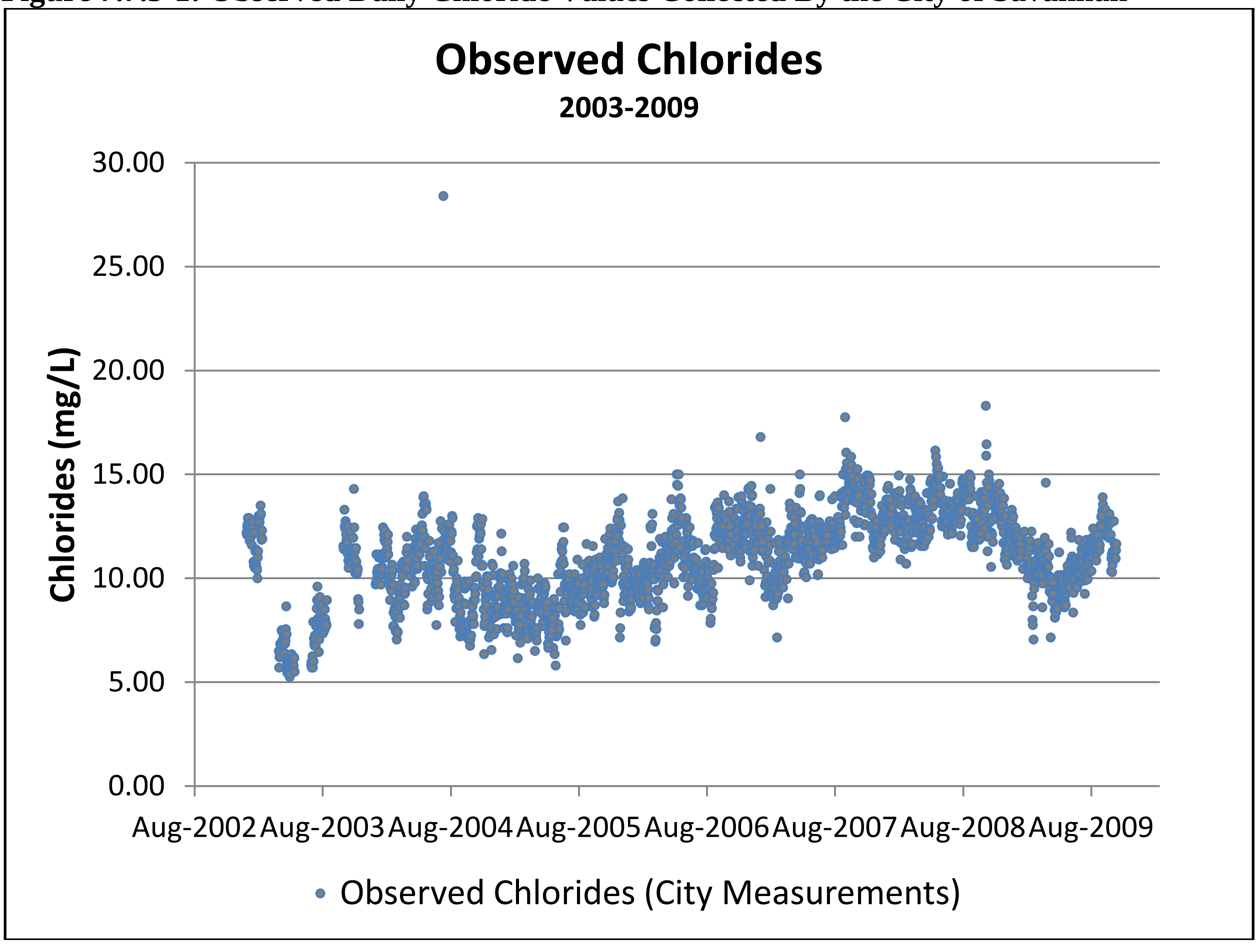
Abercorn Creek is part of the tidal freshwater system with semi-diurnal tides. Currently, salinity concentrations on Abercorn Creek are less than 0.5 parts per thousand (ppt). Salinity intrudes into the Savannah River Estuary from the ocean, and the location of the saltwater-freshwater interface is a constantly changing balance between upstream river flows and downstream tidal conditions.

Figure 7.7.2-1: Lower Savannah River Distributary System

7.7.3 Existing Chloride Regime

Based on data collected by the City of Savannah, the level of chlorides at the water intake on Abercorn Creek has historically averaged 10 to 12 mg/l. This average is based on lab analysis by the City of Savannah on a daily composite sample of the raw water prior to treatment (See **Figure 7.7.3-1**) for the period January 2003 through 2009. Prior to 2003, the lab analysis was performed on a 7 am daily grab sample. The maximum chloride value for the measured data (28.4), as shown in **Figure 7.7.4-1**, is considered suspect because there were no preceding or following high values nor accompanying low flow and spring tide. Therefore, a maximum observed value of 18.3 mg/l was used in the statistical analysis.

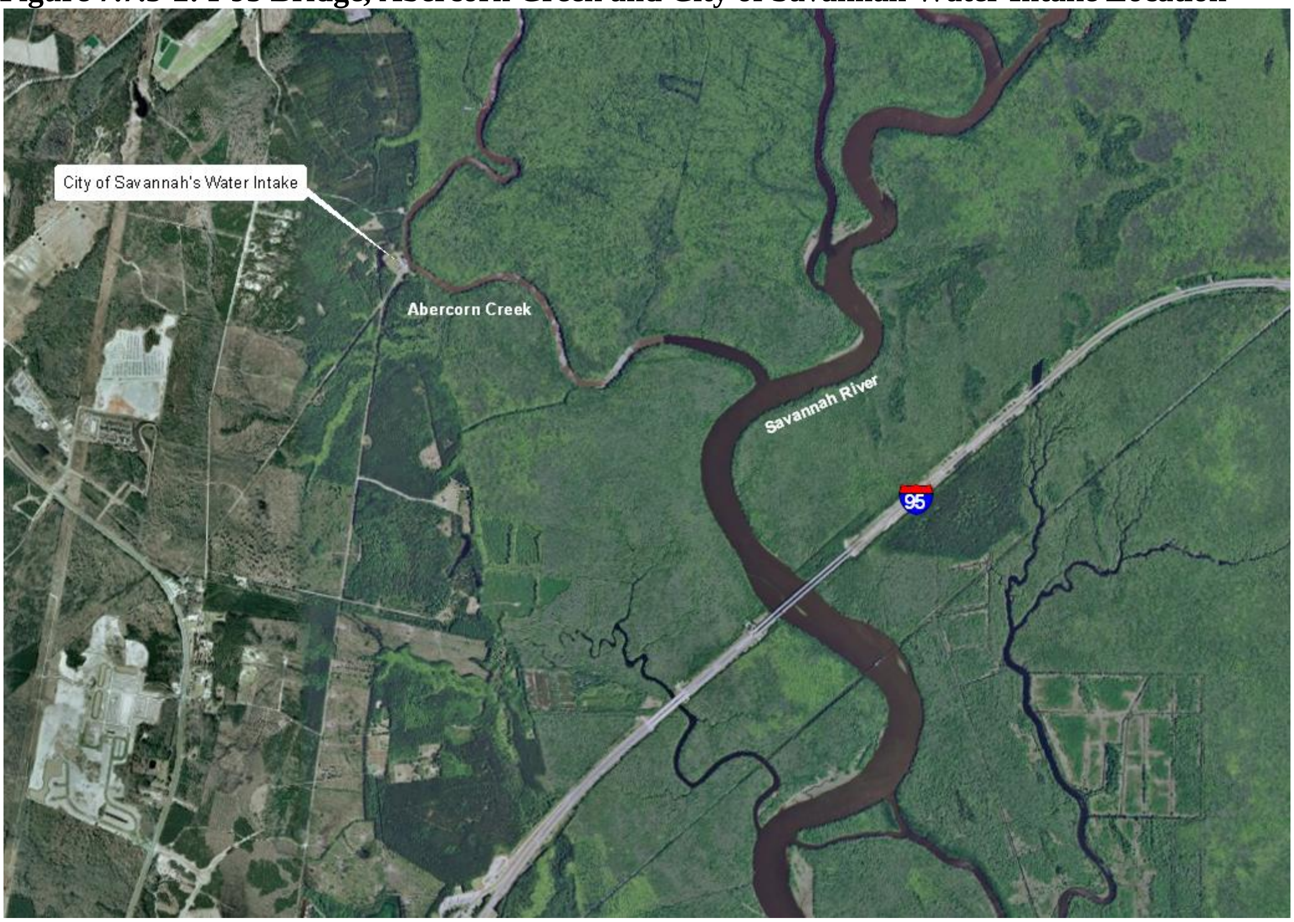
Figure 7.7.3-1: Observed Daily Chloride Values Collected By the City of Savannah



The salinity dynamics at the City's intake are significantly different from the dynamics at the I-95 Bridge, which is located approximately three miles downstream and regularly monitored for specific conductance by USGS gage 02198840. Salinity in the field is measured by specific conductance because it is much easier to measure in-situ and instantaneously. Salinity is not directly convertible to chlorides. In addition to salinity, there are other substances in water that contribute to chlorides and a temperature dependent relationship between conductivity and salinity. Normally, with brackish water, the non-salinity contribution to chlorides is so small that it is neglected; however, at Abercorn Creek which has very low chloride values, the non-salinity contribution to chlorides becomes an important consideration.

The spatial relationship of the Savannah River, the I-95 Bridge, and Abercorn Creek can be seen in **Figure 7.7.3-2**. The sharp salinity intrusions experienced at I-95 do not occur at the water intake but are dampened before reaching it. The specific conductance (field measurement for computing salinity) at I-95 shows sharp increases when streamflow measured at Clyo, GA (RM 61) decreases below 6,000 cubic feet per second (ft^3/s) and a new moon occurs (28-day cyclical spring high tide).

Figure 7.7.3-2: I-95 Bridge, Abercorn Creek and City of Savannah Water Intake Location



7.7.3.1 Low Flow Effect on Chloride Regime

Field observations at I-95 show that upriver salinity intrusion occurs only at river flows less than 6,000 cfs. Flows greater than 6,000 cfs keep the higher saline waters lower in the estuary and do not allow them to move that far upstream.

Table 7.7.3.1-1 depicts a duration analysis of flow data at the USGS Savannah River gage near Clyo, GA. The Clyo gage is located on the Savannah River at river mile 61 which is 32.5 miles above the confluence with Abercorn Creek. There are four analytical periods shown. The period 1929 – 2010 includes the entire period of record. The years 1954 – 2010 include the period of flow regulation by Thurmond Dam. The years 1987 – 2010 include the period during which the releases from Hartwell, Russell, and Thurmond reservoirs were operated using a drought plan designed to minimize impacts to all project purposes. The most recent years, 2001 – 2010, were low flow years.

Table 7.7.3.1-1: Streamflow Frequency Analysis, Savannah River at Clyo, GA

	Gage Period-of-Record	Flow Regulated by Corps Dams	Dam Releases Governed by Drought Plan	Low Flow Period
Condition	1929-2010	1953-2010	1987-2010	2001-2010
Streamflow <4,000 cfs	1%	0%	0%	0%
Streamflow <5,000 cfs	6%	3%	8%	15%
Streamflow <6,000 cfs	17%	14%	26%	42%
Streamflow <8,000 cfs	43%	43%	53%	67%
Streamflow <11,000 cfs	67%	68%	72%	80%
Average Flow	11,460 cfs	11,350 cfs	10,520 cfs	8,740 cfs

The flow records differ among these three periods as a result of reservoir regulation and implementation of drought plan rules. Both affect low flows in the Savannah River. Flows during the period 1987 – 2010 were marked by recurrent droughts and an absence of significant flood events. The time period from 2001 – 2010 includes 2 prolonged extreme droughts, including the drought of record. Flows below 6,000 cfs were experienced 26% of the time during 1987 – 2010, while flows below 5,000 cfs were experienced 8% of the time. During the extremely dry 2001-2010 period, those percentages increased to 42% and 15%, respectively. These percentages are considered to be much higher than would be expected for the long term average. From this data, it is reasonable to conclude that the number of low flow occurrences (less than 6,000 cfs) was approximately doubled during 2001-2010, when compared to what would normally be expected.

The average flow for the period 1987-2010 is 8% below the long-term average and 24% below the long-term average for the 2001-2010 period. If the average annual flows from 1929 to 2010 are sorted from lowest to highest, the seven lowest years all occurred between 1987 and 2010; the four lowest occurred between 2001 and 2010. The drought-of-record for the basin was experienced during the period March 2007 to November 2009.

The first half of the twentieth century was marked by frequent disastrous floods in the basin, while the latter half of the century and the first decade of the 21st century experienced a marked absence of floods and numerous droughts. After converting all flood events to unregulated flow, to account for the impact of the Corps' three large multi-purpose storage projects upstream, 11 of the 12 largest flood events of the twentieth century occurred prior to 1950. These historic flood events illustrate the climatic variability and show that we are presently in an unusually dry period.

7.7.4 Early Modeling Methodology & Impact Predictions

As part of the Savannah Harbor Expansion Project, Savannah District conducted a study to evaluate impacts to the City of Savannah's raw water intake on Abercorn Creek. An impact prediction tool was developed. The details of the development are outlined in the report titled *Savannah Harbor Expansion Project- Chloride Data Analysis and Model Development* dated November 15, 2006 which is included in the Engineering Investigations Supplemental Materials.

The objectives of this study were to:
Provide a statistical correlation between chloride levels at the City's intake, chloride levels at a nearby downstream station, and upstream flows.
Determine the likelihood of increased chloride levels at the City's intake.
Identify potential point and non-point sources of chlorides within the watershed.
Develop a chloride model to predict changes in concentrations at the City's intake.

Development of a statistical correlation was largely based on chloride data collected and analyzed by the City over the period 1988 to 2004. This is an extensive dataset with numerous chloride data points between 5 and 20 mg/l. The statistical correlation (equation) was developed to represent the data points. The correlation has a high level of accuracy predicting within the bounds of the data collected; however, for chlorides predicted outside of this range, the equation is less representative and has a greater margin of uncertainty.

Projection of chloride impacts due to harbor deepening and wetland mitigation using this method are documented in two reports *Chloride Impact Evaluation Impacts of Harbor Deepening Only* dated February 2007 and *Savannah Harbor Expansion Project Evaluation of Chloride Impacts with Proposed Mitigation Plan* dated December 2007, which are both included in the Engineering Investigations Supplemental Materials.

Study findings projected only negligible changes to the chloride concentrations resulting from harbor deepening. The projected impacts were less than 1 mg/l and occurred only during low river flows (less than 6,000 cfs measured at Clyo, GA).

During the review process, concerns were expressed by the independent technical reviewer (USGS) and the City of Savannah about the methodology used to identify potential project impacts and the uncertainties due to lack of chloride data. The Corps reviewed the comments and confirmed that the statistical equation used to predict project impacts was the best that could be developed with the available data. The impact analysis concluded that the impacts to chlorides levels on Abercorn Creek from a harbor deepening would not be significant. That conclusion was reported in the November 2010 Draft GRR and EIS documents.

To address the concerns about the technical reliability of the impact prediction tool, the District and GPA began an intensive campaign to collect additional chloride data. The additional data would be used to improve the accuracy and reliability of the chloride predictive tool, thereby providing a more technically robust evaluation of potential chloride impacts on Abercorn Creek due to harbor deepening.

7.7.5 Data Collection

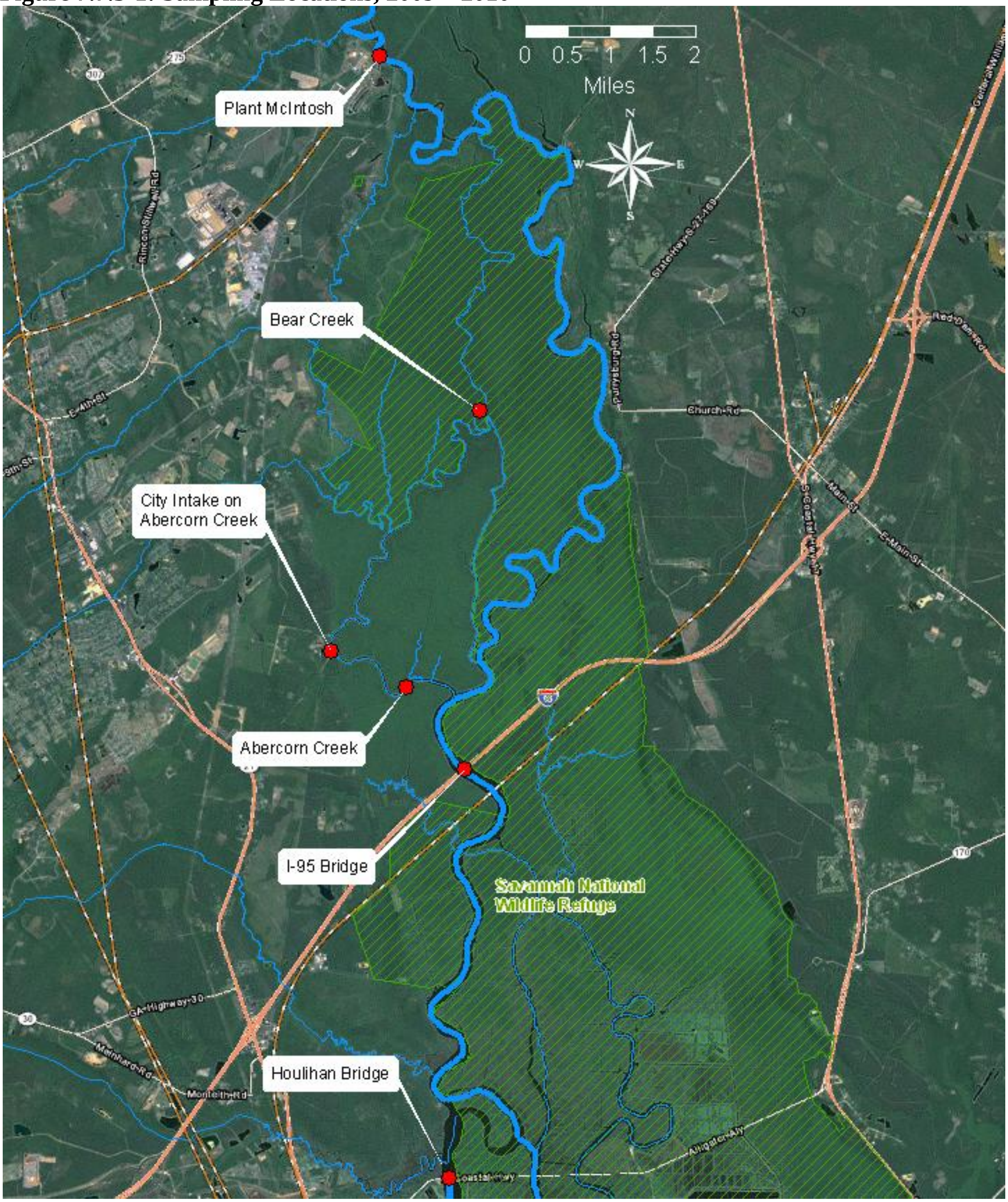
The Corps consulted with the City of Savannah to develop a scope of work for collection of additional chloride data that it could use to refine its tool to predict chloride levels with a harbor deepening. USGS and GPA also participated in development of the scope of work. Data was collected from early 2009 through summer 2010. The scope of work included collection of data using several techniques at multiple locations (see **Figure 7.7.5-1**), including:

Abercorn Creek (flows, water surface, chloride, temperature, and conductivity)
Bear Creek (flow, water surface, flow splits for Abercorn and Little Collis Creeks)
I-95 Bridge (water surface, chloride, temperature, and conductivity)
Houlihan Bridge (flow, water surface, chloride, temperature, and conductivity)
Plant McIntosh (water surface, chloride, temperature, and conductivity)
City Intake on Abercorn Creek (chloride)

Considerable effort was expended by Savannah District, USGS, and ERDC to collect additional data. Automated collection of samples at various locations followed by laboratory analysis of the samples proved successful. Efforts to record real-time chloride data were not successful due to unreliable field instruments. In addition to chloride data, velocity measurements and flow data were collected at Three Mouths, which is the confluence of Abercorn, Bear and Little Collis Creeks, in order to better calibrate the flow split in the hydrodynamic model at that location.

The Corps used this new data, the City's original chloride data, and subsequent daily chloride measurements collected by the City of Savannah to refine the modeling methodology.

Figure 7.7.5-1: Sampling Locations, 2009 – 2010



7.7.6 Updated Modeling Methodology

The new modeling methodology, development and calibration, is outlined in the report titled *Chloride Modeling Savannah Harbor Expansion Project Savannah, Georgia* prepared by Tetra Tech and Advanced Data Mining Services, dated December 31, 2010, and is included in the Engineering Investigations Supplemental Materials. The new model methodology has two parts: (1) an updated version of the EFDC model using an enhanced hydrodynamic grid to include the complicated distributary system of Abercorn Creek, and (2) an Artificial Neural Network (ANN) which uses data mining techniques. The two-pronged modeling approach provided both a mechanistic and empirical approach for predicting chloride concentrations at the City's intake and allows presentation of the findings in "bands" to better represent uncertainty associated with the data and the models. The two independent methodologies provided reasonably close agreement on chloride projections and are the best possible evidence of accuracy in the projections.

7.7.6.1 Technical Review

An Agency Technical Review (ATR) was performed on the updated model methodology and the report titled *Chloride Modeling Savannah Harbor Expansion Project Savannah, Georgia* prepared by Tetra Tech and Advanced Data Mining Services. A South Atlantic Division Regional Technical Expert for Water Resources Engineering performed an ATR of the EFDC component and ERDC staff experienced with neural networks performed an ATR of the ANN component of the chloride model.

The Independent External Peer Review (IEPR) was conducted by Battelle, Inc. Dr. Andy Stoddard of Dynamic Solutions, LLC was the principal reviewer on the Battelle team for chloride analysis. Comments from these rigorous reviews were incorporated into the modeling and analysis for chloride SHEP impacts determination and the reviewers concluded that the models were applied appropriately for this purpose.

7.7.7 Chloride Impact Predictions

7.7.7.1 Analysis of Daily Average Chloride Concentrations and Durations

Model predicted daily average and maximum daily average chloride concentrations are shown in **Table 7.7.7.1-1**. Results represent findings for two simulation periods.

The first period, 2003 to 2009, was simulated with both the EFDC and ANN models. This period was flood-free and included several prolonged drought periods, including the drought-of-record for the Savannah River Basin. Results represent impacts that could be expected during periods of extreme drought.

The second period, 1987-2009, and was simulated with EFDC only to determine the magnitude and duration of impacts over a more representative period of time and river flow conditions.

For the 1987-2009 simulation period, average chloride concentrations at the City's water intake would increase with all deepening alternatives, but the increase would be small, ranging from 0.2 to 2.9 mg/l (from the existing 10.6/10.8 mg/l). Maximum daily average chloride concentrations would also increase with all deepening alternatives, ranging from 22 to 55 mg/l (from the existing 18 mg/l). All maximum values occurred during simulation of the drought of record (2008).

Table 7.7.7.1-1: Daily Average and Maximum Daily Average Predicted Chloride Levels

	1987-2009 typical river flow	2003-2009 drought flow	2008 drought of record[4]
Project Depth Alternative	Average Daily Chloride Level, mg/l	Average Daily Chloride[3] Level, mg/l	Maximum Daily Average Chloride Level, mg/l
Existing[1]	10.6	10.8	18.3
44 ft Project	10.8	11.6	40.9
45 ft Project	11.1	12.0	48.6
46 ft Project	11.4	12.5	53.6
47 ft Project[2]	11.7	13.1	62.2
48 ft Project	12.2	13.7	73.6

[1]Existing chloride value obtained from measurements observed by the City of Savannah.
[2]NED Plan
[3]Chloride values are averages of the ANN and EFDC approaches.
[4]All maximum values occurred during simulation of the drought of record. There are 80+ years of flow data in the record and the drought of record occurred in 2008.

As the daily laboratory testing of chlorides is performed on a 24-hour composite sample, so too are the daily chloride model projections presented above a composite of modeled hourly chloride values.

The maximum number of days that daily average chloride concentrations would be at or above a specified level for the 1987-2009 and 2003-2009 simulation periods are summarized in **Table 7.7.7.1-2**. Data shown for the existing channel condition is from daily sampling and laboratory analyses performed by the City of Savannah at their Port Wentworth water treatment plant. The chloride spikes would be caused by a combination of low flow and spring high tides.

Table 7.7.7.1-2: Duration Using Daily Average Chloride Concentrations

Project Depth Alternative	Days Greater Than … % of Days Greater Than…									
	2003-2009 drought flow					1987-2009 typical river flow				
	> 5 mg/l*	> 15 mg/l	> 25 mg/l	> 40 mg/l	> 50 mg/l	> 5 mg/l	> 15 mg/l	> 25 mg/l	> 40 mg/l	> 50 mg/l
Existing	2483 100%	26 1.0%	0 -	0 -	0 -	6705 100%	111 1.7%	1 0.0%	0 -	0 -
44 ft Project	2483 100%	252 10.1%	39 1.6%	1 0.0%	0 -	8374 100%	457 5.5%	52 0.6%	1 0.0%	0 -
45 ft Project	2483 100%	331 13.3%	72 2.9%	11 0.4%	0 -	8374 100%	669 8.0%	91 1.1%	10 0.1%	0 -
46 ft Project	2483 100%	413 16.6%	112 4.5%	14 0.6%	4 0.2%	8374 100%	853 10.2%	142 1.7%	19 0.2%	4 0.0%
47 ft Project**	2483 100%	483 19.5%	156 6.3%	31 1.2%	10 0.4%	8374 100%	1051 12.6%	219 2.6%	41 0.5%	11 0.1%
48 ft Project	2483 100%	549 22.1%	206 8.3%	54 2.2%	18 0.7%	8374 100%	1301 15.5%	330 3.9%	68 0.8%	23 0.3%

**Background levels of chlorides are greater than 5 mg/l under all channel conditions.*
***NED Pla*n

Table 7.7.7.1-3 shows the percentage of time that chloride levels are projected to be above 25 mg/l and 50 mg/l for the existing 42-foot channel depth and the 47-foot depth for simulations for each year from 2001-2009. This period is not typical, as discussed previously in Section 7.7.4 and includes 2 prolonged extreme droughts, including the drought of record occurring in 2008.

Table 7.7.7.1-3: Percentage of Days Each Year that Daily Average Chlorides are Greater Than 25 mg/l and 50 mg/l

Year	Chlorides > 25 mg/l		Chlorides > 50 mg/l	
	Existing 42 ft Depth	47 ft Project Depth	Existing 42 ft Depth	47 ft Project Depth
2001	0%	6%	0%	0.0%
2002	0%	5%	0%	0.0%
2003	0%	0%	0%	0.0%
2004	0%	2%	0%	0.0%
2005	0%	0%	0%	0.0%
2006	0%	0.2%	0%	0.0%
2007	0%	9%	0%	0.5%
2008	0%	17%	0%	2.2%
2009	0%	9%	0%	0.0%

7.7.7.2 Analysis of Hourly Chloride Concentrations and Durations

The City has indicated that it is necessary to adjust some of its treatment processes on an hourly basis, and as a result, is concerned about hourly fluctuations in the chloride concentration at their raw water intake. Because there is limited mixing of the water as it moves through the Savannah I&D Water Plant process and the distribution pipelines, these hourly chloride projections are critical to the operation of the water plant.

The EFDC model used daily average chloride concentrations in its calibration process, which used data from 2001 through 2009. Since 2003, the City has analyzed chloride content of its intake water on a daily basis using a composite of samples taken hourly. Therefore, the chloride values upon which the updated EFDC model was calibrated represent a daily composite/average of hourly samples.

The District consulted with Tetra Tech, the developer of the updated EFDC model for chlorides, about the potential reliability of the EFDC calculations for hourly chloride values at the City's water intake. Tetra Tech concluded that although the daily average chloride value projections were derived from the hourly computed values, the model was not calibrated with the intent of generating hourly data and that the 90^{th} percentile predicted chloride value is a more reasonable representation of the maximum short-duration chloride level likely to be experienced with the proposed harbor deepening project.

As a result, the District used the EFDC model to predict hourly chloride values at the City's water intake. In **Figure 7.7.7.2-1**, the green line indicates the hourly maximum predicted for the day, the yellow line is the daily average, and the blue line is the minimum hourly value predicted for the day. The predicted hourly peak of 185 mg/l is substantially higher than the maximum daily average of 62 mg/l. The predicted daily minimum remains at about 15 mg/l on that peak day. The peak 90^{th} percentile chloride value is about 150 mg/l. The average being less than half the peak indicates that the lower chloride values have a longer duration than the higher chloride values. It also indicates that the high chloride levels are tidally dependent. When the tide ebbs, chloride levels are predicted to return to normal levels.

Model projections indicate that for the period 1987 – 2009, there would be 41 days where the average daily chloride concentration exceeds a threshold of 40 mg/l; however, hourly exceedances of 40 mg/l are projected to occur 445 days during this same period. Hourly exceedances of 40 mg/l typically occur 3 to 6 hours per day, with a maximum of 12 hours per day. We have little measured hourly chloride data for comparison, but the model projects that the 40 mg/l threshold has never been exceeded for even 1 hour under existing conditions.

Figures 7.7.7.2-2 through **7.7.7.2-5** show the model predicted hourly output for the existing conditions and all deepening alternatives with and without the flow-altering mitigation plan. **Figures 7.7.7.2-3** and **7.7.7.2-5** are zoomed in to show the variation in chloride concentrations during the period of November 2008 when they are at peak levels.

Figure 7.7.7.2-1: Model Predicted Daily Maximum, Minimum and Average 2001 – 2009, 47 ft Project Depth with Mitigation

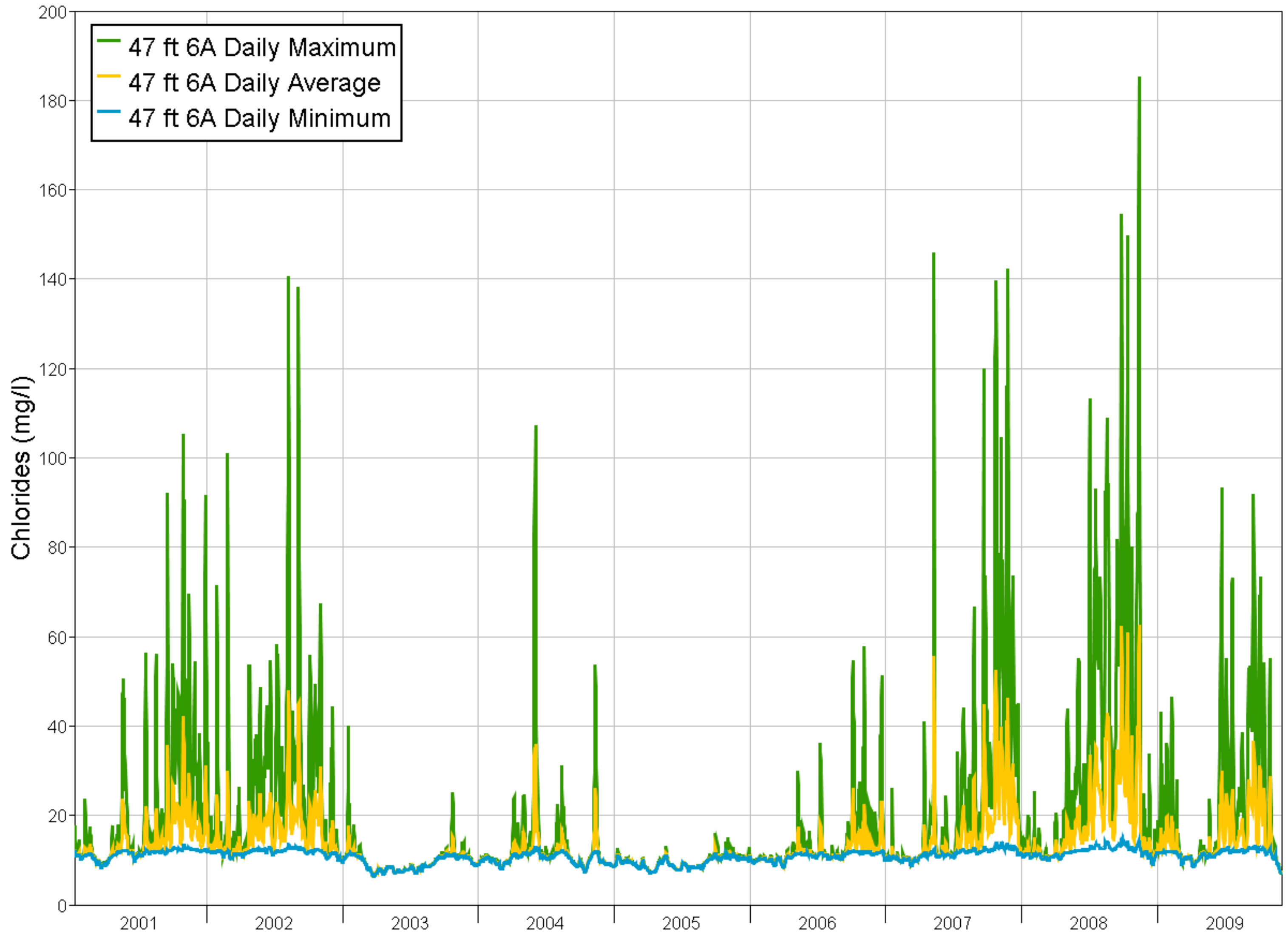


Figure 7.7.7.2-2: Model Predicted Hourly Output 2001 – 2009, Existing Conditions and All Project Depths (Deepening Only, No Mitigation)

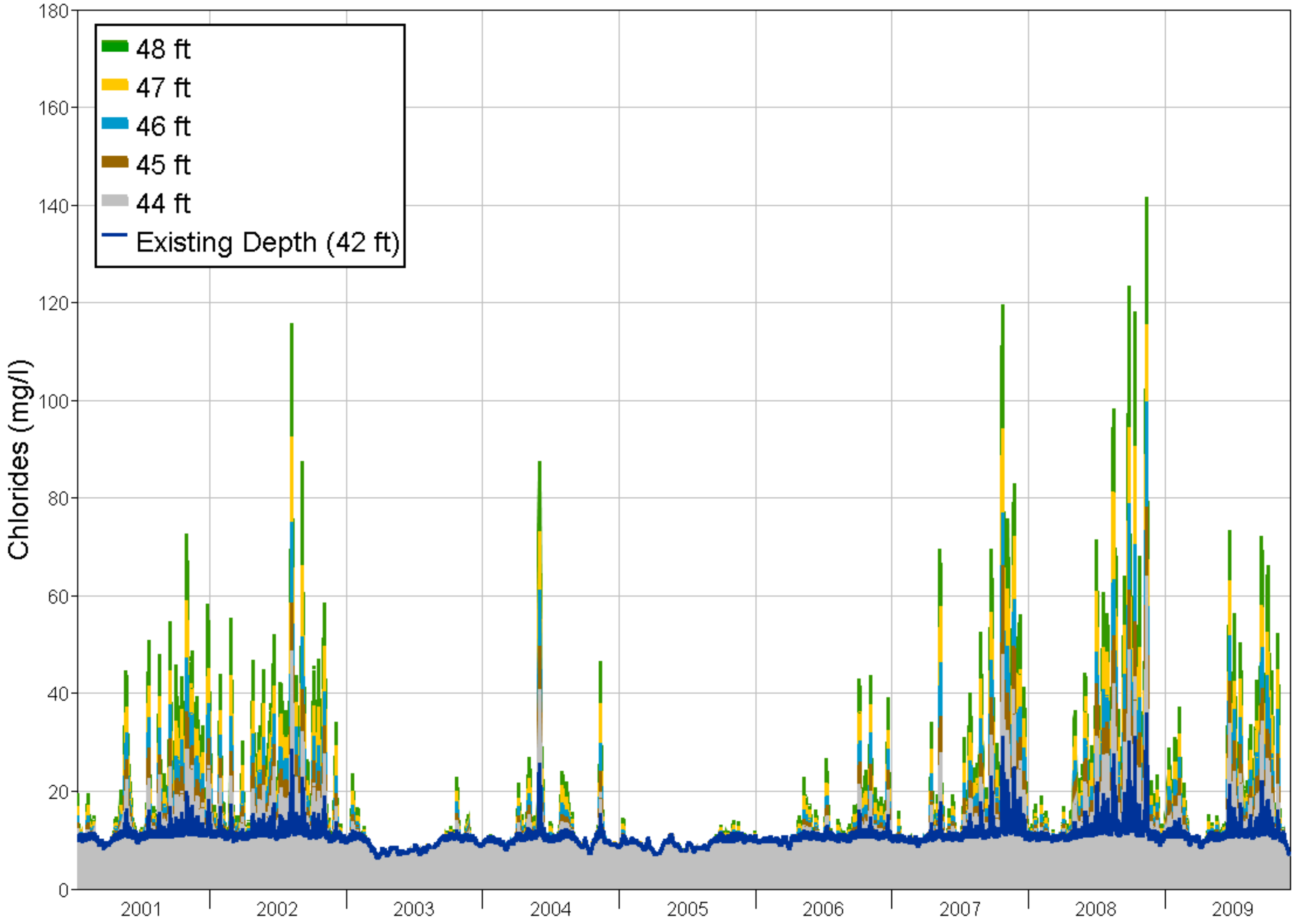


Figure 7.7.7.2-3: Model Predicted Hourly Output 2-17 November 2008, Existing Conditions and All Project Depths (Deepening Only, No Mitigation)

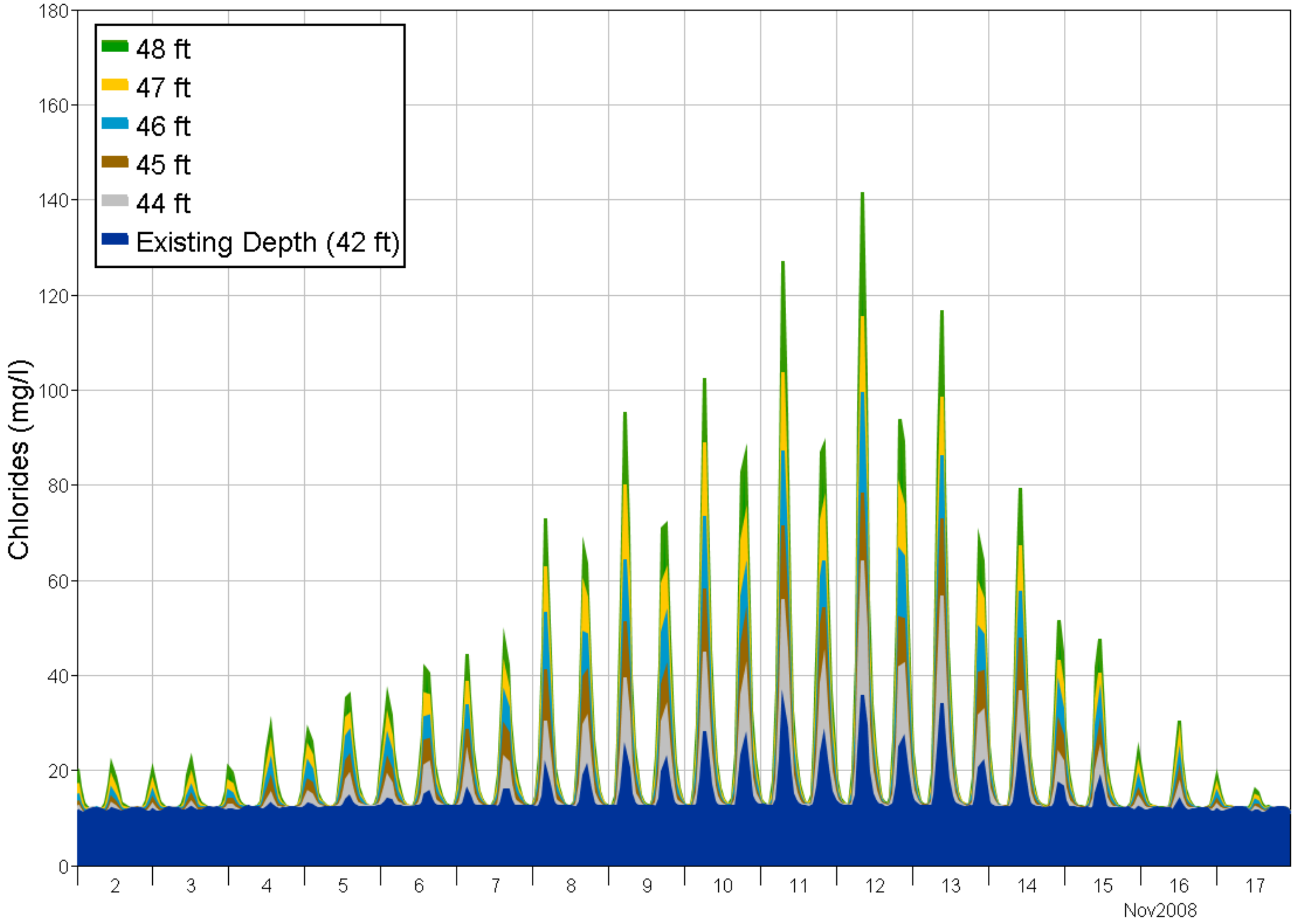


Figure 7.7.7.2-4: Model Predicted Hourly Output 2001 – 2009, Existing Conditions and All Project Depths (Deepening and Mitigation)

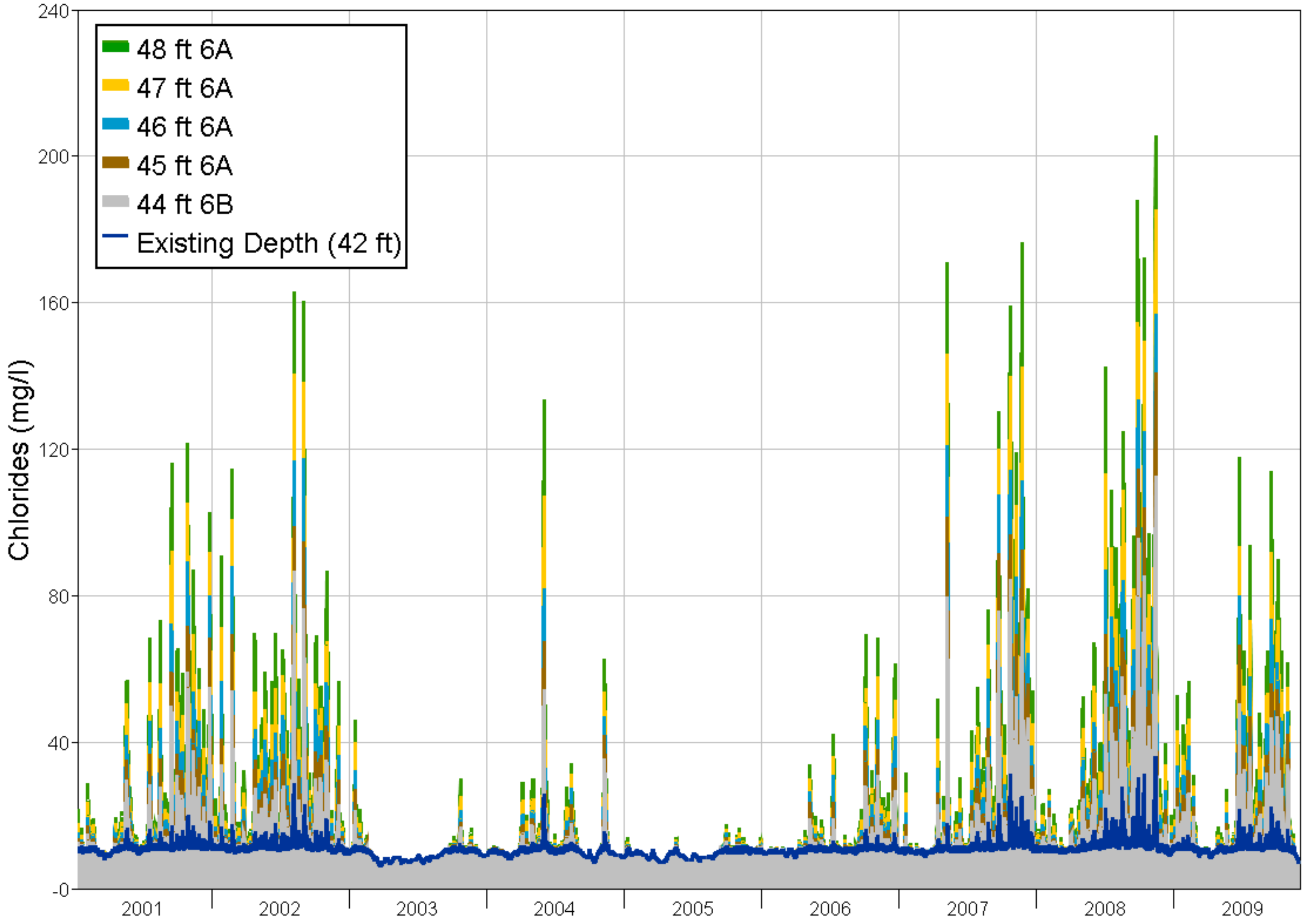
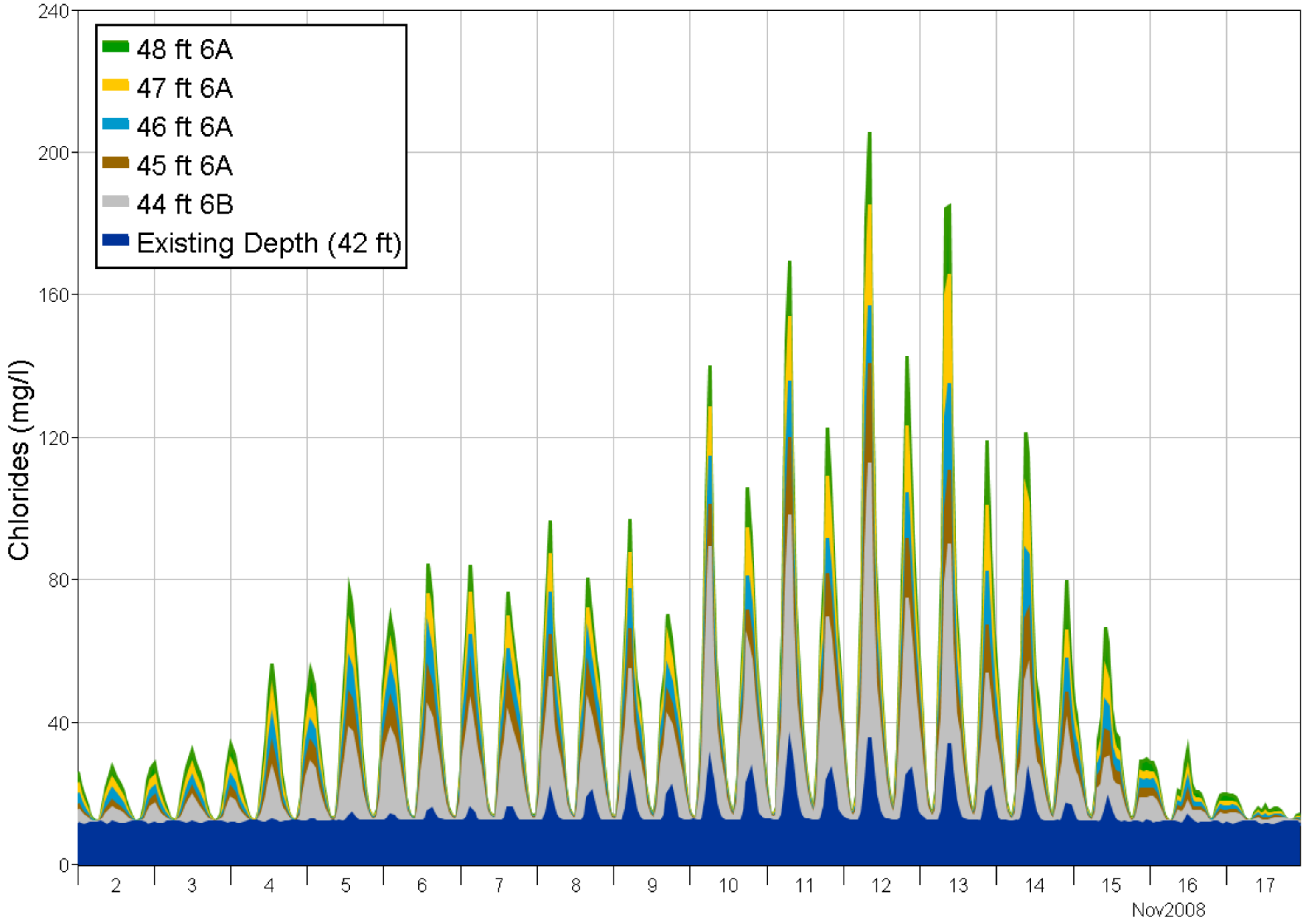


Figure 7.7.7.2-5: Model Predicted Hourly Output 2-17 November 2008, Existing Conditions and All Project Depths (Deepening and Mitigation)



7.7.8 Drinking Water Concerns

The Safe Drinking Water Act originally passed by Congress in 1974 authorizes EPA to set standards for drinking water quality to protect public health. Those standards are regulated and must be met by water suppliers to ensure drinking water is safe for human consumption. While the City of Savannah's water supply intake located on Abercorn Creek largely supplies water for industrial purposes, it also supplies municipal water and is regulated as such to ensure that the water is safe for drinking.

National drinking water regulations can be classified as primary or secondary and thresholds are specified by a maximum contaminant level (MCL). Primary standards protect public health by limiting the levels of contaminants in drinking water while secondary standards are aesthetic considerations and are not federally enforced.

The City's treatment facility in Port Wentworth currently treats approximately 30 million gallons of water per day (30 MGD) from Abercorn Creek before it is supplied to the distribution system. The plant's design capacity is 62.5 MGD and its current withdrawal permit from GA DNR-EPD is for 55 MGD.

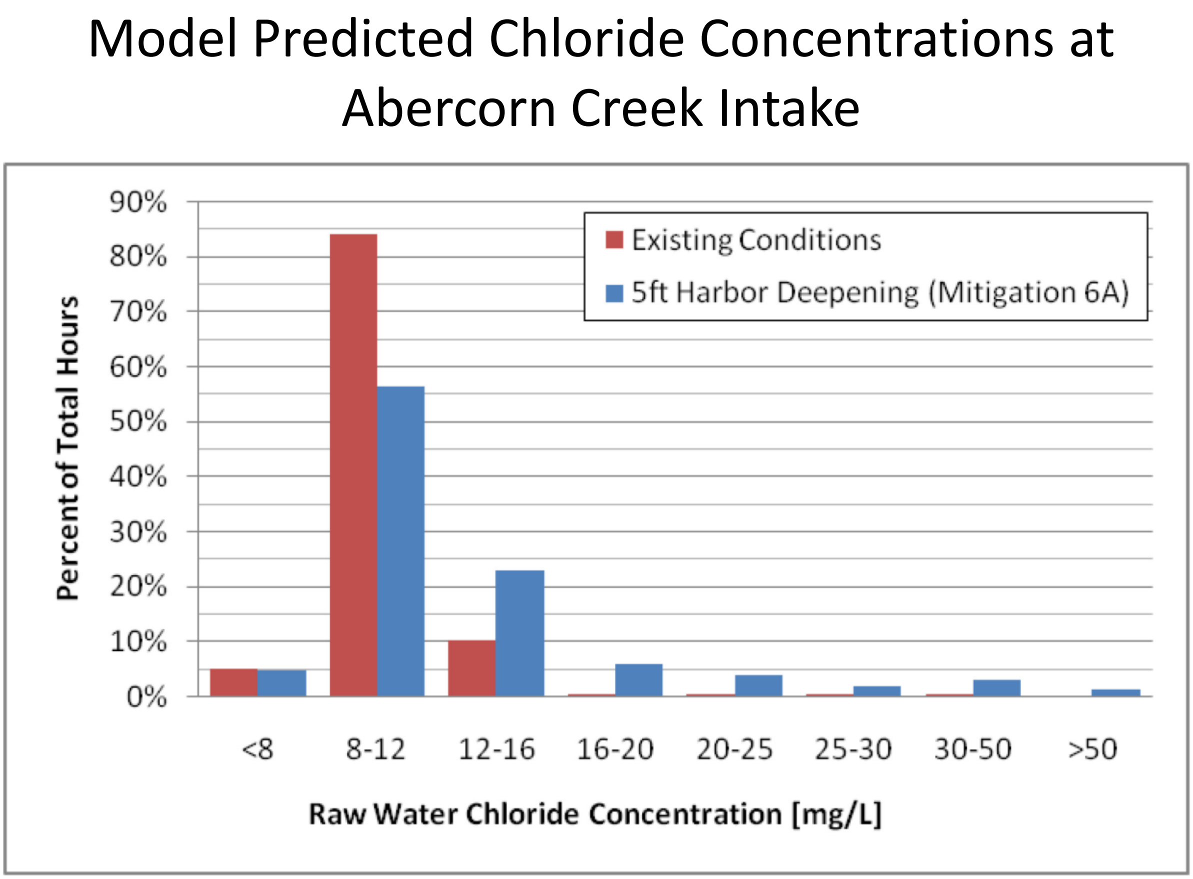
7.7.8.1 Chloride Predictions and Secondary Drinking Water Standards

The updated impact analysis indicates that the proposed harbor deepening would increase chloride levels at the City of Savannah's water intake on Abercorn Creek under drought conditions during high tide. Under those conditions, maximum daily average chloride levels are predicted to be 62 mg/l with a maximum hourly chloride level projected as 185 mg/l for the 47ft project. However, the long term average chloride level is only predicted to increase from 11 to 13 mg/l. **Figure 7.7.8.1-1**shows a comparison of Model Output for Existing Conditions and 47 ft deepening with mitigation.

The National Secondary Drinking Water Standard for chloride is 250 mg/l. This level is established as a threshold of taste and odor detection and not as a health hazard. The predicted chloride concentrations with harbor deepening do not approach that threshold, even under the worst-case drought conditions (drought of record).

Savannah District contacted the chief of the Washington Aqueduct water system (a municipal water system operated by the Baltimore District, USACE for the Washington, DC area) regarding the anticipated impacts from this type of change in water chemistry on a water treatment system. The issue was discussed with Dr. Vern Snoeyink, a leading expert in the field of water treatment. Dr. Snoeyink is a Professor of Environmental Engineering at the University of Illinois and author of a commonly used college textbook on water chemistry. They cautioned the District to avoid using the Secondary Maximum Contaminant Limit of 250 mg/L as a relevant standard upon which to evaluate the merits of the water utility's concerns as it is a taste threshold. They also indicated a concern for the water supplier's ability to achieve "simultaneous compliance". The term "simultaneous compliance" refers to a situation where addressing one regulation threatens compliance with a different regulation. In the fall of 2000, Washington Aqueduct made a change in its disinfectant to address disinfection byproduct (DBP) issues, and as a result, unknowingly affected the corrosion control in the system that manifested itself in increased lead release from lead service lines and other fixtures more than three years later. They further stated "The changes can be very subtle on the front end, but the chemistry through the plant and into the distribution system is extraordinarily complex."

Figure 7.7.8.1-1: Comparison of Model Output, Existing Conditions vs. 47 ft NED Plan with Mitigation



7.7.8.2 Corrosion and Primary Drinking Water Standards

This section discusses concerns that increased chlorides could have on:

- Corrosion of steel water distribution pipes resulting in increased life-cycle costs for the pipe distribution networks.

> In their February 2011, letter the City presented model results that show that corrosion rates of steel double for a chloride increase from 18 mg/l to 70 mg/l, on average, neglecting the influence of temperature. The City owns and maintains about 750 miles of water distribution pipeline, 60% of which is steel. They computed that a 12% decrease in life expectancy of pipelines, corresponded to an increased replacement cost of $22 million.

- Corrosion of lead and copper which could lead to unsafe levels of copper and lead ions in the water.

> National Primary Drinking Water Standards specify regulations for lead and copper through the Lead and Copper Rule. The Lead and Copper Rule is a Federal regulation which limits the concentration of lead and copper water suppliers can allow in public drinking water. It was issued by the US Environmental Protection Agency June 1991 pursuant to the Safe Drinking Water Act as amended. Lead and copper primarily enter the drinking water from corrosion of plumbing materials that utilize copper pipe and lead solder. Potential health risks can result from exposure to lead that can include brain, red blood cell, and kidney damage.

At the direction of the Savannah District, Arthur Freedman Associates performed an investigation and analysis of water system chloride concerns. This report, completed April 29, 2011, is titled *Assessment of Chloride Impact from Savannah Harbor Deepening* and is included in the Engineering Investigations Supplemental Materials. Their analysis included computer simulations (WatSim), which indicated that raising pH was a potential remedy for increased corrosion rate, and subsequent laboratory testing to confirm the model study. The conclusion of the Freedman analysis was that copper and lead corrosion were likely not an issue and that steel corrosion could be controlled by raising the pH of the treated water supplied to the distribution system. Their study also recognized that increasing pH to reduce corrosivity can result in the formation of disinfection byproducts (DBPs), such as trihalomethanes and bromates, which are suspected carcinogens and regulated by the National Primary Drinking Water Standards.

The Freedman report suggested additional laboratory analyses to confirm these conclusions. The GPA, in coordination with the District and the City of Savannah, contracted with Camp Dresser and McKee (CDM) to perform more detailed laboratory analyses on location at the water treatment plant that would replicate the City's current water treatment process and evaluate the impact of increasing chlorides on the plant water and treatment process including analysis of DBP formation. Their report titled *City of Savannah Seawater Effects Study* dated December 2011 which is included in the Engineering Investigations Supplemental Materials addressed concerns of the City's requirement for simultaneous compliance and presented evidence that there are two significant impacts to drinking water quality from increased chlorides – increased lead corrosion and formation of disinfectant byproducts (DBPs).

7.7.8.2.1 Lead Corrosion

Based on the laboratory analyses performed by CDM, lead corrosion is projected to increase considerably with increased chlorides. **Figures 7.7.8.2-1** and **7.7.8.2-2** show the effect of increased chlorides on lead, copper, and iron corrosion. They vary by the disinfectant used which mimics two distinct systems at the City's water treatment plant. While copper and iron concentrations were not shown to increase with increasing chlorides, lead concentrations in the water samples were shown to increase 2-4 times compared to the existing conditions as chloride concentrations increased from 10 mg/l to 50 mg/l. Also shown on these figures is the chloride to sulfate mass ratio (CSMR), which is a well documented indicator of corrosivity. Water utilities find this ratio useful as it can be measured at the plant prior to the water being exposed to metals within the distribution system. Research has shown that utilities with a CSMR ratio of 0.58 or less had greater tendencies to meet the action level for lead. This level is indicated by a red dashed line on **Figures 7.7.8.2-1** and **7.7.8.2-2**.

Figure 7.7.8.2-1: Lab Results of Chloride Effects on Metal Release (Free Chlorine Disinfectant)

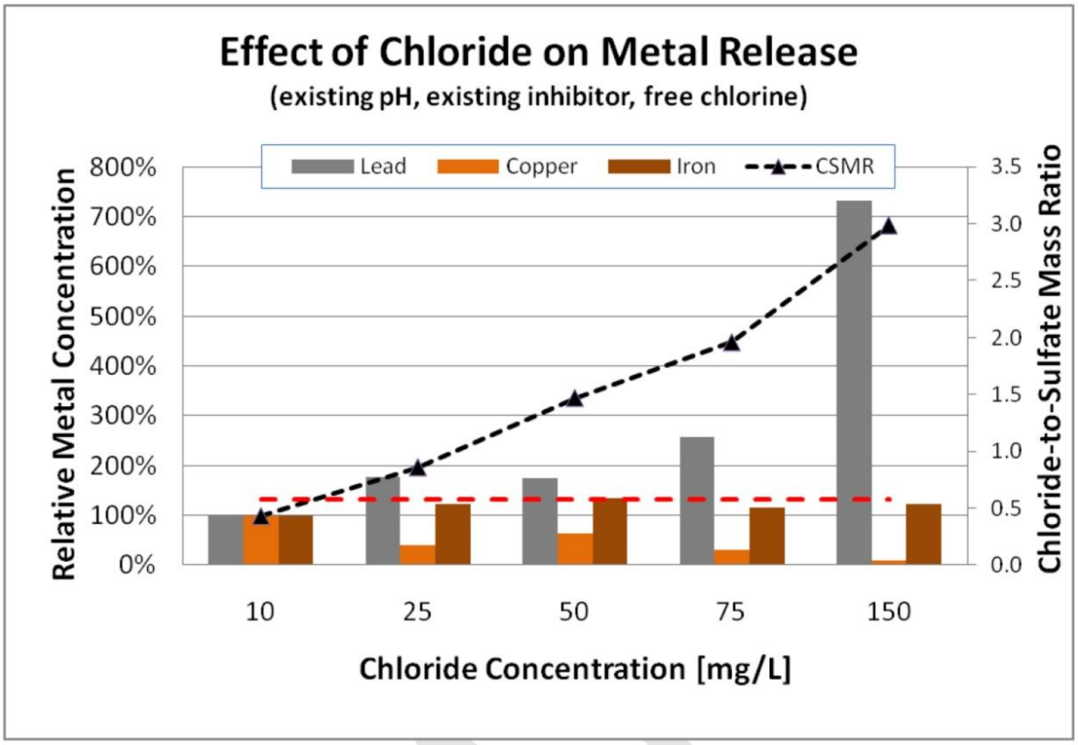
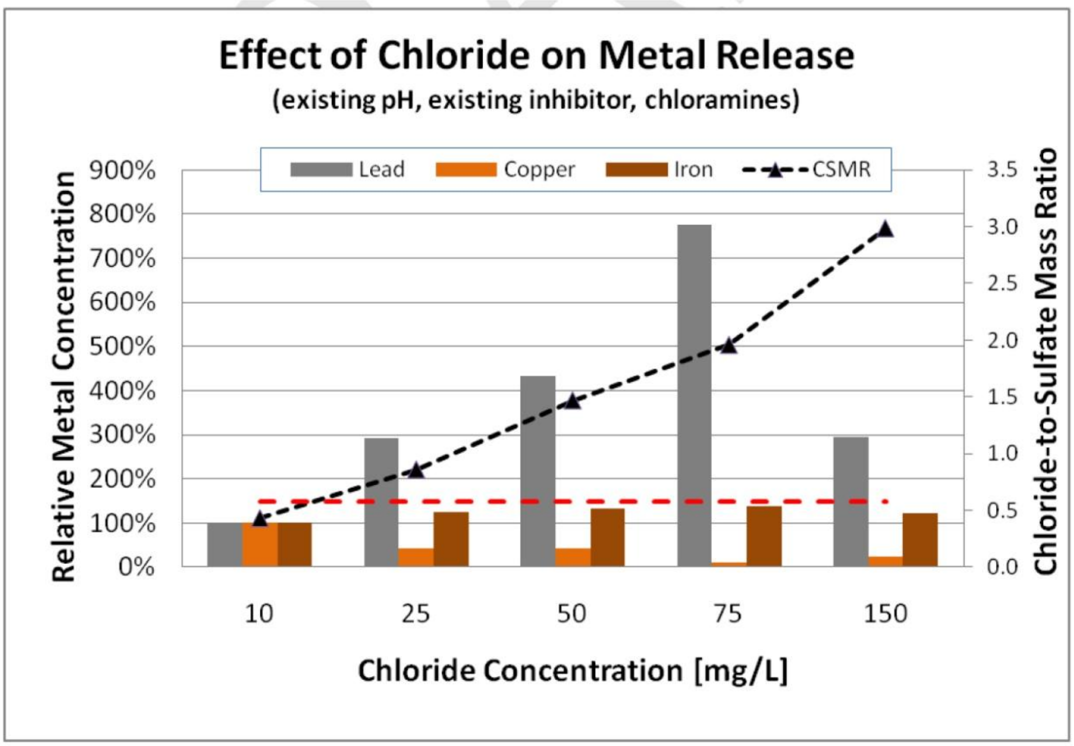


Figure 7.7.8.2-2: Lab Results of Chloride Effects on Metal Release (Chloramines Disinfectant)



Whether or not those increased levels will exceed regulatory action limits as defined by the Lead and Copper Rule cannot be determined with certainty due to the fact that regulatory sampling for lead is performed at the customer's tap and is highly dependent upon the customer's piping and the contact time in that piping. Although there are Federal regulatory limits, as outlined by the Lead and Copper Rule for lead levels in drinking water, the ideal level is zero lead. That means that it is not acceptable to cause increases in lead levels as long as the regulatory threshold is not exceeded. Any increase in lead concentration is considered an increased health risk.

7.7.8.2.2 Disinfection Byproduct Formation

They City of Savannah uses free chlorine as a disinfectant against pathogens in their water treatment process, as do many water suppliers. It is an effective disinfectant and is available at a relatively low cost. However, free chlorine can react with dissolved natural organic matter present in the water to form byproducts. These disinfection byproducts (DBPs) can be classified as trihalomethanes (THMs) and haloacetic acids (HAAs) which are regulated under the Disinfectants and Disinfection Byproducts Rule (D/DBPR). The D/DBPR is a Federal regulation which limits the concentration of DBPs water suppliers can allow in public drinking water. It was issued by the US Environmental Protection Agency December 1998 pursuant to the Safe Drinking Water Act as amended. Potential cancer, reproductive and developmental health risks can result from exposure to DBPs. Through the D/DBPR, the USEPA MCL for total trihalomethanes is 80 µg/l and the five haloacetic acids are 60 µg/l. These are Primary Drinking Water Standards and violations require notifying the public as well as reporting to the State.

The CDM lab analysis showed that DBPs are affected by increasing chlorides in two ways:

Increasing chloride concentrations due to SHEP causes an increase in chlorine required to treat the water. (See **Figure 7.7.8.2.2-1**) The additional disinfectant required to achieve treatment goals causes the formation of additional byproducts.

As chlorides are pushed further upstream with harbor deepening, bromides, which are another component of seawater, are pushed further upstream as well. Brominated compounds can react with chlorine to form bromine-containing THMs, HAAs, and other byproducts. The rate of DBP formation is also affected by the presence of bromide in the source water.

Under both of these conditions expected to occur under SHEP, total THMs are projected to increase above the permitted level when chlorides exceed about 60 mg/l (See **Figure 7.7.8.2.2-2**). HAAs are not projected to increase above regulatory limits; however, the regulated species may be expanded in the future to include brominated HAAs, at which time the chloride impacts could affect compliance.

Figure 7.7.8.2.2-1: Lab Results of Chloride Effects on Chlorine Demand

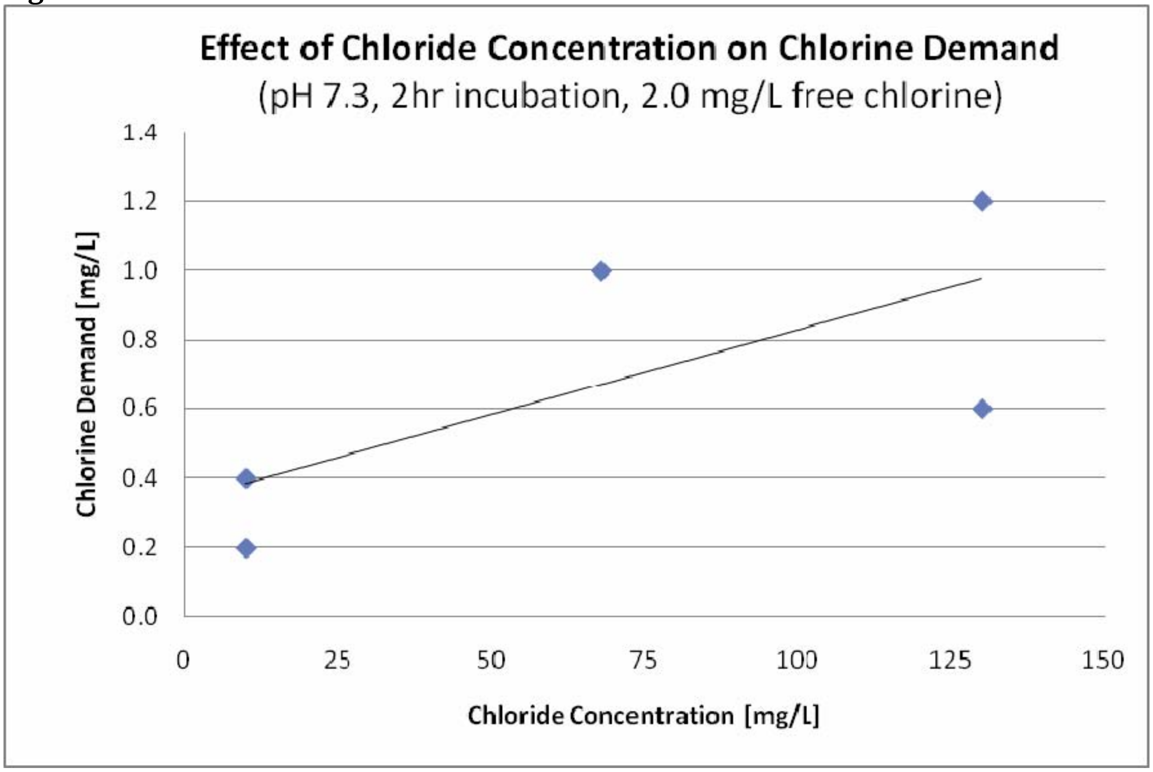
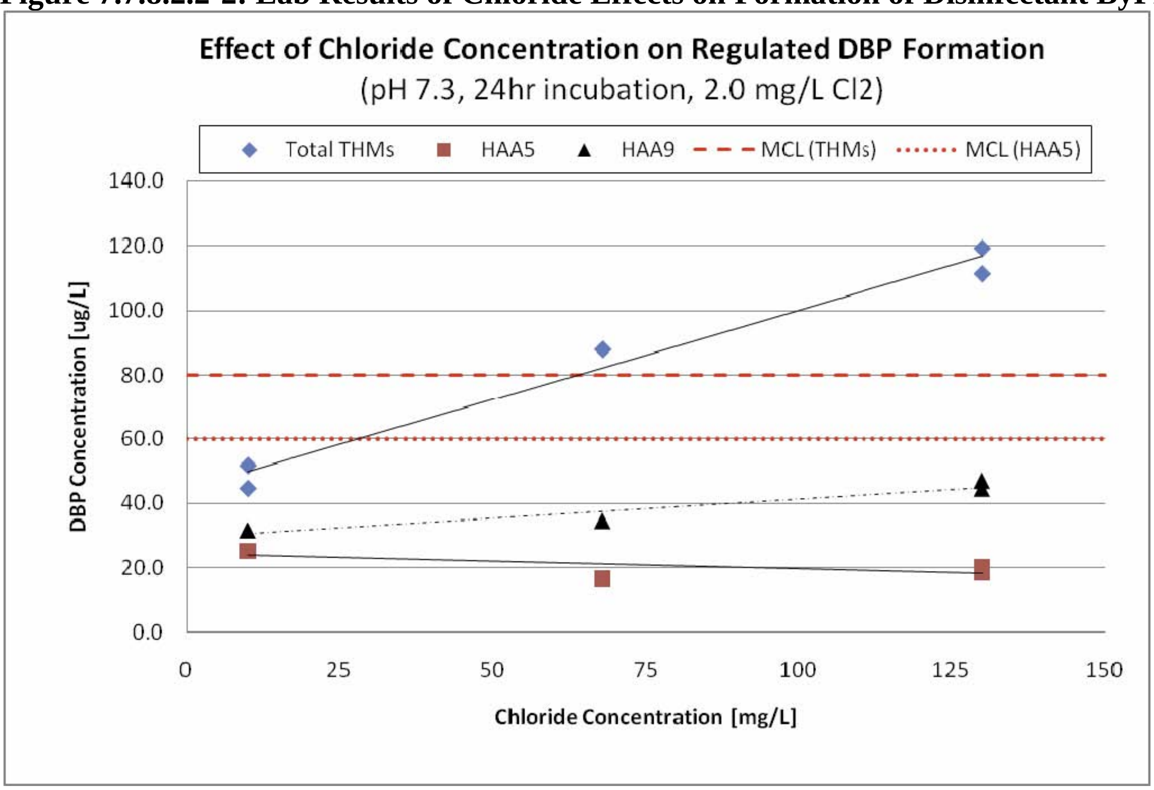


Figure 7.7.8.2.2-2: Lab Results of Chloride Effects on Formation of Disinfectant ByProducts



7.7.9 Industrial Water Supply Concerns

In a 2008 letter, the City of Savannah provided estimates of costs to the industrial users if chlorides were increased to 50 mg/l. The cost was very high but not considered to be representative since the increases to 50 mg/l are only projected to be occasional, not continuous.

7.7.9.1 Weyerhaeuser

Weyerhaeuser is the single largest user of the City's surface water supply; their demand is currently 12-13 MGD. Usage was higher, 15-16 MGD, before process water cooling towers were installed. The plant does not operate any groundwater wells. They use surface water supplied by the City supplemented by an intake they operate on-site near the Houlihan Bridge that draws 12-15 MGD of estuary water into the plant. The on-site intake water is used 1) in a large plant fire protection system and 2) to cool the black liquor surface condenser which operates with once-through cooling water. The intake is designed to draw water from near the surface; therefore, surface water model results most closely resemble the water used at the Weyerhaueser intake.

The EFDC modeling predicts that the increase in surface water chlorides at the Houlihan Bridge is about 50% (See **Table 7.7.9.1-1**). The principal concern for the Weyerhaeuser surface water intake is a reduction in the lifespan of the water distribution system. Their water distribution system for water purchased from the City is entirely separated from their surface water withdrawal system.

Table 7.7.9.1-1: Predicted Daily Average Chlorides (mg/l) at Houlihan Bridge

	Surface Layer		Bottom Layer	
	Existing 42 ft Depth	47 ft Project Depth	Existing 42 ft Depth	47 ft Project Depth
10 Percentile	290	570	1,440	7,200
50 Percentile	1,760	2,460	5,840	11,760
90 Percentile	2,970	3,890	10,170	15,460

Weyerhaeuser uses the water they purchase from the City of Savannah for boiler water, industrial process water, and cooling water. Boiler water must be demineralized before use. The demineralizer system is currently running at 50 to 60% of capacity. Additional chlorides, as well as any lime or phosphate introduced to reduce corrosion, will increase the load on the demineralizer and slow the output. Mill cooling water is cooled in cooling towers and returned to the mill circuit; however, the amount of recycling is governed by the impurity concentration. An increase in chloride will result in fewer cycles of usage for the cooling water, thus requiring an increase in the supply from the City I&D plant. The major process use of City water is in the bleaching process. Chlorides interfere with the bleaching, so increases in chloride levels could create a serious problem. The plant has an on-site storage tank for about 11 hours usage of demineralized water. If the water supply quality does not meet their requirements for more than 11 hours, a plant shutdown would likely be needed. The hourly variation in chlorides, shown in Section 7.7.9.3, indicates that the on-site storage of demineralized water may be sufficient if they are able to refill the storage tank between chloride spikes.

7.7.9.2 International Paper

Unlike Weyerhaeuser, International Paper (IP) has no on-site surface water intake. They do, however, have on-site wells that produce about 15 MGD. In the next 5 to 10 years, IP expects to be required by the GA DNR-EPD to replace their groundwater supplies with surface water. The groundwater has a high level of silica, which provides a natural corrosion protection in their process water distribution system, but which also must be removed by a demineralizer prior to being used in the boilers. Since the surface water does not have high silica content, this would enable IP to have sufficient capacity to demineralize higher levels of chloride from the City water. However, IP is concerned about the integrity of their process water distribution system. Increased chlorides may result in an increased corrosion rate which could threaten the distribution system. Replacement of the piping system would likely be very costly due to the size of the system and the numerous facilities built above the pipelines since they were originally installed. Therefore, the potential impact at IP is limited to a reduced lifespan for the water distribution system.

7.7.9.3 Other Industrial Use

IP and Weyerhaeuser are the two largest water users that the City supplies, but there are other industrial users fed by the Abercorn Creek Plant, including firms located in downtown Savannah, Garden City, Port Wentworth, Pooler, and Effingham County. No data is available on these other users, but their chloride concerns can be expected to be similar, on a smaller scale.

7.7.10 Mitigation Options

Extensive updated modeling efforts to predict chloride increases by frequency, concentration, and duration along with multi-variable bench-scale laboratory analysis on-site at the City's treatment plant have lead to the conclusion that the solution to mitigate for the impacts due to chloride increases with harbor deepening is to remove the influence of the increased seawater intrusion. That can be accomplished two ways:

- <u>Supplemental Intake and Pipeline.</u> Construct a supplemental intake and pipeline that can draw water from further upstream on the Savannah River, above the area impacted by salinity and chloride intrusion predicted with harbor deepening. Constructing a new intake pipeline would take fresh water from the Savannah River more than 10 miles further upstream from the current location on Abercorn Creek where chloride levels remain relatively constant at around 8 mg/l. The proposed pipeline route is 8.7 miles long through Chatham and Effingham County to the intake site located near Plant McIntosh.

- <u>Storage Impoundment.</u> Construct an impoundment that will store acceptable raw water for use during chloride spike events predicted to occur during very low river flow and high tides.

The storage alternative was determined to be the more cost-effective option that mitigates against both increasing lead corrosion as well as increasing DBP formation predicted with harbor deepening and is discussed further in Section 7.7.11.

All mitigation options, in addition to the alternatives discussed above, that were identified and evaluated during the study process are shown below:

- Modified Water Treatment Process. Conceptual cost estimates for modified treatment process options were developed by Freedman and Associates. Their report titled *Assessment of Chloride Impact from Savannah Harbor Deepening* is included in the Engineering Appendix Supplemental Materials. Costs outlined in the Freedman and Associates report are based on treatment of 60 MGD and are preliminary costs typical for these types of systems. Potential treatment modifications include a lime storage and feed system ($2.8 million), a granular activated carbon system ($47.2 million), and an ozonation system ($35.4 million), all at the city I&D plant. Freedman and Associates also developed cost options for reverse osmosis systems and demineralizer systems on-site at both Weyerhaeuser and International Paper, which vary by plant and expected flow rates and range from $4.4 - $30.9 million. Freedman and Associates analysis was based on water quality modeling and preliminary lab analysis. While results of their study are useful, they are not definitive and as a result they recommended further analysis be conducted regarding the corrosion of copper and lead resembling a larger on-site pilot scale study.

  A primary objective of the subsequent CDM testing was to identify a chemical process that would reduce the increased seawater corrosivity to existing levels that would work well with the City of Savannah's existing treatment plant, their water supply demands and the site specific water chemistry of the source water in Abercorn Creek. CDM explored the treatment options outlined by Freedman and Associates as well as additional options. The results of the bench-scale study indicate that neither the existing corrosion inhibitor nor pH adjustment will consistently control lead corrosion. While there are chemical treatments that could potentially address the issue of DBP formation, they would not fully mitigate for increasing chlorides as lead corrosion would remain a problem. The only treatment solution that would address both lead and DBP formation issues is advanced treatment. Under advanced treatment the conventional treatment process is amended to incorporate a range of sophisticated membrane technologies such as nanofiltration or reverse osmosis. Both of these options would remove the precursors relating to corrosion and DBP formation prior to treatment. However, either option would be very costly both in capital costs and operation and maintenance costs. Capital costs have been estimated to exceed $60 million.

- Increasing Freshwater Supply Through Bear Creek. The Bear Creek diversion structure allows a portion of freshwater from the Savannah River to be diverted down Bear Creek to Abercorn Creek providing improved water quality at the raw water intake. Bear Creek flows through a heavily wooded area which is part of the Savannah National Wildlife Refuge. Flow in Bear Creek is currently impeded by numerous fallen trees. Clearing and snagging would remove these trees and improve freshwater flow from the river. This option was shown to be effective when the Corps constructed the diversion structure in 2002 as part of the Lower Savannah River Basin Environmental Restoration Project. Clearing and snagging more of Bear Creek (than was included in the authorized Environmental Restoration Project) would be required for this measure to work effectively as a mitigation feature. Since the creek flows through the Savannah National Wildlife Refuge, approval from US Fish and Wildlife Service would be required, which would be contrary to their management plan of the refuge. This option would also require a high level of periodic maintenance (removal of fallen trees) to perform as intended.

- Desalinization. A conceptual cost estimate for desalinization treatment at the location of the four largest industrial users was developed using the methodology published by the US Department of the Interior. That estimated cost was $135 million and was determined to be cost prohibitive.

- Groundwater Supplementation. Increasing the amount of groundwater withdrawal during times of potential chloride intrusion on Abercorn Creek. This would have to be offset by greater use of surface water during higher flow periods. The Georgia Department of Natural Resources Environmental Protection Division limits the amounts of groundwater withdrawn by municipalities, complicating the use of an alternate source of water during times of drought with accompanying high chloride levels. Additionally, the City's current water supply distribution system does not have the capacity to move large quantities of water from the wells currently in place throughout its network. Construction would likely require locating and installing new wells and amending the distribution network.

- Freshwater Flow Supplementation. Instituting a variable drought plan release from Thurmond Dam. However, this produces problems for water managers and water users such as the City of Augusta, Savannah River Site, and Plant Vogtle.

- A combination of increased groundwater withdrawal and greater releases from Thurmond Dam, as described above.

- Construction of a Sill at the Mouth of Abercorn Creek to Prevent Chloride Intrusion. Modeling results have shown, and field sampling has confirmed, that the Savannah River is well mixed (not stratified) at the mouth of Abercorn Creek. Therefore a partial sill would not be effective in halting chloride intrusion. A mechanical gated structure that fully blocks inflow from the Savannah River during chloride incursion events would be required. Environmental impacts to wetlands would be excessive, and access to the gate location would be an issue. This option was not recommended for detailed study.

- Replacement of Individual Plumbing Fittings That are the Source of Lead Contamination. It is very difficult to estimate the total number of homes and businesses that would require modification. Costs would vary significantly, with much higher costs to repair slab foundation homes. Real Estate easement administration would also be very costly. Costs are conservatively estimated at $100 million, and this would not address the DBP issue.

- Use of Barges to Store Water for Supplemental Use. It would require approximately 160 water tanker barges to provide the necessary volume. The logistics of storing and maneuvering these barges on Abercorn Creek make this alternative unfeasible.

7.7.11 Proposed Mitigation – Raw Water Storage

The proposed mitigation alternative is a raw water storage impoundment which provides the means to store water for use by the City for drinking water supply during times of high chloride events. The proposed mitigation was selected as it is the least cost, environmentally acceptable plan that meets the project needs.

Design considerations for the raw water storage impoundment are:

- The GA DNR-EPD, in the Section 401 Water Quality Certification for Savannah Harbor Expansion, stated that any mitigation remedy selected shall be constructed in conjunction with the channel deepening. They also stated that mitigation shall be based on the maximum plant capacity of 62.5 mgd.

- A firm raw water pumping capacity of 75 mgd at the existing Abercorn Creek intake. Design constraints based on firm pumping capacity as opposed to the actual pumping capacity of 100 mgd is standard engineering practice and is required by the GA DNR-EPDs *Minimum Standards for Public Water Systems* published in May of 2000.

- 20% of the storage volume will be unusable due to access limitations and sedimentation.

- A performance goal of limiting the chlorides at the plant to 40 mg/l during the model predicted worst-case scenario and to 25 mg/l 99 percent of the time. As shown in the CDM lab analysis, 40 mg/l is the chloride concentration at which THMs in the distribution system can be expected to reach the MCL and potentially trigger a regulatory violation. Also shown in the analysis is that chloride concentrations as low as 25 mg/l have an adverse impact on lead corrosion.

A series of statistical analyses were used to determine the appropriate size for a raw water impoundment for use at the City's drinking water supply plant for all project depths under the design considerations noted previously. **Figure 7.7.11-1** shows the relationship between usable impoundment volume and the chloride concentration entering the I&D plant resulting from the statistical analyses. The dashed red and blue lines indicate the model-predicted maximum and 99th-percentile hourly chloride concentration under existing conditions, respectively. Each data point represents the concentration achieved when the impoundment is operated at the optimal pumping cutoff concentration, which varies by the size of the impoundment. Red circles indicate the minimum impoundment volumes needed to limit the maximum concentration to 40 mg/l and the 99th-percentile concentration to 25 mg/l, respectively. The area between the red circles represents the range of volumes recommended for consideration. Results of the analysis recommending an impoundment volume for each project depth are shown in **Table 7.7.11-1**.

Figure 7.7.11-1: Relationship Between Usable Impoundment Volume And Chloride Concentration Entering The I&D Plant

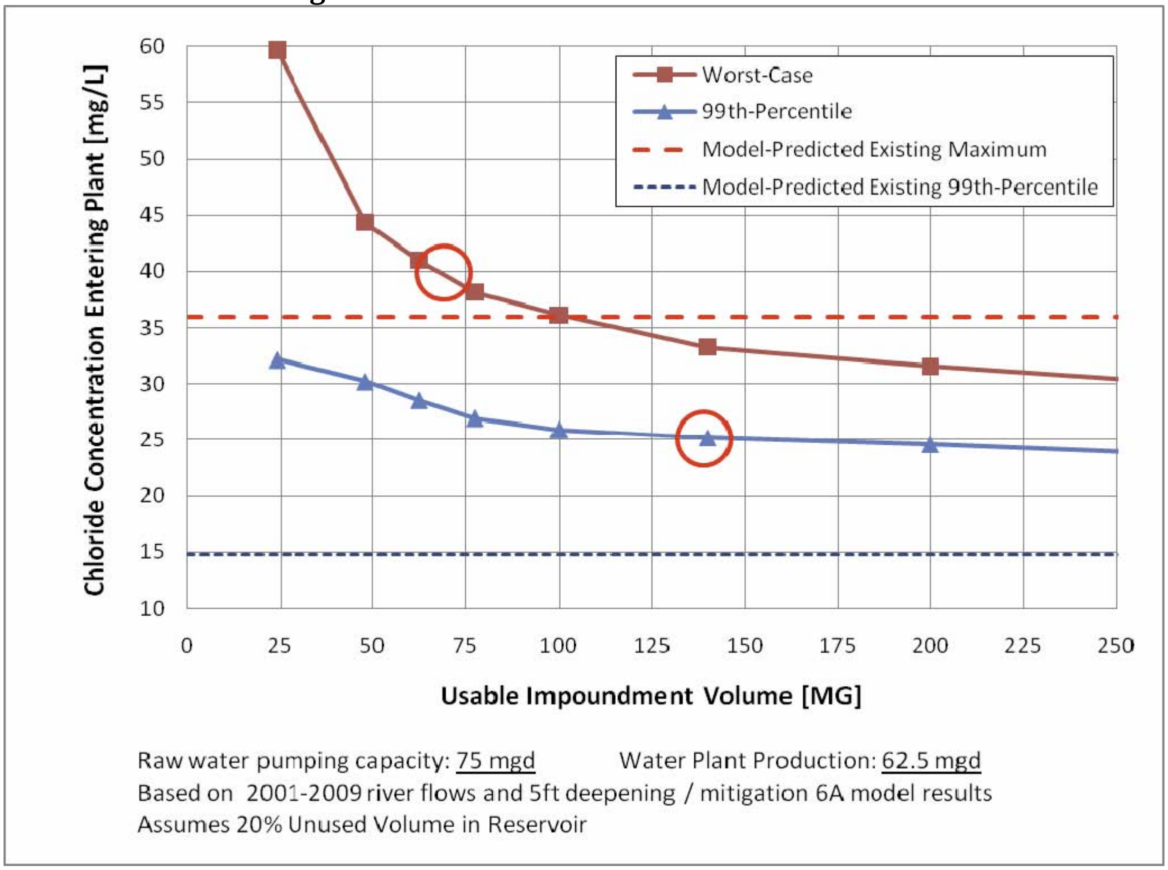


Table 7.7.11-1: Proposed Raw Water Storage Impoundment Volumes Required for Each Project Depth Alternative

Project Depth	Recommended Usable Impoundment Volume, MG	Required Total Impoundment Volume, MG
44ft	22.5	28.0
45ft	30.0	38.0
46ft	46.5	58.0
47ft	77.5	97.0
48ft	120.0	150.0

A conceptual site layout for the 47 ft depth alternative is shown in **Figure 7.7.11-1**. The preliminary layout and conceptual site plan includes the following (Details of the design can be found in the CDM report titled *City of Savannah Seawater Effects Study* included in the Supplemental Materials.):

- Dual 36" influent and effluent pipes to connect the impoundment to the existing raw water pipeline (to provide redundancy at the tie in points and allow for maintenance to occur during times when the impoundment is in use).,

- A pump station containing four vertical turbine pumps to convey flow out of the impoundment and back into the raw water lines,

- A mechanical mixer in the center of the impoundment to help maintain oxygen levels throughout the impoundment's depth reducing the likelihood of algae growth and the associated taste and odor issues,

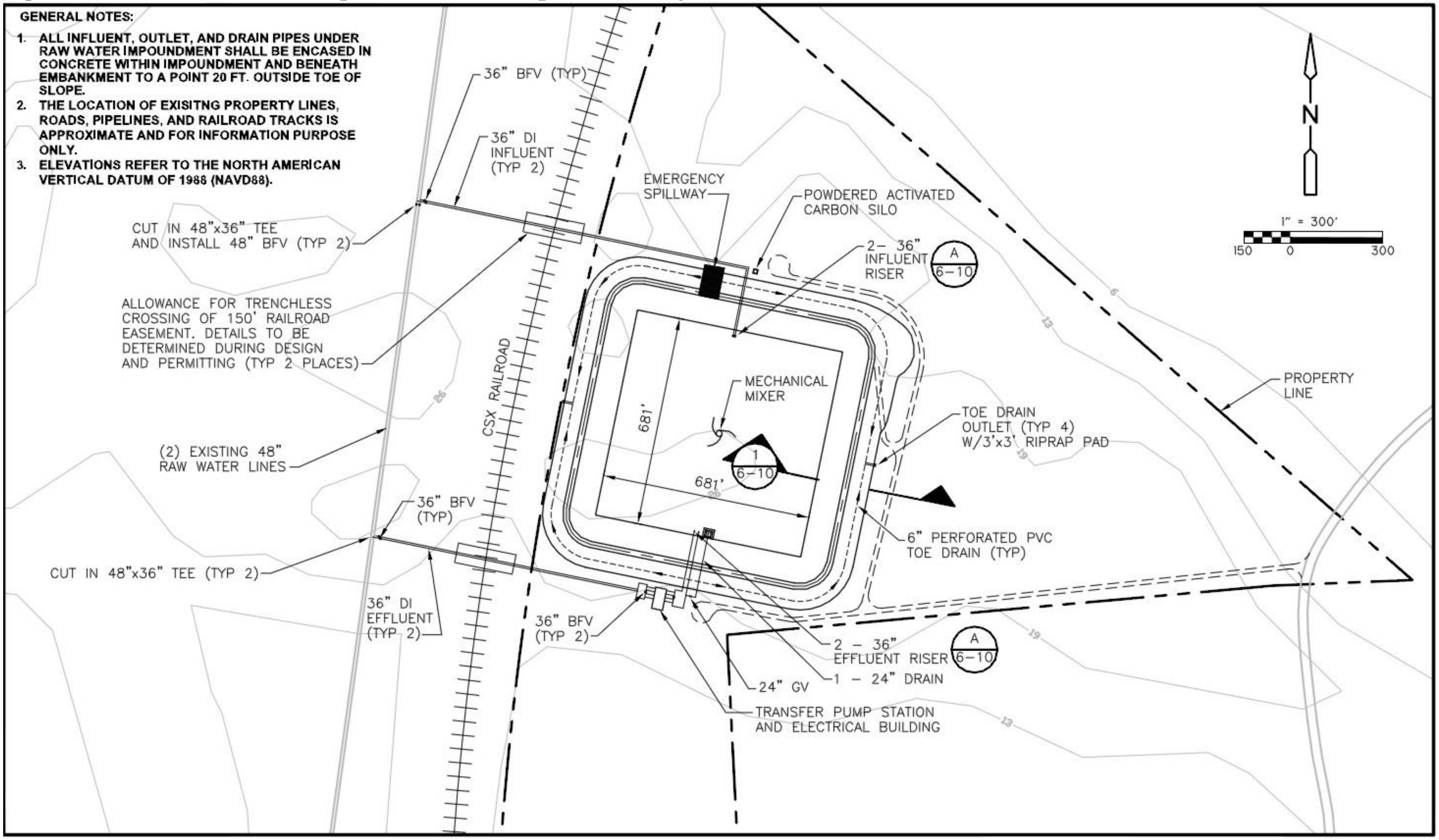
- A powdered activated carbon silo and feed system to be used on an intermittent basis during severe taste and odor episodes,

- A 24" drain pipe to be used to empty the impoundment during periodic maintenance cleaning;

- One or more in-situ chloride meters to be installed in Abercorn Creek to provide data for operational decision making.

Costs associated with this feature are detailed in Section 13.0

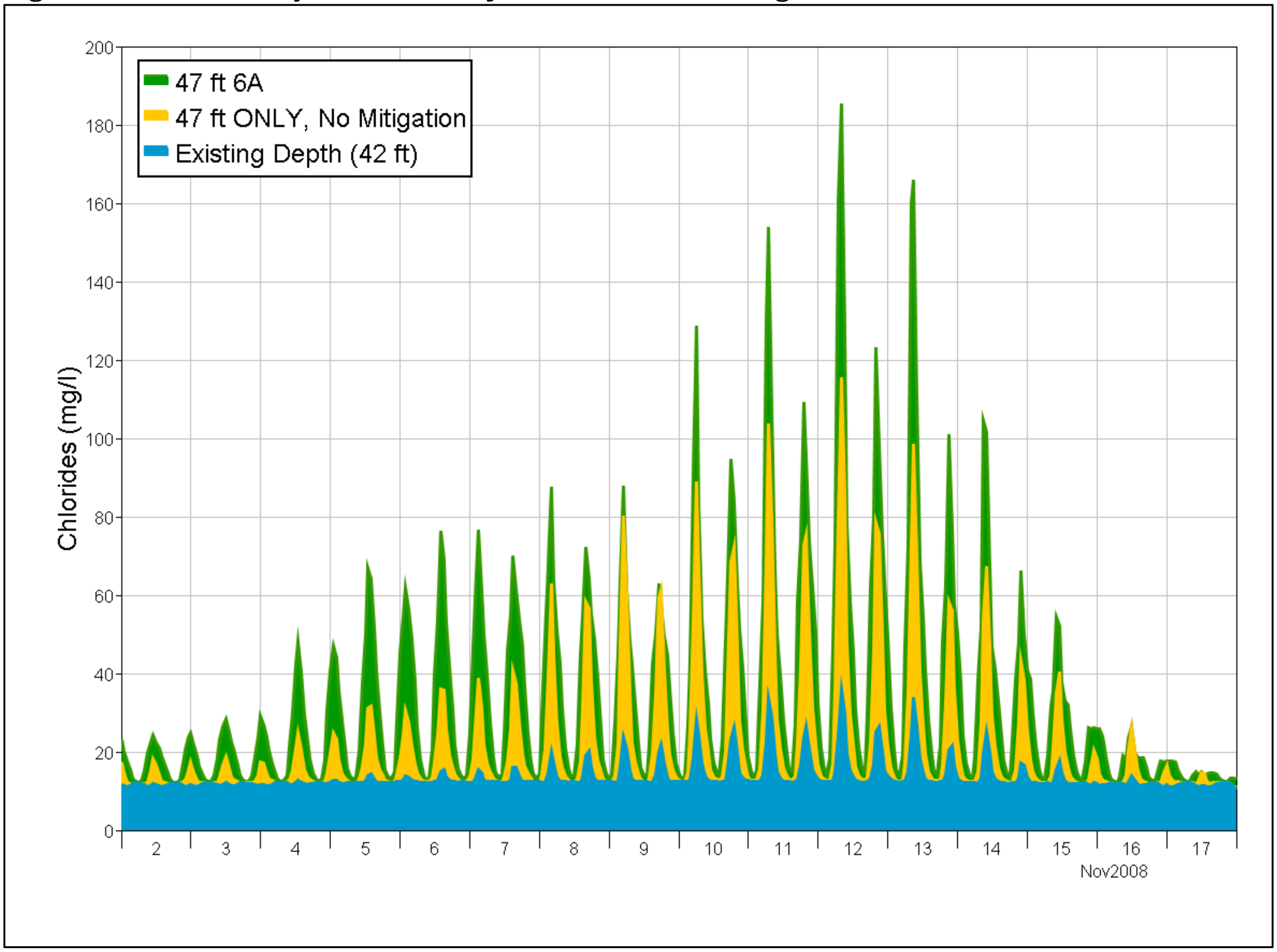
Figure 7.7.11-1: Raw Water Impoundment Conceptual Site Layout



Conceptual Site Layout from *City of Savannah Seawater Effects Study* (CDM, 2011)

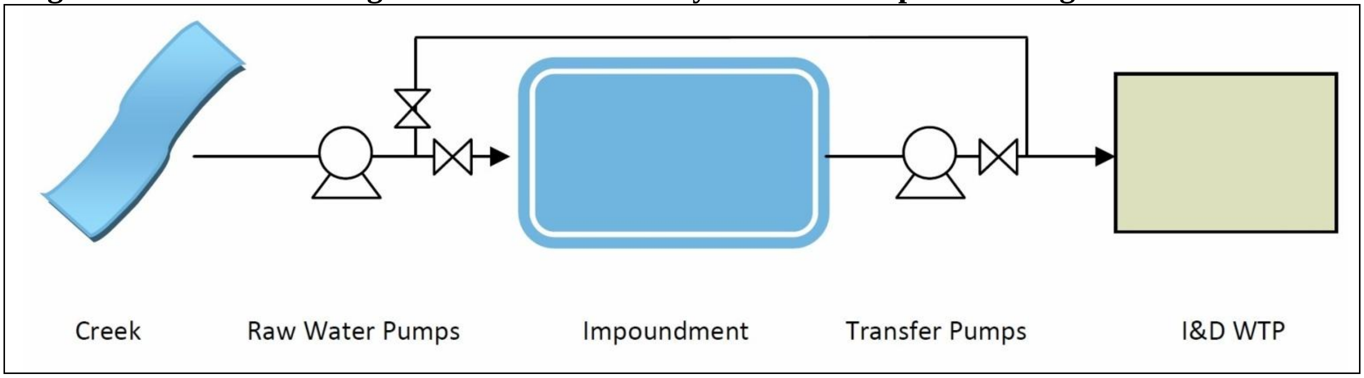
Model predictions indicate that high chlorides will occur diurnally, coincident with high tide. As shown in **Figure 7.7.11-2**, a plot of hourly projected chloride values during November 2008, chlorides return to levels of 12 to 15 mg/l twice each day, even on the peak day of predicted chlorides at 186 mg/l. An early warning system on Abercorn Creek will be required to provide data to the operator in a timely manner to know when valve and pump changes are needed.

Figure 7.7.11-2: Hourly Chloride Projections Demonstrating Diurnal Variation



Operation of the feature would require that water from the existing Abercorn Creek intake be utilized for treatment and pumped to the storage impoundment during occurrences of low chlorides. During occurrences of high chlorides, water from the impoundment will be pumped to the City's existing water treatment plant via the proposed pump station. During high chloride events at the intake, the intake pumps will be stopped and the plant will draw water from the storage impoundment thereby avoiding the high chlorides occurring in Abercorn Creek. When chloride concentrations on Abercorn Creek return to acceptable levels during low tide, the storage impoundment can be refilled and made ready for use during the next high tide. **Figure 7.7.11-3** illustrates the raw water piping and storage impoundment network.

Figure 7.7.11-3: Flow Diagram of Water Intake System with Proposed Storage



7.7.12 Conclusions

Analyses show that for normal to high river flows, there is expected to be very little change in chlorides from the existing condition for any of the deepening alternatives. However, for flows observed at the USGS Clyo, GA streamgage (RM 61) less than 6,000 cfs, the increases are statistically significant.

Mitigation to provide storage of raw water for use during high chloride events through a single impoundment optimized at 77.5 million usable gallons and the necessary supporting infrastructure is recommended. At the direction of the District, Tetra Tech has independently estimated the storage needs and has confirmed the size requirements. It is also recommended that the chloride mitigation feature be operational prior to completion of the channel deepening and the flow re-routing mitigation features associated with SHEP. Costs associated with construction of this feature are detailed in Section 13.0

Based upon flow conditions experienced in November 2008, the worst case event would require use of storage for 14 consecutive days. The total proposed storage volume represents only about 32 hours usage at plant capacity but the chloride content of Abercorn Creek will rise and fall with each tidal cycle; therefore, the impoundment will be partially refilled during low tide twice each day.

The October 1999 Chief's Report states that "If … (a) chloride mitigation feature involving modification of the City of Savannah water supply system remains a part of the project, the costs of operation, maintenance, repair, replacement, and rehabilitation of the modified City of Savannah water system will remain a City of Savannah responsibility..." Based upon this language, it is anticipated that all O&M activities would be performed by the City of Savannah.

It is anticipated this mitigation proposal would be acceptable to the City of Savannah and the project sponsor.

8.0 MITIGATION DESIGN

8.1 MITIGATION FEATURES

This section includes the design and quantity determination of each flow-altering mitigation feature necessary for estimating construction costs. Costs for each feature are detailed in Section 13.0.

Sea level change guidance described in EC 1165-2-211 was considered in the design of the mitigation features. The structural design, resiliency and integrity of each mitigation feature was evaluated under the various sea level change estimates outlined in Section 7.5.2.2 of this Appendix. The resiliency and integrity of the design were considered to ensure the structures would not be compromised or produce unintended consequences under those various sea level projections. In addition, structural features would not pose a risk to public safety under any of the sea level change projections nor would their performance be adversely affected.

8.1.1 Flow-Altering Mitigation Features (Plan 6a & 6b)

From the hydrodynamic modeling, Mitigation Plan 6a and 6b were developed. A summary of these flow-altering features along with the material quantities are summarized in **Table 8.1.1-1**. Design details of each feature are outlined following the table.

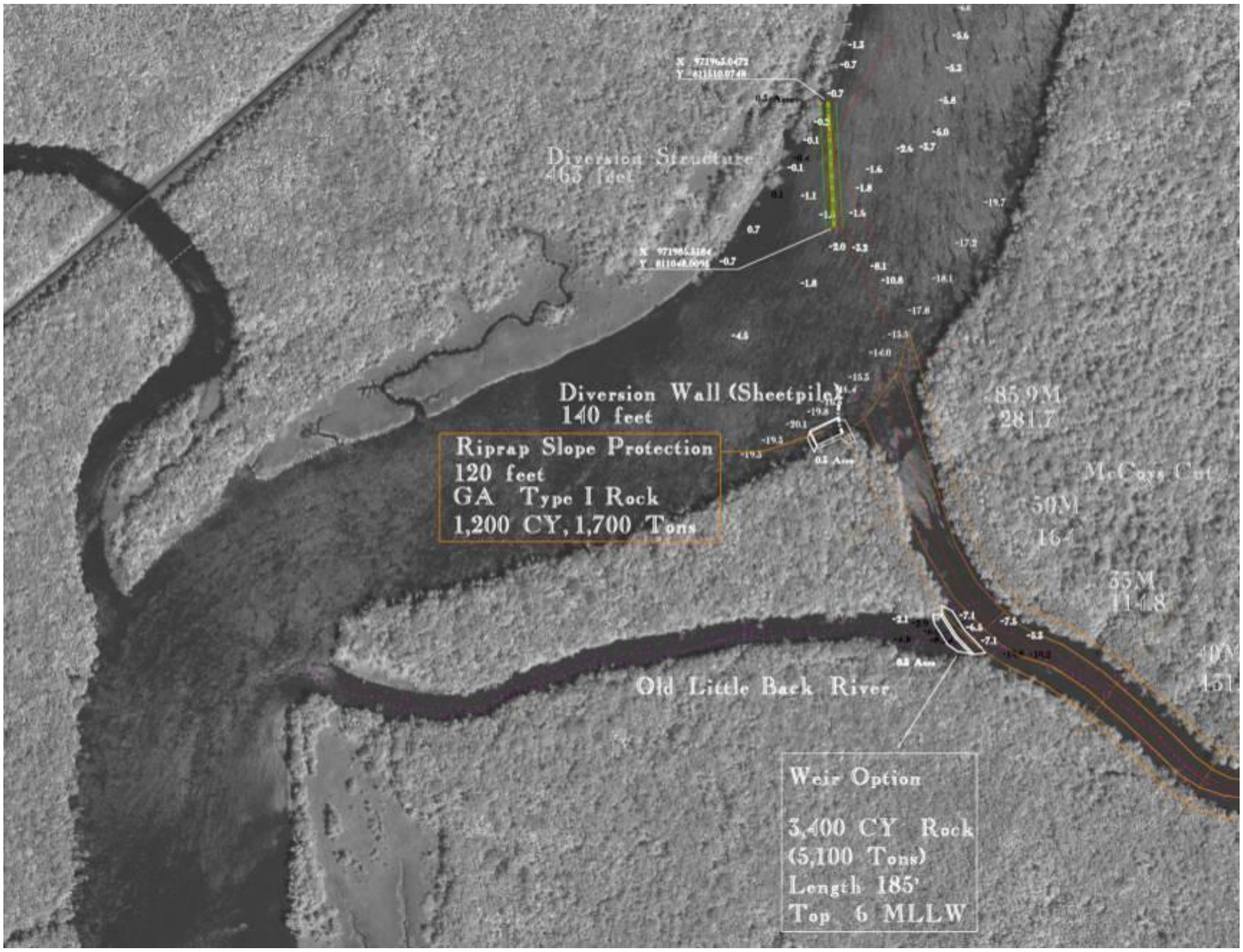
Table 8.1.1-1: Material Quantities for Flow-altering Mitigation Features (Plan 6a & 6b)

	Feature	Material Quantity	Plan
a.	Diversion Structure at McCoy Cut		6a & 6b
	Structure on northern river bank (465 ft long)	5,400 tons GA Type I Riprap	
	Structure on southern river bank (140 ft long)	1,700 tons GA Type I Riprap 12,100 sq ft Z-27 steel	
b.	Closure of Lower (western) Arm at McCoy Cut	5,100 tons GA Type I Riprap	6a & 6b
c.	Deepening McCoy Cut to Middle River/Little Back River Confluence	60,000 cy excavated	6a only
d.	Deepening upper Middle River	181,200 cy excavated	6a only
e.	Deepening upper Little Back River	74,000 cy excavated	6a only
f.	Closure of Rifle Cut	2,500 tons GA Type I Riprap 3,300 cy fill	6a & 6b
g.	Sediment Basin Broad Berm (Weir)	97,000 tons GA Type I Riprap	6a & 6b
h.	Sediment Basin Broad Berm (Fill)	1.2 million cy sandy material from 12A	6a & 6b
i.	Removal of Tidegate Abutments & Piers		6a & 6b
	Northern abutment	240,000 cy excavated over 2 acres	
	Southern abutment	785,000 cy excavated over 15.8 acres	

a) McCoy Cut Diversion Structure– The McCoy Cut diversion structure is shown in **Figure** 8.1.1-1. The design for the diversion structures actually specifies two structures, a diversion structure constructed of rock and a diversion wall constructed of sheet pile and rock. Quantity estimates for the diversion structure upstream of McCoy's Cut are 5,400 tons GA Type I Riprap, top elevation 0 MLLW. The diversion wall at the mouth of McCoy Cut specifies 140 ft length of sheet pile (9,800 sq ft steel Z-27 +2,300 sq ft Z-27 Buttress at 9 ft intervals), top elevation 11 ft MLLW, and 1,700 tons GA Type I Riprap.

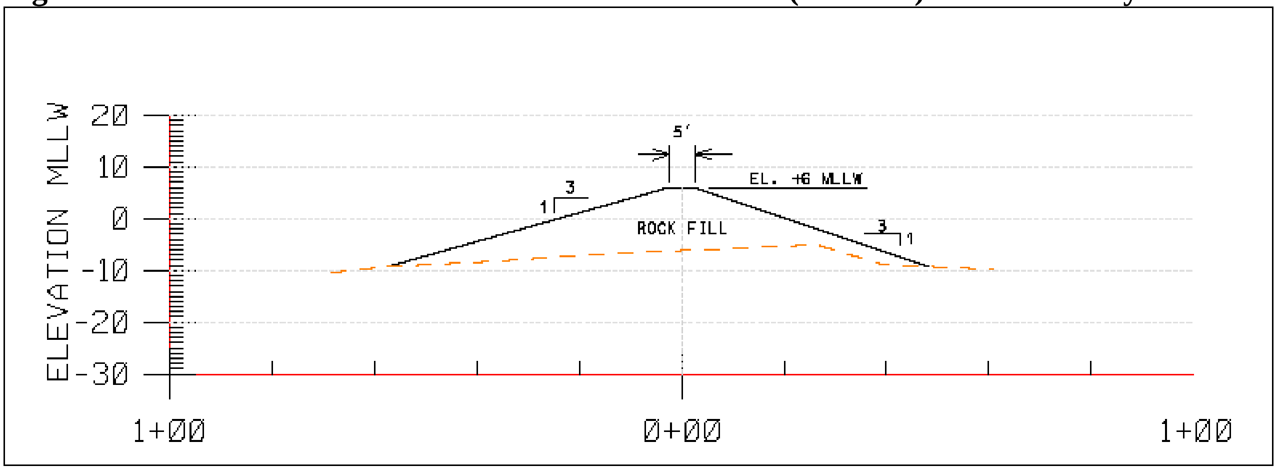
b) Closure at Lower (western) Arm at McCoy Cut – The weir structure at the closure on Lower (western) Arm at McCoy Cut (also shown in **Figure 8.1.1-1**) requires 5,100 tons of GA Type I Riprap. Top elevation is 6 ft MLLW. The center cross section of the closure structure is shown in **Figure 8.1.1-2**.

Figure 8.1.1-1: McCoy Cut Diversion Structure and Closure at Lower (Western) Arm at McCoy Cut



**The Lower (western) Arm at McCoy Cut is also known as Old Little Back River.*

Figure 8.1.1-2: Center Cross Section at Closure at Lower (Western) Arm at McCoy Cut



c) Channel Deepening on McCoy Cut – Drawings for the plan to deepen McCoy Cut and the river portion above the Middle/Little Back River confluence are shown in **Figure 8.1.1-3**. The total length of this portion is 1600 meters (5250 ft). Deepening to -4 m NGVD (-10.0 ft MLLW), as specified by the hydrodynamic model, will require 60,000 cubic yards of excavation. Two cross sections, where the template was modified to avoid taking of real estate, are shown in **Figure 8.1.1-4**.

Figure 8.1.1-3: Channel Deepening on McCoy Cut

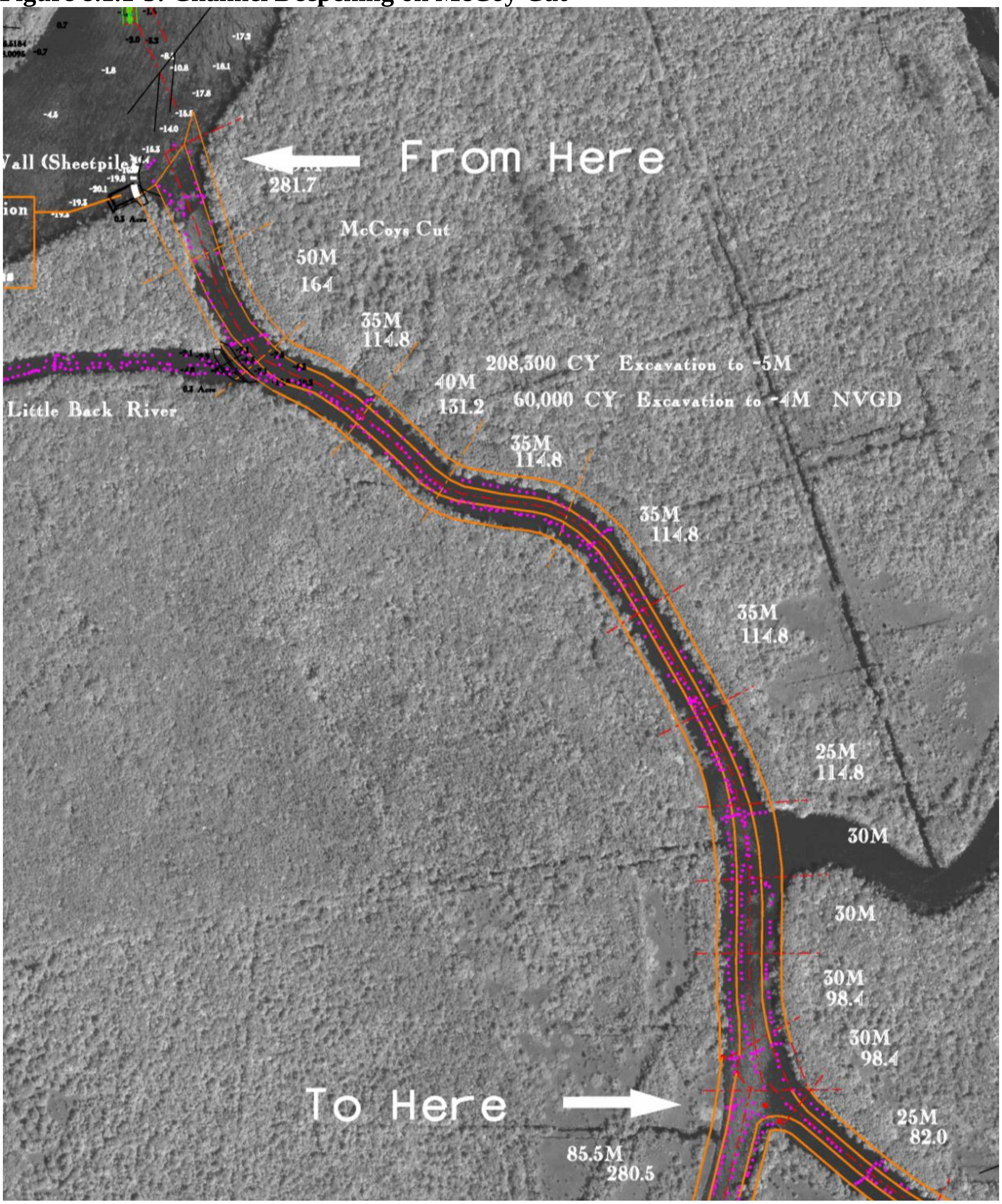
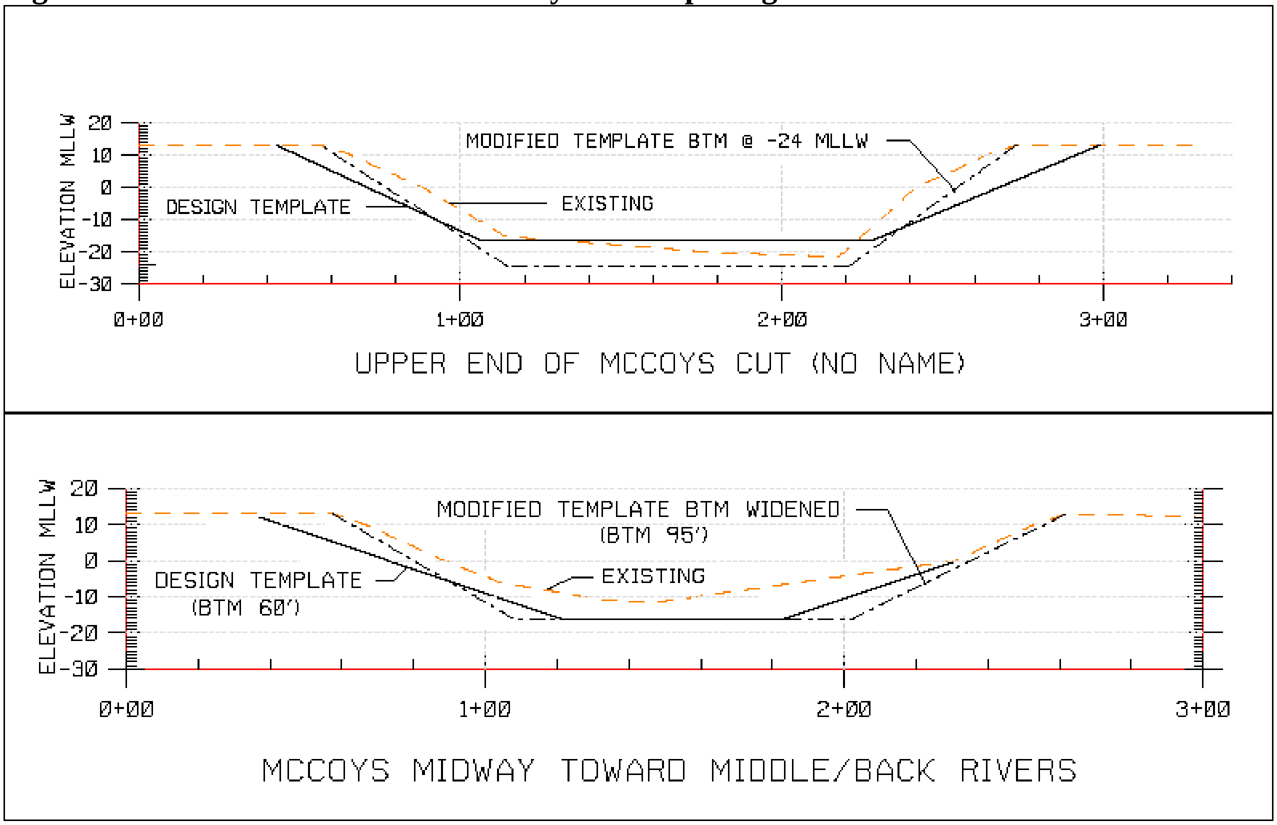


Figure 8.1.1-4: Cross Sections for McCoy Cut Deepening



d) Channel Deepening Upper Middle River – The channel deepening on Middle River is for a distance of 1700 meters (5580 ft) and is deepened to a depth of -3 m NGVD (-6.8 ft MLLW), which requires 181,200 cubic yards of excavation. See **Figure 8.1.1-5**. Two cross sections are shown in **Figure 8.1.1-6**, one at the confluence and one on Middle River midway of the planned excavation.

Figure 8.1.1-5: Channel Deepening on Middle River

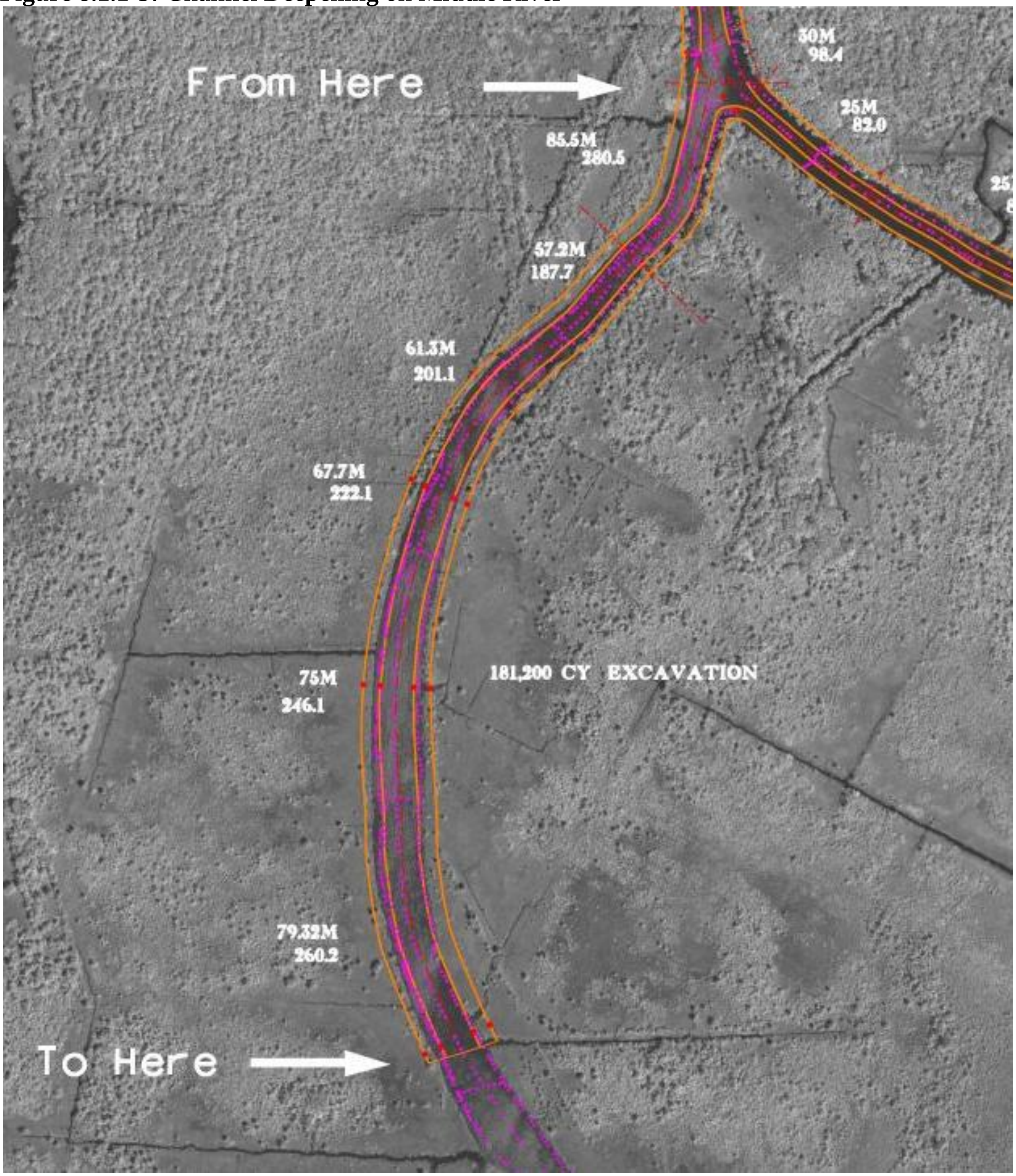
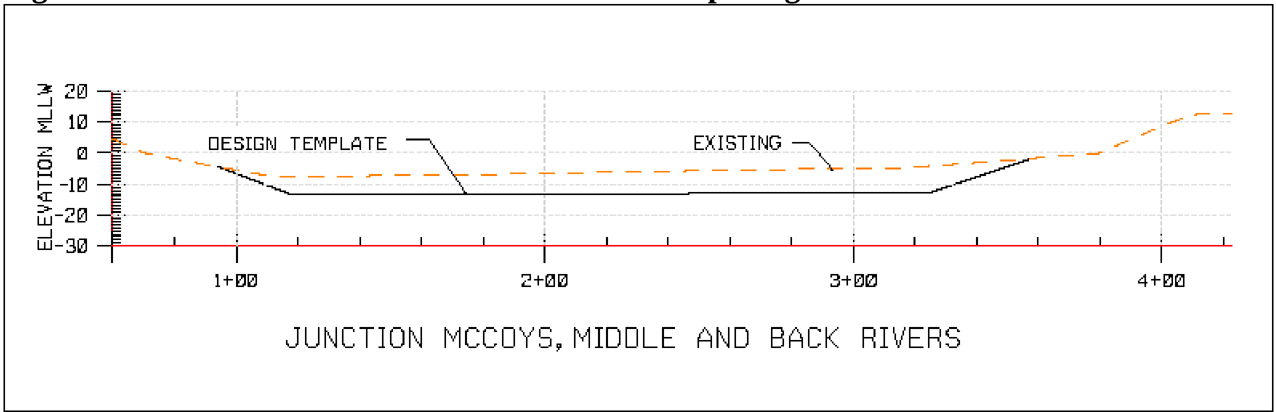
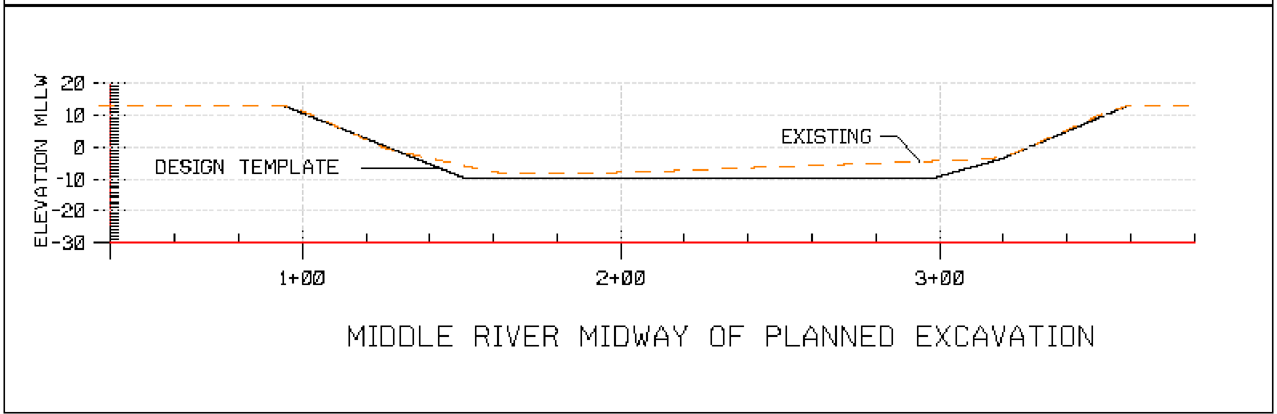


Figure 8.1.1-6: Cross Sections for Middle River Deepening





e) Channel Deepening Upper Little Back River – The channel deepening on Little Back River is for a distance of 1700 meters (5580 ft) and is deepened to a depth of -3 m NGVD (-6.8 ft MLLW), which requires 74,000 cubic yards of excavation. See **Figure 8.1.1-7**. A cross section is shown in **Figure 8.1.1-8**, on Little Back River midway of the planned excavation.

Figure 8.1.1-7: Channel Deepening on Little Back River

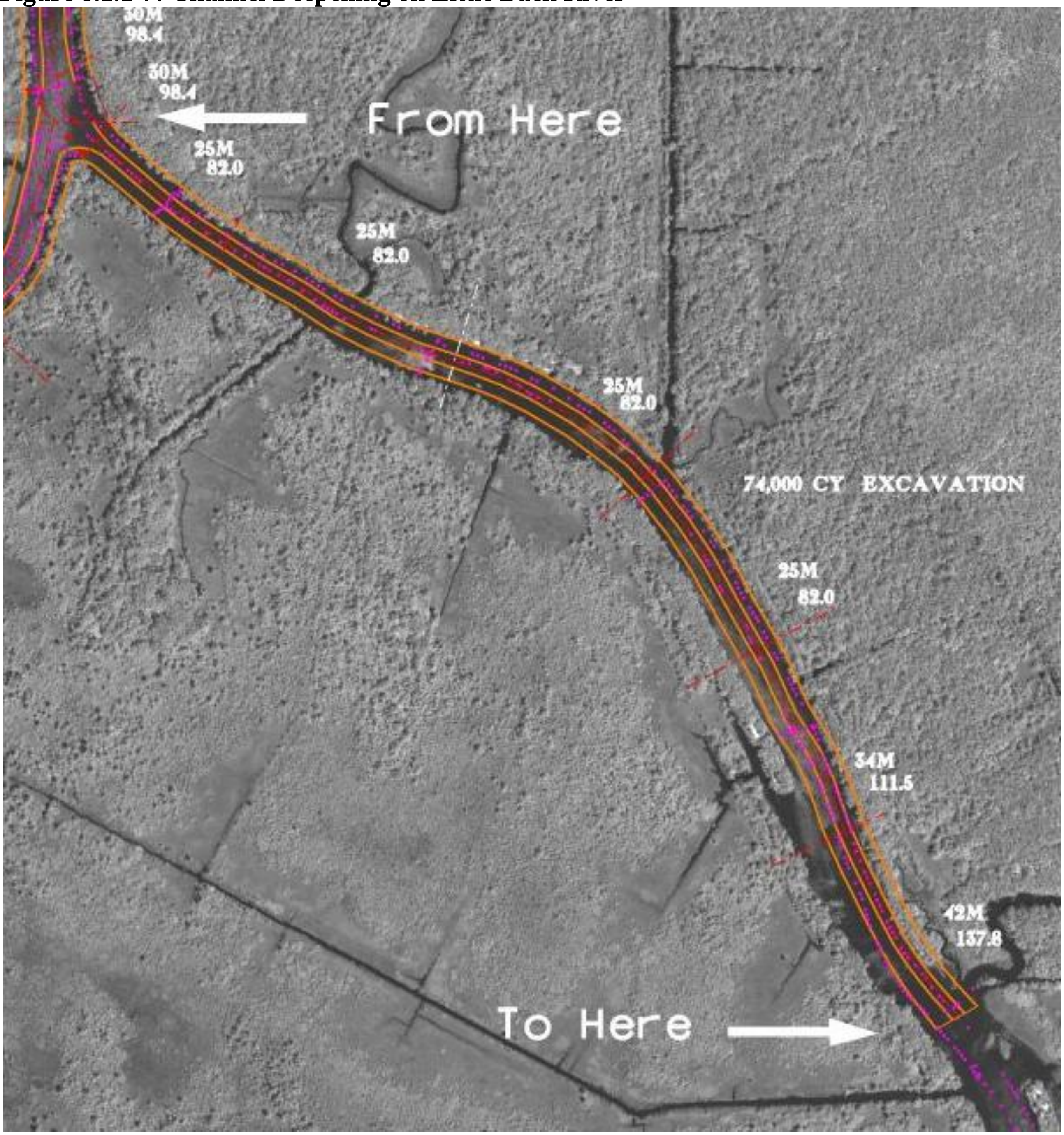
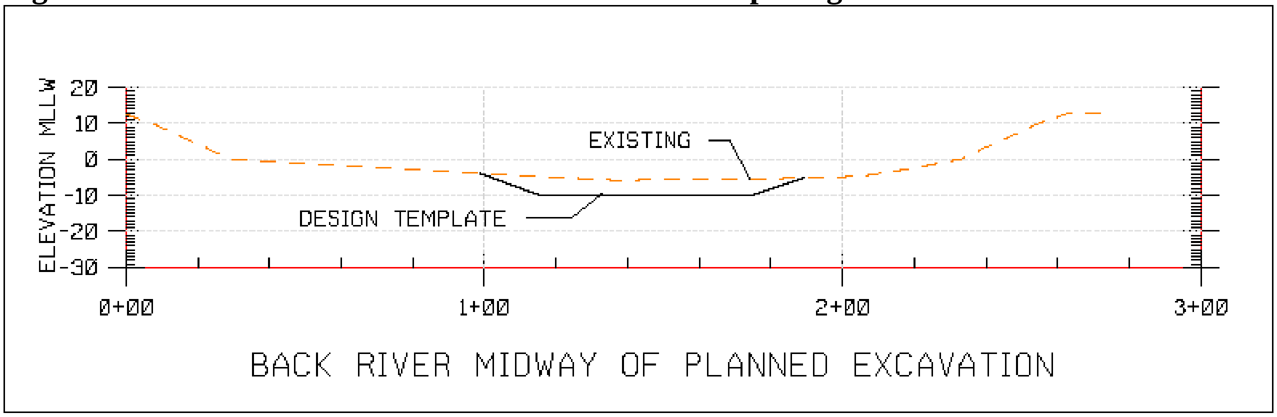


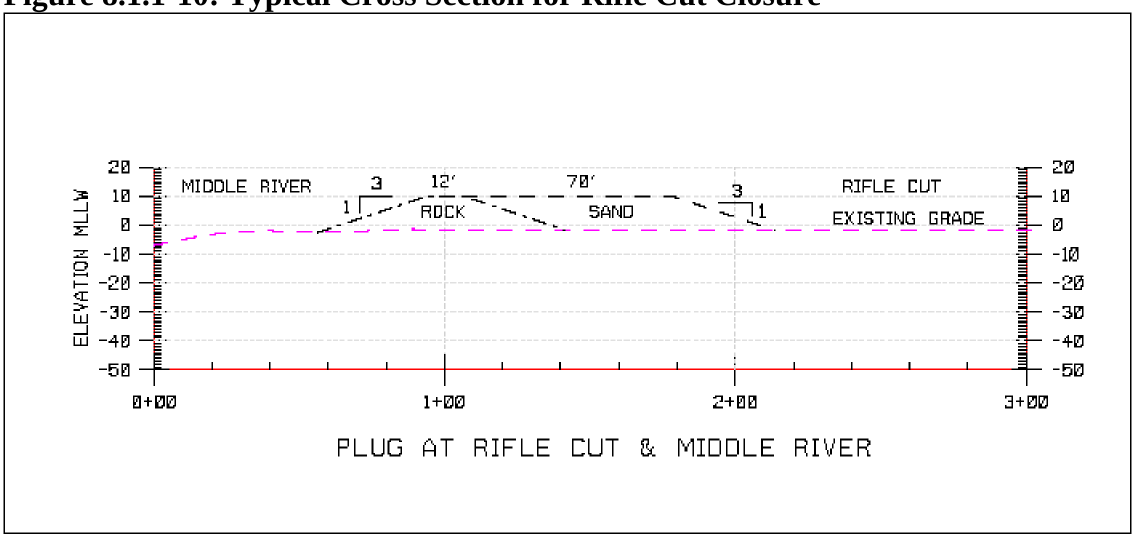
Figure 8.1.1-8: Cross Section for Little Back River Deepening



f) Closure of Rifle Cut – Closing Rifle Cut involves constructing a plug at Middle River. The required easement is 30 ft beyond the work zone. The estimated quantities are 2,500 tons GA Type I Riprap and 3,300 cy of estimated fill. See **Figure 8.1.1-9** for an overview and **Figure 8.1.1-10** for a typical cross section.

Figure 8.1.1-9: Closure at Rifle Cut

Figure 8.1.1-10: Typical Cross Section for Rifle Cut Closure

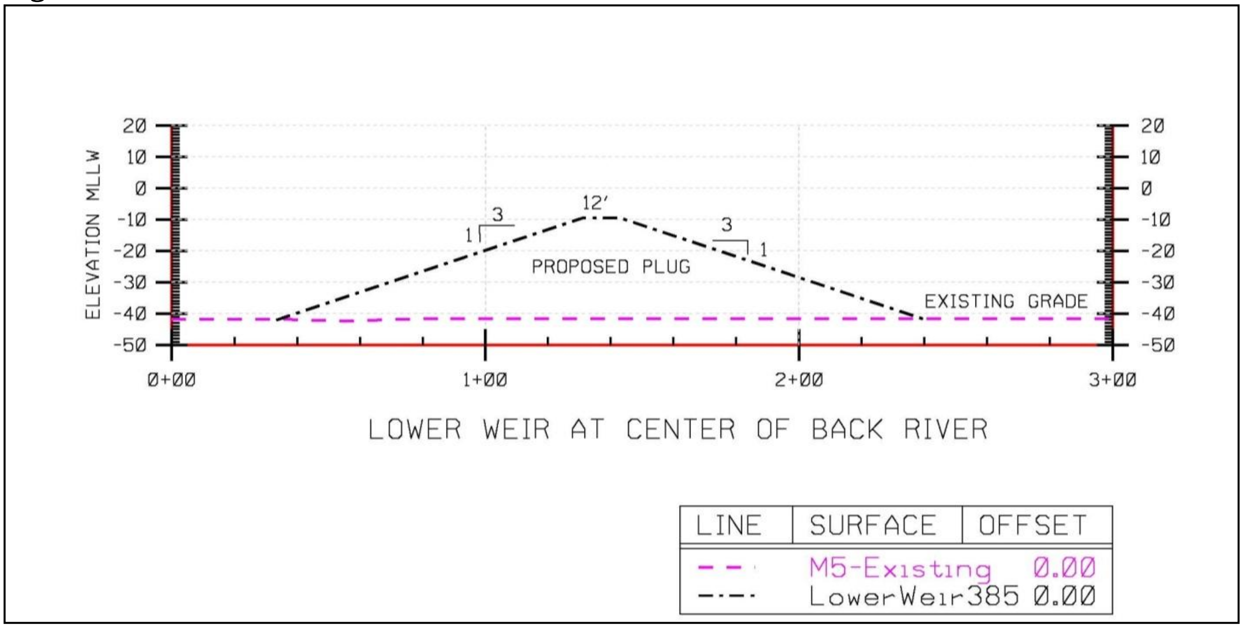


g) Sediment Basin Broad Berm (Weir) – Filling the sediment basin to elevation -3.85 m NGVD (-9.5 ft MLLW) requires construction of a broad berm across the Back River at the Back River/Front River confluence. The broad berm consists of a weir which will require 97,000 tons GA Type I Riprap. See **Figure 8.1.1-11** for an overview plan of the broad berm weir and **Figure 8.1.1-12** for a cross section of the weir.

h) Sediment Basin Broad Berm (Fill) – The broad berm is designed to eventually trap about 9.2 million cubic yards of sediment upstream to the tidegate sill at elevation -3.85 m NGVD (-9.5 ft MLLW). Once all the material is deposited to elevation -3.85 m NGVD, this mitigation feature will be considered fully functional as the design intended. To accelerate the rate of accumulation of sediments, approximately 1.2 million cubic yards of sandy material are to be placed behind the broad berm weir (within the Sediment Basin) after it is completed. This material would likely come from the construction of two other mitigation features (removal of the tidegate abutments and restoration of disposal area 1S) if that material is found to be suitable. The remaining volume needed to complete construction of the broad berm will be harvested from the interior of adjacent existing upland disposal areas.

Figure 8.1.1-11: Sediment Basin Broad Berm Weir

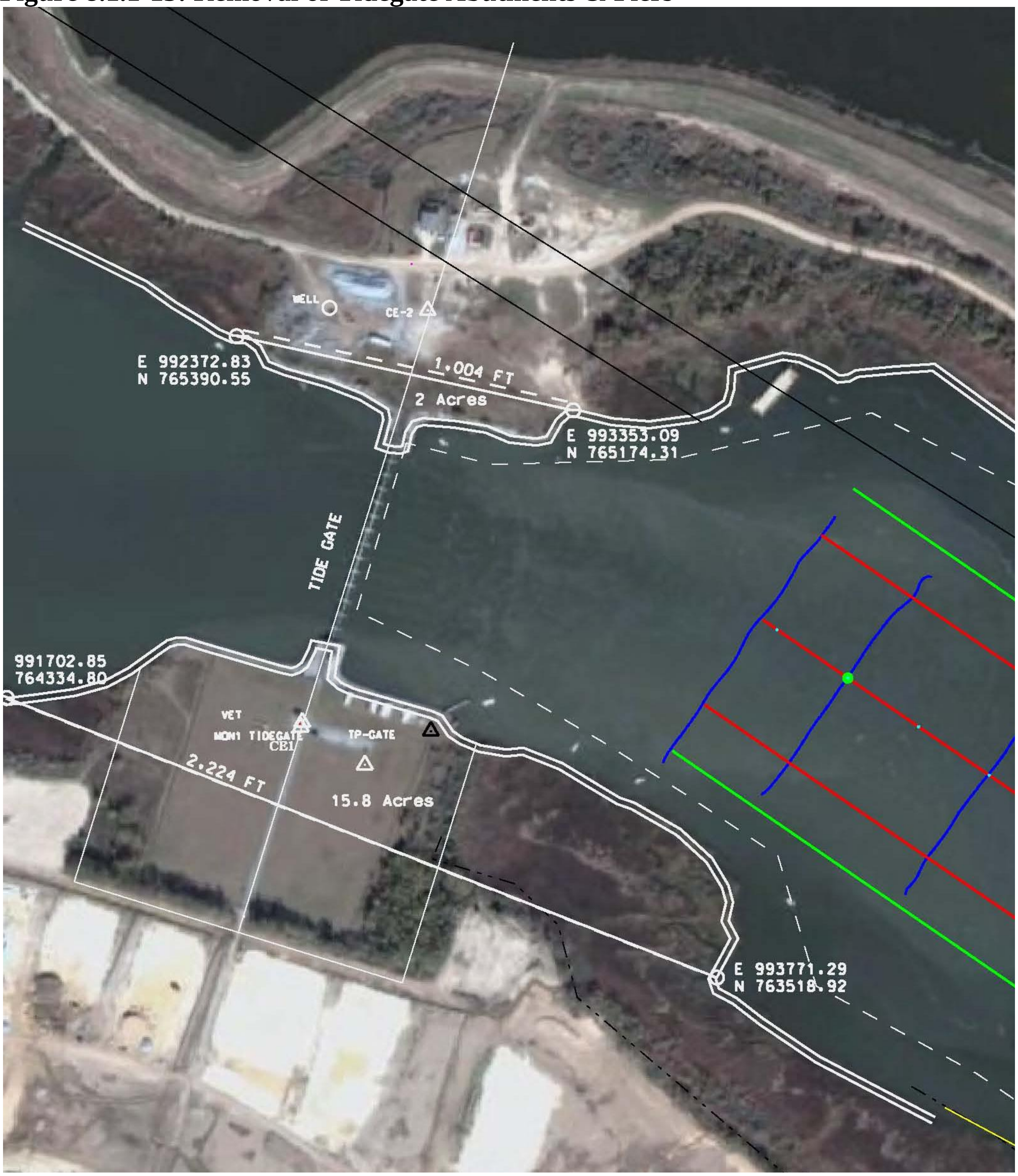
Figure 8.1.1-12: Cross Section of Sediment Basin Broad Berm Weir



Note: Left of the drawing is Back River toward the tidegate, right is toward Savannah River.

i) Removal of Tidegate Abutments & Piers – This feature requires removal of 15 concrete piers and concrete walkway to the sill elevation (-3.85 m NGVD or -9.5 ft MLLW). The sill will be left in place. Removal of the abutments, existing armor stone, existing dock, stacked gates and hardware, pump building and contents, 2 other minor structures, and relocation of utility poles with electric and phone lines is also required. The north abutment excavation requires removal of 240,000 cubic yards over 2 acres. Removal of the south abutment requires excavation of 785,000 cubic yards over 15.8 acres. Real estate will be required for this feature. See **Figure 8.1.1-13** for the taking areas designated by the white line.

Figure 8.1.1-13: Removal of Tidegate Abutments & Piers



8.1.2 Fish Passage at New Savannah Bluff Lock & Dam

The New Savannah Bluff Lock & Dam (NSBL&D) is located on the Savannah River at River Mile 187.4, approximately 13 miles downstream from the City of Augusta, Georgia. See **Figure 8.1.2-1**. This facility is the most downstream lock and dam along the Savannah River and is authorized for the purpose of improving the commercial navigation channel between the upper limits of the Savannah Harbor and the head of navigation at Augusta, Georgia. It was constructed by the Corps and was completed in 1937.

The dam structure is 360 ft long and contains five vertical spillway gates. The lock is located along the right abutment (Georgia Side) and measures approximately 56 ft wide and 360 ft long, with a maximum lift height of approximately 15 ft. The facility provides ponding upstream to support several water intakes and recreation.

While commercial traffic no longer passes through the lock, some small recreation vessel locking is provided. The facility provides little in the way of flow retention or river regulation. The project no longer serves its original authorized purpose (navigation). It is now serving other useful purposes, water supply and recreation. There are concerns that the Lock and Dam remains an obstacle to migration of spawning fish. No Shortnose sturgeon (SNS) passage has been documented at the site since the dam was constructed. Providing fish passage at this location would restore access to historic SNS spawning areas at the Augusta Shoals.

Figure 8.1.2-1: New Savannah Bluff Lock & Dam

A fish passage facility at the NSBL&D is proposed as a mitigation measure for adverse impacts to the Shortnose sturgeon, a federally listed endangered species. Salinity increases due to SHEP lead to a loss in acceptable SNS winter habitat. Incidental improvements in dissolved oxygen lead to increases in less abundant SNS summer habitat. The Corps and the natural resource agencies could not identify any acceptable mitigation in the harbor or estuary for those impacts.

In the November 2010 draft SHEP GRR and EIS, the Corps proposed a Horseshoe Rock Ramp Bypass around the South Carolina side of the NSBL&D to provide fish passage past that structure, which is the first dam fish encounter as they move up the Savannah River to spawn. The Horseshoe Bypass would capture 5 percent of the river flow. The Feasibility Study for the Horseshoe structure along with preliminary Design and Cost Estimates are included in the Engineering Investigations Supplemental Materials. The two reports are titled, *New Savannah Bluff Lock and Dam Project Savannah River Georgia and South Carolina Fish Passage Facility Engineering Report* prepared by Framatome ANP DE&S, Inc. and *Revised 35% Design Construction Cost Estimate New Savannah Bluff Dam Fish Passage Facility* prepared by BTG, Inc.

During review of the Draft SHEP documents, the Corps received comments from natural resource agencies expressing a lack of confidence in the success of the Horseshoe Bypass design. Several stated that they believed the bypass would need to carry more of the river flow to successfully pass SNS. During an April 2011 interagency workshop, several potential SNS mitigation features were considered, including removal of the dam. After screening the alternatives within the Corps and with the resource agencies, a revised off channel rock ramp was selected and is now incorporated in the project as a mitigation measure for adverse impacts to SNS. The fish passage structure design described below includes refinements that go beyond the alternatives analysis completed during the summer of 2011. To achieve mitigation goals regarding flow velocities and depths through the structure, the design will be further refined during the PED phase.

The Off-Channel Rock Ramp (See **Figure 8.1.2-2**) consists of a rock ramp constructed around the South Carolina side of the lock and dam. All five gates will remain operational to allow for adjustments in the pool during high water events. Gates 1 and 5 require structural modification so that they function as lift gates rather than overflow gates. These modifications allow for 100% of the flow to pass through the fishway up to a river flow of 9,000 cfs. Flows over 9,000 cfs would flow through the gates on the dam. Based on a flow frequency analysis, this design would accommodate 100 % of the river flow for up to 71 % of the days of February through June. In the late spring months of May and June, when downstream passage is more critical, the 100 % flow capacity of the Off-Channel Rock Ramp increases to 83 % of the time. The range of river flows is shown in **Table 8.1.2-1**.

The velocities expected on the ramp vary depending on river flows ranging from approximately 5.5 feet/second at flows of 3,100 cfs to about 8.6 feet/second at 10,000 cfs. These are average velocities and depths; placement of numerous rock boulders to form pools up the rock slope will provide areas or pockets of lower velocity. During the design phase, a low section that connects the pools will be incorporated which will increase the maximum depth by at least a foot over the average depth shown in **Table 8.1.2-1**.

Figure 8.1.2-2: Off-Channel Rock Ramp

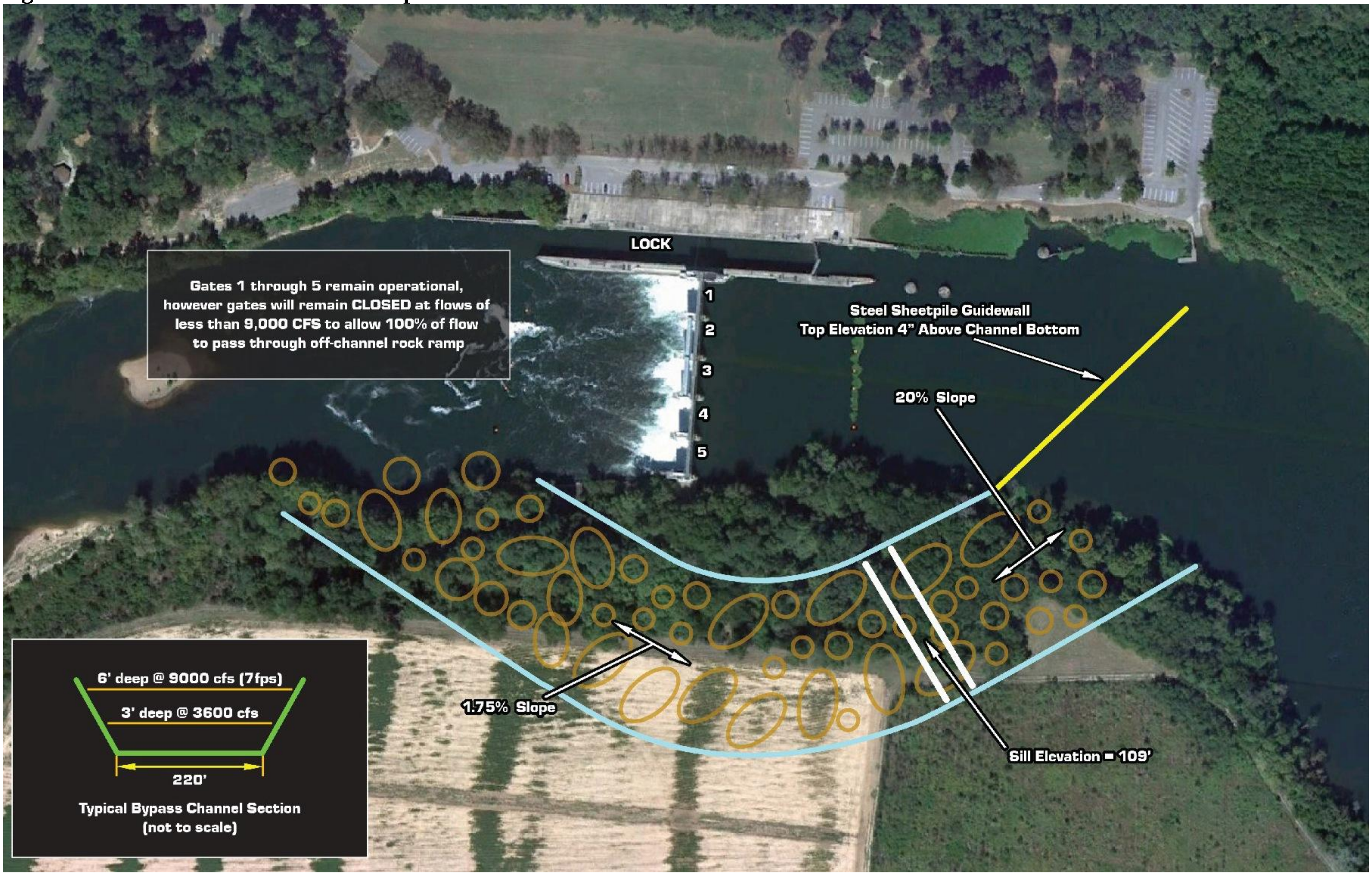


Table 8.1.2-1: Off-Channel Rock Ramp Flow Frequency

Flow (cfs)	Upper Pool Elevation (feet)	Depth of Flow Over Rock Ramp (feet)	Percent of Flow	Velocity at Crest (fps)
3,100	112.1	2.3	100%	5.5
3,600	112.4	2.5	100%	5.9
4,300	112.8	2.8	100%	6.3
5,000	113.1	3.0	100%	6.6
6,000	113.6	3.4	100%	7.1
8,000	114.5	4.0	100%	7.9
10,000	115*	4.6*	94%*	8.6*
12,000	115*	4.6*	78%*	8.6*
15,000	115*	4.6*	63%*	8.6*
20,000	115*	4.6*	47%*	8.6*
25,000	115*	4.6*	38%*	8.6*
30,000	115*	4.6*	31%*	8.6*

*estimated values

The rock ramp design is sloped up to a minimum crest elevation of elevation 109 ft at a 1.75% slope (1:50) on the downstream side and a 20% slope (1:5) on the upstream side. The design crest width is 25-feet wide, which would provide water depths of about 3.5 feet. Operational plans at the NSBL&D will be developed to ensure pool elevations are adjusted to pass high flows for elevations exceeding 115 ft (9,400 cfs) while minimizing velocities through the gates.

A submerged sheet pile wall is designed at a height of 3 to 4 feet above the river bottom or above the rock ramp. This wall guides the bottom-oriented SNS out of the deep river channel and through the ramp for downstream passage. Additionally, a small amount of dredging would be performed to shape the channel bottom so that the thalweg flows to the rock ramp. See **Table 8.1.2-2** for estimated construction quantities.

Table 8.1.2-2: Off-Channel Rock Ramp Estimated Construction Quantities

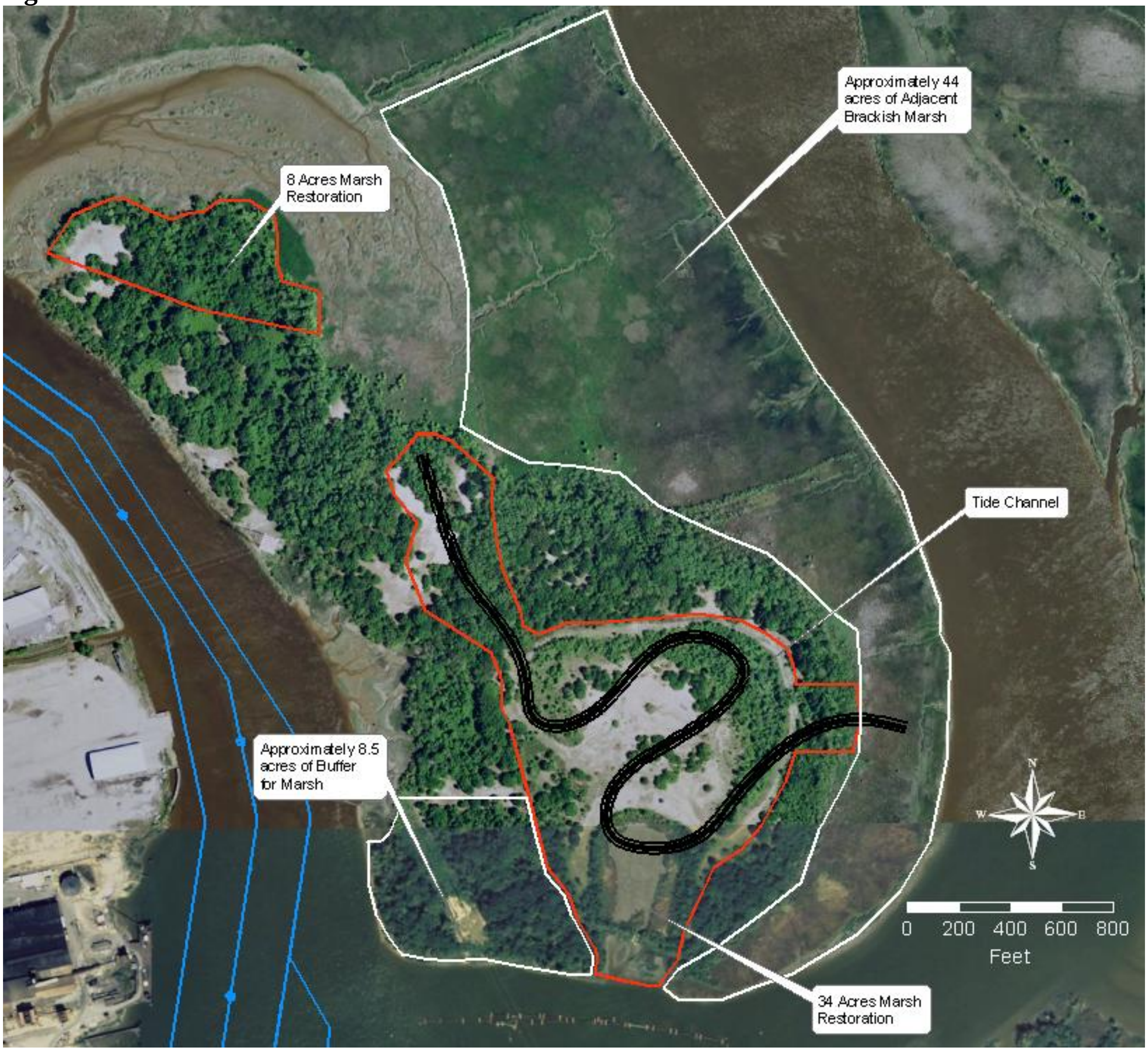
Item	Quantity
Mobilization/Demobilization	1
Armor Stone(tons)	88,000
Bedding Stone(tons)	20,000
Weir Stone 10% of armor	8,800
Clearing and Grubbing(acres)	18
Gate Replacement (each)	2
Sheet Pile Wall (lf)	1,300
Excavation(c.y.)	275,000
Access Road Construction (c.y.)	15,000

This design requires the least modification to the existing dam of the alternatives considered by the District. None of the gates would need to be removed from the dam; however, the two end gates would need to be modified. The present ability of the Lock & Dam project to reduce flood levels in upstream areas would be retained. The dam itself would not require modification. The lock and its operation would be unaffected. The Off-Channel Rock Ramp would reduce the work that would need to be performed if funds become available to rehabilitate the Lock & Dam. The rock ramp would remove the requirement to construct a fish passage structure, since it would provide the same function which would reduce the cost of the rehabilitation project. The dam would still need to be rehabilitated, to stabilize its structure and ensure its function continues to be provided in the future. The lock and its control house would still require the same amount of rehabilitation. Lands presently obtained for the Lock & Dam project would be needed to construct and operate the rock ramp around the South Carolina end of the dam. Those lands are presently wooded and are not used to operate the existing project. They provide structural stability to the dam and serve a limited security function. Those purposes would not be affected by construction and operation of the Off-Channel Rock Ramp. Additional lands would also need to be acquired to construct the rock ramp and for an access road to the site. Those lands would be acquired as part of the SHEP and not as part of the NSBL&D project. For details on additional real estate needs see Appendix B.

8.1.3 Marsh Restoration at Area 1S (Onslow Island)

Marsh restoration at Disposal Area 1S, also known as Onslow Island and owned by US Fish & Wildlife, is designed in such a way that tidal flow will be established across the entire restored site. The restoration design involves removing approximately 434,700 cubic yards of material to restore approximately 42 acres to an elevation suitable for growth of marsh vegetation (7.6 ft MLLW). See **Figure 8.1.3-1** for a plan view of this mitigation feature.

Figure 8.1.3-1: Marsh Restoration at Area 1S



Marsh restoration is broken into two pieces across the site. The northern area requires about 20,500 cubic yards of material to be removed. Clearing and grubbing for this area is approximately 8 acres. The southernmost area requires removal of about 405,000 cubic yards of material. Clearing and grubbing for this area is approximately 34 acres. A 4,360 ft long tidal conveyance feature will be constructed to provide tidal water exchange between Middle River and the restored marsh. This conveyance feature will require an additional 9,200 cy to be removed. If the excavated material is found to be suitable, it will be placed in the sediment basin to construct the broad berm. Unsuitable material will be pumped into Disposal Area 12A. **Table 8.1.3-1** summarizes the material quantities for this feature.

Table 8.1.3-1: Material Quantities for Marsh Restoration at Area 1S

Feature	Material Quantity
Northern area	20,500 cy removed 8 acres clearing & grubbing
Southern area	405,000 cy removed 34 acres clearing & grubbing
Tidal Conveyance Feature	9,200 cy removed 3 feet deep bottom width 10 ft 4,360 feet long

8.1.4 Hutchinson Island Boat Ramp

Due to the proposed closure of Rifle Cut as a mitigation feature to preserve freshwater marshes/wetlands, recreational boater access routes will be interrupted. For recreational boaters using the public access boat ramp at Houlihan Bridge on Front River, the route to move from this ramp to Back River will lengthen substantially with the closure of Rifle Cut. See **Figure 8.1.4-1**. To mitigate for this additional impact, a public access boat ramp on Hutchinson Island is proposed. This public access ramp would provide additional access to the Back River for recreational boaters.

The proposed boat ramp is located on government owned land at the tidegate on the North side of Hutchinson Island, adjacent to Back River. The proposed 2 lane concrete boat ramp will include the following: floating dock, 20 space trailer parking, handicap accessible and parking, and parking spaces for 12 single cars. See **Figure 8.1.4-2**. After construction of the public access ramp is complete, the property will be transferred to Chatham County, and the ramp will be their responsibility to operate and maintain.

Figure 8.1.4-1: Recreational Boating Routes

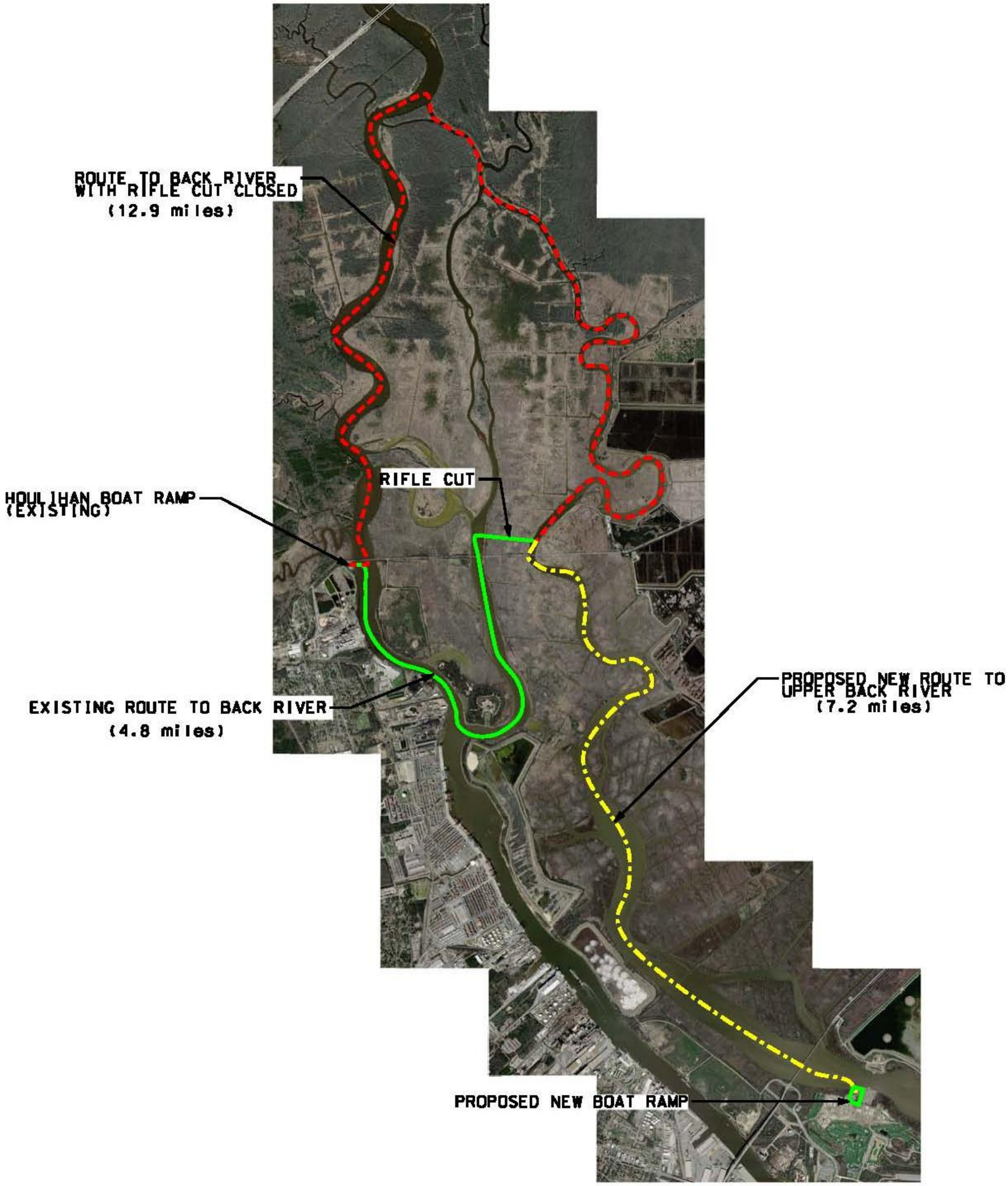
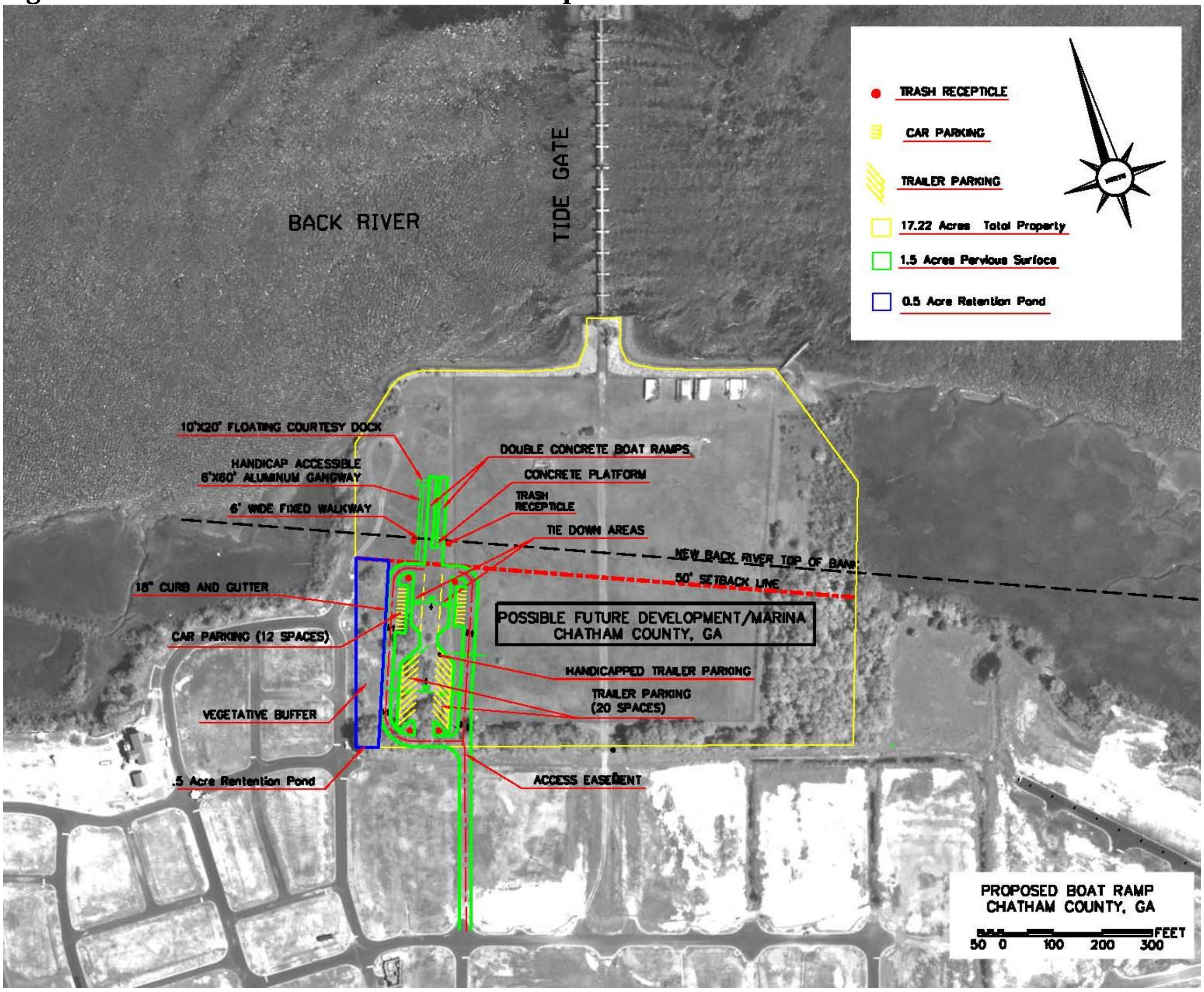


Figure 8.1.4-2: Hutchinson Island Boat Ramp



8.1.5 Dissolved Oxygen Injection System

To mitigate for D.O. impacts in the harbor resulting from SHEP, a plan has been developed to inject superoxygenated water into the estuary using Speece Cone technology. This plan was developed by Tetra Tech and results are documented in the report titled *Design of Dissolved Oxygen Improvement Systems in Savannah Harbor* which is included in the Engineering Investigations Supplemental Materials. The preliminary design of the oxygen injection system is also included in the Tetra Tech report. The design was completed under sub-contract by Eco-Oxygen Technologies, LLC.

The D.O. injection system will be land-based, with water being withdrawn from the river through pipes, then super-saturated with oxygen and returned to the river. The water intake structure would include screens to reduce the intake of trash and other suspended solids. The screens would be sized to keep flow velocities from exceeding 0.5 foot per second to minimize entrainment of fish larvae. The intake and discharge would be located along the side of the river and not extend out into the authorized navigation channel.

9.0 SHORELINE EFFECTS

9.1 SHIP FORCES ON THE SHORELINE

9.1.1 Background

A study was conducted by the Coastal and Hydraulics Laboratory at the US Army Engineer Research and Development Center to determine the impacts of the proposed deepening on the adjacent shoreline. The 2007 report is entitled *Ship Forces on the Shoreline of the Savannah Harbor Project* and is included in the Engineering Investigations Supplemental Materials. This report was updated in July 2011 to more accurately represent the vessel fleet anticipated to be calling on the port of Savannah for the future with project condition and the future without project condition. The ship speed model and ship wave equation were updated using more recent guidance and input from the harbor pilots.

Ship forces having the potential to cause shoreline erosion were evaluated at the Savannah Harbor to compare the without project (existing) and the with-project (deepened) channels. Results of this study were used by the Savannah District in a separate study to evaluate shoreline erosion (see Section 9.2, Bank Erosion). An analysis of ship forces requires determination of comparable ship speeds in the without project (existing) and with project (deepened) channels. Field data were used to determine ship speed in the without project (existing) channel. An analytical model for ship speed, along with the assumption of equal ship operation including equal power setting in the without project and with project channels, was used to determine ship speed in the with-project channel.

9.1.2 Report Updates

Based on updated sailing draft distribution and fleet forecast, the ship forces at the shoreline were reanalyzed in the report *2011 Reanalysis of Ship Forces at the Shoreline in Savannah Harbor(July 2011)* which is included in the Engineering Investigations Supplemental Materials. . In the 2007 study, a typical/average draft and a large/design draft ship were used to evaluate ship forces. In this reanalysis, the typical ship compared in the existing and deepened channels is the 50th percentile or median ship draft from the sailing draft distributions. In this reanalysis, the large draft ship compared in the existing and deepened channels is the 95th percentile ship draft from the sailing draft distributions. Because distributions were not available for sub Panamax and their draft is not affected by deepening, the typical draft ship used was the average draft determined in the 2005 field study. The large draft Sub Panamax ship has draft equal to the design draft used in the 2007 study. In the 2007 study, a post Panamax ship having a beam of 140 ft was the design ship. In this reanalysis, the design ship has not changed but post Panamax beams have been refined to Generation 1 having average beams of 131.7 ft and Generation 2 having average beams of 142.9 ft.

Because of the changes in draft, ship speed had to be recomputed along with drawdown, return velocity, and wave height. In addition, the ship speed model and the ship wave equations were updated. Due to the updated draft information and improved ship speed and wave height models, all conclusions in the 2011 revised report supersede conclusions in the 2007 report.

9.1.3 Ship Operation and Speed Trends along Savannah Harbor

In the 2007 study, predicting ship speed in the deepened channel was based on existing speeds, change in ship draft, engine power setting, and increase in channel area. While those parameters are important in certain portions of the channel, there are three areas of the channel where ships must slow down to control their wake. Along the Savannah Harbor channel, ships must slow down at the Coast Guard, the LNG facility if a ship is docked, and beginning at Old Fort Jackson to the docks in Savannah. These three wake reduction areas affect a large portion of the channel because it takes significant channel distance for a ship to slow down and speed up. In these wake reduction areas, large ships must slow down more than small ships to result in the same wake effects. In these wake reduction areas, the requirement for safe wake has a far greater effect on ship speed than deepening of the channel.

9.1.4 Summary of the Revised Report

This reanalysis of ship forces addressed the following items:

- Updated draft information in the form of sailing draft distributions was used to define ship drafts to compare in the existing and deepened channels for each vessel class.

- The trends of speeds along the channel were identified and showed three wake reduction areas and 3 reaches where ships typically use full bell. The full bell is defined as the highest manuevering setting with the highest speed a pilot can use and the one they prefer to use if conditions permit. This represents the most critical case for ship wake formation.

- Ship speeds were determined in the wake reduction areas based on maximum allowable drawdown to prevent moored ship effects.

- A revised ship speed model was developed and described. The model was validated against restricted channel data by Norrbin and shallow water effects on speed by Schlicting and Norrbin. Empirical coefficients were determined using Savannah Harbor ship event data collected in a 2005 field study.

- The ship speed model was used to determine ship speed in the three reaches where ships navigate at full bell. Drawdown and return velocity were determined in the reaches where full bell is used.

- Plots of speed along the entire Savannah Harbor channel were developed for each vessel class using the wake reduction speeds and the speeds in reaches using full bell.

- The 2007 ship wave model was revised, and a ship wave model by Kriebel and Seelig (2005) was obtained.

- Wave height was determined at the three wake reduction areas and the three full bell reaches for both existing and deepened channels using both the Kriebel and Seelig equations and the equation from the 2007 report that was revised in the reanalysis.

- Using forecasts of ship calls for existing and deepened channels, composite values of drawdown, return velocity, and wave height were determined that combine all ship classes into a single number for comparison.

- Drawdown and wave height were evaluated using total number of ships in each ship class and drawdown and wave height squared to reflect wave power.

9.1.5 Results and Conclusions

Evaluation of ship forces from drawdown, return velocity, and waves must be based on accurate ship speed because these effects vary based on the value used for speed, which is raised exponentially depending on which equation is used to predict ship force. Just as important as accurate speed prediction is consideration of operational constraints on ship speed. Ship pilots are responsible for the wake of their ship, and there are three areas along the Savannah Harbor channel where ship wake must be controlled. Because of the distance required to slow down and accelerate a ship, the wake reduction areas affect a significant length of the channel.

Long period drawdown is the best indicator of the effect of passing ships on moored vessels. Existing ships along the Savannah Harbor at wake reduction areas slow down such that their drawdown is about 0.7 ft or less. Ship speed in each wake reduction area was determined for each ship class and draft using the 0.7 ft drawdown limit.

The ship speed model used in the 2007 study was updated and a full description is provided in the reanalysis. In addition, the ship speed model was validated with independent restricted channel results from Norrbin and also compared to shallow water effects of Schlicting and Norrbin.

Previous studies have shown that ship pilots at Savannah Harbor and other channels will typically travel at full bell if not confronted by operational constraints such as wake reduction areas or Right Whale restrictions. The ship speed model was used to determine ship speeds at the three areas in Savannah Harbor where ships can travel at full bell.

Using speeds in the full bell reaches and the wake reduction areas, speed plots were presented for each ship class and both 50th percentile and 95th percentile drafts. The speed plots show that Sub Panamax ships, whose draft is not affected by channel deepening, will go faster in the deepened channel. Panamax ships, whose drafts increase by up to 1.5% in the deepened channel, will go slightly faster in the deepened channel where the cross sectional area increased by about 5%. Generation 1 post Panamax ships, whose drafts increase by about 11% in the deepened channel, will go slower in the deepened channel where the cross sectional area increased by about 5%. Generation 2 post Panamax ships, whose drafts increase by about 12-15% in the deepened channel, will go slower in the deepened channel where the cross sectional area increased by about 5%.

The ship wave model developed in the 2007 study was updated for the reanalysis and a second ship wave equation by Kriebel and Seelig (2005) was also used to predict ship wave heights.

Based on updated drafts, speeds along the channel, drawdown, return velocity, and wave height, ship forces have been recomputed and are described at various locations along the channel in the following paragraphs. From the viewpoint of changes in ship forces leading to changes in bank erosion, the three reaches where the ships are operating at full bell and thus highest speeds are the areas of primary concern.

At the full bell reach at Tybee Island, the presence of the south jetty and the 3500 ft distance from shoreline to navigation channel are significant factors in the magnitude and prediction of ship forces at the shoreline. Composite drawdown in the channel, return velocity in the channel, and wave height at the shoreline based on a single ship to represent all ship classes were less than or equal with the deepened channel when compared to the existing channel. Wave heights at Tybee Island shoreline from the Kriebel and Seelig equation are less in the deepened channel for all vessel classes and drafts. During the period of speed restrictions due the presence of Right Whales (Nov 15 to Apr 15), equal 10 knot speeds in existing and deepened channels will result in negligible secondary wave heights and lesser drawdown and return velocity in the deepened channel. Summing secondary wave power for all ships shows decreased wave power in the deepened channel from both wave equations.

Fort Pulaski was treated as a full bell location in the 2007 study. Based on this reanalysis and improved understanding of speeds along the Savannah Harbor channel, speeds at Fort Pulaski are strongly affected by the wake reduction area at the Coast Guard Docks. Fort Pulaski is in a location where speeds are dictated, not primarily by the power and size of the ship, but by the requirement of the pilot to control the ship's wake at the Coast Guard. In addition, during 6 months of the year, speeds are restricted by the Right Whale 10 knot limit. In the 2007 study conducted before the Right Whale speed restriction, Fort Pulaski was an analysis location where it was shown that wave power increased by up to 19% in the deepened channel. Wave height determined in this reanalysis using the Kriebel and Seelig wave height equation decreased for all ship classes and drafts in the deepened channel. Summing secondary wave power for all ships shows decreased wave power in the deepened channel from both wave equations.

At the full bell reach between Confined Disposal Facilities (CDFs) and Old Fort Jackson(OFJ), ships typically do not reach their maximum speed for full bell but do achieve speeds that are close to the full bell speed. Composite drawdown, return velocity, and wave height based on a single ship to represent all ship classes are up to 1.4% greater in the deepened channel. The 1.4% increase in composite wave height corresponds to a 3% increase in wave power. When considering total numbers of ships by summing wave power for all ship classes, wave power decreases in the deepened channel from both wave equations. When considering a relative measure of power using drawdown to represent transverse stern waves, sum of power from all ship classes is less in the deepened channel.

The full bell reach between the Coast Guard Docks (CG) and the Liquifed Natural Gas (LNG) dock is the only reach not restricted by operational constraints like the Coast Guard or Right Whale or limited channel length like the reach from the CDFs to OFJ. The CG to LNG reach has the highest magnitude of composite drawdown, return velocity, and wave height of all reaches. Composite values combining all ship classes of drawdown, return velocity, and wave height are less than or equal in the deepened channel when compared to the existing channel. When considering total numbers of ships by summing wave power for all ship classes, wave power decreases in the deepened channel from both wave equations. When considering a relative measure of power using drawdown to represent transverse stern waves, the sum of power from all ship classes is less in the deepened channel.

Summarizing, in the three reaches where the ships are operating at full bell and thus highest speeds, the reanalysis shows the sum of power from all ships to be less in the deepened channel than the existing channel; therefore, bank erosion due to ship wake should not be increased as a result of deepening the channel.

9.2 BANK EROSION

This analysis estimates the impacts to the shoreline (i.e. bank erosion) due to ship wakes as a direct result of SHEP deepening near City Front, the Bight Section, Fort Pulaski property and the northern beach of Tybee Island. The results of the bank erosion study are documented in two reports; 1) *Savannah Harbor Expansion Bank Erosion Study, Fort Pulaski and North Tybee Island, Georgia* and 2) *Savannah Harbor Expansion Bank Erosion Study Update*. Both are included in the Engineering Investigations Supplemental Materials. The evaluation is based on the difference between the ship wakes of today versus the ship wakes of the future, considering the without project condition compared with the deepened project condition. The total estimated shoreline erosion (due to all causes) is based on aerial photography from 1964 to 2003.

Analysis is based on the updated Fleet Forecast (June 2011) developed by the Economics team and included in the Economics Appendix (Appendix A), available soils information, bathymetry, topographic surveys, aerial photographs, historical information, observation/review of channel side slopes resulting from previous harbor widening and deepening projects, and information from previous dredging works regarding channel side slope performance. Direct correlations were made using the *Ship Forces on the Shoreline of the Savannah Harbor Project* report completed by USACE ERDC and the project fleet forecast.

The latest fleet forecast (June 2011) predicts the following fleet mix for the SHEP:

- For the year 2017: 4,285 calls or 8,570 passing events were counted for the existing 42 ft channel depth and 4,133 calls or 8,266 passing events were predicted (by estimation) for the 48 ft project depth alternative. This indicates a notable reduction of passing events that could contribute to bank erosion for Savannah Harbor.

- For the years 2030 and beyond: 7,204 calls or 14,408 passing events are predicted for the without project (42 ft) condition and 6,714 calls or 13,428 passing events are predicted for the 47 ft to 48 ft project depth alternatives. This represents about a 10% reduction of ship passing events which reduces erosion forces interacting with the banks of Savannah Harbor when compared to the without project condition.

The study concluded that the effects of deepening the navigation channel will not impact the City Front, the North Tybee Site, the confined disposal areas within the Bight Section, or the unprotected areas of Fort Pulaski with regard to erosion of bank materials at any location.

Unprotected portions of Fort Pulaski are subject to shoreline erosion measurable from 0.5 to 3.1 ft per year, depending on specific location. The majority of erosion is due to tide, flows, river mechanics, shape, and the present day ship traffic events. Future ship traffic without deepening is estimated to have a minimal to no measurable impact given the predicted fleet mix and volume. Zero additional erosion is predicted by forecast for any deepening increment or depth up to and including the 48 ft depth alternative.

Given the current traffic predictions and forecasts, no known or measureable bank erosion impacts are expected to be directly attributed to SHEP.

9.3 COASTAL EROSION

A study titled *Impacts of Savannah Harbor Expansion Project* was conducted jointly by the ERDC Coastal and Hydraulics Laboratory and the Charleston District (draft, October 2006) and is included in the Engineering Investigations Supplemental Materials. This study evaluated the impact of the proposed deepening of the Savannah Harbor navigation channel. A bathymetry and volume change analysis was conducted to provide the historical perspective of the Savannah nearshore evolution and to address review comments related to a previous ATM report (Applied Technology and Management, Inc. 2001, *Draft Savannah Harbor Beach Erosion Study: Savannah Harbor Expansion Project*). Numerical modeling of circulation, waves, and sediment transport was performed to compare pre- and post-deepening of the channel impacts on the coastal processes.

Deepening of the navigation channel, beginning in the 1870s, and subsequent construction of dual parallel jetties to stabilize the position of the navigation channel (1886-1897 time frame) are most likely the trigger for the changes in Tybee Island shoreline position that were observed between 1854/63 and 1900, and for the pattern of changes that has taken place since that time. There are no historical shoreline and bathymetric data available between 1854/63 and 1900 that would allow the details of that evolution, any variability in its rate of change during that time period, and a more direct linkage between navigation project construction and shoreline/morphology changes to be examined or discerned further. However, results of the historic shoreline analysis and volume change analysis, in addition to results from the circulation and GTRAN sediment transport models, provide a consistent picture of circulation and sediment transport processes at work. The following hypothesis is the most likely explanation for evolution of the inlet since construction of the navigation project.

Deepening of the navigation channel and construction of the dual parallel jetties appear to have concentrated the ebb tidal flow into a narrower, more concentrated, stronger ebb tidal jet. The navigation project also has fixed the location of the channel, as intended, as has construction of the submerged breakwater. Fixing the location of the channel reinforces the morphological response that occurs in response to the ebb and tidal currents that enter and flow through the channel. Concentration of the flow by the project produces higher ebb currents within the jet. Prior to project construction, ebb velocities in this vicinity were probably weaker. Associated with enhanced ebb jet formation is increased gyre formation adjacent to the jet, i.e. on either side of the jet. This flow feature is observed at many structured inlets with high ebb currents. Enhanced gyre formation produces, at peak ebb within the channel, a return current on the flanks of the inlet that flows back toward the inlet entrance, in the same direction as that experienced during typical flood flow conditions. During the flood flow here, currents are directed toward the entrance throughout the ebb shoal region. Thus, a situation is created where currents adjacent to the channel are directed toward the entrance (in the flood flow direction) a much larger percentage of the time. The duration of flood flow is greater than the duration of ebb flow along the flanks of the inlet. The North Tybee Island Shelf is the southern flank of the inlet. This change in predominance of flow direction adjacent to the ebb jet is expected to produce a corresponding change in the sediment transport patterns and morphology in these same regions, compared to conditions that existed prior to deepening and jetty construction. These changes to the circulation and sediment transport patterns, created by the navigation project, are thought to explain the shoreline and morphologic changes that have occurred on Tybee Island, along with changes to sediment supply to the island from Barrett Shoals.

At present, the Tybee Island Shelf is in the flank of the jet and is influenced by the return flow of the gyre. The GTRAN results show that the sediment transport regime in the channel and immediately adjacent to it are strongly ebb directed. The strong transport computed for the channel is consistent with the ebb dominance of this inlet system as evidenced by its massive ebb tidal delta. GTRAN results also show that a short distance away from the channel sediment, net sediment transport is directed toward the entrance, both north and south of the channel, albeit with much smaller transport rates than those computed in the channel. The GTRAN results also indicate that on the Tybee Island Shelf to the east of northern Tybee Island, sediment transport is directed toward the northwest in a net sense. This direction of transport suggests that the sediment from this region moves toward the North Tybee Shoal. The accumulation of sediment in the North Tybee Shoal (and associated migration of the South Channel to the north which has likely happened in response to the sediment accumulation there) and deflation of the Tybee Island Shelf are consistent with the results of the GTRAN model. It is likely that the sediment transport regime in this region has been altered compared to the regime that existed prior to construction of the navigation project, in response to changes in the circulation patterns and local dominance of ebb/flood flow. Additional simulations of the wave, circulation, and GTRAN models for an inlet configuration representing the pre-construction condition could shed additional light on the working hypothesis for the shoreline and beach evolution observed on north Tybee.

The GTRAN results reflect conditions in deeper water where waves tend to agitate and suspend sediment and currents tend to move the sediment, not the shallow surf zone region along the Tybee Island shoreline. The very shallow nearshore region is much more wave dominated than the offshore region for which GTRAN was applied. Shoreline change modeling to examine this inner surf zone region was not part of the scope of this effort, but the minor changes in the nearshore wave results indicate that the deepening will have little impact on the shoreline. The historical shoreline change analysis showed the presence of a nodal zone around 2nd Street, north of which sand appears to be transported to the north by wave action, and south of which sand appears to be transported to the south. The hot-spot is in the region of the nodal zone, a divergent zone in which sediment tends to leave the region in both directions, in a net sense. The erosion within this nodal zone along north Tybee, is likely to be caused both by the ebb tidal deflation that occurs offshore as well as the gradients in alongshore transport created by the wave-dominated transport in the very nearshore zone.

Another possible factor in the evolution of the north half of Tybee Island is the unusually frequent and severe tropical storm activity during the period from 1879 to 1899. This increase in storm activity could have exacerbated the situation created by initial project construction and produced unusually high rates of sediment transport and morphology change. However, it seems unlikely that a single extreme event, or sequence of hurricane/tropical storm events in the region alone are capable of stimulating the shoreline and offshore morphology changes that have occurred on Tybee Island and in the Tybee Island Shelf. The GTRAN sediment modeling results indicate that the sediment transport regime during a single episodic event, like Hugo (which was re-tracked to have a much larger impact on the area than it actually had), is similar to the pattern seen for non-hurricane conditions but is greater in magnitude in some locations (not all) than any of the months of normal winter storm activity that were examined. But where the magnitude was greater, the increase is only a factor of 1 to 5 greater. For this magnitude of difference, and in light of the infrequence and short duration of episodic events, the more typical month-to-month forcing in response to an increasingly deeper channel and tighter ebb tidal jet appears to be a more likely trigger for the shoreline and beach evolution that has taken place.

GTRAN results show that the net sediment movement on the southern half of the Tybee Island Shelf is toward the west, i.e. toward the island. This computed result is consistent with the relative health of the southern half of Tybee Island, compared to the northern half. Historic shoreline changes showed the southern half of the island to be accretionary for a significant time following construction of the project. Observations from other inlets suggest that as the ebb delta grows the down drift attachment bar migrates to the south. The attachment bar is the location on the ebb delta where sand moving around the ebb delta, from updrift to downdrift in a net sense, feeds the nearshore zone of the downdrift beach. This process may also be occurring at Tybee Island, and at present, the feeding appears to be occurring at the southern half of the island. Some of the sediment eroded from the bulge in shoreline (present in 1854/1863, but not in 1900) appears to have moved south and accreted on the southern half of the island following project construction. The shoreline position of south Tybee prior to beach fill placement was well seaward of its position prior to construction of the navigation project. The current navigation channel appears to be a nearly complete sink for any sediment moving from north to south along the shelf (suggested by pre- and post-dredging surveys and the consistency of dredging volumes following deepenings). Placement of dredged material back into the nearshore zone of Tybee Island would be a means for restoring this supply of sand to the Tybee beach system.

The circulation and wave modeling indicate very small changes associated with the proposed deepening. The GTRAN results provide insight about what this deepening will do in terms of sediment transport regime, which is expected to be similar to that of past deepenings. GTRAN results for the existing condition and the with-deepened-channel condition indicate that the additional channel deepening will not change the general overall pattern of sediment transport in the region. The most noticeable changes were computed in the channel.

Transport in the Tybee Knoll Bar Channel reach showed decreases in magnitude for all four of the typical months of simulation, but conditions remain strongly ebb-dominant. The magnitude of change was greatest here (15-20 percent), compared to changes throughout the rest of the system (changes elsewhere were generally quite small, several percent). Some small increases to shoaling rates in this sector of the channel might be expected in light of these decreases, but gradients in transport (which dictate accumulation rates) do not appear to be altered very much. For the extreme hurricane event (expected to be a very rare event), conditions remain ebb dominant and transport rates increase (factor of 2 greater) rather than decrease as they do for the other four months.

Transport in the Tybee Roads Channel reach consistently shows increases for each of the typical months, but only very slight increases of a few percent. No significant changes in sedimentation are expected in this reach of the navigation channel. Transport rates increase in this portion of the channel for the re-tracked Hugo event (about a factor of 2 or greater), as they do for all sections of the main channel. Transport remains strongly ebb dominant in this reach.

In the Tybee Range Channel reach, the outer limits of the navigation channel, changes in transport rate are also very small. The deepening increases rates for one month, shows zero change for one, and shows slight decreases for two of the months. All increases or decreases are small (a few percent). For re-tracked Hugo, the deepening only increases transport by about 20%, compared to a factor of 2 in the other channel reaches. This section of channel remains ebb-dominant. These changes do not suggest any significant changes in channel shoaling, and they suggest that the channel region will remain strongly ebb-dominant in terms of sediment transport direction.

Average transport within the Tybee Island Shelf region consistently shows slight increases or no change for each of the four typical months and an increase for the extreme event. No decreases were computed. Patterns appear unchanged, and the net direction of movement appears to remain to the northwest. Slight increases suggest a tendency for sediment to be transported from the shelf region to the northwest at a higher rate. This would be consistent with the hypothesized model for how sand has been moving to the northwest in response to initial project construction and subsequent deepening. This proposed deepening seems to produce a result consistent with that hypothesis. However, the magnitudes of change are quite small, in the range from 0 to 2 percent for all months and even for the extreme hurricane event. Zero change was computed for two of the four months. Computations show that channel deepening will have negligible effect on the Tybee Island Shelf.

For North Tybee Shoal region, computations show a consistent decrease in transport rate for all four months, about 5 percent or less; however, for the extreme hurricane event, transport rates are increased by about 40%. These changes suggest that sediment being transported into the north Tybee shoal region will have less tendency to leave the region, but this is more dictated by changes to the transport gradients. Such a trend would be consistent with historic accumulation of sediment in these shoals.

10.0 SEDIMENTATION ANALYSIS

A study was conducted to determine the effects of SHEP regarding sedimentation rates, volumes, and patterns. The results, which are summarized here, are documented in full in the report titled *Sedimentation Analysis* which is included in the Engineering Investigations Supplemental Materials. In the evaluation, the effects of past dredging and construction activities were analyzed to gain an understanding of how sedimentation in the navigation channel responds to changes. Understanding how the system responded to past changes was used as the basis to make predictions on sediment volume and shoaling rates and patterns with the proposed project.

10.1 SEDIMENT SOURCES & TYPE

The major sources of sediment to Savannah Harbor are: 1) sediments suspended in the freshwater flows carried downstream by the river and 2) the offshore sediments carried into the harbor by tidal currents. Current velocities and the location of the mixing zone between fresh and salt water influence the distribution of the shoaling from these two sources.

Results of an analysis of a suspended sediment sample taken at Clyo, Georgia, located approximately 65 miles above the mouth of the Savannah River, indicate that sediment supplied by the river and deposited throughout the inner harbor is primarily fine silt and clay. The bed load material transported by the river is deposited in the extreme upper reaches of the Savannah Harbor above Station 103+000. The shoal materials in these reaches are principally sand and account for no more than 5 % of the total volume material dredged from the harbor.

Grain size distributions for the inner harbor and entrance channel are shown in **Table 10.1-1** and graphically in **Figures 10.1-1** and **10.1-2**. The inner harbor sediments are primarily silts and clays between Stations 103+000 and 56+000. The reach between Stations 56+000 and 25+000 is a transition reach that has a higher percentage of sand in its distributions than the sediment distributions of the upstream reach. A notable exception is in the vicinity of Station 36+000, which has a high percentage of silt and clay and almost no sand. This location is near the confluence of the inner harbor channel with both Elba Island and Fields Cuts (Atlantic Intracoastal Waterway crossing). The lower harbor channel sediment distributions between Stations 25+000 and 0+000 are primarily sand, which indicates that the source of sediment for this reach is offshore. The entrance channel sediments are primarily sand with exceptions between the jetties and at Station -45+000(B), which have large silt and clay components. The upstream source of sediment for the upper river reaches and the ocean source for the lower river reach are consistent with the observation that essentially all of the shoaling material from upstream sources is being trapped within the system.

Table 10.1-1: Grain Size Distribution

ID*	Station	Channel	% SAND	% SILT	% CLAY
NB-13	-45+000	Entrance	8.7	66.8	24.5
NB-12	-35+000	Entrance	83.9	0	16.1
NB-11	-25+000	Entrance	78.2	5.8	15.8
NB-10	-15+000	Entrance	67.1	12.3	20
NB-9	-5+000	Entrance	30.8	53	16
IH-8	5+250	Inner	94.4	4.4	0.3
IH-7	15+000	Inner	88.2	2	9.1
IH-6	25+000	Inner	93.7	0	2.5
IH-5	35+000	Inner	12.1	62.4	25.5
SH-7	36+000	Inner	0	65.3	34.7
IH-4	44+000	Inner	32.2	32.3	35
IH-3	55+750	Inner	27.2	45.5	27
SH-6	56+000	Inner	10.6	75.7	13.7
SH-5	61+500	Inner	13.4	53.5	33.1
IH-2	64+000	Inner	2.7	59.8	37.5
SH-4	67+250	Inner	78.4	13.5	8.1
IH-1	75+000	Inner	5.7	49.8	44.5
SH-3	90+000	Inner	9.6	55.6	34.6
SH-1	99+000	Inner	14.4	54	31.6
SH-8	2+750	Sediment Basin	17.5	49.4	33.1
SH-9	5+250	Sediment Basin	0	72.8	27.2
SH-10	8+000	Sediment Basin	0	66.9	33.1
SH-11	10+500	Sediment Basin	17.6	46.2	36.2

See **Figure 10.1-1 and 10.1-2 for ID locations.*

Figure 10.1-1: Grain Size Distribution Station 90+000 to 5+000

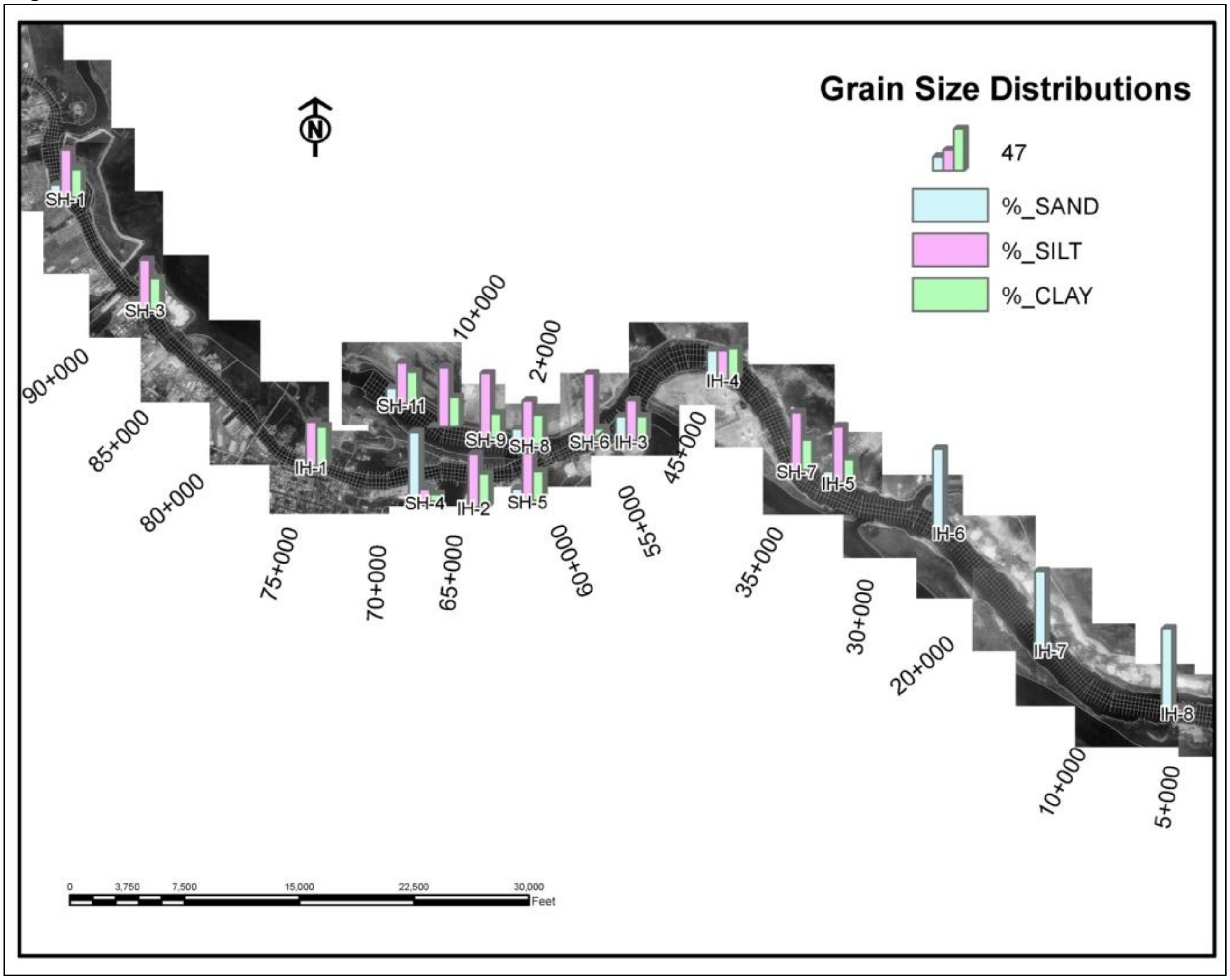
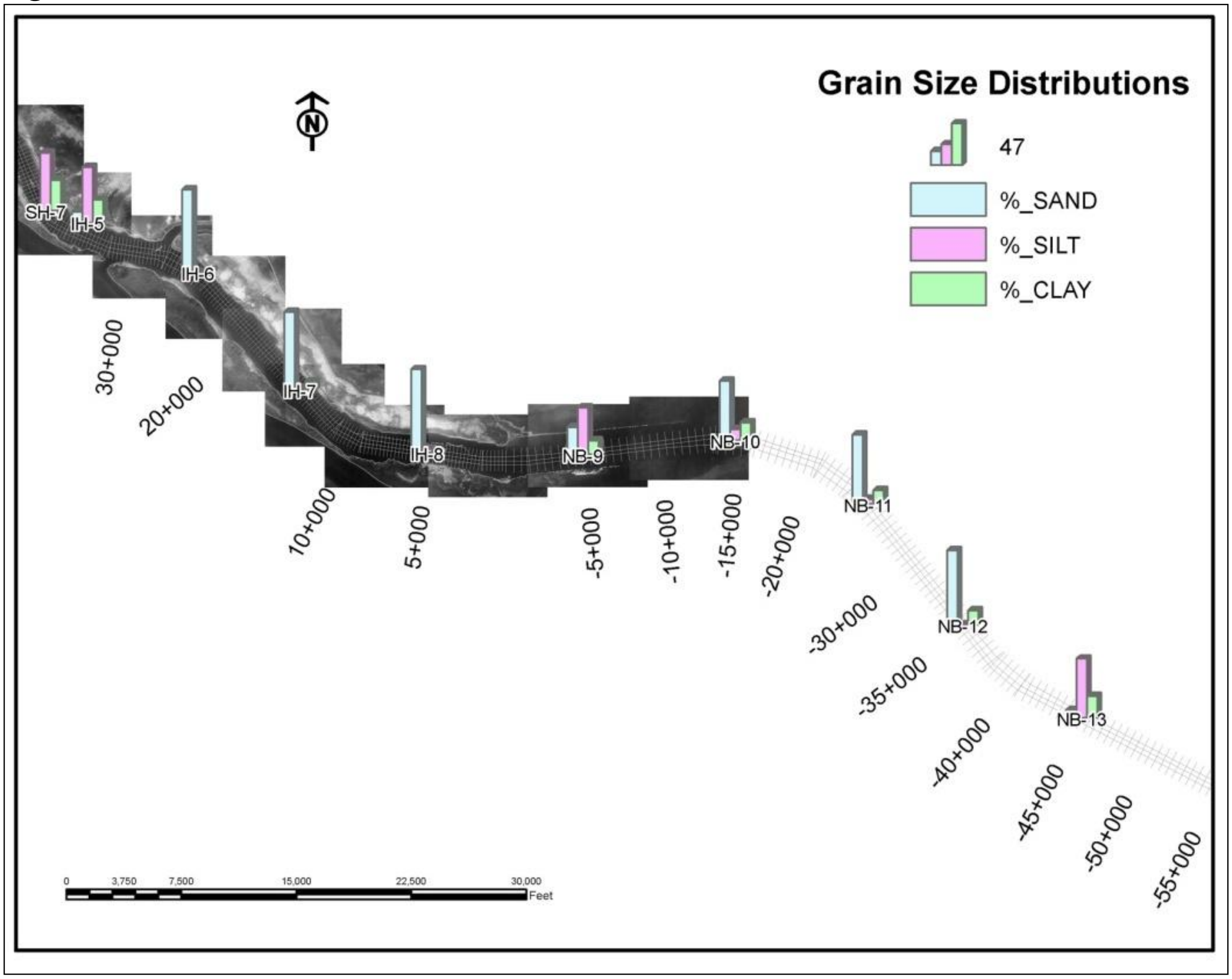


Figure 10.1-2: Grain Size Distribution Station 30+000 to -55+000



10.2 THE SHOALING PROCESS

Past channel deepenings have modified the estuary's tidal conveyance to the point where the full tidal prism reaches the upstream of the harbor. Analysis shows that the last channel deepening did not change the shoaling volume or distribution, indicating that the channel already captures all of the sediment that enters the harbor. The shoaling patterns of these sediments are outlined in the following paragraphs.

In the middle harbor reach, Stations 28+000 to 67+000, and upper harbor reach, Stations 67+000 to 103+000, silts and clays form low-density shoals in areas with low velocities or eddies. Areas with low velocities have low bottom shear stresses which allow deposition and the circular flow of the eddies promote flocculation and trap sediment. Salinity affected currents cause a flow converging area at the location of zero net bottom flow, which becomes an area of high shoaling. The salinity effects also cause the bottom flows in the lower harbor to have a net upstream flow, which traps ocean derived sand.

The source of shoaling material in the upper and middle reaches of the harbor is silt and clay eroded by rain runoff in the piedmont. The clay particles have a negative charge and a diffuse layer of positive ions surrounds the particle. The diffuse layer that surrounds the clay particles makes them mutually repulsive. The individual clay particles have an extremely slow settling rate, which allows them to travel hundreds of miles downstream from the piedmont to the estuary. The saline water of the estuary has a high ionic concentration, which compacts the diffuse layer and allows the particles to come closer together. When the particles enter a mixing flow they collide and the attractive forces, which exist between all colloidal particles, enables the particles to form aggregates. The aggregates can become relatively large and settle rapidly. When the aggregates are carried to an area with weak currents, they form low-density shoals. The delicate structure of the aggregates and low density of the shoals make them responsive to the strength of the flow velocities. An area that has weak currents or eddies will be high shoaling area and an area of strong currents will be a low shoaling area. The nature of these low-density shoals can make volumetric analysis of shoaling patterns difficult. If a low-density shoal were disbursed and the aggregates broken up, new shoals that form from the shoal material could be denser or so thinly spread out that they are not recorded at all.

Savannah Harbor is in a partially mixed estuary in which the vertical mixing of salt and fresh water is not complete over the length of salt water intrusion. Surface salinities are appreciably less than the bottom salinities, and there is a large zone of mixing between fresh and salt water. Seaward of this mixing zone, the net bottom flow over a tidal cycle is upstream. Landward of this mixing zone, the net bottom flow is downstream. The converging bottom flows carries shoaling material to the location of no net bottom flow, which tends to be an area of high shoal volumes.

Shoaling in the Savannah inner harbor channels below Station 28+000 is due to sand carried into the channel from the ocean by the strong bottom flood currents. The shoal material in the lower harbor is almost entirely sand while the shoal material upstream of Station 28+000 is silt and clay. The sand is deposited during slack tide and the weaker bottom ebb currents cannot carry the sand back to the ocean source. Results from the prior physical model tests indicate that the bottom flood currents at Station 4+000 are a foot per second faster than the bottom ebb currents and that the net bottom flow in the lower portion of the harbor is upstream.

10.3 FINDINGS

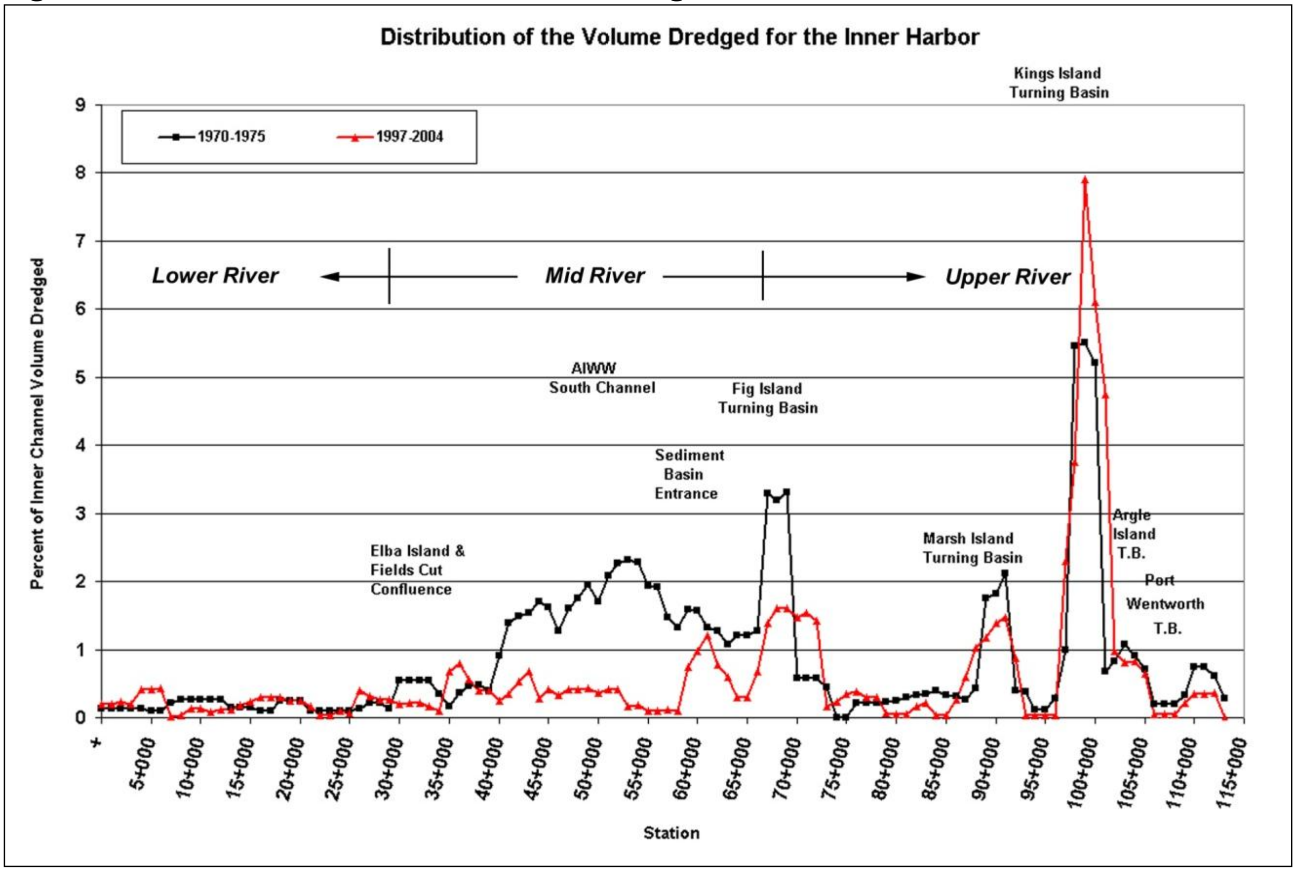
10.3.1 Shoaling Response to Inner Harbor Channel Depth Increases

To determine the changes to the amount and distribution of the inner harbor dredging volumes due to potential channel depth increases, the changes due to the 4-foot depth increase in 1994 were used as a predictor. In addition to the shoaling response to past changes, the velocities as predicted by the three-dimensional hydrodynamic model (EFDC) were used to check for potential shifts in shoaling distribution due to potential depth increases. No changes to the shoaling volume or distribution are predicted for the inner harbor due to depth increases.

<u>Past Dredging Volumes as an Indicator</u> – A comparison between the inner harbor dredging volume distributions between the periods 1970-1975 and 1997-2004 does not indicate any shoaling changes that can be attributed to a depth increase. There were multiple construction related changes between the two time periods, which subsequently affected O&M practices. However, the dredging and construction related changes reflected in the distributions are inner harbor depth increase from -38 ft to -42 ft between Stations 103+000 and 0+000, the widening of the channel from 400 ft to 500 ft between Stations 100+000 and 70+000, the enlargement of the Kings Island Turning Basin from 900 x 1,000 ft to 1,500 x 1,600 ft and the operation of the sediment basin. The tidegate and New Cut were not

operational during either the 1970-1975 or the 1997-2004 time periods. Despite changes in dredging related to these activities, there are no changes to the total volume dredged or to the shoaling distribution that can be attributed to the 1994 channel deepening and widening. See **Figure 10.3.1-1**.

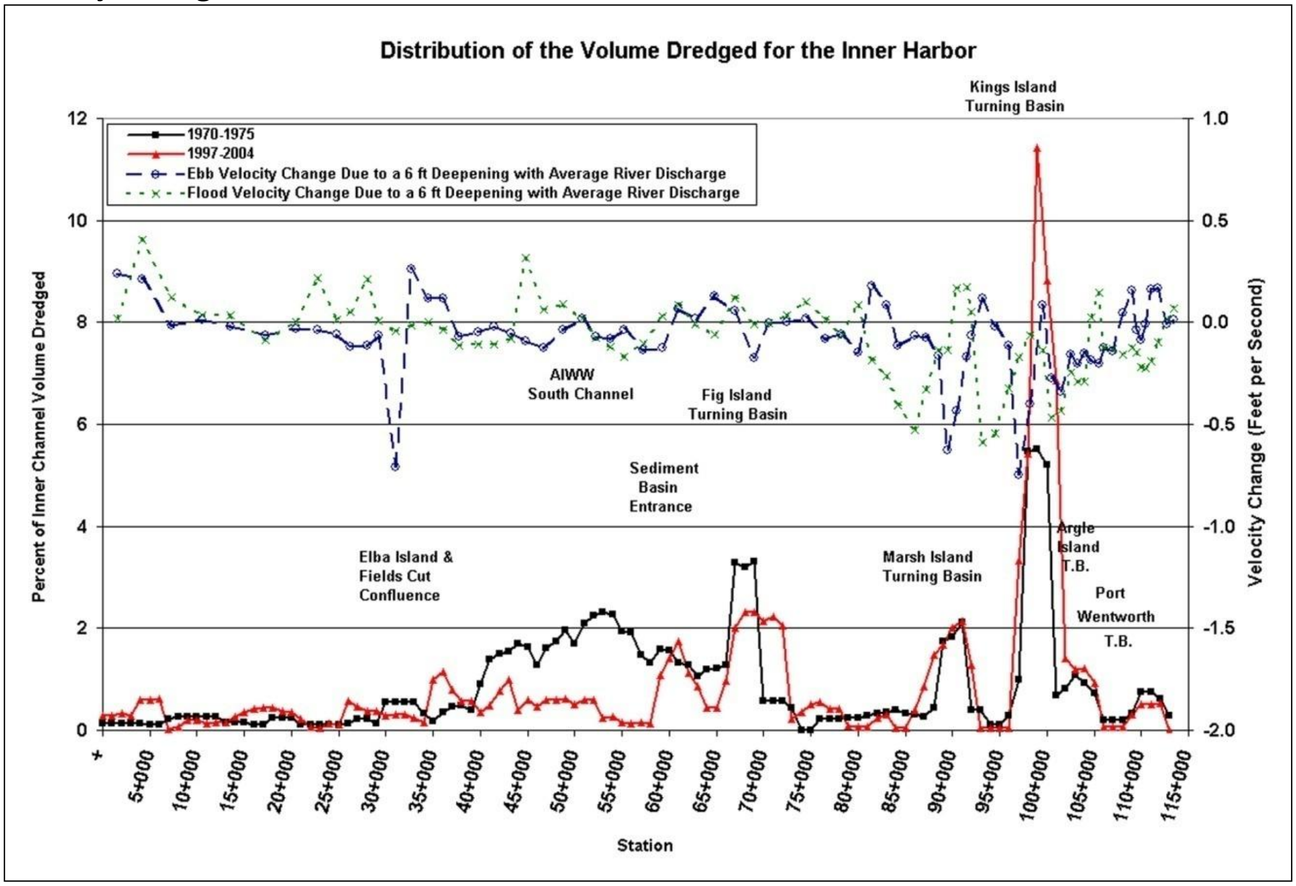
Figure 10.3.1-1: Distribution of the Volume Dredged for the Inner Harbor



Model Predicted Velocities as an Indicator – To determine if proposed depth increases would change the shoaling pattern in the river, the EFDC model was run for low, average and high flow conditions with existing project depths, a 3-foot depth increase and a 6-foot depth increase. The low flow runs used historic river discharges starting on April 1, 1999 and ran for 214 days. The average flow runs used historic river discharges starting on August 1, 1997 and ran for 91 days. The high flow runs used historic river discharges starting on July 1, 1998.

There are specific locations where velocity changes occur as a result of the proposed deepening. The velocity changes due to a 6-foot deepening with average flow conditions best represents the locations where changes occur. Velocity changes are predicted at locations where there is high shoaling already occurring and no shift in shoaling pattern is predicted with two possible exceptions. See **Figure 10.3.1-2**. The first exception is a small shoal at Station 35+000. Based on the predicted ebb velocity changes, this shoal may shift toward Station 31+000. The second exception is the spreading out of the Marsh Island turning basin shoal based on predicted flood velocity changes. If these exceptions did occur, they would cause small changes in the shoaling distribution, but the response of the volume-dredged distribution to the last deepening in 1994 indicates that these changes will not occur.

Figure 10.3.1-2: Distribution of the Volume Dredged for the Inner Harbor with Model Predicted Velocity Changes



Tidal Range as an Indicator – Another indication that there will not be major flow induced changes due to potential channel deepening is that the past depth increases have improved the conveyance of the channel to the point where a full tidal range is presently moving up the channel to the upstream end of the harbor. The mean tidal range at the entrance to Savannah Harbor, Fort Pulaski, is 6.9 ft and the mean tidal range at the upstream end of the harbor is 7.0 ft at Port Wentworth. Additional deepening would not significantly affect the tidal flow or salinity related shoaling that is associated with the location of no net bottom flow.

Since 1) there were no changes to the shoaling volume or distribution attributed to the last channel deepening, 2) the channel already captures essentially all of the sediment that enters the harbor, and 3) the future depth increases will extend along the existing channel side slopes which will decrease the bottom width of the channel, no major change to the shoaling volume or distribution is predicted. The one small exception is the meeting area extending into an existing shoal upstream of the sediment basin entrance.

10.3.2 Shoaling Response to Meeting Areas

In addition to deepening, two meeting areas are included in the project design. **Figures 10.3.2-1** and **10.3.2-2** show the location of the Oglethorpe and Long Island meeting areas. In these areas, shoaling is anticipated to shift from the adjacent channel into the meeting area, particularly in the Oglethorpe meeting area where a large amount of material currently shoaling in the Sediment Basin is predicted to move.

Figure 10.3.2-1: Oglethorpe Meeting Area Location

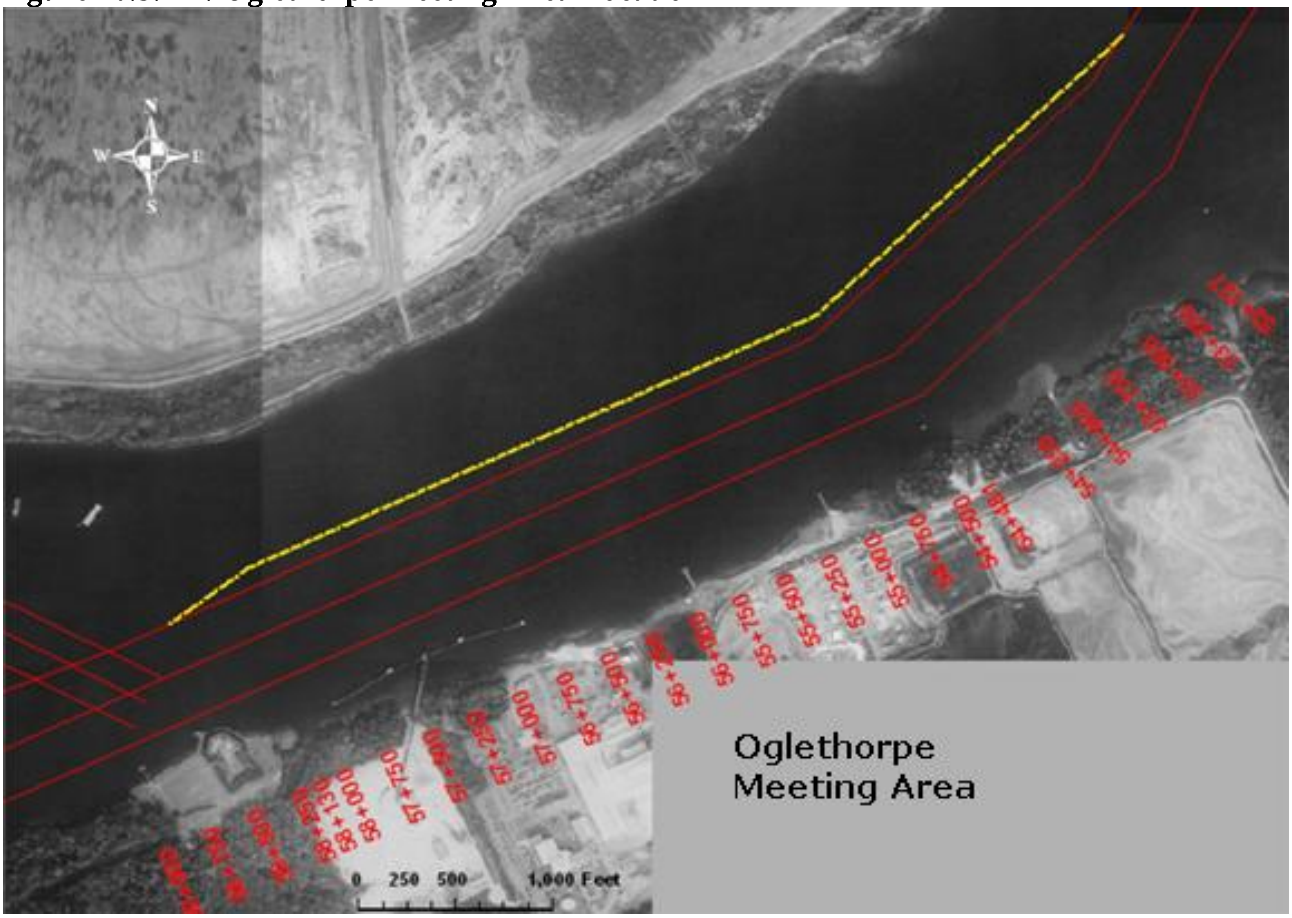
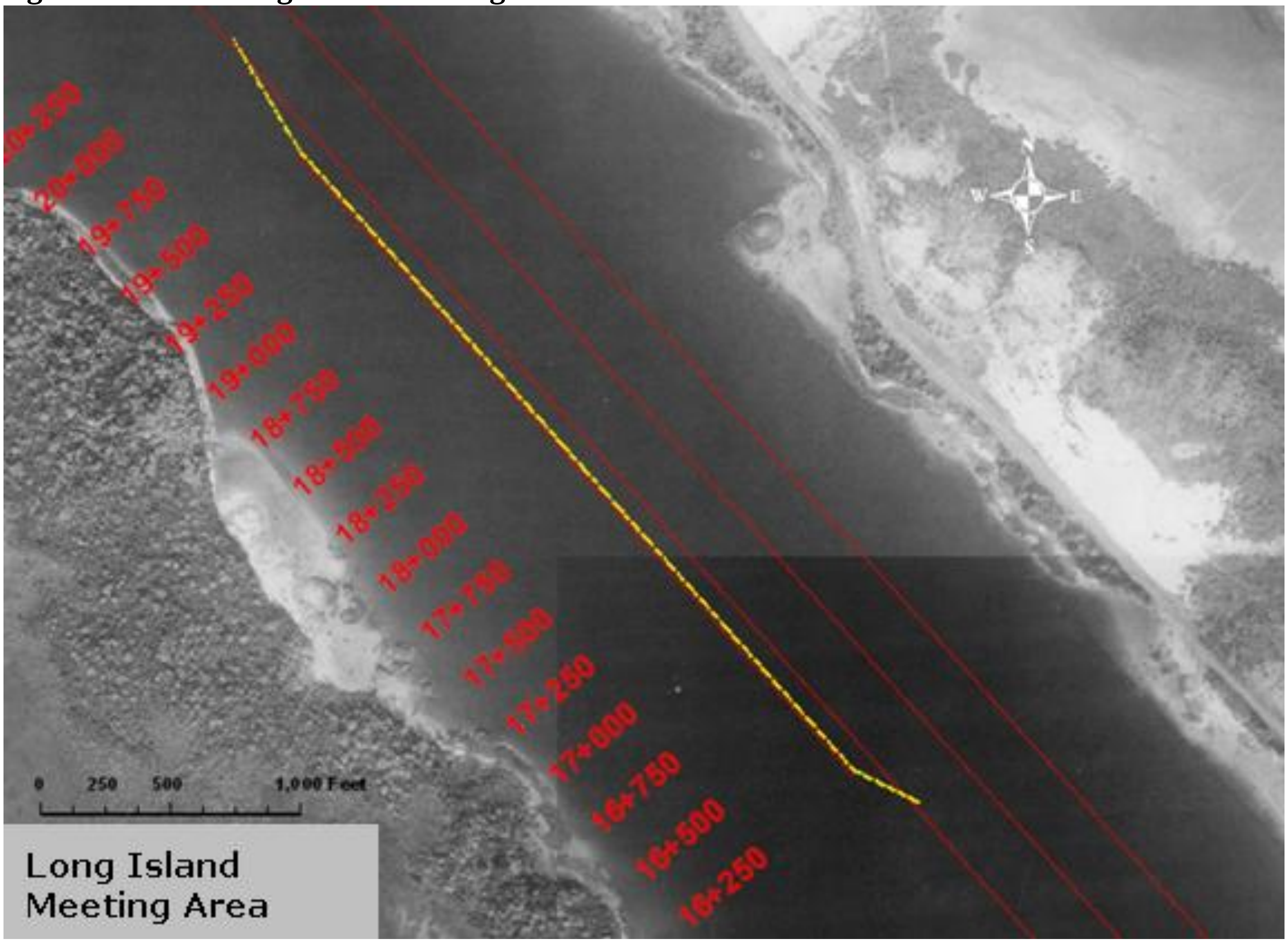


Figure 10.3.2-2: Long Island Meeting Area Location



10.3.3 Shoaling Response to Mitigation Features

Features of mitigation Plan 6a were evaluated for sedimentation impacts were evaluated from two perspectives 1) the shoaling effects resulting from the discontinued use of the sediment basin as an O&M feature and 2) the overall salinity regime changes in Front River associated with implementing all of the mitigation features included in Plan 6a.

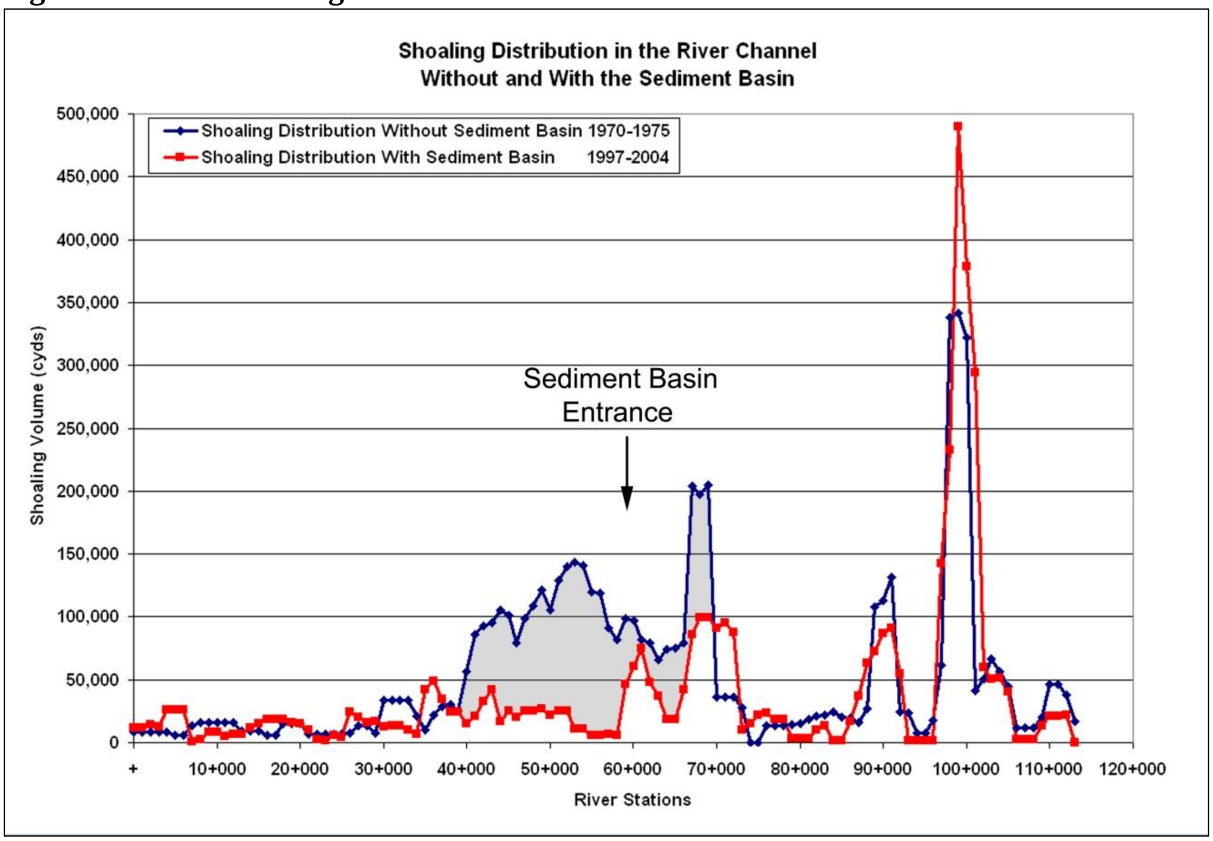
10.3.3.1 Discontinued Use of the Sediment Basin

The sediment basin is an O&M feature, located at the mouth of Back River, which allows for cost effective dredging maintenance of the channel. Details of this feature regarding O&M practices are outlined in Section 3. With regards to sedimentation and shoaling, an examination of sediment basin efficiency both currently and with the proposed channel depth increases was evaluated for this project and are documented in the report titled *Sedimentation Analysis* which is included in the Engineering Investigations Supplemental Materials.

Despite the sediment basin's usefulness as an O&M feature, discontinuing its use is part of the proposed mitigation plan. The sediment basin currently acts as a trap for sediments but also traps salinity which, due to the depths in the basin, can more readily move upstream through the tidegate into the Back River system. This is especially true during dry times when freshwater inflows coming downstream are low. By discontinuing the use of the sediment basin as a maintenance feature and allowing it to fill naturally, the current upstream elevation on Back River could be extended through the sediment trap to the confluence with Front River. The modeling results show that filling the basin reduces salinity concentrations upstream in Back River by allowing more mixing and flushing of the area on each tidal cycle. Also, salinity concentrations are reduced by limiting the interaction between the Back River and the lowest portions of the water column on Front River that have the highest concentrations of salinity.

If the use of the sediment basin is discontinued, the sediment that is annually trapped in the sediment basin will begin to settle in the river channel in a pattern similar to that which occurred before the construction of the sediment basin. The river channel shoaling distributions, before and after construction of the sediment basin, are plotted in **Figure 10.3.3-1**. After construction of the sediment basin, the river channel shoaling volume between Stations 40+000 and 69+000 was reduced by 2,050,000 cubic yards. The 1,906,000 cubic yards that shoaled in the sediment basin, which is adjacent to the reach of river between Stations 40+000 to 69+000, was responsible for the majority of the shoaling reduction in the river channel. The remainder of shoaling reduction, between Stations 40+000 and 69+000, is accounted for in the enlarged turning basins upstream.

Figure 10.3.3-1: Shoaling Distribution Without and With the Sediment Basin



If the sediment basin is not maintained, the sediment that would have settled in the sediment basin will now be deposited in the river channel between Stations 40+000 and 69+000. The distribution is shown in **Table 10.3.3.1-1**.

Table 10.3.3.1-1: Channel Shoaling Increases Due to Discontinued Use of the Sediment Basin

Station	Long Term Shoaling Average cy Existing Conditions	Long Term Shoaling Average cy Without Sediment Basin	Station	Long Term Shoaling Average cy Existing Conditions	Long Term Shoaling Average cy Without Sediment Basin
0	11,866	11,866	38+000	24,392	24,392
1+000	11,866	11,866	39+000	24,867	24,867
2+000	13,946	13,946	40+000	15,157	50,776
3+000	12,266	12,266	41+000	21,054	78,654
4+000	25,921	25,921	42+000	32,835	79,121
5+000	25,921	25,921	43+000	42,262	77,469
6+000	26,184	26,184	44+000	17,155	100,139
7+000	1,026	1,026	45+000	25,345	91,506
8+000	2,445	2,445	46+000	20,211	71,338
9+000	8,337	8,337	47+000	25,524	89,525
10+000	8,160	8,160	48+000	25,158	99,451
11+000	5,337	5,337	49+000	26,665	111,788
12+000	6,781	6,781	50+000	22,060	97,610
13+000	6,888	6,888	51+000	25,242	120,877
14+000	11,533	11,533	52+000	25,242	131,767
15+000	14,790	14,790	53+000	10,667	142,640
16+000	18,308	18,308	54+000	11,369	139,781
17+000	18,756	18,756	55+000	6,245	119,986
18+000	18,764	18,764	56+000	5,751	119,605
19+000	15,669	15,669	57+000	6,683	90,569
20+000	15,186	15,186	58+000	6,011	81,041
21+000	9,740	9,740	59+000	46,565	79,048
22+000	2,516	2,516	60+000	60,347	70,804
23+000	1,858	1,858	61+000	74,865	74,865
24+000	6,023	6,023	62+000	47,994	57,927
25+000	4,572	4,572	63+000	37,084	49,992
26+000	24,413	24,413	64+000	18,925	67,354
27+000	19,840	19,840	65+000	18,679	68,045
28+000	16,054	16,054	66+000	42,012	60,960
29+000	16,521	16,521	67+000	85,725	168,061
30+000	12,758	12,758	68+000	99,524	154,911
31+000	13,158	13,158	69+000	99,524	162,270
32+000	13,158	13,158	70+000	91,460	91,460
33+000	10,511	10,511	71+000	95,409	95,409
34+000	6,595	6,595	72+000	87,680	87,680
35+000	42,128	42,128	73+000	9,810	9,810
36+000	49,315	49,315	74+000	14,794	14,794
37+000	34,386	34,386	75+000	21,570	21,570

Table 10.3.3.1-1: Channel Shoaling Increases Due to Discontinued Use of the Sediment Basin (continued)

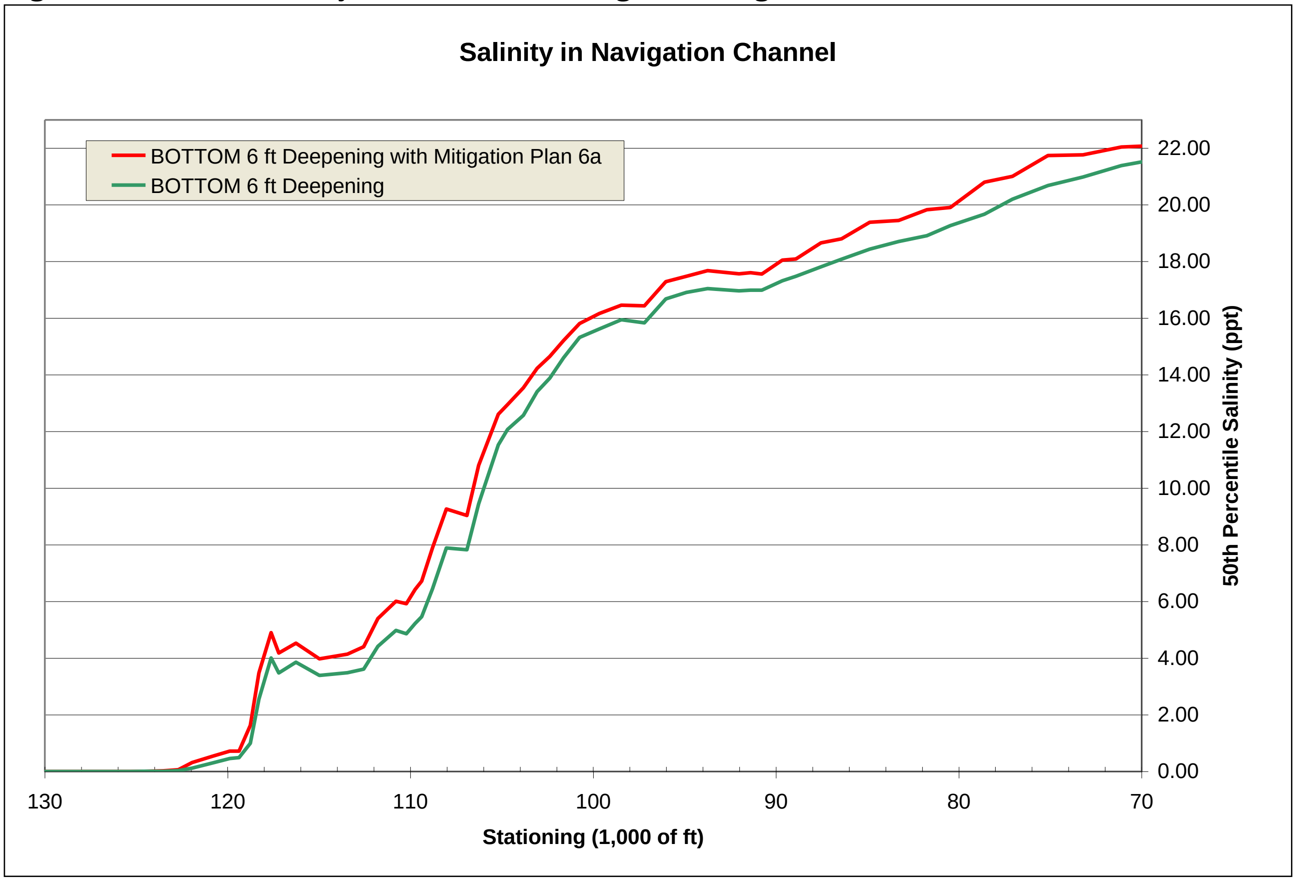
Station	Long Term Shoaling Average cy Existing Conditions	Long Term Shoaling Average cy Without Sediment Basin
76+000	23,825	23,825
77+000	18,418	18,418
78+000	18,418	18,418
79+000	3,746	3,746
80+000	2,959	2,959
81+000	2,959	2,959
82+000	9,995	9,995
83+000	13,606	13,606
84+000	1,586	1,586
85+000	1,586	1,586
86+000	16,780	16,780
87+000	36,899	36,899
88+000	63,586	63,586
89+000	72,272	72,272
90+000	86,497	86,497
91+000	91,420	91,420
92+000	54,434	54,434
93+000	2,050	2,050
94+000	2,050	2,050
95+000	2,050	2,050
96+000	1,693	1,693
97+000	142,429	142,429
98+000	232,556	232,556
99+000	489,662	489,662
100+000	378,601	378,601
101+000	293,983	293,983
102+000	60,257	60,257
103+000	50,618	50,618
104+000	51,599	51,599
105+000	40,057	40,057
106+000	2,684	2,684
107+000	2,684	2,684
108+000	2,684	2,684
109+000	13,305	13,305
110+000	21,336	21,336
111+000	21,336	21,336
112+000	22,334	22,334

10.3.3.2 Implementation of Mitigation Plan 6a

To predict if there will be a shift of the shoaling distribution due to the implementation of Plan 6a, the numerically modeled shift of the salinity distribution associated with the implementation of the mitigation Plan 6a was compared to the measured shift of the salinity distribution and the corresponding shoaling shift in a physical modeled of a deepened Savannah River. The use of a shift in the salinity distribution as an indicator of a shift in the shoaling distribution is valid since not only is the shoaling process sensitive to salinity concentrations, but the hydrodynamics that affect the shoaling distribution also affect the salinity concentration.

The salinity distributions, as predicted by the three-dimensional hydrodynamic model (EFDC), are shown in **Figure 10.3.3.2-1** for the 6ft deepening with and without Plan 6a. The average shift in the 10 ppt concentration is on the order of 500 ft. The predicted salinity shift is much less than the physical model salinity shift which did not produce a significant change in the shoaling distribution. Based on the small predicted change in the salinity distribution for Plan 6a, implementation of Plan 6a will not change the shoaling distribution from the 6 ft deepening. The predicted salinity changes for the other deepening alternatives are less than what is represented in **Figure 10.3.3.2-1**; therefore, they also will not affect the without mitigation shoaling distribution.

Figure 10.3.3.2-1: Salinity Distributions Along the Navigation Channel



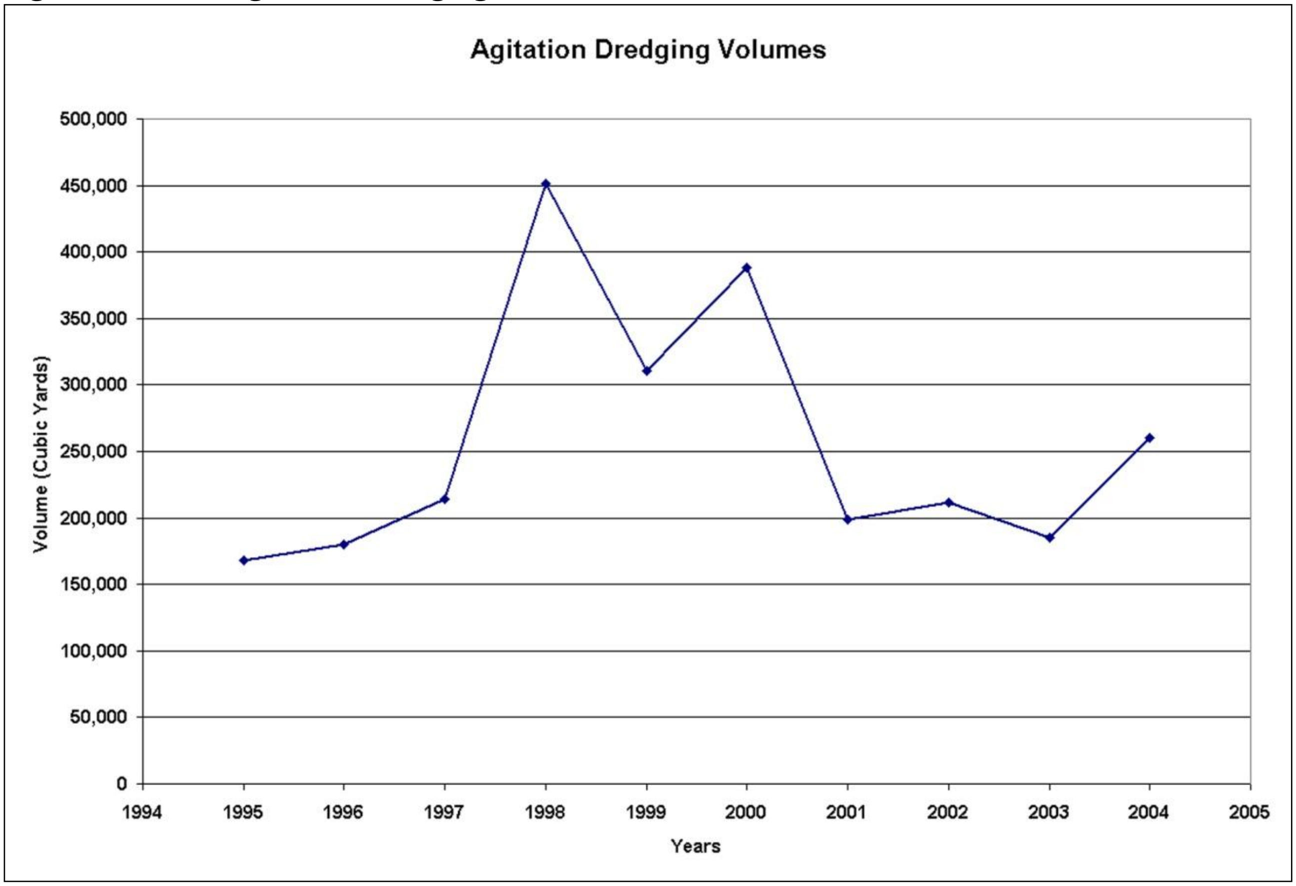
10.3.4 Shoaling Response and Berth Maintenance

The berths in Savannah Harbor are maintained by agitation dredging. **Table 10.3.4-1** lists the agitation dredging permit holders and their approximate location. Records of the agitation volumes go back to 1995. The total volume removed by agitation is plotted in **Figure 10.3.4-1** for the period 1995 to 2004. While the existing shoaling pattern is not predicted to change with project depth increase, the mitigation plan will cause a shift in shoaling in the harbor. By allowing the sediment basin to fill, the material that would have settled in the basin will now be deposited in the river channel between Stations 40+000 and 69+000. Berths falling within this area may experience an increase in shoaling. However, it is expected that the shoaling will be concentrated in the deeper channel adjacent to the berths.

Table 10.3.4-1: Agitation Dredging Permit Holders

Permit Holders	Approximate Channel Station
S.T. Services	60+500
Conoco Phillips	61+000
S.T. Services Dock 2	62+300
G P Gypsum	63+600
East Coast Term.	68+450
Ga. Ports Auth. O.T.	78+000-82+000
Colonial Oil (Plant 1)	83 + 424
Gobal Ship Systems	84+000
Colonial Oil (Plant 2)	85 + 594
International Paper	88+500
Citgo	90+000
Colonial Ga. Kaolin	91+000
Conbulk Mar. Term. (S. Bulk)	91+872
Ga. Ports Auth. G.C.	92+000-102+000
Savannah Sugar	104+100
G.P.A. Berth 7	109+000

Figure 10.3.4-1: Agitation Dredging Volumes



10.3.5 Shoaling Response to Entrance Channel Depth and Length Increases

At the present project depth of 44 ft, the entrance channel is a sediment sink, which is a total interdiction of the littoral transport. Increases in depth will not increase the channel's ability to capture sediment. The average annual shoaling volume record did show an apparent increase in shoaling after the last deepening, but this was due to the short post-deepening record not including both a high and low shoaling period as did the pre-deepening record. A small volume increase is predicted based on an increase in channel length.

The entrance channel is presently 42 ft deep and 500 ft wide from Station 0+000 to -14+000. See **Figure 10.3.5-1**. From Station -14+000 to -60+000, the channel is 44 ft deep and 600 ft wide. The entrance channel is a trap for all of the sediment that is transported to it. To substantiate that the entrance channel is a sediment sink, the depth of closure, or the depth beyond which the bottom doesn't change with storms, was calculated.

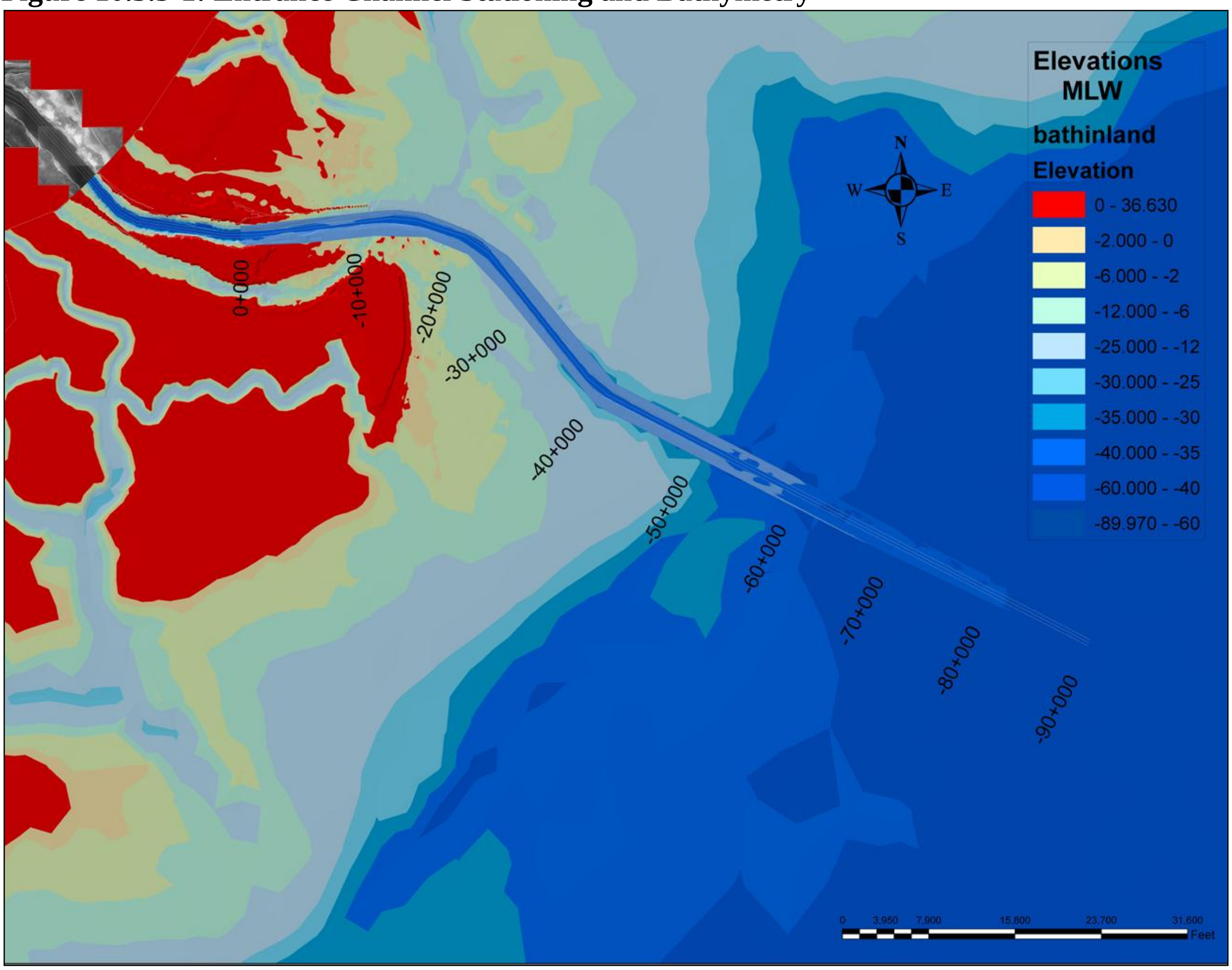
$h_c = 2\,H + 11\,\sigma_H$

where:
h_c is the depth of closure
H is the annual mean significant wave height
σ_H is the standard deviation of significant wave height

Using, in the above equation, an annual mean significant wave height of 3.28 ft and a standard deviation of 1.64 ft from Station 368 of the Wave Information Study database, http://frf.usace.army.mil/wis, produces a depth of closure of 24.6 ft. As will be shown in the following section on advance maintenance, the shoals in the entrance channel rarely rise above elevation -40 ft, MLLW. Sediment that is presently transported into the entrance channel remains there until it is dredged, and an increase in depth will not make the entrance channel a more effective trap. The shoaling in the entrance channel is a function of the amount of sand transported to it, which is then trapped in it.

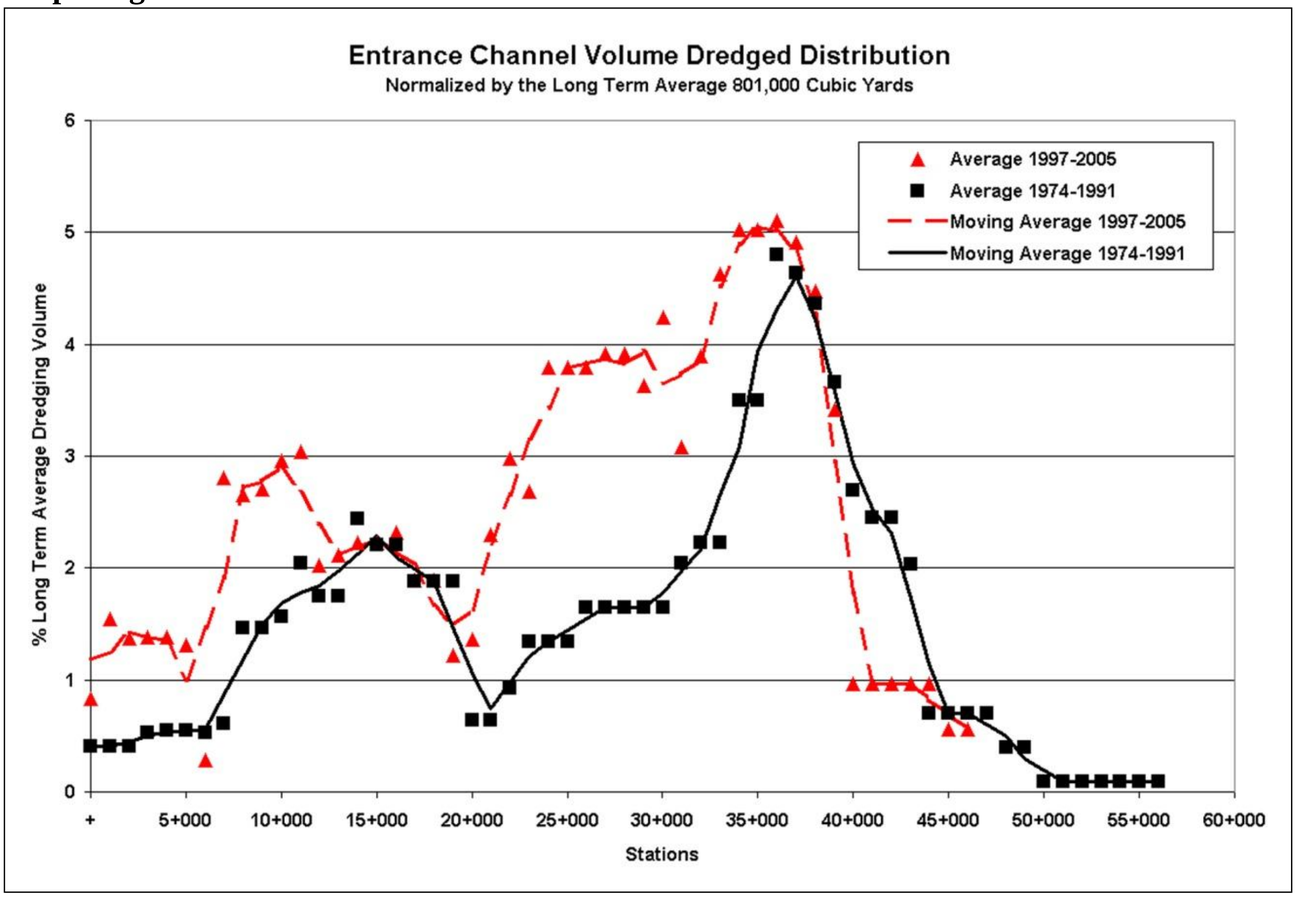
Another indication that the entrance channel is a total interdiction of the sediment entering the inlet environment is that the entrance channel completely cuts through the inlet's ocean bar. The ocean bar is the end product of the integrated effects of tidal currents, wave action and the associated sediment transport and deposition. Channel depths deeper than the depth at which the seaward tip of the ocean bar meets the offshore sea bottom will cause the channel to be a total interdiction of the littoral drift. The ocean bar does not extend beyond the 30 to 35 foot depth band. This is in agreement with the shoaling distribution along the entrance channel, which has virtually no shoaling beyond Station -50+000. See **Figure 10.3.5-1**.

Figure 10.3.5-1: Entrance Channel Stationing and Bathymetry



Maintenance dredging records for the entrance channel were available for the period 1974-2005. Dredging volumes for the entrance channel were also available from annual reports for the period 1975-2002. The data from the dredging records reflect the total volume dredged in a calendar year. The annual reports give the volume dredged that was paid for in a fiscal year. There is a general agreement between the two data sets, but the dredging record data appears to be missing several years. Both data sets show an increase in dredging of over 100,000 cubic yards from the pre- to post-1994 deepening periods. See **Figure 10.3.5-2**. The explanation for the apparent shoaling increase is due the difference in length of records of the pre- and post-deepening periods. The pre-deepening period is 19 years long and contains a cycle of both high and low shoaling periods. The post-deepening period is 8 years and contains only a high shoaling period with shoaling magnitudes comparable to those of the pre-deepening high shoaling period.

Figure 10.3.5-2: Entrance Channel Volume Dredged Distribution Pre- and Post- the 1994 Deepening



The entrance channel will be lengthened as a result of deepening. The channel length increases beyond Station -60+000 are shown in **Table 10.3.5-1** for a range of channel depths. The average annual volume dredged from the entrance channel reach between Stations -50+000 and -60+000 is 300 cubic yards per 1,000 ft of channel. Applying this rate for the channel increases gives the volume increases shown in **Table 10.3.5-1** for each incremental depth.

Table 10.3.5-1: Entrance Channel Extension and Estimated Annual Maintenance

SHEP Entrance Channel Depth Alternative (ft below MLLW)	Length (Ft)	Length (Mi)	Estimated Annual Maintenance (Cy)
46	35,700	6.8	10,710
47	36,900	7.0	11,070
48	37,500	7.1	11,250
49	37,700	7.1	11,310
50	38,600	7.3	11,580

10.3.6 Shoaling Response and Advance Maintenance Features

Advance maintenance is authorized for Savannah Harbor to reduce the overall maintenance costs by decreasing the frequency of dredging. Dredging frequency is determined based on monthly project condition surveys. The condition surveys are taken along the four centerlines of the channel's quarters. When a shoal two ft or more above the project depth occurs in two adjacent quarters, a contractor is directed to remove the shoal. Analysis of the minimum depths shown on condition surveys taken between January 1997 and January 2005 indicate the existing advance maintenance areas, with the exception of Kings Island range, are providing acceptable navigation depths. The shoaling pattern is not predicted to change with future depth increases and the present advance maintenance areas do not need to be shifted due to future depth increases.

However, as discussed previously, discontinued use of the sediment basin, a mitigation feature under consideration to reduce the salinity moving up Back River, will cause changes in the shoaling pattern. The maximum shoal increases along the centerline of each quadrant for each reach, in the affected area, was identified and are listed in **Table 10.3.6-1**. Further analysis of these predicted shoaling amounts resulted in the associated advance maintenance depth increases shown in **Table 10.3.6-1**. The current advance maintenance relative to project depth in the Bight Channel Range will be adequate without the operation of the sediment basin. The other three ranges impacted have additional advance maintenance depth increases of 2 or 4 ft.

Figures 10.3.6-1 to **10.3.6-3** illustrate the predicted shoal increase above the project depth. The area where the advance maintenance may need to be increased is shown as a shaded area.

The predicted shoaling changes may necessitate changes to the advance maintenance program. However, the future shoaling cannot be predicted precisely. By allowing the harbor to stabilize after SHEP construction, the shoaling patterns would likely be established, and at that point, a determination can be made to adjust the advance maintenance program to benefit O&M practices. If that determination is made, approvals to implement modifications to the program will be initiated through the normal business process.

Table 10.3.6-1: Maximum Shoal Increases of Each Channel Quadrant & Advance Maintenance Increases

	Left Outside Quadrant (ft)	Left Inside Quadrant (ft)	Right Inside Quadrant (ft)	Right Outside Quadrant (ft)	Advance Maintenance Increase (ft)
The Bight Channel 41+000 to 50+000	5.41	3.08	3.63	4.74	None
Ft. Jackson Range 50+000 to 54+000	6.90	5.06	7.51	10.24	2
Oglethorpe Range 54+000 to 61+000	7.28	6.09	7.81	9.36	4
Wrecks Channel 61+000 to 70+000	8.41	4.61	3.63	3.58	2

Figure 10.3.6-1: Fort Jackson Range Predicted Shoal Thickness Above Project Depth

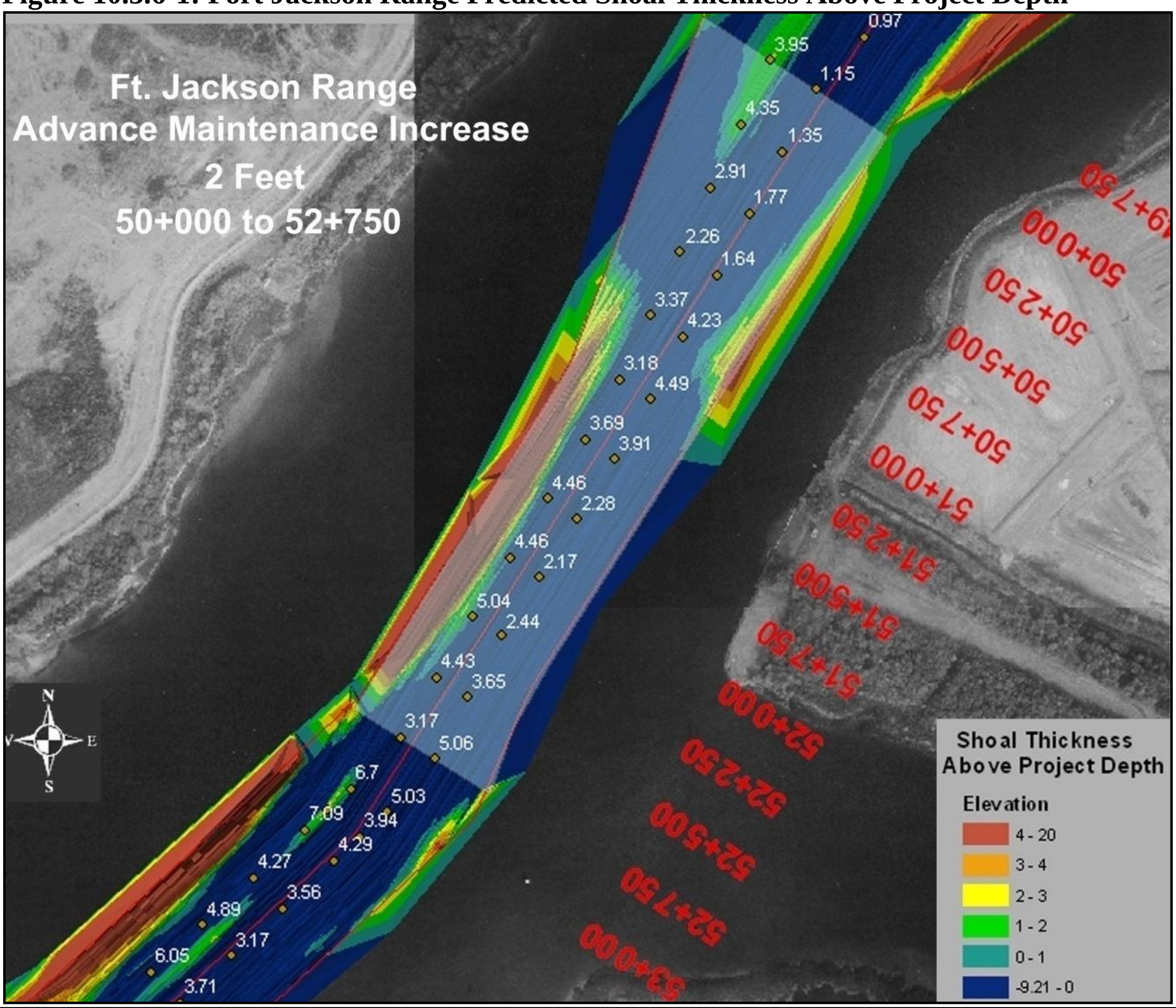


Figure 10.3.6-2: Oglethorpe Range Predicted Shoal Thickness Above Project Depth

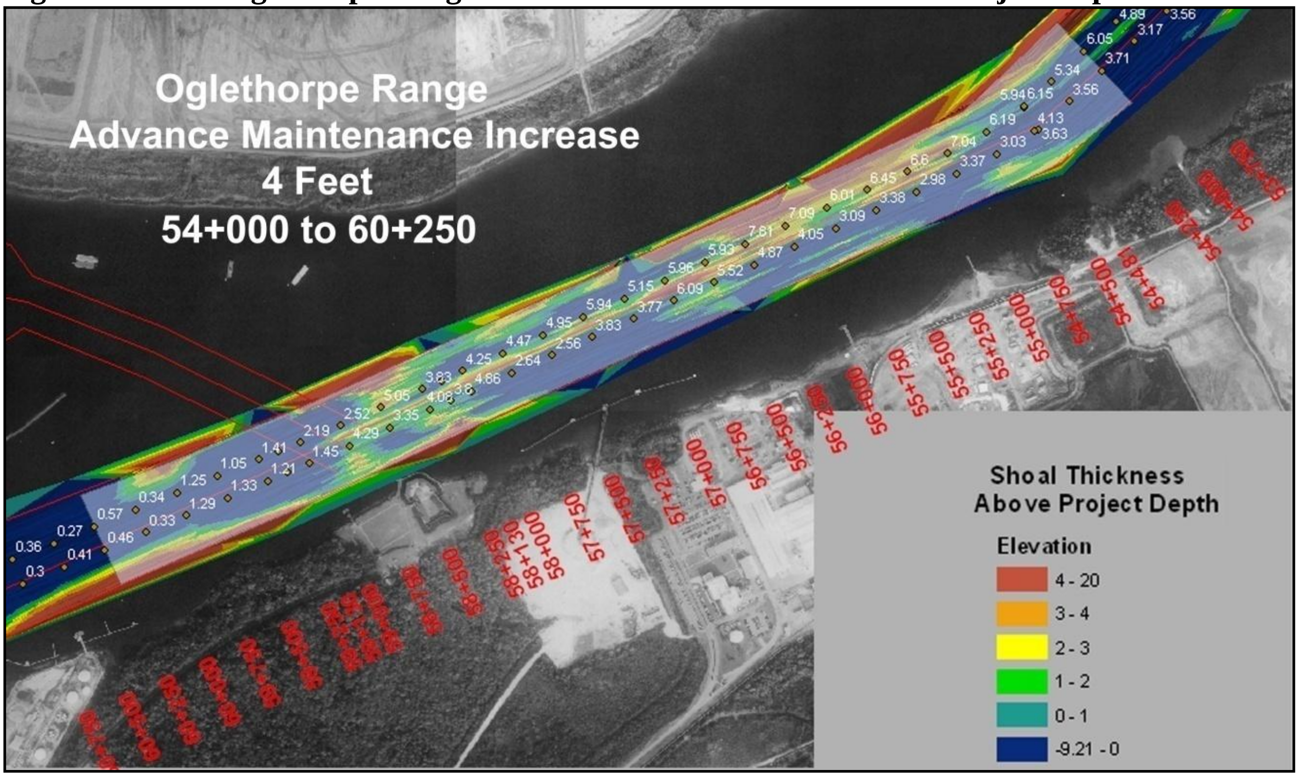
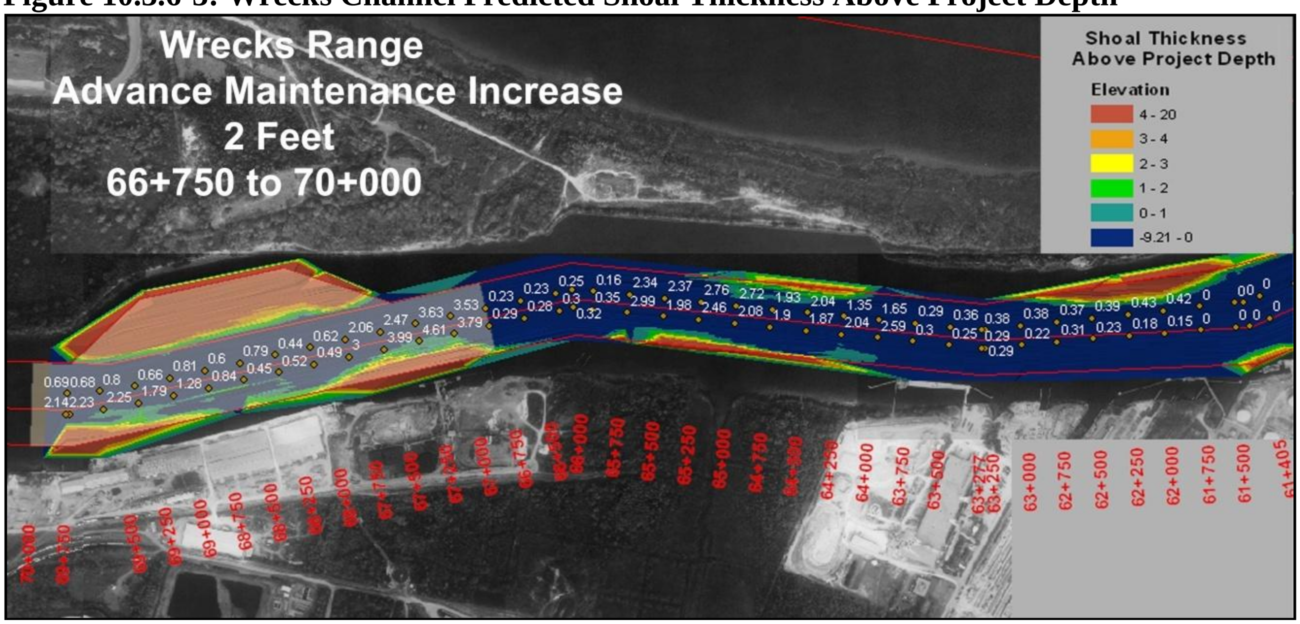


Figure 10.3.6-3: Wrecks Channel Predicted Shoal Thickness Above Project Depth



10.3.7 Shoaling Response and Lazaretto Creek

Lazaretto Creek is located near the mouth of the Savannah River in the northeastern section of Chatham County, GA. It connects the South Channel of the Savannah River with the Tybee Creek. The northern entrance to Lazaretto Creek is across from the Fort Pulaski National Monument on Cockspur Island and the south entrance is opposite the Shad River on Wilmington Island. Lazaretto Creek is approximately 7 miles long and is surrounded by saltwater marsh. There are commercial fishing and recreational boating docks on Lazaretto Creek at its northern end, on both sides of the Highway 80 Bridge. See **Figure 10.3.7-1**.

The South Channel is incorporated within the hydrodynamic model grid. Velocity comparisons with and without SHEP were made on South Channel at the entrance to Lazaretto Creek to determine if the SHEP impacts shoaling in this area. Model output indicates no major velocity or current pattern changes within this area; therefore, shoaling patterns are not expected to change from the current condition.

This area is not typically surveyed as part of the Federal navigation project. However, hydrographic surveys taken in 1997 indicate depths at the mouth of Lazaretto Creek at the South Channel confluence range from 7.5 ft below MLLW to 32.7 ft below MLLW. Additional surveys within this area taken in 2006 do not indicate major shoaling shifts.

Figure 10.3.7-1: Lazaretto Creek

10.4 CONCLUSIONS

- The Savannah Harbor Project captures all of the sediment that enters the harbor. The last channel deepening did not change the inner harbor shoaling volume, and future depth increases are predicted not to increase the shoaling volume.

- The entrance channel is a sediment sink that is a total interdiction of the littoral transport. Increases in depth will not increase the channel's ability to capture sediment.

- Based on the small predicted change in the salinity distribution for Plan 6a, implementation of Plan 6a will not change the shoaling distribution from the 6 foot deepening plan without mitigation.

- The existing advance maintenance areas, with the exception of the Kings Island range, are generally allowing an annual maintenance cycle without unacceptably encroaching above the authorized channel depth.

- If the operation of the sediment basin is discontinued, the advance maintenance depth in sections of the Fort Jackson Range, the Oglethorpe Range, and the Wrecks Channel Range may need to be deepened 2 to 4 ft.

- Shoaling is anticipated to shift from the adjacent channel into the proposed meeting areas.

11.0 DREDGING AND DISPOSAL OF NEW WORK MATERIAL

The following sections summarize the *Dredged Material Management Plan* (DMMP) for SHEP. The full DMMP can be found in the Engineering Investigations Supplemental Materials.

11.1 INNER HARBOR

11.1.1 Inner Harbor Dredging Volumes

Table 11.1-1 summarizes the volume of material to be dredged from the inner harbor for each depth alternative based on the recommended channel alignment including meeting areas, bend wideners, currently authorized advance maintenance, and over-depth.

Table11.1-1: New Work Inner Harbor Dredging Volumes

Stations	44 ft Project (cy)	45 ft Project (cy)	46 ft Project (cy)	47 ft Project (cy)	48 ft Project (cy)
4+000 to 6+375	48,128	87,346	130,559	174,073	217,263
6+375 to 30+000	913,871	1,372,897	1,840,479	2,308,262	2,775,041
30+000 to 45+000	684,583	1,052,928	1,426,462	1,802,866	2,181,609
45+000 to 51+000	324,752	508,740	699,013	892,307	1,088,128
51+000 to 57+000	464,032	602,960	743,122	883,874	1,025,450
57+000 to 67+000	415,297	621,905	828,620	1,035,585	1,242,896
67+000 to 80+125	444,210	691,727	944,611	1,196,291	1,446,786
80+125 to 90+000	380,724	570,368	759,169	946,436	1,132,066
90+000 to 103+000	1,438,457	1,803,823	2,169,594	2,533,434	2,895,175
Channel Subtotal (cy)	5,114,054	7,312,694	9,541,629	11,773,128	14,004,414
Long Island Meeting Area	350,859	384,096	417,783	450,941	484,231
Oglethorpe Meeting Area	362,348	384,089	405,461	426,336	446,541
Meeting Area Subtotal	713,207	768,185	823,244	877,277	930,772
Total New Work (cy)	5,827,261	8,080,879	10,364,873	12,650,405	14,935,186

11.1.1.1 Cadmium Laden Dredged Material

Sediment testing and analyses in 1997 and 2001 for the Savannah Harbor Expansion Project indicated a potential for elevated levels of cadmium associated with the Miocene clay layer. Because of this potential, Phase 2 sediment testing in 2005 looked at the concentration and distribution of cadmium within new work sediments. Analysis of the sediment testing data resulted in the following recommendations:

- New work sediments from the reaches 6+375 to 45+000, 51+000 to 57+000 and 80+125 to 90+000 should be isolated within a CDF and capped/covered with sediment from another reach. The high cadmium sediments should not be disturbed further and should not be allowed to be later excavated and placed in any exposed upland area.

- Once a CDF is selected to receive the sediment, the high cadmium sediments should be pumped into the area first. Once placement of sediments from this reach is completed, markers should immediately be placed on the surface of the sediment, to allow easy determination of when the proper cap/cover depth has been attained. At least 1 foot of additional sediment from another reach should then be placed in the area as soon as practicable, but as part of the SHEP, to ensure minimal environmental impacts from birds feeding within the CDF. Due to expected variability in construction techniques, the project design will use 2 ft of capping/covering sediments.

The locations and volumes of material with elevated cadmium concentrations are shown in **Table 11.1.1.1-1**. Current plans call for using disposal facilities 14A and 14B for the placement of cadmium-laden material with the emphasis of placing as much material as possible in 14A. During the design phase further analysis could result in a consolidation of all the cadmium-laden sediments in 14A. The current, conservative placement plan for dredge material containing cadmium for each project depth can be found in **Tables 11.2-1** through **11.2-5** in the next Section.

Table 11.1.1.1-1: Reaches with Elevated Cadmium Levels

Station	Volume of Material (cy)
6+375 to 45+000	4,562,069
51+000 to 57+000	1,101,114
80+125 to 90+000	946,436
Total	6,609,619

11.1.2 Inner Harbor Disposal

Material dredged from the inner harbor will be placed in upland confined disposal sites. The sites designated for placement of material were based on availability due to scheduled dike raisings, drying phases during construction, and annual channel maintenance needs. The disposal facilities that will be used for the deepening project over the construction period are 12A, 14A and 14B and Jones/Oysterbed Island (JOI). Areas 2A and 13A will not be utilized to dispose of new work material. They will exclusively hold annual maintenance dredged material when not in a drying phase or undergoing dike raising. See **Tables 11.2-1** through **11.2-5** for calculated dredged quantities and designated disposal facilities for each project depth.

Table 11.2-1: 48 ft Project Depth Dredging Sequence (designating CDFs 14A and 14B for Cadmium)

Description	Inner Harbor Stations	Dredged Material (cy)	Confined Disposal Facility (CDF)	Notes
O&M	4+000 to 57+000	3,563,754	12A	
New Work	4+000 to 6+375	217,263	JOI	
New Work	6+375 to 30+000	3,259,272	14B	Cadmium
New Work	30+000 to 45+000	2,181,609	14A	Cadmium
New Work	51+000 to 57+000	1,251,494	14A	Cadmium
New Work	80+125 to 90+000	1,555,112	14A	Cadmium
New Work	45+000 to 51+000	1,088,128	12A	
New Work + O&M	57+000 to 67+000	2,631,872	12A	
New Work + O&M	67+000 to 80+125	2,387,645	12A	
New Work + O&M	90+000 to 93+280	1,259,825	12A	
New Work + O&M	93+280 to 98+140	1,860,000	14B	Cap for 14B
New Work + O&M	98+140 to 103+000	1,860,000	14A	Cap for 14A
Total		23,115,974		

Table 11.2-2: 47 ft Project Depth Dredging Sequence (designating CDFs 14A and 14B for Cadmium)

Description	Inner Harbor Stations	Dredged Material (cy)	Confined Disposal Facility (CDF)	Notes
New Work + O&M	4+000 to 6+375	248,815	JOI	
New Work + O&M	6+375 to 36+000	4,663,278	14B	Cadmium
New Work + O&M	36+000 to 45+000	1,825,726	14A	Cadmium
New Work + O&M	51+000 to 57+000	1,857,054	14A	Cadmium
New Work + O&M	80+125 to 90+000	1,369,482	14A	Cadmium
New Work + O&M	45+000 to 51+000	1,698,443	12A	
New Work + O&M	57+000 to 67+000	2,021,557	12A	
New Work + O&M	67+000 to 80+125	2,528,753	12A	
New Work + O&M	90+000 to 103+000	898,084	12A	
New Work + O&M	90+000 to 103+000	1,860,000	14B	Cap for 14B
New Work + O&M	90+000 to 103+000	1,860,000	14A	Cap for 14A
Total		20,831,192		

Table 11.2-3: 46 ft Project Depth Dredging Sequence (designating CDFs 14A and 14B for Cadmium)

Description	Inner Harbor Stations	Dredged Material (cy)	Confined Disposal Facility (CDF)	Notes
New Work + O&M	4+000 to 6+375	205,301	JOI	
New Work + O&M	6+375 to 30+000	2,972,789	14B	Cadmium
New Work + O&M	30+000 to 45+000	2,638,870	14A	Cadmium
New Work + O&M	51+000 to 57+000	1,707,141	14A	Cadmium
New Work + O&M	80+125 to 90+000	1,182,215	14A	Cadmium
New Work + O&M	45+000 to 51+000	1,505,149	12A	
New Work + O&M	57+000 to 67+000	2,214,851	12A	
New Work + O&M	67+000 to 80+125	1,865,100	12A	
New Work + O&M	90+000 to 103+000	534,244	12A	
New Work + O&M	90+000 to 103+000	1,860,000	14B	Cap for 14B
New Work + O&M	90+000 to 103+000	1,860,000	14A	Cap for 14A
Total		18,545,660		

Table 11.2-4: 45 ft Project Depth Dredging Sequence (designating CDFs 14A and 14B for Cadmium)

Description	Inner Harbor Stations	Dredged Material (cy)	Confined Disposal Facility (CDF)	Notes
New Work + O&M	4+000 to 6+375	162,088	JOI	
New Work + O&M	6+375 to 30+000	2,482,781	14B	Cadmium
New Work + O&M	30+000 to 45+000	2,254,075	14A	Cadmium
New Work + O&M	51+000 to 57+000	1,557,444	14A	Cadmium
New Work + O&M	80+125 to 90+000	993,414	14A	Cadmium
New Work + O&M	45+000 to 51+000	1,314,876	14B	Cap for 14B
New Work + O&M	57+000 to 67+000	545,124	14B	Cap for 14B
New Work + O&M	57+000 to 67+000	1,860,000	14A	Cap for 14A
New Work + O&M	67+000 to 80+125	1,563,391	12A	
New Work + O&M	90+000 to 103+000	3,888,473	12A	
Total		16,621,666		

Table 11.2-5: 44 ft Project Depth Dredging Sequence (designating CDFs 14A and 14B for Cadmium)

Description	Inner Harbor Stations	Dredged Material (cy)	Confined Disposal Facility (CDF)	Notes
New Work + O&M	4+000 to 6+375	122,870	JOI	
New Work + O&M	6+375 to 30+000	2,003,174	14B	Cadmium
New Work + O&M	30+000 to 45+000	1,873,074	14A	Cadmium
New Work + O&M	51+000 to 57+000	1,408,733	14A	Cadmium
New Work + O&M	80+125 to 90+000	803,770	14A	Cadmium
New Work + O&M	45+000 to 51+000	1,130,888	14B	Cap for 14B
New Work + O&M	57+000 to 70+000	729,112	14B	Cap for 14B
New Work + O&M	57+000 to 70+000	1,612,556	14A	Cap for 14A
New Work + O&M	70+000 to 73+100	247,444	14A	Cap for 14A
New Work + O&M	73+100 to 80+125	553,320	12A	
New Work + O&M	90+000 to 103+000	3,523,107	12A	
Total		14,008,048		

Table 11.2-6 details the dredge material for the 47 ft project depth along with the CDF capacity before and after the material is placed.

Table 11.2-6: CDF Capacity With & Without Project Dredging Operations

	Without Project		CDF Present Capacity (cy)	47 ft Project		CDF Present Capacity (cy)
Inner Harbor Stations	**O&M Material (cy)**	**Confined Disposal Facility (CDF)**		**Dredged Material (cy)**	**Confined Disposal Facility (CDF)**	
0+000 to 4+000	76,000	13B	11,900,000	-	-	
4+000 to 6+375	26,719	13B	11,900,000	248,815	JOI	7,200,000
6+375 to 36+000	605,781	13B	11,900,000	4,663,278	14B	9,100,000
36+000 to 45+000	406,500	13B	11,900,000	1,825,726	14A	6,200,000
51+000 to 57+000	622,800	13A	32,400,000	1,857,054	14A	6,200,000
80+125 to 90+000	318,633	13A	32,400,000	1,369,482	14A	6,200,000
45+000 to 51+000	553,800	13B	11,900,000	1,698,443	12A	24,000,000
57+000 to 67+000	0	13A	32,400,000	2,021,557	12A	24,000,000
67+000 to 80+125	1,038,000	13A	32,400,000	2,528,753	12A	24,000,000
90+000 to 103+000	641,700	13A	32,400,000	898,084	12A	24,000,000
90+000 to 103+000	1,706,067	13A	32,400,000	1,860,000	14B	9,100,000
90+000 to 103+000	229,000	2A	4,400,000	1,860,000	14A	6,200,000
Total	6,225,000			20,831,192		

SHEP will place 12.65 million cy of new work material in CDFs which were established for the placement of sediments for the Savannah Harbor Navigation Project. As part of that placement, the Expansion Project will replace the sediment storage capacity that it used to place the new work material.

The following tables (**Table 11.2-7** and **11.2-8**) detail the disposal area rotation for the annual maintenance material disposal plan for the inner harbor with and without-project for the 50 year project life. The schedule is based on the reaches (station to station) and quantities laid out in **Table 11.2-2** for the 47 ft project depth. Annual maintenance dredging will be going on in conjunction with the new work dredging and will not impact the required 20-year capacity as required by the DMMP. Any loss in capacity to new work material will be replaced by the new work project. Deposition of the new work sediments is shown beginning in 2014 and ending in 2016 (background color = orange).

Table 11.2-7: Inner Harbor New Work and Annual Maintenance Material Disposal Plan 2010 -2066 (47 ft MLLW Project Depth)

	Confined Disposal Area							TOTALS		
Fiscal Year*	2A	12A	13A	13B	14A	14B	J/O	O&M	New Work	O&M and New Work
2010	229,000	4,431,000	BUILD DIKE Breach X dike	782,500	BUILD DIKE	782,500	DRYING	6,225,000		
2011	229,000	DRYING	4,431,000	1,565,000	BUILD DIKE	DRYING	BUILD DIKE	6,225,000		
2012	229,000	DRYING	4,431,000	1,565,000	BUILD DIKE	BUILD DIKE	BUILD DIKE	6,225,000		
2013	229,000	BUILD DIKE	4,431,000	782,500	DRYING	DRYING	782,500	6,225,000		
2014	DRYING	4,828,500	DRYING	782,500	DRYING	2,299,000	248,815	5,611,000	2,547,815	8,158,815
2015	DRYING	2,323,084 1,685,000	DRYING	26,719	4,575,780	2,364,278	0	1,711,719	9,263,142	10,974,861
2016	DRYING	4,823,753 2,370,000	BUILD DIKE	DRYING	2,336,482 DRYING	1,860,000 DRYING	1,489,000	3,859,000	9,020,235	12,879,235
2017	FULL	DRYING	4,660,000	DRYING	DRYING	DRYING	1,565,000	6,225,000		
2018		DRYING	4,660,000	DRYING	BUILD DIKE	BUILD DIKE	1,565,000	6,225,000		
2019		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000		
2020		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2021		4,660,000	DRYING	DRYING	782,500	782,500	BUILD DIKE	6,225,000		
2022		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000		
2023		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2024		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2025		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000		
2026		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2027		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2028		4,660,000	DRYING	782,500	DRYING	DRYING	782,500	6,225,000		
2029		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2030		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2031		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000		
2032		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		

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Table 11.2-7: Inner Harbor New Work and Annual Maintenance Material Disposal Plan 2010 -2066 (47 ft MLLW Project Depth) (continued from previous page)

	Confined Disposal Area							TOTALS		
Fiscal Year*	2A	12A	13A	13B	14A	14B	J/O	O&M	New Work	O&M and New Work
2033		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2034		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000		
2035		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2036		DRYING	4,660,000	782,500	BUILD DIKE	DRYING	782,500	6,225,000		
2037		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000		
2038		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2039		4,660,000	DRYING	BUILD DIKE	782,500	782,500	DRYING	6,225,000		
2040		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000		
2041		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2042		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2043		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000		
2044		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2045		4,660,000	DRYING	DRYING	782,500	782,500	BUILD DIKE	6,225,000		
2046		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000		
2047		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2048		DRYING	4,660,000	782,500	DRYING	BUILD DIKE	782,500	6,225,000		
2049		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000		
2050		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2051		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2052		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000		
2053		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2054		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2055		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000		
2056		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2057		4,660,000	DRYING	BUILD DIKE	782,500	782,500	DRYING	6,225,000		

Continued on next page

Table 11.2-7: Inner Harbor New Work and Annual Maintenance Material Disposal Plan 2010 -2066 (47 ft MLLW Project Depth) (continued from previous page)

	Confined Disposal Area							TOTALS		
Fiscal Year*	**2A**	**12A**	**13A**	**13B**	**14A**	**14B**	**J/O**	**O&M**	**New Work**	**O&M and New Work**
2058		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000		
2059		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2060		DRYING	4,660,000	782,500	DRYING	BUILD DIKE	782,500	6,225,000		
2061		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000		
2062		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2063		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000		
2064		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000		
2065		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		
2066		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000		

Reach	**Maintenance Volume**	
0 to 50	1,565,000	JOI, 14B, 14A, 13B: Split Annual Volume in half to two available areas each year
50 to 102	4,431,000	12A, 13A: Use Area Available
102 to 112	229,000	2A or 12A, 13A: Use 2A when available
Total	6,225,000	

**Based on Federal Fiscal Year 1 October to 30 September.*

***2A will be full in FY 2017 and no longer available for use.*

Table 11.2-8: Inner Harbor Annual Maintenance Material Disposal Plan 2010 -2066 (Without Project Conditions)

Fiscal Year*	Confined Disposal Area							TOTAL O&M
	2A	12A	13A	13B	14A	14B	J/O	
2010	229,000	4,431,000	BUILD DIKE	782,500	BUILD DIKE	782,500	DRYING	6,225,000
2011	229,000	DRYING	4,431,000	1,565,000	BUILD DIKE	DRYING	BUILD DIKE	6,225,000
2012	229,000	DRYING	4,431,000	1,565,000	DRYING	DRYING	BUILD DIKE	6,225,000
2013	229,000	BUILD DIKE	4,431,000	782,500	DRYING	DRYING	782,500	6,225,000
2014	DRYING	4,660,000	DRYING	782,500	DRYING	BUILD DIKE	782,500	6,225,000
2015	DRYING	4,660,000	DRYING	782,500	BUILD DIKE	DRYING	782,500	6,225,000
2016	DRYING	4,660,000	BUILD DIKE	DRYING	DRYING	DRYING	1,565,000	6,225,000
2017	FULL	DRYING	4,660,000	DRYING	DRYING	DRYING	1,565,000	6,225,000
2018		DRYING	4,660,000	DRYING	DRYING	DRYING	1,565,000	6,225,000
2019		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000
2020		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2021		4,660,000	DRYING	DRYING	782,500	782,500	BUILD DIKE	6,225,000
2022		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000
2023		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2024		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2025		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000
2026		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2027		4,660,000	DRYING	DRYING	782,500	782,500	BUILD DIKE	6,225,000
2028		4,660,000	DRYING	782,500	DRYING	DRYING	782,500	6,225,000
2029		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2030		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2031		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000
2032		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2033		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2034		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000
2035		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000

Continued on next page

Table 11.2-8: Inner Harbor Annual Maintenance Material Disposal Plan 2010 -2066 (Without Project Conditions) (Continued)

Fiscal Year*	Confined Disposal Area							TOTAL O&M
	2A	12A	13A	13B	14A	14B	J/O	
2036		DRYING	4,660,000	782,500	BUILD DIKE	DRYING	782,500	6,225,000
2037		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000
2038		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2039		4,660,000	DRYING	BUILD DIKE	782,500	782,500	DRYING	6,225,000
2040		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000
2041		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2042		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2043		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000
2044		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2045		4,660,000	DRYING	DRYING	782,500	782,500	BUILD DIKE	6,225,000
2046		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000
2047		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2048		DRYING	4,660,000	782,500	DRYING	BUILD DIKE	782,500	6,225,000
2049		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000
2050		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2051		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2052		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000
2053		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2054		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2055		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000
2056		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2057		4,660,000	DRYING	BUILD DIKE	782,500	782,500	DRYING	6,225,000
2058		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000
2059		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2060		DRYING	4,660,000	782,500	DRYING	BUILD DIKE	782,500	6,225,000

Continued on next page

Table 11.2-8: Inner Harbor Annual Maintenance Material Disposal Plan 2010 -2066 (Without Project Conditions) (Continued)

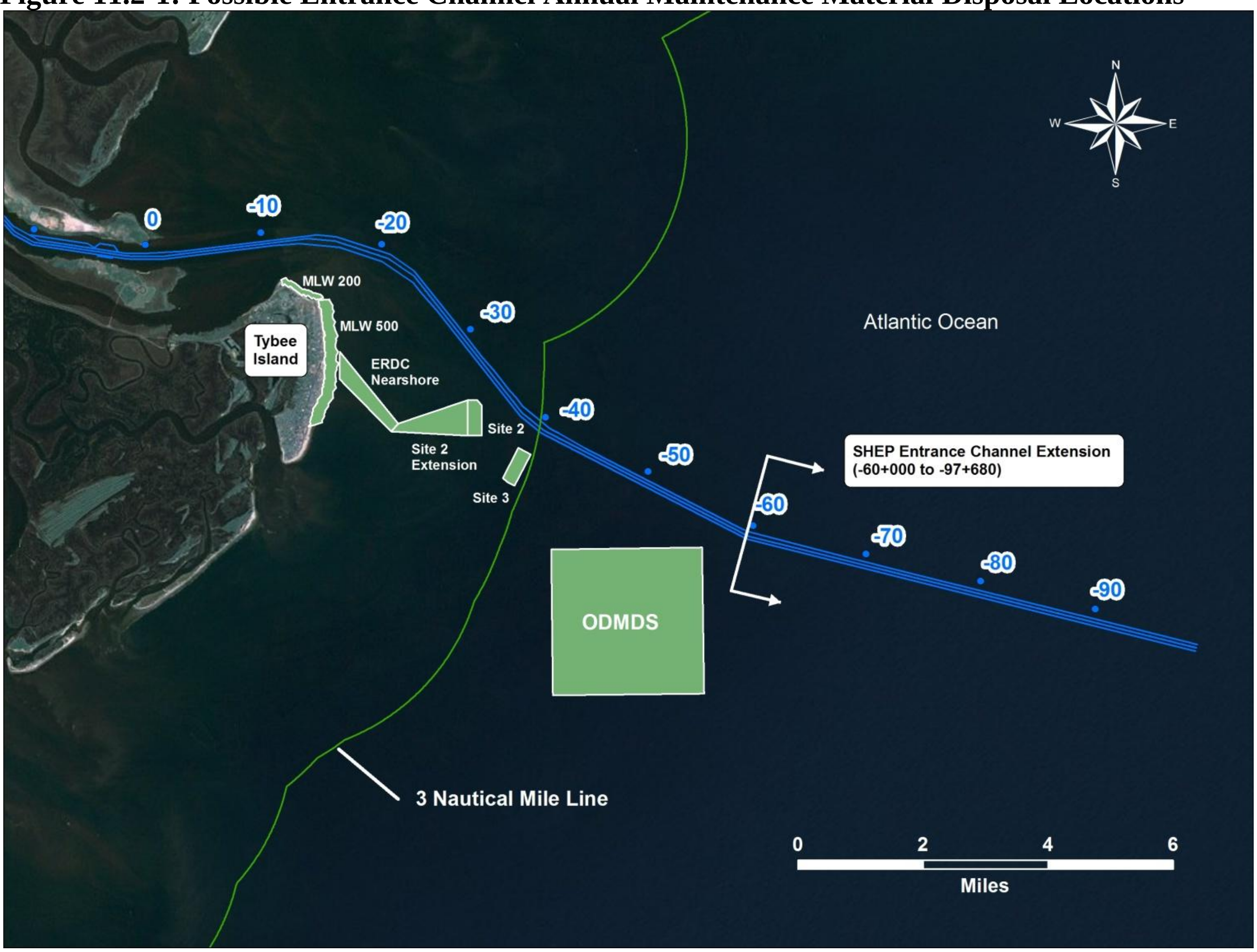
Fiscal Year*	Confined Disposal Area							TOTAL O&M
	2A	12A	13A	13B	14A	14B	J/O	
2061		BUILD DIKE	4,660,000	DRYING	782,500	782,500	DRYING	6,225,000
2062		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2063		4,660,000	DRYING	DRYING	782,500	782,500	DRYING	6,225,000
2064		4,660,000	BUILD DIKE	782,500	DRYING	DRYING	782,500	6,225,000
2065		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000
2066		DRYING	4,660,000	782,500	DRYING	DRYING	782,500	6,225,000

11.2 ENTRANCE CHANNEL DISPOSAL

Currently, annual maintenance dredged material removed from the Entrance Channel is placed in the EPA-approved Ocean Dredged Material Disposal Site (ODMDS). See **Figure 11.2-1**. All dredged sediments from the SHEP Entrance Channel would be placed into the ODMDS or potentially upland CDFs.

In the Draft SHEP GRR, placement of materials in the nearshore was proposed as a beneficial use of dredged materials. Material with a fines content of 20 percent or less would have been placed into the nearshore sites and feeder berms in addition to placing material in the ODMDS. See **Figure 11.2-1**. The Georgia DNR-CRD and the City of Tybee Island have requested that these sites not be used for sediment placement from the project because they prefer material with a fines content of 10 percent or less. Consequently, these dredged material placement sites for new work were removed from the project. The Corps may use sites previously approved in the 1996 LTMS for placement of maintenance material (Site 2, Site 3, Site 2 Extension, ERDC Nearshore, MLW 200, MLW 500).

Figure 11.2-1: Possible Entrance Channel Annual Maintenance Material Disposal Locations



The volume of new work dredged sediment to be removed by project depth for the entrance channel including the extension is shown in **Table 11.2-1**. This material will be placed in the EPA approved ODMDS site.

Table 11.2-1: Entrance Channel Volumes by Station of New Work Material for all Project Depths

Station	44 ft	45 ft	46 ft	47 ft	48 ft
-98+600 to -57+000	1,667,123	2,242,371	2,925,432	3,736,308	4,613,909
-57+000 to -53+500	156,623	235,127	313,391	391,437	469,252
-53+500 to-40+000	646,796	975,843	1,304,385	1,632,346	1,959,186
-40+000 to -30+000	505,693	771,105	1,038,620	1,305,921	1,573,800
-30+000 to -20+000	529,910	801,974	1,076,638	1,352,115	1,628,379
-20+000 to -10+000	473,047	746,614	1,028,399	1,311,222	1,594,871
-10+000 to 0+000	346,997	532,621	723,394	917,064	1,110,713
0+000 to +4+000	101,482	166,705	235,626	305,674	375,403

**Station -98+600 is the extended channel stationing for the 48 ft project depth. Channel stationing for the 47 ft project depth across the ocean bar terminates at Station -97+680.*

11.2.1 ODMDS Capacity Analysis

Two approaches were taken to determine future capacity of the ODMDS after placement of materials from the Savannah Harbor Expansion offshore dredging:

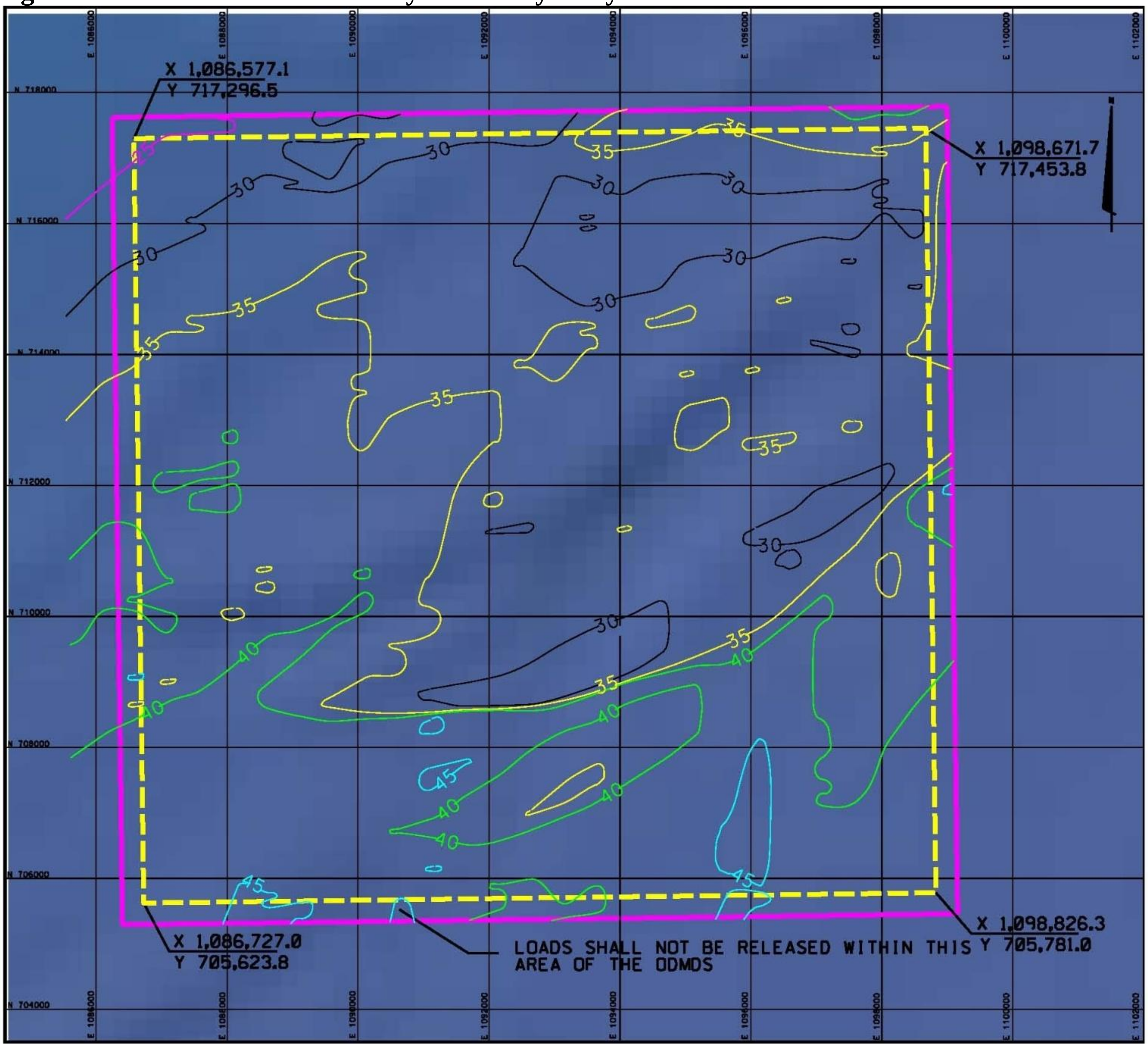
- The first analysis assumed that dredged material would primarily consist of sands (with no bulking factor) and that all deposited material would remain in the ODMDS area after placement.

- The alternative approach was to examine the last period of time when the ODMDS was used consistently and compare the dredged volumes removed from the channel that were placed in the ODMDS with the change in capacity of the ODMDS based on placement surveys over the same time period.

Differences in capacity were determined by using software in Bentley InRoads to determine surface areas and to make volume computations. Actual dredging volumes were obtained from dredging reports maintained in the Savannah District office.

The ODMDS boundary is shown in pink on **Figure 11.2.1-1**. To comply with 40 CFR §227.28 disposal shall occur no less than 330 feet (100 meters) inside the site boundaries. The ODMDS footprint (shown in yellow) for placement of material is 3,242 acres.

Figure 11.2.1-1: ODMDS Boundary and Bathymetry



The quantity of material shown in **Table 11.2.1-1** dredged from the entrance channel from 2002 to 2007 as well as the change in capacity of the ODMDS based on placement surveys made over the same time period. Results show that roughly 93% of the material dredged from the channel remains in the ODMDS after placement, possibly because of the presence of fines in the dredged material that do not settle out or may leave the area before settling or losses during the dredging process.

Table 11.2.1-1: Material Dredged Versus ODMDS Change in Capacity

Year	Quantity Dredged (cy)	ODMDS Capacity* (cy)	
2002	186,537	57,836,270	
2003	635,163		
2004	620,642		
2005	888,101		
2007	973,463	54,766,930	
Total Dredged	3,303,906		
Change in Capacity		3,069,340	
		percentage placed	93%

*ODMDS capacity is determined from the surface to -25 ft MLLW

Tables11.2.1-2 and **11.2.1-3** shows results of the remaining capacity of the ODMDS for existing conditions and assuming the material placed from the Savannah Harbor Expansion is either 1) the volume placed is equivalent to the volume dredged or 2) volume placed is equivalent to 93% of the material dredged, based on the results from **Table 11.2.1-1**. Results of both methods are based on the upper limit for material placement in the ODMDS as 25 ft below MLLW. Table 11.2.1-3 shows results for both the 47 ft and 48 ft project. The capacity life is reduced for the deeper project because of the greater volume of material required to be removed/placed and increased maintenance as the deeper project requires a longer channel. The average annual Operations and Maintenance (O&M) dredge material volumes were developed previously based on historical dredging records. The volume of available undredged material that remains after dredging (primarily due to funding constraints) is also considered.

Table11.2.1-2: ODMDS Capacity (Existing Conditions)

ODMDS capacity (2012)	57,087,926 cy
Average Annual O&M Dredge Volume	1,057,721 cy
Years Remaining until Capacity is Exceeded (without deepening)	55

Given the constraints for the Savannah Harbor ODMDS of a surface area of 3,242 acres and upper height limit of 25 ft below MLLW, with materials added from the offshore channel for the 48 ft project depth, the remaining capacity for the ODMDS would last between 36 and 40 years depending on the volume of material placed. For the 47 ft project, capacity would last between 39 and 42 years. Without material from the Savannah Harbor Expansion Project there would be a 55 year capacity for holding annually dredged O&M material.

Table11.2.1-3: ODMDS Capacity (With Project)

	46 ft depth (44 ft project)	46 ft depth (44 ft project)	47 ft depth (45 ft project)	47 ft depth (45 ft project)	48 ft depth (46 ft project)	48 ft depth (46 ft project)	49 ft depth (47 ft project)	49 ft depth (47 ft project)	50 ft depth (48 ft project)	50 ft depth (48 ft project)
Percentage of Dredged Material on ODMDS Floor	100% placed	93% placed	100% placed	93% placed	100% placed	93% placed	100% placed	93% placed	100% placed	93% placed
2014 Capacity (after 2 yrs of existing O&M)	54,972,484	54,972,484	54,972,484	54,972,484	54,972,484	54,972,484	54,972,484	54,972,484	54,972,484	54,972,484
New Work Quantity	4,326,189	4,326,189	6,305,655	6,305,655	8,410,259	8,410,259	10,646,413	10,646,413	12,950,110	12,950,110
2017 Capacity (after new work material)	50,646,295	50,949,128	48,666,829	49,108,225	46,562,225	47,150,943	44,326,071	45,071,320	42,022,374	42,928,881
Average Annual O&M Dredging Volume (after expansion)	1,066,299	1,066,299	1,066,587	1,066,587	1,066,738	1,066,738	1,066,778	1,066,778	1,067,000	1,067,000
O&M Dredging Volume (3 yrs during expansion)	3,198,896	3,198,896	3,199,761	3,199,761	3,200,213	3,200,213	3,200,333	3,200,333	3,201,000	3,201,000
2017 Capacity (after expansion new work and O&M)	47,447,399	47,974,155	45,467,068	46,132,447	43,362,012	44,174,745	41,125,738	42,095,010	38,821,374	39,951,951
Years Remaining until Capacity is Exceeded	44	48	43	47	41	45	39	42	36	40

12.0 OPERATION & MAINTENANCE IMPACTS

SHEP analysis includes an examination of the possible impacts to the USACE Operations and Maintenance (O&M) Program. A detailed report of these findings titled *Savannah Harbor Expansion Project Impacts to O&M* is included in the Engineering Investigation Supplemental Materials. The annual impacts to the program include the following:

- Increased dredging costs for the Inner Harbor, Bar Channel, and Mitigation Areas
- O&M costs for the NSBL&D fish passage structure
- O&M costs for the Oxygen injection system
- Long term monitoring
- Curation costs for the CSS *Georgia* (removal of the CSS *Georgia* from the project area will be accomplished with SHEP construction funds, and the cost will be included in the project B/C ratio determination)

12.1 MITIGATION – DISSOLVED OXYGEN SYSTEM

Impacts to D.O. throughout the Savannah River Estuary due to SHEP were determined through studies outlined in Section 7.0 Hydrodynamic Modeling of this report and the EIS. These studies evaluated the impacts to D.O. in the Harbor due to SHEP and outlined a plan for mitigation of these impacts. Mitigation for these impacts will involve construction of a dissolved oxygen injection system using Speece Cone technology and on-site Oxygen generation. Once construction of this system is complete, it will be operated and maintained by the USACE, Savannah District.

The annual impacts to O&M by the Dissolved Oxygen Injection System are for upkeep of the landside facilities to house the system and for the operating costs to produce the oxygen and are shown by project depth in **Table** 12.1-1 below. Also included in the annual O&M costs are the replacement costs for the Speece cone and intake and discharge lines at 40 year intervals; and replacement of the oxygen flow control, oxygen generator, and side stream pump at 20 year intervals.

The expected annual costs for operating and maintaining the D.O. injection system is based on continued operation for a period up to 180 days per year (May - October). The operational costs are expected to be uniform throughout that 180-day period. **Table 12.1-1** shows the associated costs for the mitigation feature.

Table 12.1-1: Dissolved Oxygen Injection System Expected Annual Operation and Maintenance Costs

Project Depth Alternative (ft below MLLW)	Expected Annual O&M Costs
44	$1,110,000
45	$908,500
46	$1,110,000
47	$1,210,400
48	$1,311,000

12.2 INNER HARBOR MAINTENANCE

Current Dredging Market – Past dredging work in Savannah and Brunswick Harbors was performed by as many as 4 different small business contractors; however, current plans advertise this work in the unrestricted category, since the competitive field of dredging contractors capable of dredging Savannah Harbor has been reduced. Currently, we are working near the limit of the small dredges (18 inch cutterhead.) capability due to the depth of the channel, the length of the dredge pipeline to the dredged material containment areas, and the height of containment areas. It is the District's contention that using larger dredges will be more cost effective (22 to 30 inch cutterhead) to perform this work. Starting in FY08, Savannah Harbor maintenance dredging work has been advertised in the unrestricted category.

Current Project Conditions **–** In order to project shoaling conditions after the Savannah Harbor Expansion project is completed, CESAW-EN conducted an investigation of historical Inner Harbor shoaling rates and dredging volumes per river reach both With and Without the project (harbor deepening and mitigation features) in existence. With authorization and implementation of the Savannah Harbor Expansion Project the Sediment Basin would be filled. The filling of the Sediment Basin is a critical feature in mitigation planning for the Savannah Harbor Expansion Project as it restricts further upstream saltwater incursion.

Mitigation Feature Impacts & Post-Project Conditions **–** The selected mitigation plan as depicted in includes, among other things, demolition of the tidegate structure, removal of the northern and southern earthen abutments and construction of a sill across the throat of the Sediment Basin which will permit the Basin to fill with sediment to pre-harbor improvement conditions, thus reducing impacts to wetlands in the Back River by creating a partial barrier to saltwater intrusion up that river.

The end result of this mitigation action will be a return of the Inner Harbor shoaling regime to Pre-tidegate conditions, which is discussed in greater detail in Section 10.0. **Figure 12.2-1** below illustrates how the shoaling distribution in the harbor changes with and without operation of the Sediment Control Works. The green area in **Figure 12.2-1** represents the inner harbor shoaling patterns since the decommissioning of the tidegate. The shoaling pattern shown in blue represents the inner harbor prior to constructing the tidegate.

Figure 12.2-1: Annual Historic Dredging Volumes

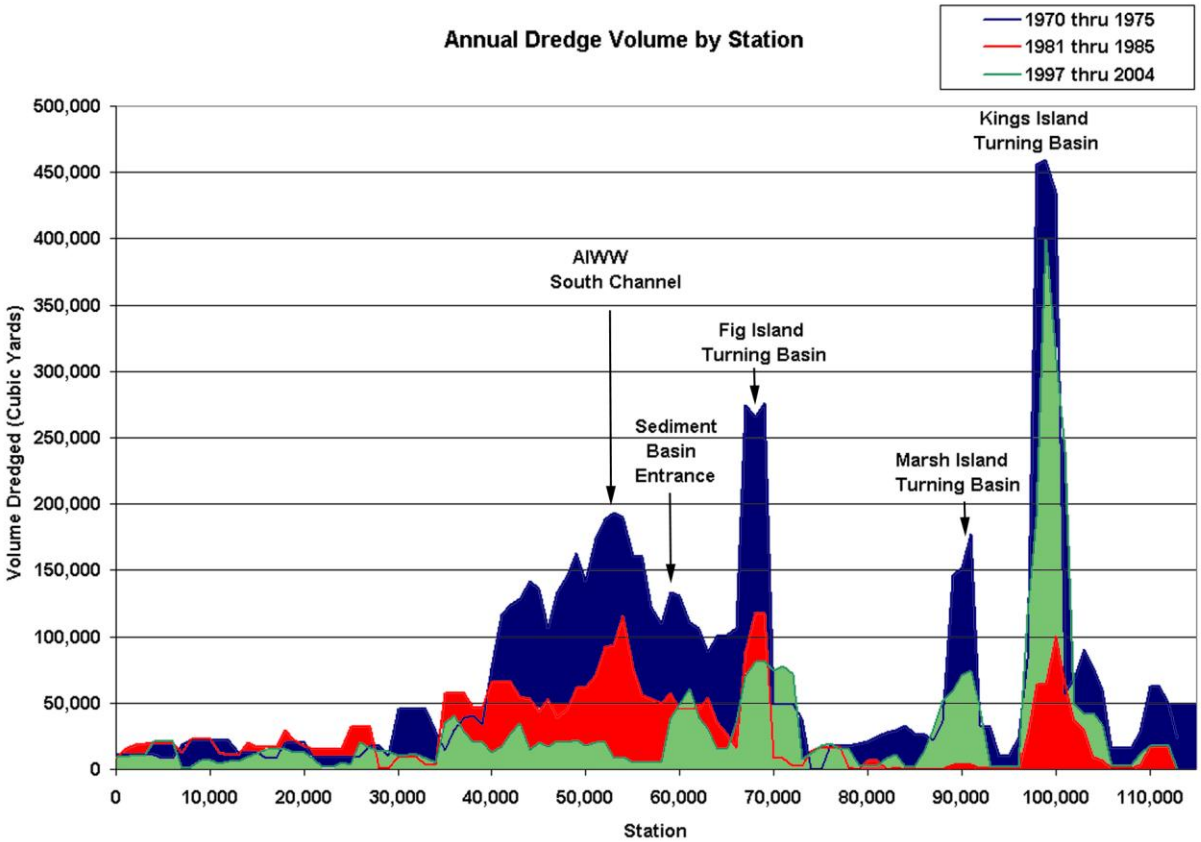


Table 12.2-1 compares historic average annual dredging quantities and costs for the middle harbor (Station 24+000 through Station 70+000) to projected quantities using the same removal costs. The total volume of sediment removed in the inner harbor is not expected to change with a harbor deepening, but the location of the shoaling will be different. Future maintenance costs for the middle harbor, using recent average annual costs, will be almost two and a half times greater than existing costs based purely on a different distribution of the sediments. This is a direct result of having to dredge 1,932,000 cubic yards of materials from the channel instead of the Sediment Basin. The difference in cost results from the increased pumping distances to disposal areas and the loss of dredge efficiency due to reduced effective working time caused by passing vessels. There is also a loss of dredge efficiency related to the decreased dredging bank height in the channel (6-8 feet) as compared to 16-20 feet in the Sediment Basin. It should also be noted that if this area experiences periods of above normal precipitation costs could rise considerably.

Table 12.2-1: Historic vs. Projected Sediment Volumes and Associated Costs

Stations	Historic Channel O&M Volume with Sediment Basin	Historic Cost	Projected Channel O&M Volume without Sediment Basin	Cost
24 to 40+000	328,000	$734,720	364,000	$815,360
40 to 50+000	260,000	$624,000	900,000	$2,160,000
50 to 70+000	820,000	$2,328,800	2,076,000	$5,895,840
Total	1,408,000	$3,687,520	3,340,000	$8,871,200

The mitigation plan also includes deepening of McCoy's Cut and construction of a freshwater diversion structure on the Savannah River near the cut. These features will facilitate more freshwater flow into the Back and Middle Rivers adjacent to the Savannah National Wildlife Refuge. Maintaining the deepened reaches on Back and Middle Rivers is expected to include dredging approximately 70,763 cubic yards of material from this area every 10 years. Dredging costs for this effort are estimated to be $1,415,000 every 10 years ($141,500/year or $7,075,000 for the 50-year life of the project). These mitigation features are consistent for all project depths, with the exception of deepening McCoy's Cut which is not included in the 44 ft depth alternative, and costs are not expected to vary by depth.

Maintenance of the fish passage structure at NSBL&D will require clearing and snagging every 2-3 years to maintain the flow, which is estimated at $50,000/year.

<u>Current Concerns</u> – The major concerns related to maintenance of the authorized depth in the inner harbor after SHEP are 1) the limited availability of O&M funds, 2) the anticipated change in shoaling locations, 3) the availability of a dredge capable of dredging in the Savannah Harbor, and 4) the limited dredging time due to environmental windows. The historical solution to all four of these concerns has been the systematic application of advance maintenance in the harbor and concentrating dredging on the middle half of the channel. Since the 1970's, the Savannah District has justified the need to dredge below the congressionally authorized depth in nearly every reach of the harbor for the purpose of maintaining the authorized depth for shipping, necessitated by the rapidity with which the channel shoals accrete. Through a series of justifications and approvals based on shoaling and dredging records, the District proved that advance maintenance was both an environmentally and economically sound method of providing navigable depths.

<u>Recommendations</u> – Advance maintenance for the currently authorized project is shown in **Table 12.2-2**. For the SHEP recommended depth alternative and mitigation plan, this table also shows three locations (identified by an asterisk and highlighted in yellow) where additional material is expected to accumulate and additional advance maintenance may be needed in order to maintain existing levels of service. These three locations were identified during the sedimentation analysis for the SHEP; details of this analysis can be found in Section 10.0 of this report. This need is based on the assumption that the same level of dredge plant will be required.

These predicted changes may require changes to the advance maintenance program. However, the future shoaling cannot be determined precisely by modeling. By allowing the harbor to stabilize after SHEP construction, the shoaling patterns would likely be established and at that point the District may determine that some adjustments to the advance maintenance program would be beneficial. If that determination is made, the District will seek approval to implement modifications to the program through the normal business process which would include an economic justification and environmental assessment.

Table 12.2-2: Advance Maintenance for Existing and SHEP Recommended Project Depths

	Currently Authorized Project			**SHEP Project**
Stations	**Authorized Project Depth (ft below MLLW)**	**Advance Maintenance (ft)**	**Maintenance Dredging Depth (ft below MLLW)**	**Estimated Advance Maintenance (ft)**
-98+600 to -60+000**	Not applicable			0
-60+000 to -14+000	44	0	44	0
-14+000 to 24+000	42	2	44	2
24+000 to 35+000	42	4	46	4
35+000 to 37+000	42	6	48	6
37+000 to 50+500	42	4	46	4
50+500 to 52+750	42	4	46	6 *
52+750 to 54+000	42	4	46	4
54+000 to 60+250	42	4	46	8 *
60+250 to 66+750	42	4	46	4
66+750 to 70+000	42	4	46	6 *
70+000 to 102+000	42	2	44	2
102+000 to 103+000	42	0	42	0
Kings Island Turning Basin	42	8	50	8

** Estimated changes in advance maintenance program due to SHEP recommended depth alternative and mitigation plan to maintain existing levels of service.*
***Station -98+600 is the extended channel stationing for the 48 ft project depth. Channel stationing for the 47 ft project depth across the ocean bar terminates at Station -97+680.*

12.2.1 Advance Maintenance Justification

The current advance maintenance program for the Savannah Harbor Navigation Project was developed in response to changes in the harbor after the 1992 Harbor Deepening Project (from 38 feet to 42 feet below MLLW) and the removal of the tidegate from operation in March 1991. The program is based on the shoaling patterns observed since completion of the 1992 Harbor Deepening Project and the discontinued operation of the tidegate. The program covers the entire length of the harbor and was developed to provide a more efficient, cost effective, and environmentally acceptable maintenance program for the harbor. The program does not provide additional navigational depth to the navigation channel since shoaling will continue to occur above authorized project depth before periodic maintenance dredging commences.

The current advance maintenance program for Savannah Harbor was authorized in the Deepening Project in accordance with House Document 102-394, 102d Congress, 2^{nd} Session. Implementation of the advance maintenance program allowed the District to maximize our use of the Dredged Material Management Plan (DMMP) (April 2003) with the 3-year rotation plan for the dredged material containment areas (DMCA). The 2003 DMMP in conjunction with the LTMS (1996), projected an average annual cost savings of $1.1 million per year and an extension of DMCA system life from 5 to 20 years upon implementation. Additionally, more efficient dredging procedures allows full use of one pipeline dredge on an annual basis for the Inner Harbor dredging contract. Currently, one dredge is used, but even with the current advance maintenance program, three to four dredge movements to cut down on fast rising shoals are required. Savings associated with each movement of a dredge during the contract are estimated to be $30,000 for the actual move and 2 days of dredging lost while moving the dredge. Performing advance maintenance saves a minimum of four dredge movements during each contract. Therefore, a cost avoidance of least $120,000 and 8 days lost production per year are realized. Under the program, about 48% of the harbor navigation channel will require advance maintenance of at least 2-ft, 19% will require advance maintenance of at least 4 ft, and 8% will require advance maintenance of at least 6ft Two of the seven harbor turning basins, Fig Island and Kings Island, will require advance maintenance of 4 ft and 8 ft, respectively.

Savannah District's goal is to dredge the navigation channel on an annual basis with a minimum cost and maximum efficiency of operation of both dredging and containment area operation. An annual cycle will provide the ability to better coordinate our actions with the local assurer and better fulfill our environmental commitments. The current advance maintenance program is represented in **Table 12.2.1-1**. Please note, currently, advance maintenance is authorized in two of the five turning basins. See **Table 12.2.1-2**. **Tables 12.2.1-2** and **12.2.1-3** describe the locations of the turning basins and the DMCAs respectively.

Table 12.2.1-1: Currently Authorized Advance Maintenance Depth by Station

Station	Authorized Depth (ft below MLLW)	Authorized Advance Maintenance (ft)	Allowable Over-depth (ft)	Maximum Sediment Removal Depth (ft below MLLW)
112+500 to 105+500	30	2	2	34
105+500 to 103+000	36	2	2	40
103+000 to 102+000	42	0	2	44
102+000 to 70+000	42	2	2	46
70+000 to 37+000	42	4	2	48
37+000 to 35+000	42	6	2	50
35+000 to 24+000	42	4	2	48
24+000 to 0+000	42	2	2	46
0+000 to -14+000	42	2	2	46
-14+000 to -60+000	44	0	2	46
Jones/Oysterbed Turning Basin	40	0	2	42
Fig Island Turning Basin	34	4	2	40
Marsh Island Turning Basin	34	0	2	36
Kings Island Turning Basin	42	8	2	52
Argyle Island Turning Basin	30	0	2	32
Port Wentworth Turning Basin	30	0	2	32

Table 12.2.1- 2: Specifications & Locations of Turning Basins

Turning Basin	Length (ft)	Width (ft)	Maintenance Depth* (ft below MLLW)	River Mile	Station
Port Wentworth	600	600	30	20.9	111+363 to 109+757
Argyle Island	600	600	30	19.6	104+185 to 103+085
Kings Island	1,600	1,500	50	18.8	101+298 to 97+750
Marsh Island	900	1000	34	17.1	91+610 to 89+485
Fig Island	1500	1000	38	13.0	69+740 to 67+386
Jones/Oysterbed Island	1050	1200	40	0.7	4+395 to 2+345

**Maintenance Depth excludes Allowable Over-Depth.*

Table 12.2.1-3: Locations of Dredged Material Containment Areas (DMCA)

DMCA	Station*	DMCA acreage
1N	107+500 to 112+500	130
2A	93+000 to 103+000	240
12A	6+000BR to 10+500BR	1040
13A	47+800 to 6+600BR	1307
13B	43+000 to 47+000	525
14A	37+000 to 47+000	647
14B	28+000 to 37+000	703
Jones/Oysterbed	10+000 to 27+000	803

** Back River Channel Station*

Advance maintenance dredging is authorized in Savannah Harbor to efficiently maintain the required projects depths. The importance of advance maintenance in a rapidly shoaling, silty harbor such as Savannah Harbor cannot be overstated. Given the lack of sufficient contractor equipment, environmental windows and limited funding, advance maintenance is a necessity. For example, prior to 1996, the Kings Island Turning Basin without 8 feet of advance maintenance routinely shoaled to 34 feet in a four month period causing major problems for pilots turning vessels. The District simply could not dredge the basin and the channel in a timely manner because of the striped bass window and dissolved oxygen requirements. Since the advance maintenance was instituted, we have been able to insure turning basin depths for longer periods of time and have not faced the crisis modes of the years prior to 1996.

Maintaining harbor channel project depth at the level of service required by the pilots and the USCG without advance maintenance would be impossible due to the sedimentation rate in the harbor. Providing project depth would require increased funds and the need for additional dredging plant. Even with these assets, the existing environmental windows would prevent dredging to the degree necessary to maintain depths adequate for navigation.

The Savannah Harbor annual maintenance dredging budget has increased from approximately $8.5 million in 1990-91 to $13.9 million in 2000 and has remained at this level to this date. Historical dredging records indicate that shoaling rates in the harbor have not changed, but the locations of shoals since the tidegate decommissioning have shifted locations to the upper harbor where disposal area capacity is negligible thus requiring longer pumping distances and increased costs. Loss of advance maintenance would exacerbate this problem due to the requisite number of dredge re-mobilizations.

In **Figure 12.2.1-1**, developed for the SHEP sedimentation analysis, the area in blue represents Savannah Harbor shoaling patterns prior to the construction of the tidegate (1970 – 1975), the area in red represents Savannah Harbor shoaling patterns during the operation of the tidegate (1981 – 1985), and the area in light green represents Savannah Harbor shoaling patterns since the tidegate was taken out of operation in March 1991 (1997 – 2004). Please note that the current (1997 – 2004) shoaling patterns are beginning to revert back to the shoaling patterns prior to the construction of the tidegate (blue color). Also note that the areas highlighted in yellow on **Table 12.2.1-4** need to be dredged on a 6-month cycle. These areas conform closely with those areas in blue (pre-tidegate shoaling patterns). This is an indication that the current approved advance maintenance program needs to be preserved if we are going to keep the harbor operating efficiently.

Figure 12.2.1-1: Annual Historic Dredging Volumes

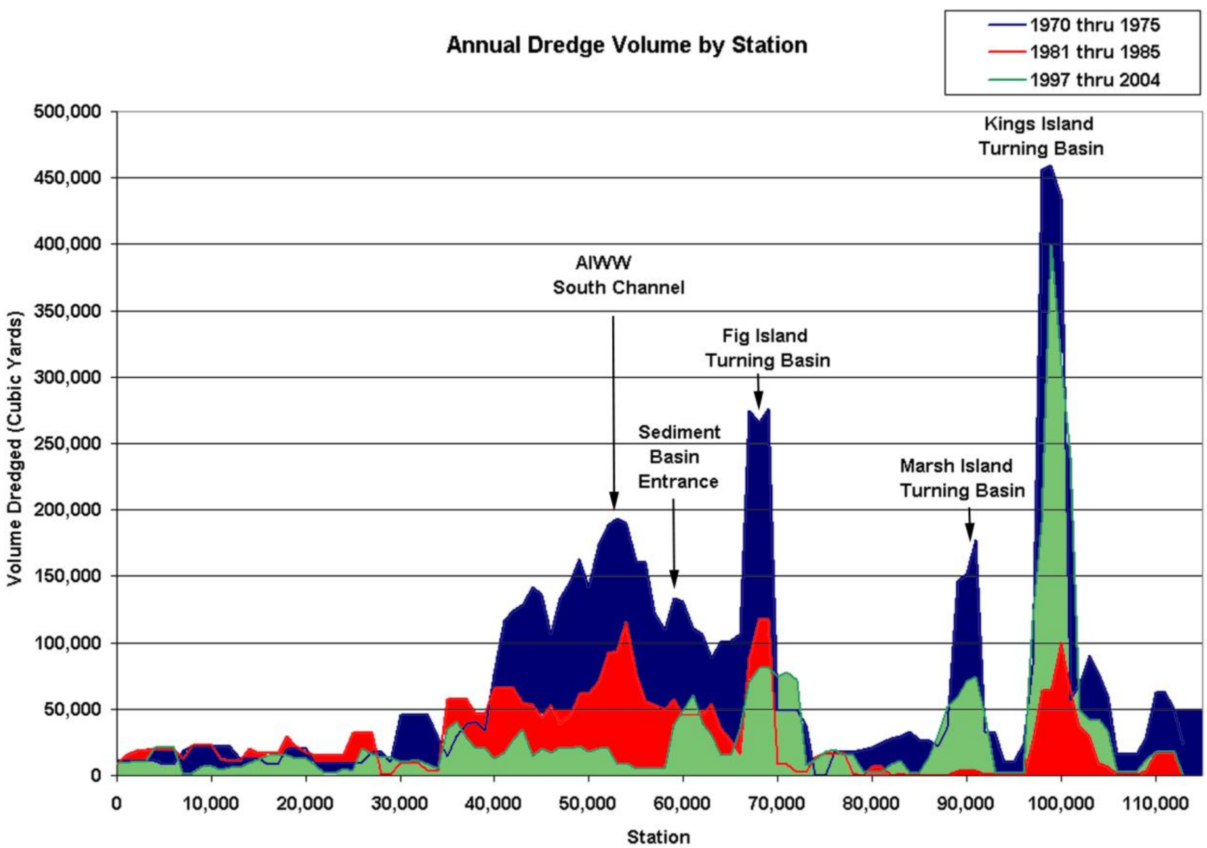


Table 12.2.1-4: Shoaling Average (cy) by Station for 2008 - 2009

Station	Shoaling Volume (cy)	Station	Shoaling Volume (cy)	Station	Shoaling Volume (cy)
0	11,866	38+000	24,392	76+000	23,825
1+000	11,866	39+000	24,867	77+000	18,418
2+000	13,946	40+000	15,157	78+000	18,418
3+000	12,266	41+000	21,054	79+000	3,746
4+000	25,921	42+000	32,835	80+000	2,959
5+000	25,921	43+000	42,262	81+000	2,959
6+000	26,184	44+000	17,155	82+000	9,995
7+000	1,026	45+000	25,345	83+000	13,606
8+000	2,445	46+000	20,211	84+000	1,586
9+000	8,337	47+000	25,524	85+000	1,586
10+000	8,160	48+000	25,158	86+000	16,780
11+000	5,337	49+000	26,665	87+000	36,899
12+000	6,781	50+000	22,060	88+000	63,586
13+000	6,888	51+000	25,242	89+000	72,272
14+000	11,533	52+000	25,242	90+000	86,497
15+000	14,790	53+000	10,667	91+000	91,420
16+000	18,308	54+000	11,369	92+000	54,434
17+000	18,756	55+000	6,245	93+000	2,050
18+000	18,764	56+000	5,751	94+000	2,050
19+000	15,669	57+000	6,683	95+000	2,050
20+000	15,186	58+000	6,011	96+000	1,693
21+000	9,740	59+000	46,565	97+000	142,429
22+000	2,516	60+000	60,347	98+000	232,556
23+000	1,858	61+000	74,865	99+000	489,662
24+000	6,023	62+000	47,994	100+000	378,601
25+000	4,572	63+000	37,084	101+000	293,983
26+000	24,413	64+000	18,925	102+000	60,257
27+000	19,840	65+000	18,679	103+000	50,618
28+000	16,054	66+000	42,012	104+000	51,599
29+000	16,521	67+000	85,725	105+000	40,057
30+000	12,758	68+000	99,524	106+000	2,684
31+000	13,158	69+000	99,524	107+000	2,684
32+000	13,158	70+000	91,460	108+000	2,684
33+000	10,511	71+000	95,409	109+000	13,305
34+000	6,595	72+000	87,680	110+000	21,336
35+000	42,128	73+000	9,810	111+000	21,336
36+000	49,315	74+000	14,794	112+000	22,334
37+000	34,386	75+000	21,570		

Note: Stations highlighted in yellow require dredging twice a year.

12.2.1.1 Advance Maintenance Justification by Station

Station 112+500 to Station 103+000

Advance maintenance in this segment is currently 2 feet which was justified after the 1992 Deepening Project. Sediments in this segment were originally deposited in DMCA 2A, but since this DMCA has limited storage capacity, the sediments are now pumped to DMCAs 12A or 13A (an additional two booster pumps) depending on the rotation schedule. The objective is to remove the predominantly sandy materials before they travel to Kings Island Turning Basin (KITB) and further downstream to an area where sand waves, which are a hazard to navigation due to their density, form. Relocating the sand to DMCA 12A (or 13A) will maximize the life of DMCA 2A and will help minimize future sand waves causing problems with harbor operations between Station 79+600 and KITB.

Kings Island Turning Basin (Station 102+000 to 97+000) and Adjacent Channel

Initial indications are that the current approved advance maintenance (8 feet), while somewhat effective, may need to be increased since the District is still dredging the basin on a 6-month rotation rather than a yearly rotation. Current advance maintenance has proved effective in extending the time between dredging intervals to a limited extent and providing a safer area in which to turn ships, but additional advance maintenance may be requested in the future. This basin is the only turning basin capable of handling the post-Panamax design vessel The pilots must use the upper end of the basin when turning to maintain safe maneuverability while turning the ship in a strong ebb tide. Biennial dredging reduces the time that large military and commercial ships risk damage due to grounding while using the basin.

The current requirement of a 6-month dredging cycle even with the 8-foot of advance maintenance is costly. With DMCA 2A no longer available, we are required to use DMCAs 12A or 13A depending on the rotation. The use of DMCA 12A or 13A requires two booster pumps which triples the dredging costs. While the 8 feet of advance maintenance was estimated to give us a yearly dredging cycle, the result has been a 6 month cycle which has been far more costly. This, however, may be more due to the fish window and the efficiency of the dredge rather than the number of times we could dredge the basin.

Future cost savings would result from reducing the costs associated with having to move a dredge on station the second time during a contract and the elimination of one dredging cycle using two booster pumps.

Station 97+000 to 79+600

The existing two feet of advance maintenance must be maintained, and an additional 4 feet needs to be approved on the south side of the channel for Station 86+000 to 92+000 (Marsh Island Turning Basin and below). This channel segment is an area of sand wave formation. The advance maintenance needs to be approved to minimize the impact of sand waves. Even though sand waves are not large formations, the waves are a hazard to navigation due to their density. Measurement and dredging of the wave is difficult because the waves are perpendicular to the channel. Since the 1992 Deepening Project, we have dredged the sand waves on an annual basis.

Part of the advance maintenance has been dredged, and indications are that the sand waves are peaking below the authorized channel depth except on the south side of the channel between Station 86+000 and 92+000. Shoaling between Station 86+000 and 92+000 requires additional advance maintenance. We expect minimal sand waves above the authorized channel dimensions in the future. However, some sand waves may continue to exist and must be removed due to the hazard to navigation that they pose. DMCAs 12A and 13A are available for placement of material from this channel segment.

Station 79+600 to 73+000

We have dredged the channel segment on an annual basis since the 1992 deepening project. We are currently experiencing transient shoaling in the channel segment during our ongoing maintenance dredging contract and cannot estimate a shoaling rate for the segment. We believe the current two feet of advance maintenance will be sufficient to prevent the transient shoals from becoming an obstruction to navigation. Cost savings would be the costs associated with moving a dredge on station for the second time during the contract.

Station 73+000 to 70+000

The two feet already in place is fine, but we may need to request additional advance maintenance (five additional feet) in order to get back to a yearly cycle rather than the 6-month cycle we are in now. Shoaling is occurring on the north side of the channel at a rate of two feet per month requiring us to dredge the segment at about 6-months interval. The cost savings for dredging the segment on an annual basis would result from the lack of need to have a dredge on standby to dredge the shoal and the costs associated with moving a dredge on station the second time during the contract.

Fig Island Turning Basin (Station 69+740 to 67+386)

The four feet of advance maintenance currently approved in the basin should be maintained. The basin is located adjacent to a heavily shoaling channel segment that needs to be dredged on a 6-month cycle. The material removed from the basin also helps increase the time between dredging cycles in the channel.

Station 70+000 to 59+000

We currently have four feet of advance maintenance in this segment of the channel which was approved in the 1992 Deepening Project. Shoaling rates in the channel are up to a foot per month. We are dredging the segment on a 6-month cycle. Cost savings for an annual dredging cycle would be the costs associated with moving a dredge on station the second time during the contract.

Station 59+000 to 37+000

The four feet of advance maintenance currently approved should be maintained. Shoaling is continuing at a rate that the advance maintenance is necessary.

Station 37+000 to 35+000

Currently, six feet of advance maintenance has been approved for the channel segment. The channel segment is shoaling at a rate of a foot per month. Cost savings would be the costs associated with moving a dredge on station the time during the contract.

Station 35+000 to 24+000

The four feet of advance maintenance currently approved should be maintained. Shoaling is continuing at a rate that justifies advance maintenance.

Station 24+000 to 0+000

The two feet of advance maintenance currently approved should be maintained. The unit cost of dredging the channel segment is much higher ($6.40/CY vs. $5.20/CY) than the rest of the Inner Harbor Channel due to the type of material (sand) in the segment. We need the advance maintenance to give us flexibility in scheduling our dredging of the channel segment to a time we have funds available from reduced costs in another channel segment. We expect the reduced costs will result from the rotational use of the containment areas (Jones/Oysterbed versus 14B) and the shortened pumping distances that occur during the rotational cycle.

Station 0+000 to -14+000

We currently have two feet of advance maintenance for this channel segment which is located in the Entrance Channel and can only be dredged by a hopper dredge during December through March. The shoal in the channel segment develops across the channel to a height of two feet above project depth in less than the yearly maintenance cycle. Hopper dredging at more frequent intervals is not allowed throughout the year due to environmental constraints and use of a cutterhead dredge would require a seagoing cutterhead dredge. Mobilization costs for a seagoing cutterhead dredge could be expected to be in excess of $1,000,000 and thus be prohibitive.

Station 14+000 to -60+000

Current shoaling rates indicate no advance maintenance is required at the present time.

With the construction of the tidegate, the shoals moved out of the turning basins in the upper harbor; Kings Island, Marsh Island, and Fig Island; and City Front into the newly constructed Sediment Basin. With the removal of the tidegate and the filling in of the Sediment basin, the shoals are beginning to return to the turning basins and the city front. These shoaling areas are highlighted in yellow in **Table 12.2.1-4** shown previously. This is a strong indication that the currently requested advance maintenance in those areas may have to be increased even more for future dredging operations.

12.2.1.2 Advance Maintenance Recommendation

Shoaling patterns have changed as a result of Harbor widening and deepening, removal of the tidegate from operation, and low flows from the Savannah River Below Augusta (SRBA) as a result of long-period drought conditions. Future dredging without advance maintenance would require increased O&M funding due to the inefficiencies in dredging – more dredges would be required with an increased number of contracts, decreased dredging efficiency, and increased mobilizations. There may not be sufficient dredges available to meet the demands of the channel without advance maintenance as multiple dredges will be required to dredge many areas of the channel at once. Based on the District's experience with dredging the harbor during the pre and post tidegate operational period, the advance maintenance program is absolutely vital to the provision of a safe, reliable, and cost effective channel.

12.3 ENTRANCE CHANNEL MAINTENANCE

Current Annual Maintenance Cycle – Maintenance dredging is presently performed on an annual basis. The current annual maintenance dredging costs for the entrance channel are shown in **Table 12.3-1**. This annual dredging process entails hopper dredges removing the dredged material from the entrance channel and placing it in the ocean dredged material disposal site (ODMDS). SHEP will require the re-alignment and extension of the entrance channel from Station -60+000 to Station -97+680, which will result in an increase in annual maintenance. See **Table 12.3-2**.

Coastal Zone Management Act (CZM) – Construction of the proposed project would be fully consistent with both Georgia and South Carolina's coastal zone management programs. The new work sediments associated with dredging the entrance channel will be placed in the ODMDS and an existing upland CDF.

Due to their cost effectiveness, hopper dredges are expected to be the primary equipment used to maintain depths in the entrance channel. Those dredges would generally deposit maintenance sediments in the ODMDS, Site 2 or Site 3 because the other nearshore placement areas are too shallow for direct access to hopper dredges. This material could be placed in the nearshore placement areas if a non-Federal interest paid the incremental cost that would be required.

Table 12.3-1: Current Entrance Channel Annual Maintenance Costs

Stations	Disposal Site	O&M Dredge Costs $/cy (Oct 2007 Price Level)	Project Annual Maintenance Volume (cy)	Projected O&M Costs
-85+000 to -57+000	ODMDS	$4.35	10,000	$43,500
-57+000 to -53+000	ODMDS	$4.35	3,000	$13,050
-53+000 to -40+000	ODMDS	$4.35	54,000	$234,900
-40+000 to -30+000	ODMDS	$4.35	325,000	$1,413,750
-30+000 to -20+000	ODMDS	$4.35	281,000	$1,222,350
-20+000 to -10+000	ODMDS	$4.35	163,000	$709,050
-10+000 to 0+000	ODMDS	$4.35	155,000	$674,250
0+000 to 4+000	ODMDS	$4.35	76,000	$330,600
		TOTAL	1,067,000	$4,641,450
			Mob & Demob	$690,000
			Multiple Mobs	$40,000
			Total Mob & Demob	$730,000
			PED	$40,000
			S&A	$100,000
			TOTAL	$5,511,450

Table 12.3-2: Projected SHEP Entrance Channel Annual Maintenance Costs

Stations	Disposal Site	O&M Dredge Costs $/cy Oct 2007 Price Level	Project Annual Maintenance Volume (cy)	Projected O&M Costs
-98+600 to -57+000	ODMDS	$4.35	21,310	$92,699
-57+000 to -53+000	ODMDS	$4.35	3,000	$13,050
-53+000 to -40+000	ODMDS	$4.35	54,000	$234,900
-40+000 to -30+000	ODMDS	$4.35	325,000	$1,413,750
-30+000 to -20+000	ODMDS	$4.35	281,000	$1,222,350
-20+000 to -10+000	ODMDS	$4.35	163,000	$709,050
-10+000 to 0+000	ODMDS	$4.35	155,000	$674,250
0+000 to 4+000	ODMDS	$4.35	76,000	$330,600
		TOTAL	1,078,310	$4,690,649
			Mob & Demob	$690,000
			Multiple Mobs	$40,000
			Total Mob & Demob	$730,000
			PED	$40,000
			S&A	$100,000
			TOTAL	$5,560,649
			Impact to O&M	$49,199

12.4 SUMMARY OF OPERATION AND MAINTENANCE IMPACTS

Currently, Savannah District receives approximately $13M for O&M dredging and maintenance of the upland disposal areas. This does not include funds for dike raising which are described in **Table 12.4-1**. Current practice is to conduct maintenance dredging of critical shoals to the limit of funding and to seek additional funding for the remainder of critical shoals and other shoals growing in the sides of the channel. However, under current conditions, with the sediment basin operational, if the entire channel prism were maintained the cost would be $24,368,190 (current price levels) an increase of approximately $11.4 million (59%) over the present O&M funding.

This information is based on a Sedimentation Analysis done for SHEP; details are in Section 10.0 of this report. The sill constructed at the throat of the Sediment Basin as a mitigation feature along with the discontinued practice of maintaining the Sediment Basin will cause it to fill. This will result in the sediment currently being captured in the Sediment Basin (~1,932,000 cy) being deposited in the navigation channel mainly in the range from Station 24+000 to Station 70+000. This will result in an increase of O&M dredging and maintenance costs to $27,861,490 or an increase attributable to the project of $3,493,300. Of this total, $3,442,927 was attributed to the Inner Harbor while $50,373 was attributed to the Bar Channel. See **Table 12.4-2**.

Table 12.4-1: SHEP Annual Work Plan – Dike Raising Schedule

Year	Project	Total Estimated Costs	Federal Costs	Non-Federal Costs
2010	13A Dikes Raising	$3,078,880	$2,001,272	$1,077,608
	Dike Maintenance	$590,000	$560,000	$30,000
	Mosquito Control	$300,000	$300,000	$0
	Bird Island Construction	$900,000	$600,000	$300,000
2011	JOI Dike Raising	$11,950,000	$7,767,500	$4,182,500
	Dike Maintenance	$597,000	$567,000	$30,000
	Mosquito Control	$300,000	$300,000	$0
2012	Dike Maintenance	$615,000	$615,000	$0
	Mosquito Control	$300,000	$300,000	$0
2013	12A Dike Raising	$9,000,000	$5,850,000	$3,150,000
	Dike Maintenance	$633,000	$633,000	$0
	Mosquito Control	$300,000	$300,000	$0
2014	Dike Maintenance	$640,000	$640,000	$0
	Mosquito Control	$300,000	$300,000	$0
2015	Dike Maintenance	$650,000	$650,000	$0
	Mosquito Control	$300,000	$300,000	$0

**The raising of dikes is coordinated with the rotation schedule of the CDFs which is included in the DMMP Annual Work Plan.*

Table 12.4-2: Average Annual Current & Post SHEP Construction Harbor Maintenance Costs

Range	O&M Dredge Costs $/cy	Current Annual Maintenance Volume (cy)	Current Annual Maintenance Costs	Projected Annual Maintenance Volume (cy)	Projected Annual Maintenance Costs
-98+600 to -57+000*	$4.35	10,000	$43,500	21,580	$93,873
-57+000 to -53+000	$4.35	3,000	$13,050	3,000	$13,050
-53+000 to-40+000	$4.35	54,000	$234,900	54,000	$234,900
-40+000 to -30+000	$4.35	325,000	$1,413,750	325,000	$1,413,750
-30+000 to -20+000	$4.35	281,000	$1,222,350	281,000	$1,222,350
-20+000 to -10+000	$4.35	163,000	$709,050	163,000	$709,050
- 10+000 to 0+000	$4.35	155,000	$674,250	155,000	$674,250
0+000 to +4+000	$4.35	76,000	$330,600	76,000	$330,600
Entrance Channel Total		1,067,000	$4,641,450	1,078,580	$4,691,823
4+000 to 24+000	$5.95	225,000	$1,338,750	225,000	$1,338,750
24+000 to 40+000	$3.25	328,000	$1,066,000	364,000	$1,183,000
40+000 to 50+000	$2.93	260,000	$761,800	900,000	$2,637,000
50+000 to 70+000	$2.80	820,000	$2,296,000	2,076,000	$5,812,800
70+000 to 79+000	$2.40	294,000	$705,600	294,000	$705,600
79+000 to 97+750	$4.06	605,000	$2,456,300	605,000	$2,456,300
97+750 to 102+000	$3.67	1,456,000	$5,343,520	1,456,000	$5,343,520
102+000 to 103+000	$3.67	51,000	$187,170	51,000	$187,170
Inner Harbor Total		4,039,000	$14,155,140	5,971,000	$19,664,140
Sediment Basin	$1.30	1,932,000	$2,511,600	0	$0
Projected Total without MOB & DEMOB		7,038,000	$21,308,190	7,049,580	$24,355,963
MOB & DEMOB Entrance Channel - One Hopper			$690,000		$690,000
MOB & DEMOB Inner Harbor - Two Pipeline Dredges			$2,200,000		$2,200,000
MULTIPLE MOBS			$40,000		$40,000
MULTIPLE MOBS			$40,000		$40,000
Total MOBS & DEMOBS			$2,970,000		$2,970,000
PED			$40,000		$40,000
S&A			$50,000		$50,000
Projected Total for All Costs			$24,368,190		$27,415,963
			Projected Impacts to O&M		$3,047,773

**Station -98+600 is the extended channel stationing for the 48 ft project depth. Channel stationing for the 47 ft project depth across the ocean bar terminates at Station -97+680.*

Annual Savannah Harbor Expansion Project impacts to O&M are shown in **Table 12.4-3**, while **Table 12.4-4** contains the annual impacts to O&M for the NED Plan (47-ft) and the annual O&M project costs after SHEP construction. Cost sharing is 100-percent Federal above 45 feet and 50/50 Federal/non-Federal below 45 feet Cost indexing was accomplished using EM 1110-2-1304 (March, 31 2000) *Civil Works Construction Cost Index System (CWCCIS) (CWBS Code = 12 – Navigation Ports & Harbors)*.

Table 12.4-3: Total Annual Impact to O&M

44 ft Project Depth			
	Cost	Federal Cost	Non-Federal Cost
Oxygen Injection System	$1,110,000.00	$1,110,000.00	$0.00
Inner Harbor O&M Dredging	$2,672,080.00	$2,672,080.00	$0.00
Channel Extension	$46,589.00	$46,589.00	$0.00
Mitigation Features Dredging	$114,000.00	$114,000.00	$0.00
CSS Georgia Curation	$20,000.00	$20,000.00	$0.00
Fish Passage O&M	$50,000.00	$50,000.00	$0.00
Long Term Monitoring	$428,400.00	$428,400.00	$0.00
Total	**$4,441,069.00**	**$4,441,069.00**	**$0.00**
45 ft Project Depth			
	Cost	Federal Cost	Non-Federal Cost
Oxygen Injection System	$908,500.00	$908,500.00	$0.00
Inner Harbor O&M Dredging	$2,672,080.00	$2,672,080.00	$0.00
Channel Extension	$48,155.00	$48,155.00	$0.00
Mitigation Features Dredging	$114,000.00	$114,000.00	$0.00
CSS Georgia Curation	$20,000.00	$20,000.00	$0.00
Fish Passage O&M	$50,000.00	$50,000.00	$0.00
Long Term Monitoring	$428,400.00	$428,400.00	$0.00
Total	**$4,241,135.00**	**$4,241,135.00**	**$0.00**
Continued on next page			

Table 12.4-3: Total Annual Impact to O&M (continued)

46 ft Project Depth			
	Cost	Federal Cost	Non-Federal Cost
Oxygen Injection System	$1,110,000.00	$1,009,250.00	$100,750.00
Inner Harbor O&M Dredging	$2,672,080.00	$2,672,080.00	$0.00
Channel Extension	$48,938.00	$48,546.50	$391.50
Mitigation Features Dredging	$114,000.00	$114,000.00	$0.00
CSS Georgia Curation	$20,000.00	$20,000.00	$0.00
Fish Passage O&M	$50,000.00	$50,000.00	$0.00
Long Term Monitoring	$428,400.00	$428,400.00	$0.00
Total	**$4,443,418.00**	**$4,342,276.50**	**$101,141.50**
47 ft Project Depth			
	Cost	Federal Cost	Non-Federal Cost
Oxygen Injection System	$1,210,400.00	$1,059,450.00	$150,950.00
Inner Harbor O&M Dredging	$2,672,080.00	$2,672,080.00	$0.00
Channel Extension	$49,199.00	$48,677.00	$522.00
Mitigation Features Dredging	$114,000.00	$114,000.00	$0.00
CSS Georgia Curation	$20,000.00	$20,000.00	$0.00
Fish Passage O&M	$50,000.00	$50,000.00	$0.00
Long Term Monitoring	$428,400.00	$428,400.00	$0.00
Total	**$4,544,079.00**	**$4,392,607.00**	**$151,472.00**
48 ft Project Depth			
	Cost	Federal Cost	Non-Federal Cost
Oxygen Injection System	$1,311,000.00	$1,109,750.00	$201,250.00
Inner Harbor O&M Dredging	$2,672,080.00	$2,672,080.00	$0.00
Channel Extension	$50,373.00	$49,264.00	$1,109.00
Mitigation Features Dredging	$114,000.00	$114,000.00	$0.00
CSS Georgia Curation	$20,000.00	$20,000.00	$0.00
Fish Passage O&M	$50,000.00	$50,000.00	$0.00
Long Term Monitoring	$428,400.00	$428,400.00	$0.00
Total	**$4,645,853.00**	**$4,443,494.00**	**$202,359.00**

Table 12.4-4: Annual O&M Cost Increase & Total O&M Annual Costs due to SHEP (47-Ft Project)

ANNUAL O&M INCREASE COST DUE TO EXPANSION										
	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019
Inner Harbor O&M Dredging	$2,672,080	$2,709,489	$2,752,841	$2,799,639	$2,847,233	$2,895,636	$2,944,862	$2,994,925	$3,045,838	$3,097,618
Bar Channel O&M Dredging	$49,199	$49,888	$50,686	$51,548	$52,424	$53,315	$54,222	$55,143	$56,081	$57,034
Oxygen Injection System	$1,210,400	$1,227,346	$1,246,983	$1,268,182	$1,289,741	$1,311,667	$1,333,965	$1,356,642	$1,379,705	$1,403,160
Mitigation Features O&M Dredging	$114,000	$115,596	$117,446	$119,442	$121,473	$123,538	$125,638	$127,774	$129,946	$132,155
Fish Passage O&M	$50,000	$50,700	$51,511	$52,387	$53,277	$54,183	$55,104	$56,041	$56,994	$57,963
CSS Georgia Curation	$20,000	$20,280	$20,604	$20,955	$21,311	$21,673	$22,042	$22,416	$22,798	$23,185
Long Term Monitoring	$428,400	$434,398	$441,348	$448,851	$456,481	$464,242	$472,134	$480,160	$488,323	$496,624
	$4,544,079	$4,607,696	$4,681,419	$4,761,003	$4,841,940	$4,924,253	$5,007,966	$5,093,101	$5,179,684	$5,267,738
ANNUAL O&M PROJECT COST AFTER EXPANSION										
	2010	2011	2012	2013	2014	2015	2016	2017	2018	2019
Inner Harbor O&M Dredging	$21,487,100	$21,787,919	$22,136,526	$22,512,847	$22,895,565	$23,284,790	$23,680,631	$24,083,202	$24,492,617	$24,908,991
Bar Channel O&M Dredging	$3,416,423	$3,464,253	$3,519,681	$3,579,516	$3,640,367	$3,702,254	$3,765,192	$3,829,200	$3,894,297	$3,960,500
Oxygen Injection System	$1,210,400	$1,227,346	$1,246,983	$1,268,182	$1,289,741	$1,311,667	$1,333,965	$1,356,642	$1,379,705	$1,403,160
Mitigation Features O&M Dredging	$114,000	$115,596	$117,446	$119,442	$121,473	$123,538	$125,638	$127,774	$129,946	$132,155
Fish Passage O&M	$50,000	$50,700	$51,511	$52,387	$53,277	$54,183	$55,104	$56,041	$56,994	$57,963
CSS Georgia Curation	$20,000	$20,280	$20,604	$20,955	$21,311	$21,673	$22,042	$22,416	$22,798	$23,185
Long Term Monitoring	$428,400	$434,398	$441,348	$448,851	$456,481	$464,242	$472,134	$480,160	$488,323	$496,624
Estimated $ to Fully Maintain	$26,726,323	$27,100,492	$27,534,099	$28,002,179	$28,478,216	$28,962,346	$29,454,706	$29,955,436	$30,464,678	$30,982,578
Current Funding Appropriation	$13,000,000	$13,000,000	$13,000,000	$13,000,000	$13,000,000	$13,000,000	$13,000,000	$13,000,000	$13,000,000	$13,000,000
Additional Need to Fully Maintain	$13,726,323	$14,100,492	$14,534,099	$15,002,179	$15,478,216	$15,962,346	$16,454,706	$16,955,436	$17,464,678	$17,982,578

** All costs are totals assuming a 47' project. No breakout of Federal vs. Sponsor cost have been applied. (100% Fed to 45', 50/50 Fed/Non-Fed from 45' to NED).*

13.0 COST ENGINEERING

13.1 CONSTRUCTION COST NARRATIVE - CURRENT WORKING ESTIMATE

The Cost Engineering narrative was prepared to describe the Current Working Estimate (CWE) of alternative plan evaluations for the Savannah Harbor Expansion Project (SHEP) deepening. This narrative also provides details for the selected plan of a 47-ft design depth. The alternatives evaluated were to deepen the harbor design depth, existing 42 ft, to a design depth between 44 ft and 48 ft below MLLW. The costs are summarized and listed below in the Code of Accounts format and are based upon October 2010 price levels.

A CWE for construction (dredging & mitigation) and non-construction features were developed for each design depth alternative of 44 ft to 48 ft. The existing (design) depth for the Savannah Harbor is 42 ft below MLLW. Existing advance maintenance depths range from 2 ft to 8 ft below the design depth at various locations. Each alternative includes 2-ft of allowable overdepth dredging quantities. A summary of costs for all alternative design depths (44 ft to 48 ft) is shown in **Table 13.1.1** below and Table 10-5 of the General Reevaluation Report (GRR).

Evaluations of impacts and benefits compared to costs determined the National Economic Development (NED) plan to be a 47-ft design depth (plus 2-ft of allowable overdepth).

The CWE for the selected plan of 47-ft design depth includes new deepening quantities to be dredged, 2-feet allowable overdepth quantities, dredging of river wideners for ship meeting areas, required mitigation features including onsite dissolved oxygen injection systems, river salinity mitigation features, raising existing disposal area dikes, marsh restoration, cadmium sediment removal, adaptive management, environmental monitoring, removal of the Confederate vessel CSS Georgia, Real Estate, Planning, Engineering and Design, Construction Management, and the addition of Navigation Aids.

The CWE and Code of Account features for 47-ft design depth are further broken down into more detail in the Microcomputer Aided Cost Estimating System (MCACES) MII estimate discussed below and shown in Attachment 2 of this appendix (Appendix C: Engineering Investigations).

A Total Project Cost Summary (TPCS), Attachment 2 of this appendix (Appendix C: Engineering Investigations), identifies the CWE for the 47-ft design depth for October 2010 as $483,738,000 ($604,673,000 with 25% contingency) and fully funded to midpoint of construction as $688,118,000 with contingency.

The TPCS estimate was reviewed by Cost Center DX for certification during the ATR. The TPCS and COST CENTER DX certification is included with Attachment 2 of this appendix (Appendix C: Engineering Investigations).

Overall construction midpoint for dredging and mitigation will vary depending on contract acquisition method but overall is estimated to be October 2014. Construction completion is estimated to be OCTOBER 2016 with post construction monitoring and adaptive management to follow construction completion.

Cost Estimates were prepared under guidance given in the Corps of Engineers Regulation ER 1110-2-1302, *Civil Works Cost Engineering* and ETL 1110-2-573, Cons*truction Cost Estimating Guide for Civil Works*, dated 30 Sep 2008. The Corps of Engineers Dredge Estimating Program (CEDEP) was used in developing dredging costs throughout the project for medium hopper dredges and large pipeline cutterhead dredges. A Cost Risk and Schedule Analysis (CRSA) was conducted with the Project Delivery Team and the Center of Expertise at Walla Walla District to support a 25% contingency percentage for risk and uncertainty. A final copy of the Cost Risk Report, Risk Register, Cost models, and sensitivity analysis is included as Attachment 4 of this appendix (Appendix C: Engineering Investigations).

Table 13.1-1: Cost Summary for All Alternative Project Depths (44 ft to 48 ft)

WBS NO.	FEATURE DESCRIPTION	44 FT	45 FT	46 FT	47 FT	48 FT
1	REAL ESTATE	$4,701,250	$15,553,250	$17,666,250	$18,605,625	$21,825,625
6	FISH & WILDLIFE FACILITIES - Mitigation	$198,592,370	$202,598,595	$206,231,095	$212,292,345	$215,777,345
	Fish Passage – New Savannah Bluff Lock & Dam	$29,577,470	$29,577,470	$29,577,470	$29,577,470	$29,577,470
	McCoy Cut Diversion Channel	$2,905,103	$2,905,103	$2,905,103	$2,905,103	$2,905,103
	Deepen McCoy Cut, Upper Middle & Little Back Rivers	$0	$9,109,975	$9,109,975	$9,109,975	$9,109,975
	Rock Berm At Mouth of Back River	$21,030,061	$21,030,061	$21,030,061	$21,030,061	$21,030,061
	Close Rifle Cut	$828,914	$828,914	$828,914	$828,914	$828,914
	Remove Tidegate Piers And Abutments	$3,575,643	$3,575,643	$3,575,643	$3,575,643	$3,575,643
	Embankment At Tidegate Removal	$17,969,583	$17,969,583	$17,969,583	$17,969,583	$17,969,583
	Close Lower McCoy's Cut Western Arm	$1,421,946	$1,421,946	$1,421,946	$1,421,946	$1,421,946
	Boat Ramp	$624,953	$624,953	$624,953	$624,953	$624,953
	Construct Marsh At Disposal Island 1S	$17,594,949	$17,594,949	$17,594,949	$17,594,949	$17,594,949
	Water Impoundment	$25,187,500	$25,187,500	$25,187,500	$25,187,500	$25,187,500
	Broad Berm Sediment Basin - Back River	$8,362,500	$8,362,500	$8,362,500	$8,362,500	$8,362,500
	Mitigation Costs For Striped Bass	$2,085,000	$356,250	$613,750	$3,300,000	$3,410,000
	Construct Dissolved Oxygen Injection Sys	$67,428,750	$64,053,750	$67,428,750	$70,803,750	$74,178,750
12	NAVIGATION, PORTS AND HARBORS	$160,137,581	$193,721,068	$222,579,256	$251,169,942	$279,735,740
	Mobilization And Demobilization	$26,500,000	$27,625,000	$27,625,000	$29,625,000	$30,500,000
	Dredging	$118,464,265	$149,259,623	$175,073,167	$199,292,615	$224,612,951
	Disposal Area Requirements - Dike Raises	$1,188,316	$2,851,445	$5,896,089	$8,267,328	$10,637,789
	Cadmium Sediment - Dike Raises 14A & 14B	$11,875,000	$11,875,000	$11,875,000	$11,875,000	$11,875,000
	Debris Removal	$2,110,000	$2,110,000	$2,110,000	$2,110,000	$2,110,000
31	MITIGATION MONITORING and ADAPTIVE MGT	$58,792,500	$59,818,750	$60,160,000	$60,195,000	$60,468,750
18	CULTURAL RESOURCES - CSS Georgia	$13,914,375	$13,914,375	$13,914,375	$13,914,375	$13,914,375
30	PLANNING, ENGINEERING & DESIGN	$21,933,037	$23,873,642	$25,522,693	$27,257,824	$28,887,677
31	CONSTRUCTION MANAGEMENT	$10,966,519	$11,936,821	$12,761,346	$13,628,912	$14,443,838
PROJECT FIRST COSTS WITH CONTINGENCIES		**$469,037,632**	**$521,416,501**	**$558,835,015**	**$597,064,023**	**$635,053,350**
	Berthing Areas	$2,216,414	$2,401,299	$2,551,629	$2,583,564	$2,854,826
	Navigation Aids	$5,025,000	$5,025,000	$5,025,000	$5,025,000	$5,025,000
PROJECT FIRST COST WITH ASSOCIATED COSTS & CONTINGENCIES		**$476,279,046**	**$528,842,799**	**$566,411,644**	**$604,672,587**	**$642,933,176**
TOTAL - ROUNDED OCT 2010 PRICE LEVEL		**$476,279,000**	**$528,843,000**	**$566,412,000**	**$604,673,000**	**$642,933,000**

Current MCACES Estimate Prepared: January 4, 2012.

13.2 CODE OF ACCOUNTS

A CODE OF ACCOUNTS format is established to provide a consistent organization of major costs by task and type. Summary of the Code of Accounts for the 47 ft project depth is shown in **Table 13.2-1**.

Table 13.2-1: Summary of Code of Accounts for the 47 ft Project Depth

Code Of Accounts	47 ft Depth $ millions
Lands and Damages (01) – Real Estate	$ 15
Fish and Wildlife Facilities (06) Fish Passage – New Savannah Bluff Lock & Dam Mitigation Plan 6a – McCoy's Cut Diversion Structure Close Western McCoy Cut Connection Deepening McCoy's Cut – Little Back and Middle Back River Close Rifle Cut Channel Tidegate Removal Broad Berm Stone Weir Broad Berm Fill Tidegate Embankment Removal Marsh Restoration at Island 1S New Boat Ramp Raw Water Impoundment Dissolved Oxygen Generation Sites	$170
Navigation, Ports, and Harbors (12) Outer Bar Channel Dredging Inner Bar Channel Dredging Misc - Raise Existing Disposal Dikes Navigation Aids	$207
Cultural Resource Preservation (18) – Recovery of CSS Georgia	$ 11
Planning, Engineering and Design (30) – Investigations, Plans and Specifications	$ 22
Construction Management (31) – Contract Admin, Construction Inspection & Adaptive Management	$ 59
TOTAL	**$484**
Project Cost and Schedule Risk Analysis: Contingency Summary 25%	$121
TOTAL with Contingency	**$605**

Lands and Damages (01) – The estimated cost was furnished by the Real Estate Division, Savannah District, and is discussed in the Real Estate Appendix.

Fish and Wildlife Facilities (06) – This account includes costs for mitigation including a fish passage structure at New Savannah Bluff Lock and Dam, salinity mitigation for deepening the harbor as outlined by Mitigation Plan 6a, creation of a marsh island habitat at Disposal Area 1S, removal of the Tidegate structure, a 97 MG raw water storage impoundment (a 25 acre footprint with 20 ft high earthen berm) with mechanical mixing, and an on-site oxygen generated D.O. injection systems at two (2) locations along the harbor.

Emphasis was placed on accuracy of quantities, material characteristics, and detail costs during evaluation of alternative plans to develop the CWE Plan. The reasonableness of costs developed was evaluated based on historical data, discussions with industry, crew production rates and construction methods based on similar projects.

A Fish Passage at New Savannah Bluff Lock and Dam (FPSBLD) will be constructed to allow fish passage upstream of the dam. The structure would consist of a rock ramp constructed around the side of the dam. The primary construction tasks involve excavation of approximately 275,000 cubic yards of material around the end of the dam and placement of 116,000 tons of rip rap and boulder weir stone. A submerged sheet pile wall would be constructed at a height of 3 to 4 feet above the river bottom to guide fish in both upstream and downstream directions. Existing gates 1 and 5 would also be raised from 12 ft to 15 ft.

A combination of features will be constructed to mitigate for potential salinity impacts in the river as described by Mitigation Plan 6a. More detail of all these features are described in Chapter 8.1 of the Engineering Appendix. McCoy's Cut Diversion feature is diverting a portion of flow from Front River into the upstream areas at McCoy's Cut. This feature includes constructing a diversion structure with 168 tons of sheet pile wall and 7,100 tons of GADOT Armor Stone at the entrance to McCoy's Cut. Additionally a portion of lower western McCoy's connection will be closed using over 5,100 tons of GADOT TYPE 1 stone.

McCoy's Cut and upper portions of Middle River and Little Back River will be deepened to accommodate the additional diverted flow. Estimated deepening quantities excavated for McCoy's Cut, Middle and Little Back Rivers will be 315,000 cy. This material will be disposed of in the existing Dredge Confined Disposal Facilities used for dredged material disposal.

Rifle Cut waterway will be closed using 3,300 cy of fill sediment and 2,500 tons of GADOT, Type 1 rip rap to improve flow diversion.

Additional features include removal/demolition of the Tidegate structure piers and abutment (elevated walkways and associated appurtenances) as well as the Tidegate embankment removal to widen the river. The approximately 1,000,000 cubic yards of embankment at the Tidegate abutments will be removed and disposed of in either the Broad Berm fill area, if it is suitable fill, or existing confined dredge disposal areas.

A submerged Broad Berm stone weir will be constructed at the conversion of Back River with Front River. The stone weir is located downstream of the Tidegates and sedimentation basin. The stone weir will be constructed of 97,000 tons of GADOT, TYPE 1 stone to -9.5 MLLW.

In addition, a Broad Berm fill, upstream or behind the stone weir, will require 1.2 million cubic yards of suitable (sandy) fill material. The suitable fill is assumed to come from existing dredge confined disposal areas and/or the embankment removal at the Tidegate.

A marsh island habitat restoration of 42 acres will be developed and restored at the current location of Disposal Area 1S to an elevation of +7.6 ft MLLW. Construction includes excavation/grading of 435,000 cy of material with assumed disposal into existing confined disposal areas.

A proposed public access Boat Ramp on Hutchinson Island includes a 2 lane concrete boat ramp with floating dock, 20 space trailer parking, handicap accessible and single car parking spaces.

Onsite dissolved oxygen injection systems will be developed and constructed at two locations along the river. The two sites will have multiple Speece cones to generate oxygen into the harbor.

A 97 million gallon (77.5 MG usable) Raw Water Storage Impoundment will be constructed in order to stabilize and reduce any necessary chloride concentrations before being pumped into the City of Savannah's water treatment plant. An earthen embankment with HDPE liner, mechanical mixing system, four (4) pump stations rated at 21 MGD each, and a powdered activated carbon treatment system will be included in the construction.

Navigation, Ports, and Harbors (12) – The 47-ft NED plan includes dredging of approximately 23,603,000 cy of material from 38 miles of both the Ocean Bar entrance channel and Inner River Harbor.

River Stations are labeled 4+000 to 103+000 for an Inner Harbor design depth of 47-ft MLLW plus 2-ft of allowable overdepth. The Outer Bar includes 2 ft additionally below MLLW, or 49-ft plus 2-ft of allowable overdepth in outer ocean bar 4+000 to -97+680.

Also included in Account 12 are costs for raising existing disposal area dike elevations/capacity, debris removal, new navigation aids, and removal/containment of any cadmium sediment dredged from the river. Pricing was developed assuming both medium hopper dredges and large pipeline cutterhead dredges may be used.

Average bank heights for new work dredging are approximately 5-6 ft. Quantities also include 2 ft of allowable pay overdepth (to ensure a sufficient design depth of 47-ft is met for vessel traffic). Areas and depths of advanced maintenance will be maintained or remain at the same reaches/locations as they have been located historically.

The Corps of Engineers Dredge Estimating Program (CEDEP) was used to determine dredging average unit costs for each reach of location along the 38 miles. Significant factors input into CEDEP for determining unit prices and dredging time includes: Dredge Area, Dredge Depth-bank height, Non-pay overdepth cubic yardage, Material Factors – silt, sand, and clay, Pumping Distance or Haul Distance to Disposal Areas, EWT- Effective Work Time when dredging, Production cy/hr when dredging, Production cy/day average, and other monthly costs – such as land equipment, field office overhead, turtle monitoring, site specific maintenance costs, pipeline wear costs and fuel pricing. All of these factors are critical for developing a reasonable price estimate for various locations and conditions. The equipment most likely to be used for dredging excavation was assumed to be:

- A hydraulic pipeline dredge for the Inner Harbor (Stations 4+000 to 103+000), about 19 miles. Dredge material will be placed into existing confined disposal areas 13A, 14A, 14B and Jones Oyster Island.

- A combination of hydraulic pipeline (loading scows & tug hauled) and hopper dredges for the Outer Ocean Bar (Stations 4+000 to –98+600), about 19 miles. Material will be placed in the existing EPA approved ODMDS.

The cost estimate for all dredging construction anticipates two contracts and covers a four-year period of concurrent work. The Inner Harbor will be one contract and the Ocean Bar will be another contract.

Cultural Resource Preservation (18) – This account includes the costs for removal of remnants of the CSS Georgia and was provided by Savannah District archeologist. The costs are based on discussions with the various archeological firms who are experienced in removal and conservation of historical shipwrecks in tidal waters. The CSS Georgia has been impacted severely over the years due to O&M dredging operations.

Planning, Engineering and Design (30) – 5% of construction costs were included in this account and supported by those responsible for performing each activity during PED. This account includes plans and specifications, field investigations and surveys, cost estimates, engineering during construction, and project management.

Construction Management (31) – 2.5% of construction costs were included for supervision and administration of the contracts by construction management, hydrologic surveys during construction, contracting personnel during construction, environmental/physical monitoring, and adaptive management following monitoring phases.

Project Cost and Schedule Risk Analysis: Contingency – An overall project contingency of 25% was developed during a cost/risk analysis conducted with the Project Delivery Team (PDT) and Walla Walla District Cost Center of Expertise.

The updated and final CRSA which includes a Cost Risk Report, Risk Register, Cost models and sensitivity analysis is included as Attachment 4 of this appendix (Appendix C: Engineering Investigations).

13.3 ASSUMPTIONS AND SCHEDULE

Overall construction period for dredging and mitigation features is anticipated to be approximately 4 +/- years, FEBUARY 2013 to OCTOBER 2016. Construction contracts for dredging and mitigation are assumed to be concurrent during this 4 year period. Pre construction environmental monitoring and monitoring during construction will be performed. Post construction monitoring and adaptive management for mitigation will follow dredging and mitigation construction.

- GENERAL CONSTRUCTION

- DREDGING – INNER HARBOR AND OUTER BAR CHANNEL

- MITIGATION FEATURES
  Fish Passage Structure
  Rip Rap at McCoy's Cut Diversion, Rifle Cut Closure, and Broad Berm Stone Weir
  Deepening McCoy's Cut, Little and Middle Back Rivers
  Fill for Rifle Cut
  Removal of Tidegate – Abutments and Piers
  Tidegate Embankments Removal
  Broad Berm fill
  Marsh Restoration at Island 1S
  Boat Ramp
  Dissolved Oxygen injection systems
  Raw Water Storage Impoundment

- PROJECT COST AND SCHEDULE RISK ANALYSIS

13.3.1 General Construction

The initial construction will be to raise dikes to have adequate capacity for dredge disposal quantities. Construction of Dissolved Oxygen systems, raw water storage impoundment, and removal of the CSS GEORGIA will be concurrent with raising of the dikes.

13.3.2 Dredging - Outer Bar and Inner Harbor

The Ocean Bar Channel contract was assumed to begin November 2013 and disposal will be in the ODMDS. The OUTER BAR was assumed to be constructed with both hopper dredges and/or hydraulic cutterhead pipeline dredges. Either pipeline (filling dump scows and tug hauled) and/or the hopper dredges would place material in the ODMDS.

For the Outer Ocean Bar, cutterhead pipeline dredges (filling dump scows and tug hauled to the ODMDS for disposal) would not be restricted to work only during the winter dredging season. Hopper dredges would be allowed to work during the months of December 1 thru March 31.

The Inner Harbor contractor was assumed to place dredged material into existing confined disposal areas 13, 14A, 14B and Jones Oyster Island. The Inner Harbor contractor would be required to avoid upstream of Station 66+310 during the period April 1 – May 15 to avoid impacts to striped bass fishery resources. Dredging the Inner Harbor between June 1 and September 30 may also be impacted if dissolved oxygen levels fall below 4 to 5 ppm.

Several areas to be dredged within the Inner Harbor contain cadmium sediments. These areas are to be placed in disposal dikes 14A & 14B. These dikes will then be capped with non-cadmium dredged materials. Construction sequence will be important to dredge cadmium materials first and then cap with non-cadmium dredged material.

There will be limited debris removal in both contracts made necessary because of sinkers and other miscellaneous debris which may have accumulated since the last deepening.

Disposal of 12 to 15 million cys of material from the Inner Harbor, Stations 4+000 to 103+000 would be into existing Confined Disposal Facilities labeled 13A, 14A&B, and Jones Oysterbed Island.

Disposal of 11 to 13 million cys of material from the Outer Harbor Bar Stations 4+000 to -97+680 would be into the existing EPA approved ODMDS.

The average dredging production (18,000 to 21,000 cy/day) and unit costs appear reasonable when compared to historical information for production and pricing (adjusted to October 2010).

13.3.3 Mitigation Features

The Fish Passage at the New Savannah Bluff lock and dam will be constructed by excavating around the end of the dam and placement of rock ramp rip rap and weir stone. The contract time period was assumed to be 700 cal days. There will be no downstream inwater construction during the months of February through May.

RIP RAP features for the Diversion Structure at McCoy's Cut; Closure of lower Arm at McCoy's cut; Rifle cut closure; and the stone weir at Sediment Basin Broad berm were assumed to be constructed by loading rip rap onto material barges from docks and hauled by tug to the individual sites. RIP RAP will be dumped and/or placed using barge mounted cranes with rock boxes or buckets. The contract time period assumed for these contracts are listed below.

- Diversion Structure at McCoy's Cut (includes 140 LF sheetpile) – 150 cal days
- Western Lower Arm at McCoy's Cut – 150 cal days
- Rifle Cut closure (Rip rap + Fill) – 150 cal days
- Broad Berm weir stone – 420 cal days

Deepening excavation for McCoy's Cut, Middle and Little Back River channels was assumed to be by barge mounted clamshell/shovels loading material into hopper barges. Hopper barges will then be tugged to the existing confined disposal sites and unloaded or pumped out into the disposal areas. The contract time period assumed for this contract is 510 cal days.

Suitable Fill material for Rifle Cut closure was assumed to come from within existing confined dredge disposal areas. The material to be suitable must be mostly sandy. Material would be excavated from the existing disposal areas and loaded into hopper barges for transport. Hopper barges would be transported by tug and then unloaded/pumped out at the construction fill site.

Rifle Cut closure (Rip rap + Fill) – see above anticipated contract period.

Removal of Tidegate Abutments and Piers – It was assumed the 15 concrete piers, walkways, and abutments will be broken down by blasting/mechanical methods. Concrete pieces would be loaded by barge mounted cranes onto material barges and/or land for stockpiling and removal from the site. Removal of earthen abutments/embankments, existing rip rap, stacked gates, conduit hardware, lighting, handrails and utilities are also required. The contract time period was assumed to be 365 cal days.

Tidegate embankments (approx 1,000,000 cy's) are to be excavated to widen the river. Suitable embankment (sandy) material may be excavated by barge mounted equipment and placed into the nearby Broad Berm fill. One half of material (525,000 cy) was assumed to be placed onto flat or confined hopper barges and then unloaded by dumping or pumpout into the Broad Berm. Turbidity limits cannot be exceeded when performing this work. The remaining unsuitable material from embankments was assumed to be pumped or dredged into the existing confined disposal facilities. The contract time period was assumed to be 540 cal days.

Broad Berm fill material, 1,200,000 cubic yards, was assumed to come from either/or Tidegate embankment removal or existing confined dredge disposal areas. It was assumed suitable material in the disposal areas would be excavated from the existing disposal areas and loaded into hopper barges for transport. Hopper barges would be transported by tug and then unloaded/pumped out at the construction fill site areas. Small portable pipeline cutterhead dredges may also be used in the disposal areas or at the Tidegate location to pump suitable material into the fill area.

Marsh Restoration at Area 1S (Onslow Island) – Areas designated to be restored must be cleared and grubbed. It was assumed that an entrance channel will be excavated from Middle River into the island area using barge mounted crane clamshell/shovels and material loaded into hopper barges. Hopper barges will be towed to the existing confined disposal areas 12 thru 14 and unloaded/pumped out. The entrance channel excavation will continue into the interior of the island and remove approximately 425,000 cy of material with disposal into the CDA's 12 thru 14. Land equipment will then backfill the excavated area and grade to elevation +7.6 MLLW. Suitable material may be used to fill Broad Berm near existing sedimentation basin. The contract time period was assumed to be 540 cal days.

A new public access boat ramp will be constructed on the North side of Hutchinson Island (at the site where Tidegate embankment abutment is removed). The public access Boat Ramp includes a 2 lane concrete boat ramp with floating dock, 20 space trailer parking, handicap accessible and single car parking spaces. The contract time period was assumed to be 365 cal days.

The Dissolved Oxygen injection systems will be land based at two locations, with water being withdrawn from the river, super-saturated with oxygen, and returned to the river. The sites will require development of access roads, concrete platform for work areas and to support Speece cones, intake/discharge piping systems, electrical service, perimeter fencing and multiple Speece cones per site. The contract time period was assumed to be 365 cal days for each location.

A Raw Water Storage Impoundment will be constructed of earthen embankment with HDPE liner , mechanical mixing system, four(4) pump stations rated at 21 MGD each, and a powdered activated carbon treatment system will be included in the construction. The contract time period was assumed to be 365 cal days.

13.3.4 Project Cost and Schedule Risk Analysis: Contingency

An overall project contingency of 25% was originally developed during a cost/risk analysis conducted with the PDT and Walla Walla District Cost Center of Expertise. The 25% contingency represents an 80% confidence level for the project overall. Details of the cost/risk analysis are further described in the Project Cost and Schedule Risk Analysis Report (CSRA) which is included as Attachment 4 of this appendix (Appendix C: Engineering Investigations).

A contingency percentage is the amount added to an estimate to allow for uncertainty of items, conditions, or events that impact a project. Many of these items are known to exist but it is uncertain how much these items will vary depending on many factors.

The overall contingency of 25% was the result of statistical Monte Carlo simulations for many elements within each cost and schedule that could occur during the life of the project.
The best examples of risks/uncertainties are fluctuations of fuel prices, labor, material prices or availability, how work will be performed, competitive bid environment, multiple year contracts and schedules, funding constraints, site condition material factors and quantities, etc.

Experience shows that many items formulated during the estimate construction first costs will vary (be uncertain) during the life of a project and likely result (overall) in additional costs. In other words, many items are known factors with variances which will increase/decrease costs in some manner. The question is how to account for the likelihood of these items changing the project costs and what confidence level cost changes will be correct (or costs exceeded).

Questions of risk and uncertainty may be addressed by a problematic risk analysis. A risk analysis is a systematic and comprehensive method to evaluate uncertainty and risks. Risks were characterized by the magnitude of possible uncertainties and the probability of occurrence for each item or event. Details for events identifying likelihood, impact, and risk level are shown in the Risk Register, Attachment 4 of this appendix (Appendix C: Engineering Investigations).

Using computer software to conduct Monte Carlo simulations and statistical sensitivity, key risk drivers were identified as listed below:

- Risk Events I-37 & I-38 fuel increases from $2.70/gallon up to $6.00/gallon
- Risk Event I-20 & I-36 competition or competitive bid environment
- Risk Event I-41 Construct the Dissolved Oxygen Injection System
- Risk Event I-33 construction contract schedules for dredging

Together these risk driver's are the majority of the statistical cost variance.

Cost contingencies calculated for the 80% confidence level resulted in a 25% contingency. The 80% confidence level means there is an 80% certainty that project costs will not exceed the total with contingency.

14.0 VALUE ENGINEERING

A Value Engineering (VE) Study for the project was conducted in Savannah, Georgia on May 20-22, 2008. The VE team was comprised of members from Wilmington, Charleston, and Savannah Districts USACE, Georgia DOT, and Concord Project Consulting, Inc. and associated consultants. Findings and recommendations from the VE team are presented in full in their report titled *Value Engineering Study Summary Report* which is included in the Engineering Investigations Supplemental Materials.

Value Engineering is a process used to study the functions a project is to achieve. The VE Team took a critical look at how these functions are proposed to be met and it identified alternative ways to achieve the equivalent function while increasing the value and the benefit ratio of the project. In the end, it is hoped that the project will realize a reduction in cost, but increased value is the focus of the process, rather than simply reducing cost. USACE guidance on conducting VE studies in place at the time the SHEP VE study was conducted was ER 11-1-321, dated 28 February 2005. Current USACE guidance on conducting VE studies is ER 11-1-321 Change 1, dated 1 January 2011.

The VE Study identified $34,221,436 in potential savings for project construction. Each of the proposals included in the VE Study recommendations was evaluated by the Engineering team to determine constructability and performance to meet project requirements. A summary of the team's findings is included in **Table 14.0-1**. Overall, all of the major design changes proposed (for savings more than $1 million) were not considered to be viable alternative designs.

The major savings identified were for proposals 4 and 7 were both for alternative methods for constructing the McCoys Cut diversion structure. After further analysis, it was determined that an error had been made in the original rock volume for this structure. When the revised volume was carried through to the cost estimate for this structure, the actual cost for construction of a rock structure was $400,000, which was much less than the cost previously estimated for this feature or the costs estimated for proposals 4 and 7. Modification of this structure was not determined to be cost effective. Some of the smaller project recommendations, such as reuse of concrete from the tidegate or use of sand from 2A for the sand sill at Rifle Cut (proposals 3 and 6) may be cost effective and will be considered during the planning, engineering and design phase of this contract, when more site specific information is available.

Subsequent to the Value Engineering Study the design for the overall project was modified to include additional mitigation features. The following features were added to the project:

- Increase in the number of Oxygen Systems- $21 million
- Increase in the dimensions of the NSBL&D fish passage - $21 million
- Construction of a recreational boat ramp in Back River- $600,000
- Restoration of 35 acres of salt marsh in Disposal Area 1S- $18 million
- Construction of a raw water impoundment- $25 million

The methods of construction for the majority of the added features for mitigation are almost identical to the other methods of mitigation that were included during the VE Study. Since there were no VE proposals accepted as a result of the VE Study it was determined that no additional value engineering study was required at that time. Value Engineering Proposals which would result in cost savings to the project will continue to be considered throughout the design and construction of this project. In addition, USACE guidance requires the project features be re-evaluated during the design phase prior to construction.

Table 14.0-1: Evaluation of Value Engineering Proposals

Proposal No.	Proposal	Estimated Savings from VE Study	Designer Evaluation
1	Consider all cut-off structures/core to be made out of Geo-tubes with a layer of rock over	$0- Not Recommended	The use of Geo-tubes in high velocity areas and in deep water is not recommended.
2	Make all cut-off walls using rock filled gabions in lieu of rock	$11,733,000 Cost Added	Not recommended. Would result in additional cost to the project.
3	Re-use concrete from Tidegate for core of Sediment Basin Sill	$770,493	Potential cost savings during PED; however, removal of rebar would have to be added to cost.
4	Use sheet pile in lieu of rock at McCoy Cut for construction of Diversion Structure	$16,230,600	This evaluation was based on use of 81,000 tons of rock for construction of the diversion structure. After further analysis it was determined that the quantity of rock was overestimated and should only be 2,200 tons, resulting in a cost of $400,000, which would be less than the estimated cost of the sheet pile. Further design of the diversion structure is planned during PED and alternative construction methodologies will be revisited.
5	Reclaim rock from New Cut Area above mean high water, for closure structures	$384,315	This proposal is not recommended due to the disruption/destruction of established marsh in this area to gain access to the rock.
6	Use sand from the southern part of 2A for sand fill at Rifle Cut plug	$4,039	This proposal is recommended and should be included in the PED phase of the project.
7	Use precast concrete and "H" piles for closure structures at McCoy Cut Diversion Structure	$16,725,900	This is basically an alternative design for the structure discussed in proposal no. 4. The actual quantity of rock to be placed was previously overestimated and would only cost $400,000, which would be less than the estimated cost of the concrete "H" piles. Further design of the diversion structure is planned during PED and alternative construction methodologies will be revisited.
8	Install piping between DO system and river using tunnels to prevent cutting down or disturbing the ground cover	$106,089	Analysis recommends HDPE instead of DIP pipe. HDPE pipe could collapse. Tunnel option would require use of both DIP pipe and HDPE pipe. Not recommended.

15.0 RISK AND UNCERTAINTY

As part of the SHEP Engineering Investigations, uncertainty with predictions, estimates, and assumptions for project design and cost determination, including mitigation planning, were evaluated and potential risks were identified. See **Table 15-1**, on the next two pages, for a summary of SHEP risk and uncertainty discussion.

Table 15-2 outlines details on risk and uncertainty for possible sea level changes in Savannah Harbor. Analysis and rates are based on EC 1165-2-211; *Water Resource Policies and Authorities Incorporating Sea-Level Change Considerations in Civil Works Programs* dated July 1, 2009.

Table 15-1: SHEP Engineering Investigations Risk and Uncertainty

Feature	Risk/Uncertainty Short Term	Risk/Uncertainty Long Term	Methods Employed to Minimize Risk/Uncertainty	Discussion
Salt water Intrusion in the Aquifer	Low	Moderate	Use of well established industry standards for field data collection. Verification of vertical hydraulic conductivity in the field. Extensive field observations. Use of a well accepted and time proven regional flow model developed and approved by USGS. Verification of model results.	Moderate risk in the long term is not due to the predictive methods employed, but rather uncertainties in the future pumping rate for the City of Savannah, which could have a far greater impact on the aquifer than channel deepening.
Ship Simulation	Low	Low	Model is well established. Has been used extensively for navigation projects worldwide. Calibrated with detailed mariner input and evaluation of recorded data.	There is a low residual risk in the fact that a simulation is not the real world and unforeseen situations not modeled may occur.
Channel Side Slope Stability	Low	Low	Generally, in the areas of existing shoreline structures, the existing side slope will be maintained by reducing the channel width. Exceptions occur in the meeting lanes and curve wideners. Established well documented models are used to predict side slopes.	Risk and uncertainty are minimized due to the long period of observations of harbor channel conditions, the extensive number of borings that have been drilled, and the use of well established models.
Relative Sea Level Change *(See **Table** 15-2 for additional details on sea level rise risk and uncertainty.)*	Low	Moderate	There is a great deal of uncertainty about the potential sea level rise over the next 50 to 100 years throughout the scientific community. To deal with these uncertainties, the project was evaluated with a range of sea level rise scenarios. These rates were calculated using the most recent guidance and the recommended methodology.	Risk and uncertainty are low for project structures and channels. Overall, the functioning of the navigation project is not expected to be impacted significantly due to sea level rise. One area of concern it that freshwater impacts are overstated due to the use of the base year for determining impacts rather than some point in the future when sea level will impact these areas with or without the project. Although there is a risk of mitigating more than required, the impact on the environment at year one without this mitigation could be significant.
Environmental Impacts	Low	Low	Establishment of predicted environmental impacts are only as good as the predictive models and available data on existing conditions. Extensive modeling with approved models was done to predict these impacts. Extensive data documentation of existing conditions was also collected. Lastly, an extensive post project monitoring plan and adaptive management plan are included in the project to address unforeseen impacts due to the project.	Risks that remain are the predicted impacts of the project may be understated and the project will not adequately mitigate for project construction or that the predicted impacts are overstated and the project will overmitigate for project construction. The long term monitoring and adaptive management reduce this overall risk significantly.
Hydrodynamic and Water Quality Models	Low	Low	Large sets of data were used for model calibration. Uncertainty analysis was performed by KAC, independent of the model development. Uncertainty analysis results were incorporated into the model grid.	Risks remain with the predictive models, including 1) inaccuracies in predicting salinities and chloride concentrations, particularly in areas where small variations can impact critical habitat for fish or water quality for industrial use 2) inaccuracies in predicting DO, which could lead to ineffective mitigation design. Even with the best data and predictive models, some uncertainties remain in predicted impacts. The post construction monitoring and adaptive management will address these concerns.

Continued on next page

Table 15-1: SHEP Engineering Investigations Risk and Uncertainty (Continued)

Feature	Risk/Uncertainty Short Term	Risk/Uncertainty Long Term	Methods Employed to Minimize Risk/Uncertainty	Discussion
Underground Utility Pipelines	Low	Low	An inventory was taken of the pipelines underneath the navigation channel. Pipelines were researched to verify their location and ensure that adequate overburden will remain after dredging. Industry input was collected to determine the safest methodology to remove overburden.	Risk of damage to pipelines that cross the channel is inherent in the dredging and navigation process and slightly increased due to reduced overburden. Risk includes, 1) ships that lose steering capability in the navigation channel may drop and/or drag anchor to maintain position, potentially causing damage in the area of a pipeline crossing, 2) an error in dredge position, meaning dredge could "spud" (20 ton cylindrical tubes used as pivot points for dredge movement) down on top of pipeline crossing.
Ship Forces on the Shoreline	Low	Low	The study shows the sum of power from all ships to be less in the deepened channel than the existing channel; therefore, bank erosion due to ship wake should not be increased as a result of deepening the channel. Uncertainty in the prediction is low due to use of proven analytical methods and field data collection. Assumptions made about future ship traffic were conservative.	Erosion due to the ship traffic is minimal when compared to coastal wind and wave processes acting on the shoreline; therefore risk associate with ship traffic wake erosion is low.
Bank Erosion Study	Low	Low	Studies specifically addressed Fort Pulaski and North Tybee Island. Three separate studies were conducted in these areas. Two by USACE (ERDC and Savannah District) and one by Skidaway Institute of Oceanography (for Fort Pulaski). All reached similar conclusions on the magnitude of shoreline recession.	Bank erosion due to ship traffic is much lower than erosion due to other causes. Risk of underestimating erosion due to ship traffic is low relative to impacts of erosion due to other causes. It can be assumed that erosion control measures will be put in place as needed to address this erosion.
Coastal Erosion Study	Low	Low	Coastal models produced similar results to those which have occurred in the historic record. Extensive data is included in the historic record due to extensive records in this area.	Risk is low for increases in erosion due to channel deepening. Monitoring of the Federal Project will continue and adjustments to the beach renourishment project will be made if erosion rates increase after deepening.
Sedimentation and Shoaling	Low	Low	Previous deepening and widening projects have not resulted in an increase in the overall volume of material shoaling in the harbor. Uncertainty is related to where shoaling will occur in the harbor due to discontinued use of the sediment basin.	Based on the historic records as well as salinity and hydrodynamic models, shoaling locations and volumes have been predicted for the deepened project. If shoaling locations vary from those predicted, the advance maintenance for the project may be adjusted in some areas. Adequate O&M funding will be required to maintain the channel in areas outside of the sediment basin.
Cost Risk Analysis	Low	Low	A cost risk analysis was performed by the USACE Center of Expertise in the Walla Walla District to support contingency percentages for risk and uncertainty.	The risk for unanticipated costs of the project exists, however this risk is considered low due to the high level of design for this feasibility project and the contingencies included in project cost.

Table 15-2: SHEP Engineering Investigations Risk and Uncertainty for Sea Level Rise

Project Component	Year	Low Rate of Sea Level Rise (0.01 ft/yr) (Historic)		Intermediate Rate of Sea Level Rise (EC 1165-2-211, Curve 2)		High Level of Sea Level Rise (EC 1165-2-211, Curve 3)	
		SLR (ft)	Impacts/Risk	SLR (ft)	Impacts/Risk	SLR (ft)	Impacts/Risk
Dredging, Entrance Channel and Inner Harbor	1	0	Base Condition	0	No Change- No Risk	0	No Change- No Risk
	25	0.3	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom, dredging requirements will stay the same. No risk to navigation interests - Low Risk	0.4	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom, dredging requirements will stay the same. No risk to navigation interests - Low Risk	0.9	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom, dredging requirements will stay the same. No risk to navigation interests - Low Risk
	50	0.5	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom, dredging requirements will stay the same. No risk to navigation interests - Low Risk	0.9	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom, dredging requirements will stay the same. No risk to navigation interests - Low Risk	2.3	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom, dredging requirements will stay the same. No risk to navigation interests - Low Risk
Channel O&M	1	0	Base Condition	0	No Change- No Risk	0	No Change- No Risk
	25	0.3	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom. The volume that fills in as the datum is adjusted would be minimal and would be negligible when compared to annual maintenance dredging quantities. O&M dredging quantities have not previously changed due to changes in channel dimensions. All material entering the harbor from upstream is captured. This will not change, but the location of the shoals may move upstream with the salt water interface - Low Risk	0.4	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom. The volume that fills in as the datum is adjusted would be minimal and would be negligible when compared to annual maintenance dredging quantities. O&M dredging quantities have not previously changed due to changes in channel dimensions. All material entering the harbor from upstream is captured. This will not change, but the location of the shoals may move upstream with the salt water interface - Low Risk	0.9	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom. The volume that fills in as the datum is adjusted would be minimal and would be negligible when compared to annual maintenance dredging quantities. O&M dredging quantities have not previously changed due to changes in channel dimensions. All material entering the harbor from upstream is captured. This will not change, but the location of the shoals may move upstream with the salt water interface. With increases in sea level rise, this would move farther inland - Moderate Risk for Changes in Dredging Locations
	50	0.5	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom. The volume that fills in as the datum is adjusted would be minimal and would be negligible when compared to annual maintenance dredging quantities. O&M dredging quantities have not previously changed due to changes in channel dimensions. All material entering the harbor from upstream is captured. This will not change, but the location of the shoals may move upstream with the salt water interface. With increases in sea level rise, this would move farther inland, but the total sea level rise of 0.66 would have only a small impact on the location of shoaling - Low Risk	0.9	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom. The volume that fills in as the datum is adjusted would be minimal and would be negligible when compared to annual maintenance dredging quantities. O&M dredging quantities have not previously changed due to changes in channel dimensions. All material entering the harbor from upstream is captured. This will not change, but the location of the shoals may move upstream with the salt water interface. With increases in sea level rise, this would move farther inland - Moderate Risk for Changes in O&M shoal locations only.	2.3	Depth will be relative to MLLW, which is adjusted as sea level rises, gradually the channel will fill in on the bottom. The volume that fills in as the datum is adjusted would be minimal and would be negligible when compared to annual maintenance dredging quantities. O&M dredging quantities have not previously changed due to changes in channel dimensions. All material entering the harbor from upstream is captured. This will not change, but the location of the shoals may move upstream with the salt water interface. With increases in sea level rise, this would move farther inland - Moderate Risk for Changes in O&M shoal locations only.

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Table 15-2: SHEP Engineering Investigations Risk and Uncertainty for Sea Level Rise (Continued)

Project Component	Year	Low Rate of Sea Level Rise (0.01 ft/yr) (Historic)		Intermediate Rate of Sea Level Rise (EC 1165-2-211, Curve 2)		High Level of Sea Level Rise (EC 1165-2-211, Curve 3)	
		SLR (ft)	Impacts/Risk	SLR (ft)	Impacts/Risk	SLR (ft)	Impacts/Risk
Mitigation Features (Sills and Plugs)	1	0	Base Condition	0	No Change- No Risk	0	No Change- No Risk
	25	0.3	All sills and plugs are designed to tie into the adjacent shoreline and operate under variable water elevations (mean tide range of over 7.0 feet). After 25 years, all plugs would continue to function as designed - Low Risk	0.4	All sills and plugs are designed to tie into the adjacent shoreline and operate under variable water elevations (mean tide range of over 7.0 feet). After 25 years, all plugs would continue to function as designed - Low Risk	0.9	All sills and plugs are designed to tie into the adjacent shoreline and operate under variable water elevations (mean tide range of over 7.0 feet). After 25 years, all plugs would continue to function as designed, though possibly at a lower efficiency due to an additional 0.9' of water passing into the freshwater wetlands - Low Risk
	50	0.5	All sills and plugs are designed to tie into the adjacent shoreline and operate under variable water elevations (mean tide range of over 7.0 feet). After 50 years, all plugs would continue to function as designed - Low Risk	0.9	All sills and plugs are designed to tie into the adjacent shoreline and operate under variable water elevations (mean tide range of over 7.0 feet). After 50 years, all plugs would continue to function as designed, though possibly at a lower efficiency due to an additional 0.9' of water passing into the freshwater wetlands - Low Risk	2.3	All sills and plugs are designed to tie into the adjacent shoreline and operate under variable water elevations (mean tide range of over 7.0 feet). After 50 years, all plugs would continue to function as designed, however a larger volume of salt water would pass over the structures into the freshwater areas. Since the bottom layer of water is the most saline, the structures would continue to provide a benefit. Raising top elevation of a rock structure would be a simple construction task - Moderate Risk.
Mitigation Feature (Channel Dredging)	1	0	Base Condition	0	No Change- No Risk	0	No Change- No Risk
	25	0.3	Upstream creeks dredged as part of the overall flow plan are within the banks of the existing marsh. With a sea level rise of 0.3 feet, they would continue to function properly to carry freshwater flow into the Back River/Wildlife Refuge from upstream. Established dredging depth would be relative to MLLW, as adjusted for sea level rise - Low Risk.	0.4	Upstream creeks dredged as part of the overall flow plan are within the banks of the existing marsh. With a sea level rise of 0.4 feet, they would continue to function properly to carry freshwater flow into the Back River/Wildlife Refuge from upstream. Established dredging depth would be relative to MLLW, as adjusted for sea level rise - Low Risk.	0.9	Upstream creeks dredged as part of the overall flow plan are within the banks of the existing marsh. With a sea level rise of 0.9 feet, they would continue to function properly to carry freshwater flow into the Back River/Wildlife Refuge from upstream. Established dredging depth would be relative to MLLW, as adjusted for sea level rise - Low Risk.
	50	0.5	Upstream creeks dredged as part of the overall flow plan are within the banks of the existing marsh. With a sea level rise of 0.5 feet, they would continue to function properly to carry freshwater flow into the Back River/Wildlife Refuge from upstream. Established dredging depth would be relative to MLLW, as adjusted for sea level rise - Low Risk.	0.9	Upstream creeks dredged as part of the overall flow plan are within the banks of the existing marsh. With a sea level rise of 0.9 feet, they would continue to function properly to carry freshwater flow into the Back River/Wildlife Refuge from upstream. Established dredging depth would be relative to MLLW, as adjusted for sea level rise - Low Risk.	2.3	Upstream creeks dredged as part of the overall flow plan are within the banks of the existing marsh. With a sea level rise of 2.3 feet, they would continue to function properly to carry freshwater flow into the Back River/Wildlife Refuge from upstream. Established dredging depth would be relative to MLLW, as adjusted for sea level rise. The benefits of the creeks for bringing in fresh water into these areas would remain, but the fresh water brought into these areas would have a higher salinity - Moderate Risk.

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Table 15-2: SHEP Engineering Investigations Risk and Uncertainty for Sea Level Rise (Continued)

Project Component	Year	Low Rate of Sea Level Rise (0.01 ft/yr) (Historic)		Intermediate Rate of Sea Level Rise (EC 1165-2-211, Curve 2)		High Level of Sea Level Rise (EC 1165-2-211, Curve 3)	
		SLR (ft)	Impacts/Risk	SLR (ft)	Impacts/Risk	SLR (ft)	Impacts/Risk
Mitigation of Freshwater Wetland Losses	1	0	Base Condition- Freshwater wetlands requiring mitigation for 48' project- 337 acres; for 47' project- 223 acres: for 46' project- 201 acres: for 45' project- 32 acres; for 44' project- 0 acres (i.e. fully mitigated with flow altering structures). Based on the Corps policy of "no net loss of wetlands" and the Corps' Environmental Operating Principles this would present the lowest risk for environmental impacts. (known condition, lowest risk)	0	Base Condition- Freshwater wetlands requiring mitigation for 48' project- 337 acres; for 47' project- 223 acres: for 46' project- 201 acres: for 45' project- 32 acres; for 44' project- 0 acres (i.e. fully mitigated with flow altering structures). Based on the Corps policy of "no net loss of wetlands" and the Corps' Environmental Operating Principles this would present the lowest risk for environmental impacts. (known condition, lowest risk)	0	Base Condition- Freshwater wetlands requiring mitigation for 48' project- 337 acres; for 47' project- 223 acres: for 46' project- 201 acres: for 45' project- 32 acres; for 44' project- 0 acres (i.e. fully mitigated with flow altering structures). Based on the Corps policy of "no net loss of wetlands" and the Corps' Environmental Operating Principles this would present the lowest risk for environmental impacts. (known condition, lowest risk)
	25	0.3	Freshwater wetlands requiring mitigation for 48' project- 240 acres; for 47' project- 167acres: for 46' project- 151 acres: for 45' project- 0 acres; for 44' project- 0 acres (i.e. fully mitigated with flow altering structures). The risk for mitigating the project based on the base condition could result in over-mitigation by compensating for loss of 97 acres(48' project), 56 acres(47' project), 50 acres(46' project), 32 acres(45' project) that may be lost anyway due to salt water intrusion after 25 years. The risk of not mitigating for the base condition would be the temporal loss of wetlands that would not be compensated for. This would not comply with "no net loss of wetlands" and the Corps Environmental Operating Principles.	0.4	Freshwater wetlands requiring mitigation for 48' project- 168 acres; for 47' project- 116 acres: for 46' project- 100 acres: for 45' project- 0 acres; for 44' project- 0 acres (i.e. fully mitigated with flow altering structures). The risk for mitigating the project based on the base condition could result in over-mitigation by compensating for loss of 169 acres(48' project), 107 acres (47' project), 101 acres (46' project) that may be lost anyway due to salt water intrusion after 25 years. The risk of not mitigating for the base condition would be the temporal loss of wetlands that would not be compensated for. This would not comply with "no net loss of wetlands" and the Corps Environmental Operating Principles.	0.9	Loss of Freshwater Wetlands would be fully mitigated with flow altering structures. The risk for mitigating the project based on the base condition could result in over-mitigation by compensating for loss of up to 337 acres (for the 48' project) that may be lost anyway due to salt water intrusion after 50 years. The risk of not mitigating for the base condition would be the initial loss of wetlands that would not be compensated for. This would not comply with "no net loss of wetlands" and the Corps Environmental Operating Principles.
	50	0.5	Freshwater wetlands requiring mitigation for 48' project- 130 acres; for 47' project-86 acres: for 46' project- 69 acres: for 45' project- 0 acres; for 44' project- 0 acres (i.e. fully mitigated with flow altering structures). The risk for mitigating the project based on the base condition could result in over-mitigation by compensating for loss of 207 acres(48' project), 137 acres(47' project), 132 acres (46' project), and 32 acres (45' project) that may be lost anyway due to salt water intrusion after 50 years. The risk of not mitigating for the base condition would be the temporal loss of wetlands that would not be compensated for. This would not comply with "no net loss of wetlands" and the Corps Environmental Operating Principles.	0.9	Loss of Freshwater Wetlands would be fully mitigated with flow altering structures. The risk for mitigating the project based on the base condition could result in over-mitigation by compensating for loss of up to 337 acres(for the 48' project) that may be lost anyway due to salt water intrusion after 50 years. The risk of not mitigating for the base condition would be the temporal loss of wetlands that would not be compensated for. This would not comply with "no net loss of wetlands" and the Corps Environmental Operating Principles.	2.3	Loss of Freshwater Wetlands would be fully mitigated with flow altering structures. The risk for mitigating the project based on the base condition could result in over-mitigation by compensating for loss of up to 337 acres (for the 48' project) that would be lost anyway due to salt water intrusion after 50 years. The risk of not mitigating for the base condition would be the temporal loss of wetlands that would not be compensated for. This would not comply with "no net loss of wetlands" and the Corps Environmental Operating Principles.

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Table 15-2: SHEP Engineering Investigations Risk and Uncertainty for Sea Level Rise (Continued)

Project Component	Year	Low Rate of Sea Level Rise (0.01 ft/yr) (Historic)		Intermediate Rate of Sea Level Rise (EC 1165-2-211, Curve 2)		High Level of Sea Level Rise (EC 1165-2-211, Curve 3)	
		SLR (ft)	Impacts/Risk	SLR (ft)	Impacts/Risk	SLR (ft)	Impacts/Risk
Upstream Chlorides	1	0	Base Condition- No significant impact to City of Savannah Water Supply	0	No Change- No Risk	0	No Change- No Risk
	25	0.3	Moderate impacts to City water intake with or without the project due to further salt water intrusion into the Savannah Harbor.	0.4	Moderate impacts to City water intake with or without the project due to further salt water intrusion into the Savannah Harbor. Relocation of the water intake may be necessary.	0.9	Significant impacts to City water intake with or without the project due to salt water intrusion into the upper reaches of Savannah Harbor and the Savannah River. Relocation of the water intake would be necessary.
	50	0.5	Moderate impacts to City water intake with or without the project due to further salt water intrusion into the Savannah Harbor.	0.9	Significant impacts to City water intake with or without the project due to salt water intrusion into the upper reaches of Savannah Harbor and the Savannah River. Relocation of the water intake would be necessary.	2.3	Significant impacts to City water intake with or without the project due to salt water intrusion into the upper reaches of Savannah Harbor and the Savannah River. Relocation of the water intake would be necessary. Moderate risk that project funds intake relocation when sea level rise would require same relocation.