Wilmington Harbor

96 Act General Re-evaluation Report (GRR)
for Relocated Turning Basin and Remaining Mitigation Features

Alternative Formulation Briefing
Preconference Materials: **MAIN TEXT**
August 2007

Chemserve turning basin site, Northeast Cape Fear River

US Army Corps of Engineers
Wilmington District

TABLE OF CONTENTS
AFB PRECONFERENCE MATERIALS

Executive Summary ES-1

1.0 STUDY BACKGROUND 1-1
1.1 Authority 1-1
1.2 Purpose and scope 1-1
1.3 Project Area Description 1-2
1.4 Previous studies and projects 1-7

2.0 PROBLEMS, OPPORTUNITIES, FORMULATION OBJECTIVES, CONSTRAINTS AND CRITERIA 2-1
2.1 Overview 2-1
2.2 Navigation and Commerce 2-1
2.3 Environmental 2-3
2.4 Formulation Objectives, Constraints and Criteria 2-5
2.4.1 National Objectives 2-5
2.4.2 Public Concerns 2-5
2.4.3 Planning Objectives 2-6
2.4.4 Planning Constraints 2-7
2.4.5 Study Team 2-7

3.0 FORMULATION AND EVALUATION OF ALTERNATIVE PLANS 3-1
3.1 Turning Basin Alternatives Descriptions 3-1
3.2 No action 3-5
3.3 Relocation of ship turning basin in Northeast Cape Fear River 3-5
3.3.1 Alternative B1: enlarging of turning basin at upper terminus 3-9
3.3.2 Alternative C3: creation of middle turning basin, south 3-9
3.3.3 Alternative C2: creation of middle turning basin, middle 3-10
3.3.4 Alternative C1: creation of middle turning basin, north 3-11
3.3.5 Ship simulation 3-12

3.4 Mitigation for new turning basin 3-13
3.4.1 Fill-in abandoned basins 3-19
3.4.2 Restoration of Alligator Creek on Eagle Island 3-20
3.4.3 Island restorations 3-23
3.4.3.1 Island 12 restoration 3-23
3.4.3.2 Island 13 amplification 3-25
3.4.3.3 Island 18 restoration 3-26

3.4.4 Improved fish passage at the locks and dams 3-26

3.5 Fish Passage 3-30
3.5.1 Requirements 3-30

3.5.2 Fish Passage Alternatives: Deauthorization NOT REQUIRED..3-35
3.5.2.1 Improve locking at all three locks and dams 3-35
3.5.2.2 Nature-like fish bypass at Lock and Dam # 1.....3-36
3.5.2.3 Diversion rock rapids at all three locks and dams.....3-38
3.5.2.4 Rock rapids across entire downstream face of dam 3-40
at all three locks and dams

3.5.3 Fish Passage Alternatives at All Three Locks and Dams: Deauthorization REQUIRED.....3-43
3.5.3.1 Rock rapids in lock chamber(s).....3-43
3.5.3.2 Lower the dam(s) with reduced water supply capacity 3-45
3.5.3.3 Lower the dam(s) such that no fish passage is needed.....3-45
3.5.3.4 Remove the dam(s) 3-46
3.5.3.5 Place rock rapids upstream of Lock and Dams # 1 & 2 3-46

3.5.4 Combination of Fish Passage Alternatives 3-47
3.5.4.1 Rock rapids across the entire downstream face of dam and in the lock chamber 3-47
3.5.4.2 Diversion channel and rock ramp in the lock chamber 3-47
3.5.4.3 Nature-like by pass and rock ramp in the lock chamber 3-47

3.6 Alternatives not considered in detail 3-48

3.7 General environmental considerations 3-48
3.7.1 Estuarine and riverine resources 3-48
3.7.1.1 Locks and Dams 3-48
3.7.1.2 Relocated turning basin and PNA mitigation sites 3-49
3.7.1.3 Essential Fish Habitat 3-50

3.7.2 Terrestrial resources.....3-51
3.7.2.1 Locks and Dams 3-51
3.7.2.2 Relocated turning basin 3-52

3.7.3 Protected species 3-52
3.7.4 Sediment and contaminants 3-53
3.7.4.1 Locks and Dams 3-54
3.7.4.2 Ship Turning Basin in Northeast Cape Fear River 3-55

3.7.5 Wetlands.....3-56
3.7.5.1 Locks and Dams 3-56
3.7.5.2 Relocated turning basin 3-56

3.7.6 Cultural Resources 3-57
3.7.6.1 Turning Basin alternatives 3-57
3.7.6.2 Lock and Dam alternatives 3-58

3.7.7 Spawning areas for anadromous fish 3-60
3.7.7.1 Locks and Dams 3-60
3.7.7.2 Relocated turning basin 3-61

3.7.8 Saltwater intrusion 3-61
3.7.9 Recreation 3-61

3.7.10 Geotechnical and Ground Water 3-61
3.7.10.1 Geotechnical.....3-62
3.7.10.2 Ground Water 3-62
3.7.11 Shoaling.....3-62

3.8 Major design/operational considerations.....3-65
3.8.1 Water supply.....3-65
3.8.2 Potential for commercial use of the locks.....3-66
3.8.2.1 Small boat navigation.....3-66
3.8.2.2 Commercial navigation 3-66
3.8.2.3 Commercial terminal facilities 3-67
3.8.2.4 Commercial cargo trends.....3-67
3.8.2.5 Alternative commercial transportation.....3-68
3.8.2.6 Analysis of alternative transportation 3-68
3.8.2.7 Military uses.....3-69

3.8.2.8 Evaluation of commercial traffic on the Cape Fear River above Wilmington.....3-69
3.8.2.8.1 Commercial navigation 3-69
3.8.2.8.2 Commercial commodities.....3-69
3.8.2.8.3 Commercial cargo trends.....3-69

3.8.3 Water level modeling for dam alteration or removal.....3-69

3.8.4 Future ownership of locks and dams, and recreation areas.....3-71

3.8.5 Pilots' preferred turning basin alternative.....3-71
3.8.5.1 Descriptions of turning basins on the Northeast Cape Fear River ...3-71
3.8.5.1.1 Alternative B1 – modify existing North Turning Basin 3-72
3.8.5.1.2 Alternative C3 – new turning basin (Chemserve), south option 3-72
3.8.5.1.3 Alternative C2 – new turning basin (Chemserve), middle option 3-73
3.8.5.1.4 Alternative C1 – new turning basin (Chemserve), north option.....3-73
3.8.5.1.5 Build the Two Authorized Turning Basins (Almont and Southern States) 3-73
3.8.5.1.6 Comparison of Economic Costs.....3-74
3.8.5.2 Evaluation of alternative turning basins 3-74
3.9 Costs of the alternative plans.....3-76

3.9.1 Cost estimates for turning basins.....3-76

3.9.2 Cost estimates for fish passage at the Locks and Dams 3-79

3.9.3 Mitigation cost estimates for new turning basin 3-79
3.9.3.1 Island 13 amplification 3-79
3.9.3.2 Alligator Creek restoration 3-79
3.9.3.3 Island 18 Restoration 3-79
3.9.3.4 Island 12 Restoration 3-79
3.9.3.5 Improved fish passage at Lock and Dam # 1 3-79

3.10 Benefits of the alternative plans 3-79
3.10.1 Fish passage 3-80
3.10.2 Relocation of ship turning basin in northeast Cape Fear River 3-83

3.11 Identification of
NED Plan for Turning Basin and
NER Plan for Locks and Dams 3-84
3.11.1 Description of NED Plan for turning basin 3-84
3.11.2 Description of NER Plan for fish passage at Locks and Dams 3-84
3.11.3 Risk and Uncertainty 3-84
3.11.3.1 Narrow openings for fish passage at the Locks and Dams 3-85
3.11.3.2 Effects on water supply/storage pools above the dams 3-85

3.11.4 Allocation and apportionment of costs and responsibilities 3-85

4 POLICY ISSUES AND QUESTIONS 4-1
4.1 Congressional Deauthorization 4-1
4.2 Disposal of Government Property 4-1
4.3 902 Limit 4-1

5.0 STATUS OF ENVIRONMENTAL COMPLIANCE ACTIONS,
COORDINATION, DOCUMENTATION 5-1
5.1 Coordination 5-1
5.2 Cultural resources 5-1
5.2.1 Turning basin alternatives 5-1
5.2.2 Lock and Dam alternatives 5-2

5.3 Estuarine and riverine resources 5-4
5.4 Terrestrial resources 5-4
5.5 Endangered and Threatened species 5-4
5.6 Recreation and aesthetic resources 5-4
5.7 Sediment and contaminants 5-5
5.8 Dredging window 5-5
5.9 Environmental mitigation 5-6
5.10 USFWS Recommendations 5-6
5.11 NEPA Documentation 5-6

6 INDEPENDENT REVIEWS....6-1
6.1 Peer Review Work Plan....6-1
6.2 Independent Technical Review....6-1
6.3 Value Engineering....6-1

7 LEGAL REVIEW DOCUMENTATION....7-1

8 STATUS OF ONGOING TECHNICAL ACTIVITIES....8-1
8.1 Relocation of Ship Turning Basin in Northeast Cape Fear River....8-1
8.2 Fish Passage....8-1
8.2.1 Diversion rock ramp at all three locks and dams....8-1
8.2.2 Rock rapids across entire downstream face of dam at all three locks and dams....8-1
8.3 Municipal water supply intake evaluation....8-1
8.4 Planning Models....8-1

9 REAL ESTATE....9-1
9.1 Real Estate Overview....9-1
9.2 LERRDs....9-1
9.3 Sponsor responsibilities....9-1
9.4 Easement Revision....9-1
9.5 Real Estate Estimate....9-1
9.6 Remaining Real Estate Work....9-1
9.6.1 Gross Appraisal Revision....9-1
9.6.2 Staging Area Identification....9-1
9.6.3 Sponsors Capability....9-1
9.6.4 Code of Accounts....9-1

10 ENVIRONMENTAL COMPLIANCE, COORDINATION, AND RESOURCE AGENCY VIEWS....10-1

11 PROJECT MILESTONES AND COMPLETION DATES....11-1

12 STATUS OF MCACES ESTIMATE....12-1

13 HEADQUARTERS GUIDANCE MEMORANDUM....13-1

14 HEADQUARTERS GUIDANCE COMPLIANCE MEMORANDUM....14-1

15 STATUS OF NON-FEDERAL SPONSOR SUPPORT....15-1

16 REFERENCES....16-1

FIGURES Page

1.1 Vicinity Map, Cape Fear River Basin....1-4
1.2 Location Map, Wilmington Harbor GRR Study Area 1-5
1.3 Figure 1.3 Location Map: Existing Turning Basins and Turning Basin Alternatives B1, C1, C2, and C3 1-6
3.1a Turning Basin Alternative Site B1, Southern States Chemical 3-2
3.1b Locations of Turning Basin Alternatives Sites C1, C2, and C3, north of Chemserve Terminal 3-3
3.2.2a Turning Basin Alternative Locations....3-5
3.2.2b Existing Almont Turning Basin 3-5
3.3a Northeast Cape Fear River Stationing 3-8
3.3.1 Location of Alternative B1 3-9
3.3.2 Location of Alternative C3 3-10
3.3.3 Location of Alternative C2 3-10
3.3.4 Location of Alternative C1 3-11
3.4a Areas Evaluated for Turning Basin Mitigation, Alligator Creek and Islands 12, 13, and 18 3-18
3.4b Existing Almont turning basin, potentially abandoned 3-19
3.4c Existing Southern States turning basin, potentially abandoned 3-19
3.4.2a Alligator Creek, Eagle Island, 1938 aerial photograph 3-21
3.4.2b Alligator Creek restoration corridor, Cape Fear River to Redmond Creek 3-22
3.4.2c Existing access road blocking Alligator Creek....3-20
3.4.2d Alligator Creek resotration: Redmond Creek connection area....3-20
3.4.2e Alligator Creek restoration: Cape Fear River connection area 3-20
3.4.3.1a Island 12 Stream Creation connection area 3-23
3.4.3.1b Island 12 Stream Creation 3.24
3.4.3.1c Island 12 Stream Creation: Connection area at southeastern creek 3-25
3.4.3.1d West shoreline of Island 12, Federal channel in offshore waters 3-25
3.4.3.2 Island 13 – view south....3-26
3.4.3.3 Island 18 – view south from northeast point....3-26
3.4.4 aa Existing Lock and Dam No. 1 3-27
3.4.4 ab Existing Lock and Dam No. 2 3-28
3.4.4 ac Existing Lock and Dam No. 3 3-29
3.5.2.1a Lock gate in operation....3-35
3.5.2.1b Fish passage locking at Lock and Dam No. 3 3-35
3.5.2.2 Nature-like fish bypass at Lock and Dam No. 1 3-37
3.5.2.3 Diversion Rock Rapids (Lock and Dam No. 1)....3.39
3.5.2.4a Riverside Dam, Red River: Fargo, ND 3-40
3.5.2.4b Rock rapids at Lock and Dam No. 1....3-42

3.5.3.1 Rock rapids in lock (Lock and Dam No. 1) 3-44
3.7.6.2a Early days of the Cape Fear Locks and Dams 3-58
3.7.6.2b Cape Fear Locks and Dams today 3-59
3.7.6.2c Dam No. 1 3-59
3.7.6.2d Dam No. 2 3-60
3.7.6.2e Dam No. 3 3-60
3.8.5.1 Turning basin alternative locations 3-71
3.8.5.1.1 Turning basin alternative B1 3-72
3.8.5.1.2 Turning basin alternative C3 3-72
3.8.5.1.3 Turning basin alternative C2 3-73
3.8.5.1.4 Turning basin alternative C1 3-73

TABLES Page

3.3 Decision Matrix for Relocated Turning Basin, Northeast Cape Fear River, NC 3-7
3.4 a Acres of Primary Nursery Habitat Impacted for the Turning Basin Alternatives and Associated Mitigation Acres 3-15
3.4 b Summary of Cost Estimates for Fish Passage Alternatives, Turning Basins, Primary Nursery Area Mitigation 3-16
3.4 c Decision Matrix for Relocated Turning Basin, Northeast Cape Fear River, NC 3-17
3.5.1 a Decision Matrix for Fish Passage at Lock and Dam No. 1, Cape Fear River, NC 3-32
3.5.1 b Decision Matrix for Fish Passage at Lock and Dam No. 2, Cape Fear River, NC 3-33
3.5.1 c Decision Matrix for Fish Passage at Lock and Dam No. 3, Cape Fear River, NC 3-34
3.7.11 Average annual shoaling rates for the Northeast Cape Fear River 3-63
3.8.1 Water Supply Demand in Million Gallons per day (MGD) by the Surface Water Supply Users from Lock and Dam #1 through the Headwaters of Lock and Dam #3 3-65
3.8.2.2 Cape Fear River Above Wilmington, Short Tons of Cargo 3-66
3.8.2.4 Commercial Traffic to Fayetteville 3-67
3.8.3 Water Surface Elevations at 300 CFS 3-70
3.9.1 a Turning Basin Costs 3-77
3.9.1 b Turning Basin Costs 3-78
5.2.1 Known Cultural Resources Sites 5-2
5.2.2a Lock and Dam No. 1 5-2
5.2.2b Lock and Dam No. 2 5-3
5.2.2c Lock and Dam No. 3 5-3

APPENDICES
Follow Section 16

1.1	HQUSACE Authority Concurrence Email
3.3.5	Ship Simulation
3.4	Environmental Appendix – Mitigation Analysis
3.7.10.1	Geotechnical
3.7.10.2	Review of Ground Water Studies
3.7.11	Turning Basins Shoaling Analysis
3.8.1	Intake Evaluation
3.8.3	Hydraulics
6.3	Value Engineering Study Summary Report
8	Engineering Considerations
10	Resolutions
12	MCACES Estimate

AFB PRECONFERENCE MATERIALS

EXECUTIVE SUMMARY

INTRODUCTION

Wilmington Harbor is a Federal navigation project located along the Cape Fear and Northeast Cape Fear Rivers in southeastern North Carolina. With the signing of the Energy and Water Appropriations Bill on October 13, 1998 three separate projects (Wilmington Harbor - Northeast Cape Fear River project, Wilmington Harbor - Channel Widening project, and Cape Fear - Northeast Cape Fear Rivers project) were combined into one, the Wilmington Harbor, NC - 96 Act project. The dredged material disposal facilities improvements associated with maintenance of the existing navigation channel have been added to the deepening project. This element includes incrementally raising the dikes on Eagle Island to an elevation of 40 feet over a 20-year period. To date, the combined project has been constructed with multiple contracts at an estimated total project cost of $512,000,000 including $49 million for disposal area modifications for maintenance of the project. This is the current total project cost estimate inflated through construction completion. The initial construction of the project is cost shared 75% Federal and 25% non-Federal with an additional 10% paid by the non-Federal cost share partner over a thirty year period after initial construction is completed. The project's estimated annual benefits are $39 million compared to the estimated annual cost of $29 million resulting in a benefit to cost ratio of 1.4.

Project Description:

Improvements to date consist of deepening the ocean bar and entrance channels from the authorized depth of 40 feet to 44 feet; deepening the authorized 38-foot project to 42 feet up to and including the anchorage basin immediately upriver from the State Ports Authority dock, and the existing 400-foot wide channel has been widened to 600 feet over a total length of 6.2 miles including Lower and Upper Midnight and Lower Lilliput reaches; five turns and bends have been widened by 100 to 200 feet providing a total average channel width of 500 to 675 feet; the Fourth East Jetty Channel has been widened to 500 feet over a total length of 1.5 miles and mitigation to include construction of a 30-acre embayment.

Remaining items to be completed for the authorized project are: extending the anchorage basin northward by 300 feet; deepening the 32-foot channel between Castle Street and the Hilton Railroad Bridge, the 32-foot turning basin just above the mouth of the Northeast Cape Fear River on the west side to 38 feet, and the 25-foot channel from the Hilton Railroad Bridge to 750 feet upstream all to a depth of 38 feet; deepening the 25-foot channel from 750 feet upstream of the Hilton Railroad Bridge to the turning basin near the upstream limits of the project

to 34 feet, along with widening of the channel from 200 to 250 feet; and widening the turning basin from 700 to 800 feet; acquisition of about 700 acres of existing tidal swamp and upland areas for preservation of habitat to offset losses of wetlands and primary nursery areas, and construction of a fish passage structure around Lock & Dam #1 and study fish passage at Lock and Dam #2 and #3.

Harbor deepening to improve navigation was initiated in August 2000 with the award of two channel deepening contracts. Since then, the non-Federal cost sharing sponsor at the request of the Cape Fear Docking Pilots who are responsible for navigating all commercial traffic in the remaining portions of the project, requested that we revise project features to accommodate a relocated turning basin upstream of the Hilton Railroad Bridge in lieu of the turning basin just above the mouth of the Northeast Cape Fear River (Almont) and any associated mitigation requirements that have not been completed to date as authorized. A General Reevaluation Report (GRR) is being prepared to address these issues.

A feasibility report and final Environmental Impact Statement (EIS) for the Wilmington Harbor Deepening project were completed in June 1996 and supplemented by an Environmental Assessment (EA) in February 2000. Construction phase began with advanced contributed funds with award of contract to improve the Eagle Island confined disposal facility on July 31, 1997.

General Evaluation Report

Since construction began the need for two changes to the authorized project have been determined: a relocated turning basin on the Northeast Cape Fear River (NECFR) and a change in mitigation with respect to type of fish passage constructed at Cape Fear River Lock and Dam No. 1. The purpose of this General Reevaluation Report (GRR) is to address these issues in accordance with the Planning Guidance Notebook (ER-1105-2-100).

The harbor pilots have requested that a turning basin be relocated upstream of two existing bridges to avoid the need for some vessels to maneuver stern first through these bridges to reach the existing turning basin. This need is primarily for improved navigation efficiency and safety. Mitigation is required for this relocated basin since it will impact primary nursery area. Improved fish passage at Lock and Dam No. 2 is the preferred mitigation for the relocated turning basin.

The fish passage plan at Lock and Dam No.1, as coordinated in the February 2000 EA is a 3800-foot nature-like bypass, as authorized by WRDA 96. This feature is for mitigation of blasting to remove rock in the harbor. However, this plan could be improved if deauthorization of the locks and dams was considered. This could not be originally considered because the major purpose of the locks and dams was for commercial navigation, which has not occurred in years.

MAJOR CONCLUSIONS AND FINDINGS

Planning Objectives

The investigation of the problems and opportunities in the study area led to the establishment of the following planning objectives:

- Improve the navigation efficiency and safety of the turning area upstream of the Hilton Railway Bridge on the Northeast Cape Fear River (at the north end of the project), and provide mitigation for associated impacts; and
- Determine the best form of fish passage at Cape Fear River Lock and Dam No.1, taking into consideration the change in conditions for the authorized purpose of navigation.

Alternatives

Alternatives which relocate the ship turning basin in the Northeast Cape Fear River were considered as well as the no-action alternative which would maintain the existing turning basin at Almont (the authorized 38' turning basin on Northeast Cape Fear River just above the confluence with the Cape Rear River). Four alternative turning basins were evaluated in order to address the planning objectives and the problems and opportunities identified by the Sponsor, the harbor pilots, and the public. The turning basin alternatives considered were:

Lock and Dam No. 1

- Enlarging of the authorized 35' turning basin at the northern project terminus (Southern States);
- Relocation of a turning basin in NECFR north of Hilton Railway Bridge and deauthorizing existing turning basins. The three basin options at this location are
    - South location (pilots preferred alternative);
    - Mid-reach location;
    - North location.

In addition to the turning basin relocation, comprehensive plans to accomplish uncompleted environmental requirements and mitigation for the new basin impacts were evaluated. The mitigation alternatives evaluated included:

- Filling the existing turning basin(s) to be abandoned;
- Restoration of Alligator Creek on Eagle Island;
- Dredged material Disposal Island 12 restoration;

- Improved fish passage at the Cape Fear River Locks and Dams No. 2 and No. 3.

These alternatives were evaluated to ensure that project environmental requirements were met in a manner that minimizes costs and maximizes environmental outputs and quality.

Identification of the NED Plan
The Federal objective in this GRR is to contribute to the National Economic Development (NED) and have effects consistent with protecting the nation's environment, pursuant to the national environmental laws, applicable executive orders, and other planning requirements.

The investigation and analysis conducted determined that the relocation of the turning basin to the southern location just above the Hilton Railroad Bridge (the pilots preferred alternative) is the best plan to meet the NED objective. Fish passage at Lock and Dam No. 2 is the best mitigation plan for the relocated turning basin.

Recommended Plan and Locally Preferred Plan
The Recommended Plan calls for the following modifications to the existing Wilmington Harbor, NC, 96-Act project:

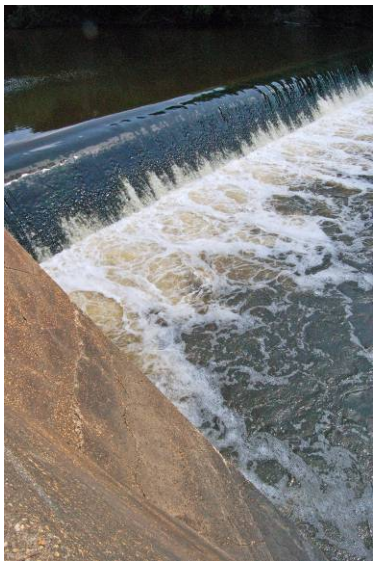
- Relocate the NECFR primary turning basin north of Hilton Railroad Bridge.
- The turning basin will be 800 feet by 800 feet. The basin corners are chamfered into the existing channel geometry.
- Turning area would be dredged to the authorized depth of -39 feet.
- Turning basins at Reach 7 (Almont) and at Reach 2 (Southern States) are recommended for de-authorization.
- Provide mitigation for the relocated turning basin by improved fish passage at the Cape Fear River Lock and Dam #2 through dam removal or construction of rock-ramp rapids downstream of the dam.
- Determination that the construction of a rock ramp at the other two locks and dams provides the best overall form of fish passage, although construction at Lock and Dam No. 3 is not required.

Mitigation for potential impacts to the endangered shortnose sturgeon resulting from blasting to remove rock is required for portions of the Wilmington Harbor 96 Act project already constructed. National Marine Fisheries Service (NMFS), condition number 8 of the August 3, 2000 Biological Opinion (BO) indicated that the USACE must:

- Construct a fish passage structure at Lock and Dam No. 1 on the Cape Fear River: the type of fish passage structure was not specified, and;
- Study the best fish passage design for Locks and Dams Nos. 2 & 3. The type of fish passage structure was not specified in the BO.

Lock & Dam No. 1

Lock & Dam No. 2



Lock & Dam No. 3
William O. Huske Lock & Dam

Features of the Recommended Plan
The Recommended Plan provides for navigation channel improvements and a comprehensive environmental plan to compensate for project impacts for the Wilmington Harbor, NC – 96 Act project.

A relocated turning basin in the NECFR above the Hilton Railway Bridge, dam removal or a rock-rapids at Lock and Dam No. 2 to mitigate for the impacts, and rock ramps at the other two locks and dams meet the planning objectives.

Environmental Compliance
The recommended plan could not be fully implemented until after the NEPA process is complete. All plan components will be evaluated for environmental impacts and a Draft Environmental Impact Statement (DEIS) (integrated with the GRR Report). The recommended plan is being evaluated in accordance with environmental requirements including but not limited to the Endangered Species Act, Sections 404 and 401 of the Clean Water Act, the National Historic Preservation Act, the Marine Protection Research and Sanctuaries Act, the Coastal Zone Management Act, and the Magnuson-Stevens Fishery Conservation and Management Act and is compliant with these requirements at the current stage of planning.

Benefits and Costs of the Recommended Plan
The basic economic benefits from the navigation improvements are derived from the safe and expeditious transit of vessels in and out of the Northeast Cape Fear River portion of the Wilmington Harbor Federal Navigation Project. The recommended plan turning basin would best serve the facilities on the Northeast Cape Fear River receiving existing traffic, but would not preclude future upstream users. Also a rock ramp for fish passage at all three locks and dams provided

the most environmental benefit without deauthorizing the project purpose of the Locks and Dams. Removal of Lock and Dam #2 for fish passage would require deauthorization of the project.

Based on the economic analyses performed during the study, total annual project benefits are estimated at $[ADD WHEN AVAILABLE]. .

PUBLIC COORDINATION

The U.S. Army Corps Engineers (USACE) and the NC Division of Water Resources have implemented a public involvement plan as a part of the study process to ensure responsiveness of the needs and concerns of stakeholders and the public. The public involvement has been an open active process. Coordination has been conducted throughout the study process primarily through an interagency Project Delivery Team (PDT). In addition to the USACE, the PDT members are from Federal, state, and local agencies and governments, water users, environmental groups, and citizen groups. An agency scoping meeting was held in June 2005. The PDT meetings have been held at least monthly beginning in December 2005 with the option of participation by telephone conference. A web site, www.saw.usace.army.mil/wilmington-harbor/GRR/GRR.htm is maintained as a repository of study information. Numerous presentations regarding the GRR have been made to municipal and citizens groups in the area.

NON-FEDERAL SPONSOR SUPPORT

The State of North Carolina, Department of Natural Resources, Division of Water Resources has expressed interest and willingness to provide 25 percent of the cost of implementing the recommendations of the GRR.

AREAS OF CONTROVERSY

The federal and state environmental agencies request fish passage at both Locks and Dams Nos. 2 and 3 for mitigation for the relocated turning basin. At this time, mitigation appears only justified for fish passage at Lock and Dam No. 2.

UNRESOLVED ISSUES

All the information is not currently available to make a determination on whether or not a recommendation should be made to deauthorize the Cape Fear River Locks and Dams. For example, a willing recipient for the federal lands has not been identified.

A rock ramp is the best form of fish passage at Locks and Dams 1 and 3, and maybe at No. 2. However, it is possible that Lock and Dam No. 2 can be removed. That determination can not be made until more information is obtained from a proposed water supply user upstream of the dam. The proposed principal user (Smithfield Foods) has been required by the State of North Carolina to move from groundwater to surface water. Also the stability analysis has not been

completed for Lock and Dam No. 3 related to a lower pool level downstream of the dam if Lock and Dam No. 2 was removed.

Wilmington Harbor 96 Act General Re-evaluation Report (GRR)

1.0 STUDY BACKGROUND

1.1 Study Authority. Wilmington Harbor is a Federal navigation project located along the Cape Fear and Northeast Cape Fear Rivers in southeastern North Carolina. The project extends from the Atlantic Ocean to the Port of Wilmington. There is a separate project called the Cape Fear River Above Wilmington which encompasses the three locks and dams located on the Cape Fear River, originally built for the purpose of extending commercial navigation from Wilmington to the Fayetteville, North Carolina area.

A Feasibility Study and final EIS for the Wilmington Harbor Deepening project were completed in June 1996 and supplemented by an EA in February 2000. In this authorization, three separate projects (Wilmington Harbor - Northeast Cape Fear River project, Wilmington Harbor – Channel Widening project, and Cape Fear - Northeast Cape Fear River project) were combined into one, the Wilmington Harbor, NC - 96 Act project. See executive summary for detailed project description.

Lock and Dam Number 1 and 2 for the Cape Fear River above Wilmington navigation project, along with an eight-foot channel, were authorized by HD 890/60/1 on June 25, 1910. The authorization for the third and final lock and dam, later renamed the William O. Huske Lock and Dam, followed on August 30, 1935 (HD 786/71/3). The Rivers and Harbor Act of August 26, 1937 (R&H Com. Doc. 17/75/1) authorized a 25-foot channel to a basin at Navassa, North Carolina (2.9 miles up the Cape Fear River above Wilmington) and, thence, an eight-foot navigation channel for the remaining 108 miles to Fayetteville, North Carolina. Recreation facilities at each lock were authorized with Section 4 of the Flood Control Act, December 22, 1944.

The construction phase on the Wilmington Harbor 96 Act project began in July 1997 and continues to the present. The Deepening and Widening Project has been completed to above the NC State Port Authority Docks to include the Vopak Terminal. Remaining portions of the project will be constructed pending the receipt of funds.

The non-Federal sponsor for the Wilmington Harbor – 96 Act and this study is the State of North Carolina Department of Environment, Health, and Natural Resources, the Division of Water Resources having the active role in representing the State.

1.2 Purpose and Scope. The GRR is to present the findings of investigations conducted to determine if there is Federal interest in making changes to the currently authorized Wilmington Harbor – 96 Act project. It has been approximately 11 years since the project was authorized under the WRDA 96.

Since construction began, the need for changes to the authorized Wilmington Harbor 96 Act project including changes to turning basins and mitigation issues have arisen. The purpose of this General Reevaluation Report (GRR) is to address these issues in accordance with the Planning Guidance Notebook (ER-1105-2-100).

The harbor pilots have requested that the previously authorized turning basin be relocated upstream of two existing bridges to avoid the need for some vessels to maneuver stern first through these bridges to reach the existing turning basin. This need is primarily for improved navigation safety and efficiency. Mitigation is required for this relocated basin since it will impact primary nursery area.

The approved fish passage plan at Lock and Dam No. 1 is a nature-like bypass. This feature is for mitigation of blasting to remove rock in the harbor. However, there is great potential for changes in the technical solutions for the required mitigation that may have a significant cost savings to the project and will best meet the commitments to provide the required mitigation. This may include recommendations to modify or remove some of the locks and dams on the Cape Fear River. This is a potential recommendation because minimal commercial traffic has used the locks and dams since 1996 and no increase in commercial traffic is expected. Because of the lack of traffic, the navigation channel to the lock and dam has not been maintained for many years.

This GRR Study will analyze the problems and opportunities, and express desired outcomes as planning objectives. Alternatives will be developed to address the planning objectives. The alternatives will include a plan of no action and various combinations of alternatives. The economic and environmental impacts of the alternatives will be evaluated and a feasible plan selected. The GRR also presents details of the USACE, Sponsor, and public participation involved in developing, and the need to implement, the plan. The GRR concludes with an appropriate recommendation for authorization.

1.3 Project Area Description. Wilmington Harbor is an approximately 37-mile federally authorized and maintained waterway located in southeastern North Carolina. This Federal navigation channel connects deep water of the Atlantic Ocean to port facilities at Wilmington and vicinity along the Cape Fear River and to the Northeast Cape Fear River upstream of Wilmington.

The Cape Fear River estuarine system - located in the southeastern quadrant of North Carolina -originates in the North Carolina Piedmont region and drains 9,140 square miles (Giese, Wilder, and Parker 1985). It is the longest river entirely within North Carolina, and it flows into the Atlantic Ocean near Cape Fear, from which it takes its name. The Cape Fear River is formed by the confluence of the Deep and Haw Rivers just below Jordan Lake, south of Raleigh, NC. It flows southeast past Lillington, Erwin, Fayetteville, and Elizabethtown, then receives the Black River approximately 10 miles northwest of

Wilmington. At Wilmington, it receives the Northeast Cape Fear River and turns south, widening as an estuary and entering the Atlantic approximately three (3) miles west of Cape Fear.

The Cape Fear River is a blackwater stream which drains extensive riverine swamp forests. The watershed has extensive agriculture areas including large swine and poultry farms.

The lower Cape Fear River connects to the Atlantic Ocean through both the mouth of the river at Southport/Bald Head Island and through Carolina Beach Inlet (via Snow's Cut). Snow's Cut and Carolina Beach Inlet - both man-made features - were constructed in 1930 (Snow's Cut) and 1952 (Carolina Beach Inlet). Snow's Cut lies approximately 13.5 miles from the Cape Fear River mouth while Carolina Beach Inlet lies approximately two miles further north. The Northeast Cape Fear River joins the Cape Fear River approximately 27.6 miles upstream of the Cape Fear River mouth.

Tidal influence extends 65 miles upstream of Bald Head Island on the Cape Fear River. Similarly, tidal influence extends 60 and 86 miles upstream of Bald Head Island on the Black River and NE Cape Fear River, respectively (Giese, Wilder, and Parker, 1985).

The major port city of Wilmington is located approximately 26.6 miles upstream of Bald Head Island just downstream of the Cape Fear River/Northeast Cape Fear River confluence. The existing Northeast Cape Fear turning basin lies approximately 28.6 miles upstream of Bald Head Island. Three locks and dams on the Cape Fear River were constructed to ensure a navigable channel for commercial barges from Wilmington to Fayetteville. In recent years, this commercial traffic has ceased operations. With the demise of commercial traffic between Wilmington and Fayetteville, the current operations are to lock anadromous fish, military vessels, and assorted other vessels if lock operator is present.

See Figures 1.1, 1.2, and 1.3 for the general location of the study area.

This GRR study encompasses two portions of the Cape Fear/Northeast Cape Fear River:

1. Wilmington Harbor, from Island 12, located in the Cape Fear River just south of Wilmington north to the Harbor terminus.

2. The Cape Fear River, from Wilmington Harbor upstream through the three Locks and Dams to Fayetteville, NC.

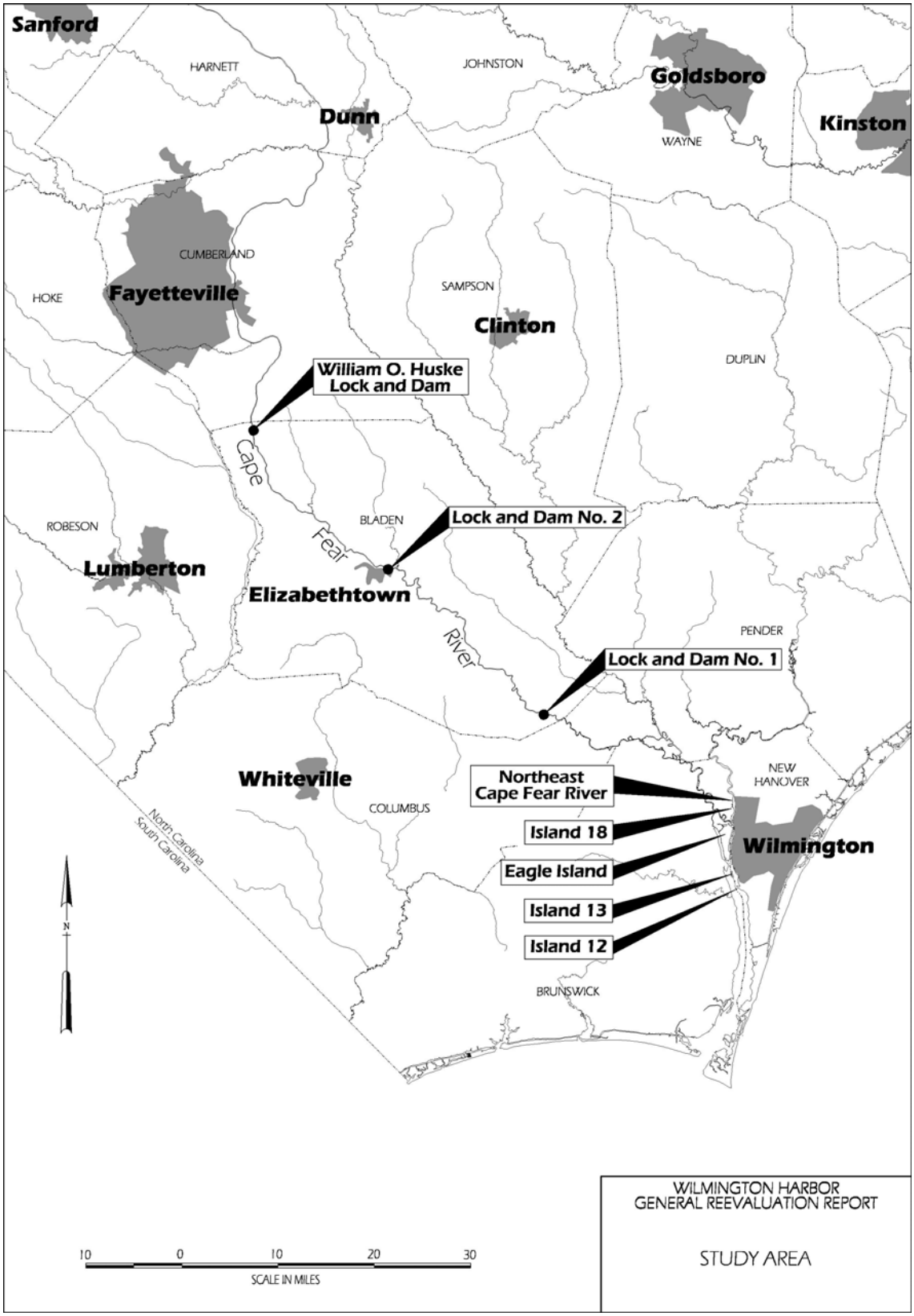


Figure 1.1 Vicinity Map: Locks and Dams, and Potential Mitigation Sites

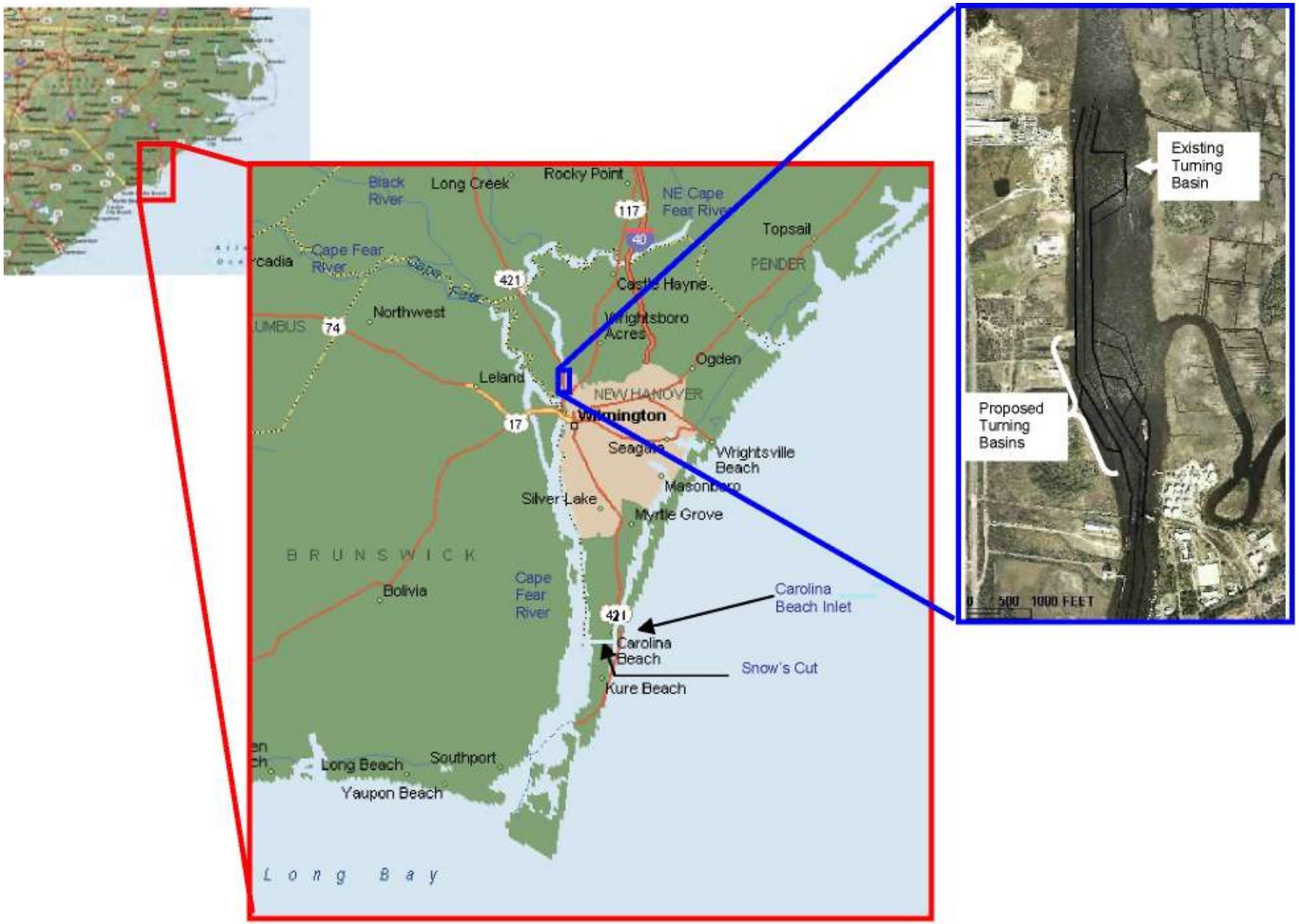


Figure 1.2 Vicinity Map: Turning Basins

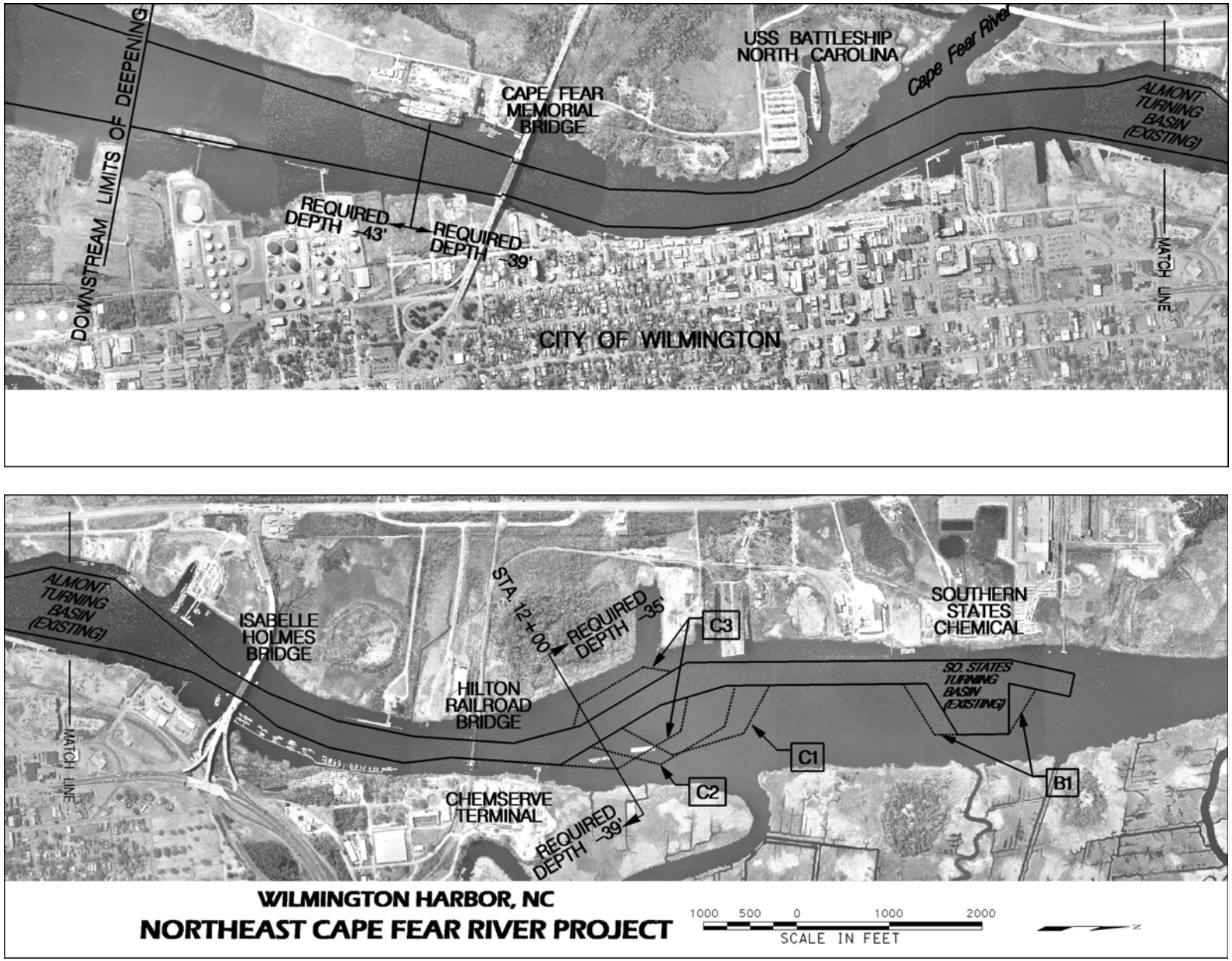


Figure 1.3 Location Map: Existing Turning Basins and Turning Basin Alternatives B1, C1, C2, and C3

1.4 Previous studies and projects.

1.4.1 US Army Corps of Engineers, Wilmington District, *Final Feasibility Report and Environmental Impact Statement on Improvement of Navigation, Cape Fear – Northeast Cape Fear Rivers Comprehensive Study, Wilmington, North Carolina, June 1996.* This report was prepared in final response to a resolution adopted 8 September 1988 by the United States House of Representatives, which directed that the existing Federal project for Wilmington Harbor be reviewed and improvements considered. Refer to the Executive Summary Introduction of this report.

1.4.2 US Army Corps of Engineers, Wilmington District, *Environmental Assessment, Preconstruction Modifications of Authorized Improvements, Wilmington Harbor, North Carolina, February 2000.* This EA addressed preconstruction modifications to harbor improvements including Ocean Bar Channel realignment, beach placement of dredged sand, rock blasting without air curtains, and a comprehensive dredging and disposal plan.

2.0 PROBLEMS, OPPORTUNITIES, FORMULATION OBJECTIVES, CONSTRAINTS AND CRITERIA

2.1 Overview. This section of the GRR covers the first step of the planning process and identifies the future without project (FWOP) problems in the existing Wilmington Harbor – 96 Act project area and potential future with project (FWP) project opportunities available if a collaborative and sustainable approach is used.

Existing water resource and environmental problems and needs in the study area will be identified through coordination and collaboration with Federal, state, local agencies, other interested groups and individuals.

2.2 Navigation and Commerce

Northeast Cape Fear River

Future Without Project Conditions
The Northeast Cape Fear River portion of the Wilmington Harbor 96 Act project has not been constructed. The two currently authorized turning basins (Almont and Southern States) in the Northeast Cape Fear River would be constructed.

The existing turning basins do not effectively serve the needs of the vessels calling on this portion of the harbor. Ships are required to back through the Hilton Railroad and Isabelle Holmes bridges stern first. This poses an elevated risk to the bridge structures and yacht anchorages in the area of the existing basins. Turning vessels in the upper portion of the harbor above the Hilton Railroad Bridge is time consuming and difficult.

The pilots prefer to maneuver the vessels through the bridge openings bow first for safety reasons. The horizontal clearance at each bridge is only 200 feet. Maneuvering a vessel, some of which may have a beam of 106 feet, through the bridge openings stern first requires careful operating procedures with tug assistance. The minimal horizontal clearance at each bridge opening leaves very little room for error. Factors such as strong currents and wind also contribute to the difficulty of safe passage through the bridge openings.

The existing traffic in these reaches primarily calls at two facilities, Chemserve Terminal and Cemex. The Chemserve Terminal has about 110 to 130 vessel calls a year, and the Cemex facility receives about 10 to 20 vessel calls a year. The existing vessel use and waterborne commerce in this portion is expected to continue or to increase in the future.

Future With Project Conditions
A relocated turning basin in the Northeast Cape Fear River portion of Wilmington Harbor would be constructed to improve vessel movements in that area. The

new turning basin would be located in the area of highest current use and north of the Hilton Railroad Bridge as well as the Isabelle Holmes Bridge. A relocated turning basin would provide opportunities to have significant construction cost savings. The existing turning basins would be recommended for deauthorization and subsequently abandoned.

The existing vessel use and waterborne commerce in this portion is expected to continue or to increase in the future.

Cape Fear Above Wilmington - Locks and Dams (CFRL&D)

Future Without Project Conditions

The existing system of the Cape Fear Above Wilmington would be maintained. The approved fish passage plan at Lock and Dam No. 1, a nature-like bypass would be implemented. The status-quo navigable condition for this part of the Cape Fear River would not change. The facilities would continue to be federally maintained.

The three existing locks and dams were constructed over a period of about 20 years, from 1915 to 1935, for the purpose of extending commercial navigation on the Cape Fear River from Wilmington to the Fayetteville, North Carolina area. Although commercial locking ceased in 1997, the locks have been maintained in operating condition for recreational and military vessels, and for fish passage. They are currently operated for fish passage from about mid-March through mid-May, the peak of the anadromous fish migration period. Lockages are made on an irregular basis for recreational and military vessels. Five municipal or industrial water intake structures now exist within pools upstream of two existing dams. All three Locks and Dams include recreational facilities. In addition, each facility includes a lock tender's office. The project land acreages associated with CFRL&D Numbers 1, 2, and 3 are 17, 30, and 20 acres, respectively.

In 1993 and 1994, all three locks were refurbished at a cost of more than $1.3 million. All of the mechanical parts and seals were refurbished and the lock gates were sand blasted and repainted. As a result of this rehabilitation, these antiquated locks are in fair structural condition. Similarly, the dams are in stable and fair condition in spite of the age of the concrete. At present, structural evaluation of the locks and dams continues on a five-year basis. Recent evaluations have focused on the scour holes that may threaten the stability of Locks and Dams 1 and 2, and quoin replacement.

The staffing and routine maintenance of the three locks and dams is handled by two lockmasters that rotate among the locations. The locks have also required periodic dredging by the Snagboat Snell to remove shoals that prevent anadromous fish from reaching the lock chambers. Clearing and snagging, minor dredging, and maintaining aids to navigation are part of the annual

activities at the Locks and Dams. Inspections, conditions surveys, recreation facility repairs, and other miscellaneous expenditures may also be required.

Future With Project Conditions
The future with project condition involves improved fish passage on Cape Fear Above Wilmington and could take two pathways: one where the navigation purpose is not affected; or the other, where the navigation purpose is discontinued (deauthorization required). Deauthorization of the locks and dams and the appropriate divestiture of those federal properties offer opportunity for disposal of the Federal project or reduction of Federal operations and maintenance expenses. This is only possible after the associated costs and impacts to the environment, water supply, recreation, and the potential for a return of commercial navigation are carefully evaluated.

2.3 Environmental

Northeast Cape Fear River

Future Without Project Conditions
The pilots would continue to turn in the existing basins with some of the shorter vessels continuing to turn upstream of the bridges near Chemserve Terminal. No primary nursery area would be directly impacted, but the vessels that are turned near Chemserve will stir up bottom sediments (suspended solids) which could adversely impact adjacent shallow primary nursery areas.

Vessel safety passing through the bridges will not be improved which would increase the likelihood of a vessel incident that could result in the spill of products harmful to the environment.

Future With Project Conditions
Relocation of the turning basin near Chemserve would impact 19 acres of primary nursery area. This impact will be mitigated by increasing fish passage at Lock and Dam No. 2. Vessels that are turned near Chemserve will still stir up bottom sediments (suspended solids) but the impact to adjacent primary nursery areas should be less. Suspended solids from the deep basin are less likely to reach the shallow primary nursery areas.

Vessel safety passing through the bridges will be improved which would decrease the likelihood of a vessel incident that could result in the spill of products harmful to the environment.

Cape Fear Locks and Dams

Future Without Project Conditions

The existing system of the Cape Fear Above Wilmington would be maintained. The approved fish passage plan at Lock and Dam No. 1, a nature-like bypass would be constructed.

The construction of the three Cape Fear River Locks and Dams (CFRL&D) from 1915 to 1935 has contributed to the decline of anadromous fish species in the Cape Fear River by reducing access by spawning fish to upstream portions of the river. Current fish passage through the locks and dams is inadequate to restore upstream spawning to levels that existed prior to construction of the locks and dams.

Mitigation for potential impacts to the endangered shortnose sturgeon resulting from blasting to remove rock is required for portions of the Wilmington Harbor 96 Act project already constructed. National Marine Fisheries Service (NMFS), condition number 8 of the August 3, 2000 Biological Opinion (BO) indicated that the USACE must:

- Construct a fish passage structure at Lock and Dam No. 1 on the Cape Fear River: the type of fish passage structure was not specified, and;
- Study the best fish passage design for Locks and Dams Nos. 2 & 3. The type of fish passage structure was not specified in the BO.

The maintenance of the current Cape Fear Above Wilmington project would provide status-quo conditions for water intakes associated with two of those facilities. Water users, including the City of Fayetteville, the City of Wilmington, the Lower Cape Fear Water and Sewer Authority, and Dupont Fayetteville depend upon the presence of the Locks and Dams 1 and 3 to provide a viable water supply from the Cape Fear River.

The Cape Fear Above Wilmington would continue to provide public park and river access at all three lock and dam locations. This provides the public an opportunity to view and enjoy the river for boating, fishing and bird watching at those locations. At least one public boat ramp exists between each lock and dam.

Future With Project Conditions

The future with project condition, improved fish passage on the Cape Fear Above Wilmington would be provided and could take two pathways: one where the navigation purpose of the Cape Fear Above Wilmington is not affected; or, one where the navigation purpose is discontinued (deauthorization required).

The existing conditions on the Cape Fear Above Wilmington provide a unique opportunity to restore anadromous fisheries resources that have been adversely affected. Improved fish passage at Lock and Dam Number 1 would provide access to about 33 river miles between Lock and Dams Number 1 and Number 2. An additional 23 river miles could be improved for anadromous fish between

Lock 2 and William O. Huske Lock and Dam 3 and about 60 additional river miles between William O. Huske Lock and Dam 3 and the historic upstream limit for anadromous fish spawning located at Smilely Falls near Lillington, North Carolina.

Plans to provide fish passage on the Cape Fear Above Wilmington facilities could affect the existing water supply intakes by lowering or eliminating the pool(s) behind one or more of the dams. This source of water is important to the area's residential, commercial, and industrial users, as well as its capacity for future growth. Other important impacts such as river water levels and water quality, salinity, currents, sedimentation, and turbidity, must be considered.

The implementation of improved fish passage may change the current recreational uses of the Cape Fear Above Wilmington facilities. Therefore, the impacts on the Cape Fear Above Wilmington public park and river access at all three lock and dam locations must be considered.

2.4 FORMULATION OBJECTIVES, CONSTRAINTS AND CRITERIA

2.4.1 National Objectives

The fundamental national objective of Federal participation in water resource development projects is to assure that optimum contribution is made to the welfare of all people. The Water Resources Council's Economic and Environmental Principles and Guidelines for Water and Related Land Resources Implementation Studies, dated March 1983 and the Planning Guidance Notebook (ER-1105-2-100) and the National Environmental Policy Act of 1969 (NEPA) provide the basis for planning Federal water resources projects. These authorities have established the procedures for formulation and evaluation of water resource projects. Additional policies and regulations, derived from executive and congressional authorities, further define the criteria for assessment of plan impacts, risks, review and coordination procedures, and project implementation. Within the Federal planning requirements, a General Reevaluation Report (GRR) evaluates currently authorized projects that are still within the construction phase in accordance with the Planning Guidance Notebook (ER-1105-2-100).

The GRR is being carried out in a manner consistent with the Corps' Environmental Operating Principles (EOPs). The principles are consistent with National Environmental Policy Act (NEPA); the Army's Environmental Strategy with its four pillars of prevention, compliance, restoration and conservation; and other environmental statutes and Water Resources Development Acts (WRDA's) that govern USACE activities.

2.4.2 Public Concerns

Public concerns have been identified during the course of the GRR study. Initial input was received through coordination with the sponsor and Federal, state, and local agencies that make up the study team. Additional public involvement will include early opportunity to provide study input as well as public review of draft and final products.

Concerns and comments received from the public related to the establishment of planning objectives and planning constraints include:

- Need a new turning basin within the Northeast Cape Fear River to allow all vessels to travel through the bridge openings bow first;
- Potential impacts to estuarine primary nursery areas affected by dredging of a new turning basin;
- Need for mitigation of adverse impacts cause by dredging within new turning basin and rock blasting;
- Wilmington Harbor 96 Act project must construct a fish passage at Cape Fear Above Wilmington, L&D 1 and study fish passage at Cape Fear Above Wilmington, L&D 2 and 3;
- Potential adverse impacts to existing water supply facilities at Cape Fear Above Wilmington, L&D 1 and 3 if changes to the Locks and Dams are implemented;
- Fish passage at all Cape Fear Above Wilmington Locks and Dams is highly desirable and a preferred mitigation measure by all resource agencies for any additional project impacts.

The USACE and the NC Division of Water Resources have implemented a public involvement plan as a part of the study process to ensure responsiveness of the needs and concerns of stakeholders and the public. The public involvement has been an open active process. Coordination has been conducted throughout the study process primarily through a Project Delivery Team (PDT). In addition to the USACE, the PDT members are from Federal, state, and local agencies and governments, water users, environmental groups, and citizen groups. An agency scoping meeting was held in June 2005. The PDT meetings have been held at least monthly beginning in December 2005 with the option of participation by telephone conference. All monthly meetings are held the third Wednesday of every month. Reminder notices of these meeting are emailed to about 100 individuals.

A web site, www.saw.usace.army.mil/wilmington-harbor/GRR/GRR.htm is maintained as a repository of study information. Numerous presentations regarding the GRR have been made to municipal and citizens groups in the area.

2.4.3 Planning Objectives

The national objectives are general statements not specific enough for direct use in plan formulation. Planning objectives reflect the problems and opportunities

and represent desired positive changes. The following are planning objectives to be used in formulation and evaluation of alternative plans for the Wilmington Harbor 96 Act General Re-evaluation Report:

- Improve the navigation efficiency and safety of the turning area upstream of the Hilton Railway Bridge on the Northeast Cape Fear River;
- Provide adequate mitigation for project impacts;
- Determine the best form of fish passage at the Cape Fear Above Wilmington Locks and Dams;

2.4.4 Planning Constraints

Unlike planning objectives that represent desired positive changes, planning constraints represent restrictions that should not be violated. Plans must be formulated with regard to addressing the problems and needs of the area taking into consideration the future without project conditions. The plans should identify tangible and intangible benefits and costs from economic, environmental, social, and regional perspectives. Institutional implementation constraints should be identified.

The formulation framework requires a systematic preparation and evaluation of alternative solutions to recognize water resource related problems and opportunities for the Wilmington Harbor - 96 Act GRR. The process requires that impacts of the proposed action be measured and results displayed or accounted for in terms of contributions to: National Economic Development (NED), environmental quality, regional economic development, and other social effects.

Interaction with other interests must be maintained throughout the planning process to avoid duplication of effort, minimize conflict, obtain consistency, and assure completeness. The following constraints apply to this feasibility study:

- Adverse impacts to fish and wildlife and the human environment should be minimized as much as possible;
- The study process and plans must comply with Federal and State laws and policies;
- Plans that resolve problems in one area should not create or amplify problems in other areas;
- USACE must construct a fish passage structure at Lock and Dam No. 1 (and study fish passage at Nos. 2 and 3) (Compliance with Biological Opinion dated August 3, 2000);

2.4.5 Study Team

The Wilmington Harbor 96 Act GRR has proceeded in a collaborative, comprehensive and integrated manner in accordance with EC 1105-2-409, Planning in a Collaborative Environment. Representatives of other Federal,

State, and local agencies have been invited to be members of the Project Development Team (PDT). Collaborating study team participants to date have included:

- US Army Corps of Engineers
- NC Division of Water Resources
- US Fish and Wildlife Service
- National Marine Fisheries Service
- NC Division of Marine Fisheries
- NC Division of Coastal Management
- NC Division of Water Quality
- NC Department of Transportation
- NC Wildlife Resources Commission
- NC State Ports Authority
- NC Coastal Federation
- City of Wilmington
- New Hanover County
- City of Fayetteville
- Fayetteville Public Works Commission
- Cumberland County
- Town of Elizabethtown
- Bladen County
- Brunswick County
- NC and US Representatives
- Lower Cape Fear Water and Sewer Authority
- Triangle J Council of Governments
- Cape Fear River Assembly
- Smithfield Foods
- Dupont Fayetteville
- International Paper
- Cape Fear Docking Pilots
- Wilmington Docking Pilots Association
- UNC – Wilmington
- UNC – Chapel Hill
- NC State University
- Coastal Conservation Association
- Cape Fear River Watch
- Chemserve Terminal
- Citizens

3.0 FORMULATION AND EVALUATION OF ALTERNATIVE PLANS

3.1 Turning basin alternatives descriptions. Following are location diagrams and photographs of the ship turning basin alternatives: Alternatives **B1**, **C1**, **C2**, and **C3**. All elevations associated with the turning basins are referenced to MLLW.

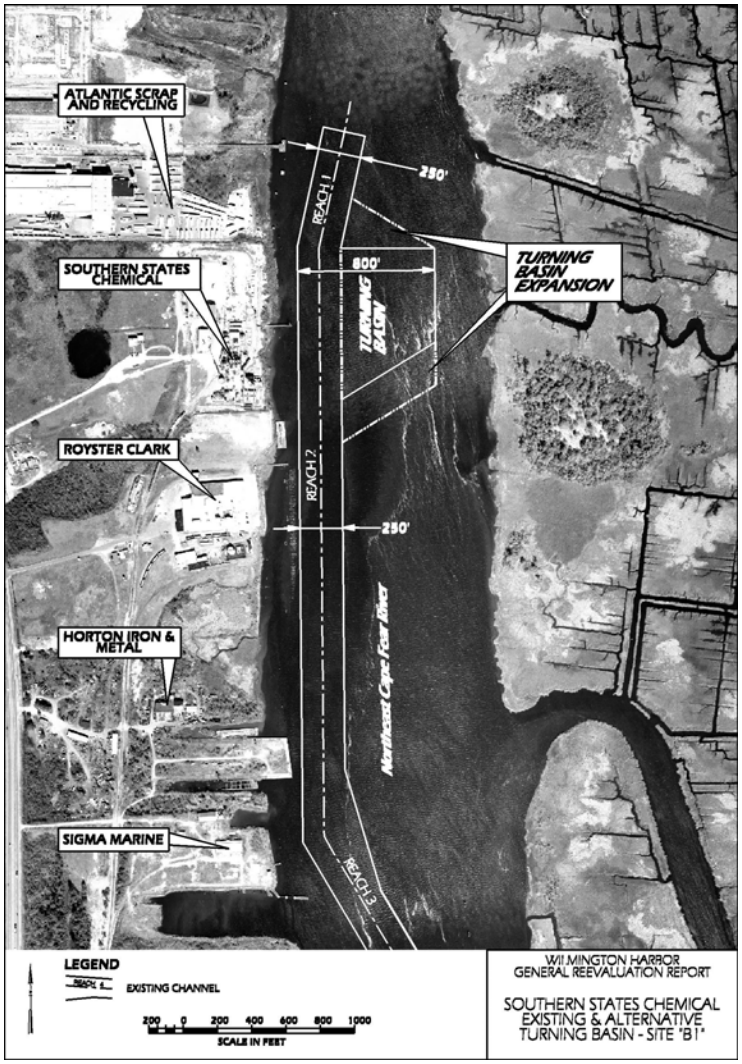


Figure 3.1a
Turning Basin
*Alternative Site **B1***
Southern States Chemical

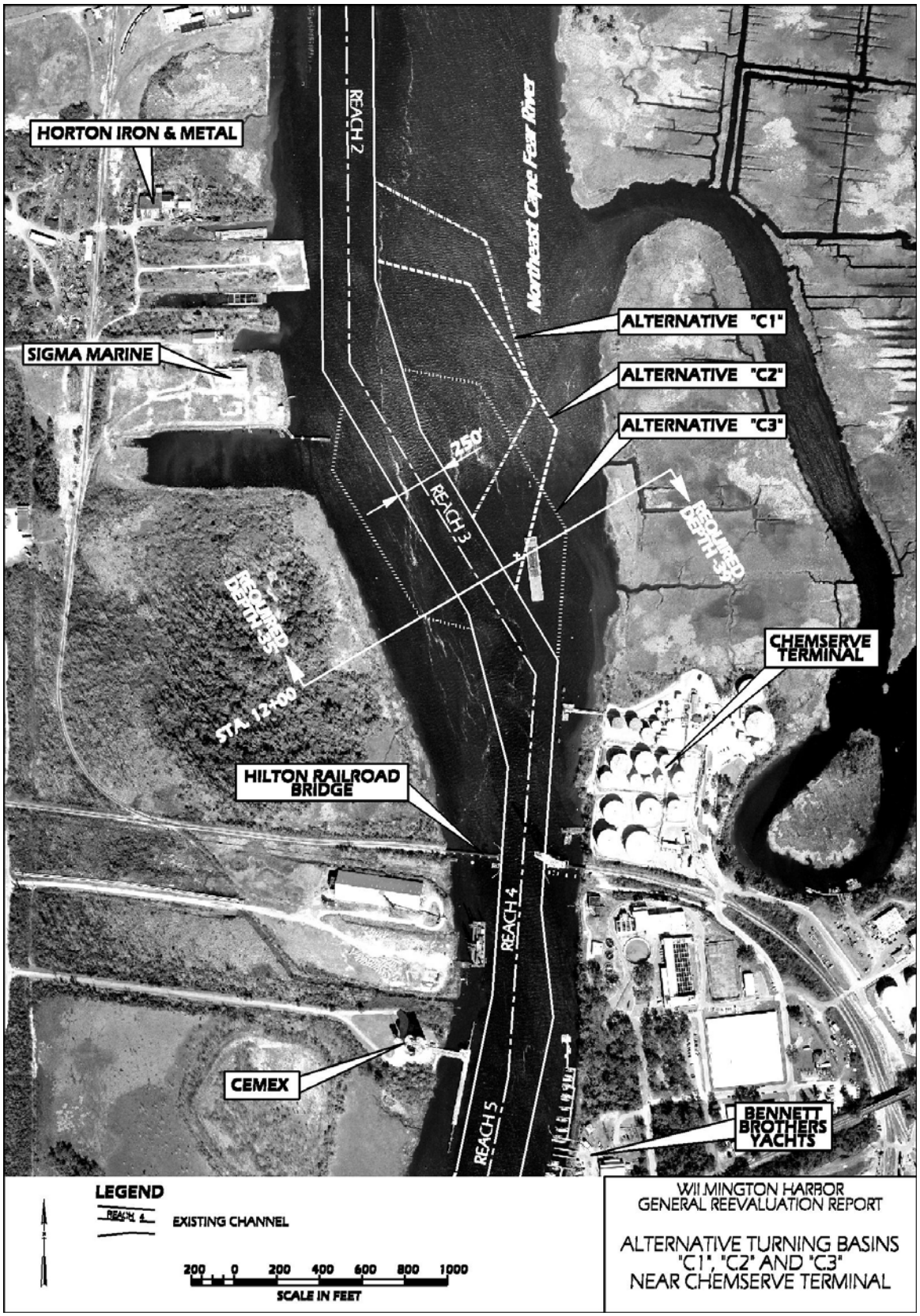
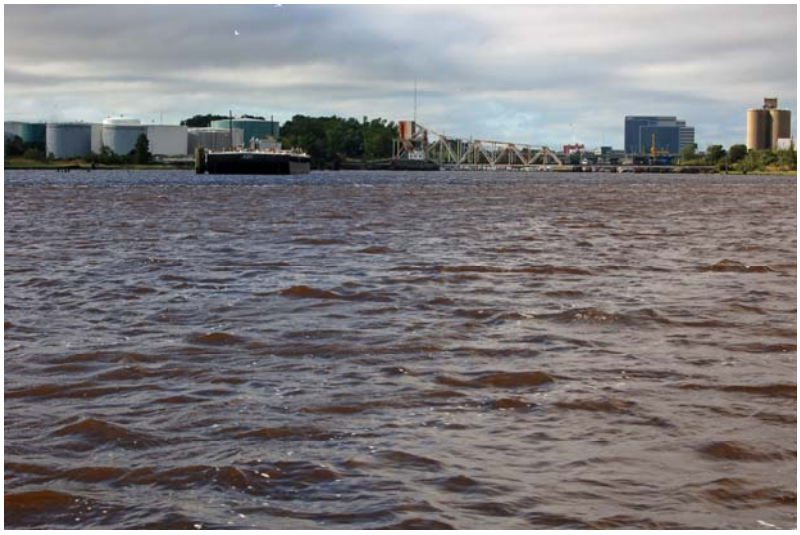


Figure 3.1b: Locations of Turning Basin Alternatives
*Sites **C1**, **C2**, and **C3**, north of Chemserve Terminal*

Turning Basin Alternative
*Site **"C3"***
South option,
north of Chemserve



Turning Basin Alternative
*Site **"C2"***
Middle option,
north of Chemserve

Turning Basin Alternative
*Site "**C1"***
North option,
north of Chemserve

3.2 No action, Turning Basin and Locks and Dams.

3.2.1 Turning Basin. The No Action alternative would maintain the existing ship turning basin at Almont and near Southern States. Most ships would continue to be required to back through the Hilton Railroad and Isabelle Holmes Bridges stern-first. Turning would continue to pose hazards to bridge structures and yacht moorings in the vicinity.

3.2.2 Locks and Dams. Under the no action alternative, the current activities at the locks and dams would continue including locking anadromous fish upstream each spring. This would not allow the anadromous fish population to fully recover and also violation the terms and conditions of the 2000 NMFS Biological Opinion. This opinion required the Corps to construction fish passage at Lock

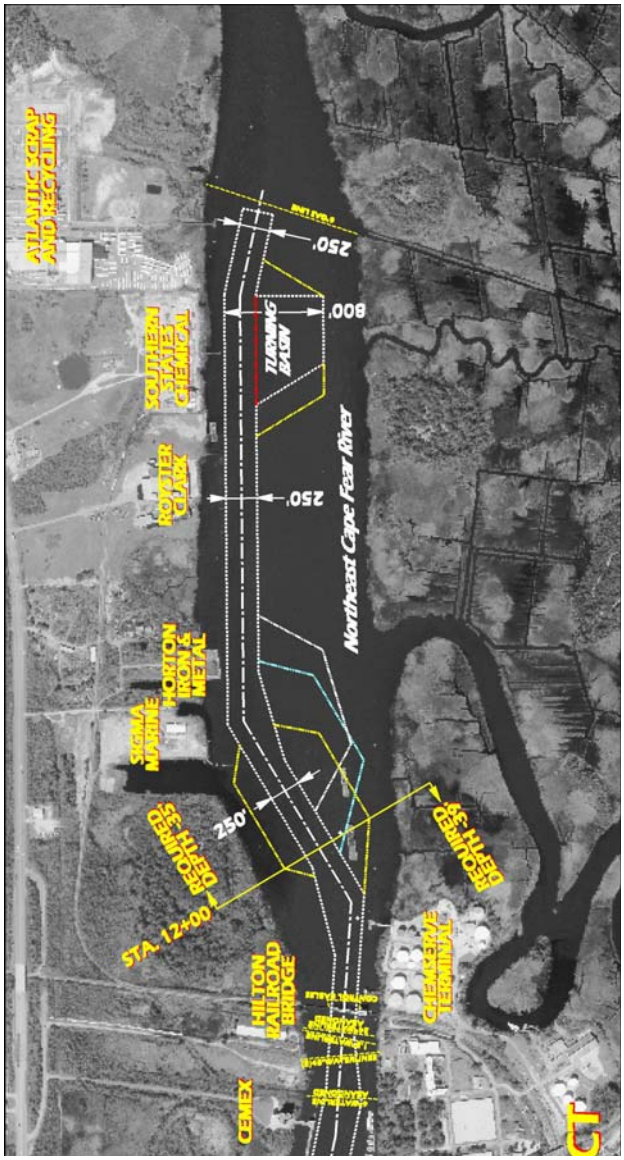


Figure 3.2.2a Turning Basin Alternative Locations

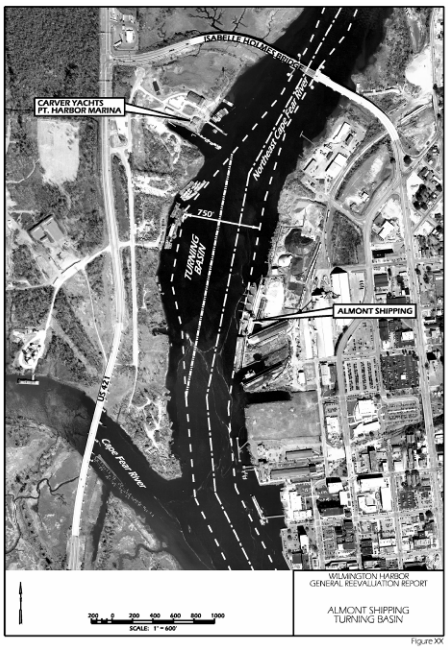


Figure 3.2.2b Existing Almont turning basin

and Dam No. 1, and study fish passage options at Locks and Dams 2 & 3 (NMFS 2000).

3.3 Relocation of ship turning basin in Northeast Cape Fear River. The Almont turning basin has been the only turning basin available in the NECF 32-foot authorized project. As the deepening studies progressed, discussions began with the river pilots regarding their need for a

turning basin that would preclude backing ships through the bridges.

The preferred choice for the turning basin by the pilots was based upon their current needs in the river. The Chemserve terminal became the preferred location for a turning basin since it has the highest current use. Other permutations of turning basins in this area were set up for evaluation as well as an alternative developed around the existing turning basin at the upper project terminus. Four alternatives emerged, all of which would serve the needs of the pilots to varying degrees. All alternatives would have construction methods of some combination of rock blasting, mechanical excavating and cutter-suction hydraulic dredging.

All dredged material will either be pumped to the existing 740 acre Eagle Island confined disposal facility about 2-3 miles from the alternative basin locations or transported to the EPA approved ocean dredged material disposal site (ODMDS) about 40 miles downstream. There are no likely beneficial uses for the dredged material. The sediments contain salt water, are over 40% silt and clay, and the rock and sediment can not be easily separated. However for the material place in Eagle Island, a dredged material management plan (DMMP) is being prepared for that facility and beneficial uses of the dried dredged material is being evaluated.

Stationing in NE CFR: refer to *Figure 3.3b, Northeast Cape Fear River Stationing*. Descriptions of the river channel in the NE CFR below refer to Reaches and stationing along the reach. The reaches are numbered from Reach 1 at the northern-most extent of the project near Southern States Chemical to Reach 14 in the Anchorage basin south of the Cape Fear Memorial Bridge. Stationing begins at 0+00 at the northern end of the centerline for each reach.

Table 3.3. Decision Matrix for Relocated Turning Basin, Northeast Cape Fear River, NC

	Turning Basins			
	Chemserve Area			
Evaluation Factors	Southern	Middle	Northern	Southern States
Mitigation Habitat Units Required[1]	2	2	1	10
(10 least impact,1 worst)				
Subtotal (Weight X 2)	**4**	**4**	**2**	**20**
Costs: Dredging (Construction & O&M) and Mitigation[2]	10	10	9	1
Subtotal (Weight X 2)	**20**	**20**	**18**	**2**
Pilot's Preference	10	5	5	1
(10 Best, 1 worst)				
Impact of Potential Need for Congressional Approval and Additional Time Requirement	5	5	5	1
(5 Best, 1 worst)				
Total[3]	**41**	**36**	**31**	**34**

1. Mitigation Habitat Units Required. Chemserve (Southern =9.8 acres; Middle =10.1 acres; Northern =10.7 acres) Southern States =4.2 acres. The ranking of the alternatives are inversely proportional to the habitat units (appendix 3.4) impacted.

2. Chemserve (C3 ~ $21 million (m); C2 ~ $22 m; C1 ~ $23) Southern States B1 ~ $29 m
The ranking of the alternatives are inversely proportional to the costs and include mitigation.

3. Alternative with the highest total is the best choice.

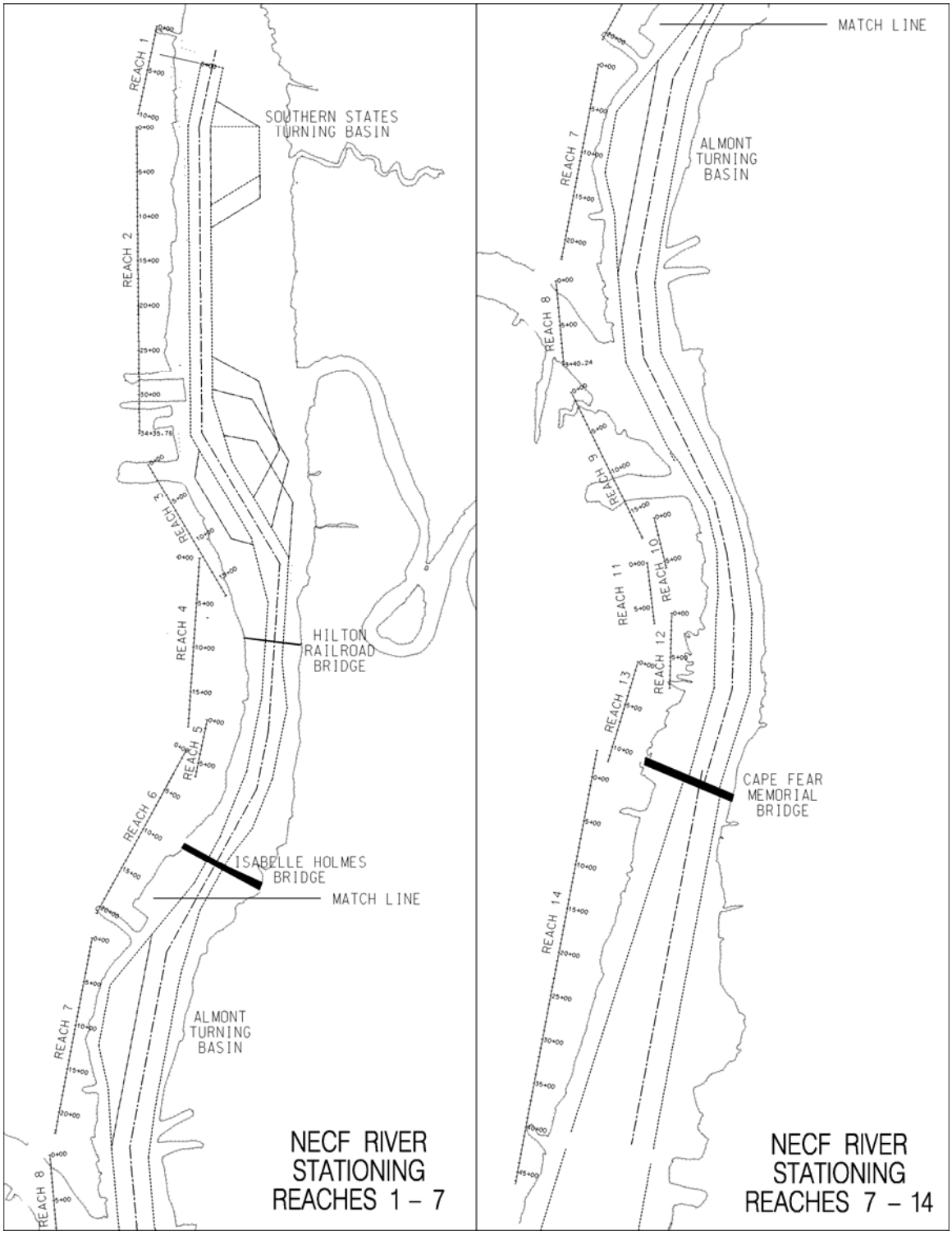


Figure 3.3a, Northeast Cape Fear River Stationing

3.3.1 Alternative B1: enlarging of turning basin at upper terminus. Locate the NECFR primary turning basin at site of existing North turning basin (-25' project) at junction of reaches 1 and 2. This basin is near the site of Southern States Chemical terminal. The basin is lengthened 250' to the south and flared into the adjacent channel at the north and south. The existing turning basin at Reach 7 (Almont) will be abandoned.

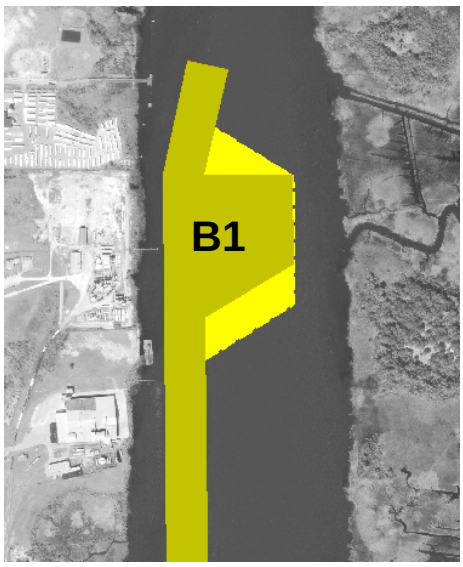


Figure 3.3.1 Location of Alternative B1

Assumptions:
For the purposes of quantity estimates shown below, NECF project is assumed deepened to -35' required (-37' allowable) <u>in the channel only</u> from Reach 3, station 12+00, northward to Reach 1, station 3+00. The existing turning basin remains at its current depth.

Description:

Channel would be deepened to -39' required (-41' allowable) from Reach 3, station 12+00, northward to Reach 1, station 3+00 (northern terminus of project). This includes 1,200 LF in Reach 3, all 3,435 LF of Reach 2 and 875 LF of Reach3. Turning basin would be widened in north-south direction to full 800' by 800' dimensions. In addition, the turning basin is flared on the north and south sides to provide the required geometry as specified in EM 1110-2-1613 for safe and effective deep-draft ship navigation.

Material Quantity Summary:

Note these quantities reflect the difference between the assumed condition and the proposed alternative. [Formula: proposed channel and widened turning basin volumes – (minus) assumed condition channel volumes]

Total sediment: 604,090 CY
Total rock: 303,739 CY
Grand total: 907,829 CY

Primary Nursery Area Impact: 8.8 Acres (measured as the difference between the currently authorized project impacts and Alternative B1 impacts)

3.3.2 Alternative C3: creation of middle turning basin, south. Locate the NECFR primary turning basin just north of Chemserve in Reach 3 – this is the southern most of the three options in this area. This has been described as the "pilots preferred alternative" because of its proximity to the Chemserve terminal and its functional geometry relating to adjacent reaches. Turning basin at Reach 7 (Almont) and at Reach 2 (Southern States) would be abandoned.

Assumptions:
For the purposes of quantity estimates shown below, NECF project is complete with -35' required (-37' allowable) in the channel only from Reach 3, station 12+00, northward to Reach 1, station 3+00. The existing Reach 2 turning basin remains at its current depth.

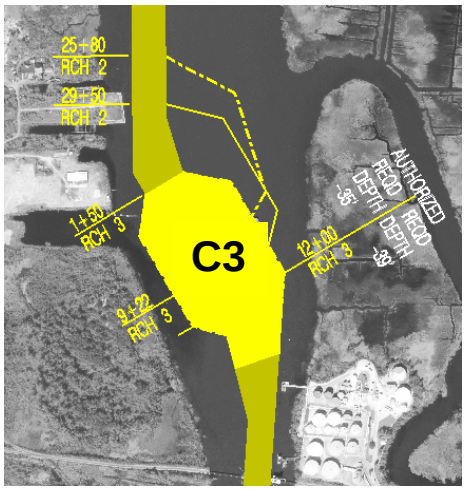


Figure 3.3.2 Location of Alternative C3

Description:

Channel would be deepened to -39' required (-41' allowable) from Reach 3, station 12+00, northward to station 1+50 to accommodate a new 800' by 800' turning basin. Basin corners are chamfered into the existing channel geometry. The 250' wide channel above Reach 3 , station 1+50 would be a -35' required project.

Quantity Summary:

Note these quantities reflect the difference between the assumed condition and the proposed alternative. [Formula: proposed channel and turning basin volumes – (minus) assumed condition channel volumes]

Total sediment: 454,791 CY
Total rock: 178,881 CY
Grand total: 633,672 CY

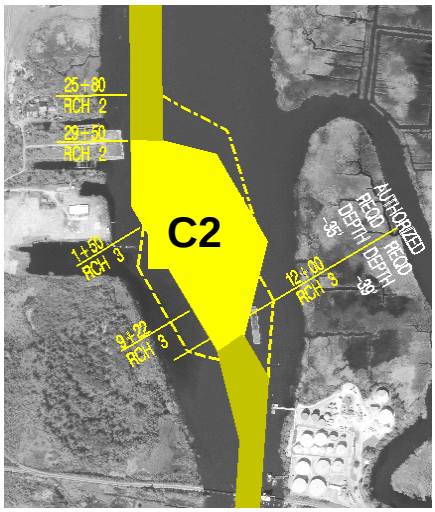


Figure 3.3.3 Location of Alternative C2

Primary Nursery Area Impact: 18.6 Acres (measured as the difference between the currently authorized project impacts and Alternative C3 impacts).

3.3.3 Alternative C2: creation of middle turning basin, middle. Locate the NECFR primary turning basin just north of Chemserve in Reaches 3 and 2 – this is the middle of the three options in this area. Turning basin at Reach 7 (Almont) and at Reach 2 (Southern States) would be abandoned.

Assumptions:

For the purposes of quantity estimates shown below, NECF project is complete with -35' required (-37' allowable) in the channel only

from Reach 3, station 12+00, northward to Reach 1, station 3+00. The existing Reach 2 turning basin remains at its current depth.

Description:

Channel would be deepened to -39' required (-41' allowable) from Reach 3, station 12+00, northward to Reach 2, station 29+50 to accommodate a new 800' by 800' turning basin. Basin corners are chamfered into the existing channel geometry. Southern extent of turning basin begins at station 13+00. The entire turning basin extends east of Reach 3. The 250' wide channel above Reach 2 , station 29+50 would be a -35' required project.

Quantity Summary:

Note these quantities reflect the difference between the assumed condition and the proposed alternative. [Formula: proposed channel and turning basin volumes – (minus) assumed condition channel volumes]

Total sediment: 597,741 CY
Total rock: 149,343 CY
Grand total: 747,084 CY

Primary Nursery Area Impact: 19.0 Acres (measured as the difference between the currently authorized project impacts and Alternative C2 impacts).

3.3.4 Alternative C1: creation of middle turning basin, north. Locate the NECFR primary turning basin just north of Chemserve in Reaches 3 and 2 – this is the upper northern most of the three options in this area. Turning basin at Reach 7 (Almont) and at Reach 2 (Southern States) would be abandoned.

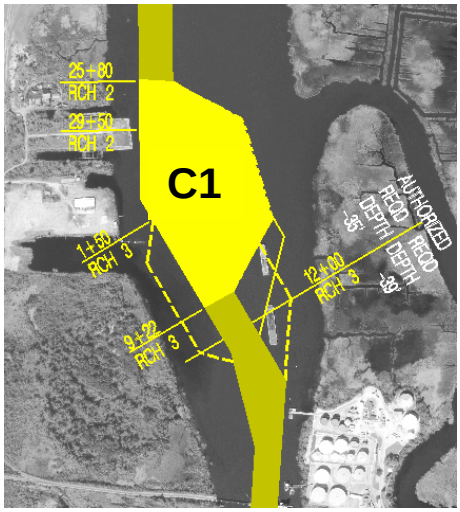


Figure 3.3.4 Location of Alternative C1

Assumptions:
For the purposes of quantity estimates shown below, NECF project is complete with -35' required (-37' allowable) <u>in the channel only</u> from Reach 3, station 12+00, northward to Reach 1, station 3+00. The existing Reach 2 turning basin remains at its current depth.

Description:

Channel would be deepened to -39' required (-41' allowable) from Reach 3, station 9+22, northward to Reach 2, station 25+80 to accommodate a new 800' by 800' turning basin. Basin corners are chamfered into the

existing channel geometry. The turning basin extends east of the junction of Reaches 2 and 3. The 250' wide channel above Reach 2 , station 25+80 would be a -35' required project.

Quantity Summary:

Note these quantities reflect the difference between the assumed condition and the proposed alternative. [Formula: proposed channel and turning basin volumes – (minus) assumed condition channel volumes]

Total sediment: 677,869 CY
Total rock: 160,101 CY
Grand total: 837,970 CY

Primary Nursery Area Impact: 19.6 Acres (measured as the difference between the currently authorized project impacts and Alternative C1 impacts).

Table 3.3 is the decision matrix used to assist the PDT in selecting the best turning basin alternative. The matrix was developed during four PDT meetings in April and May 2007. The turning basin alternatives are listed along the top of each matrix and evaluation factors are listed along the left side. The rating is an ordinal scale from 1 to 10: 1 is the worst and 10 is the best. The scale is very common and widely accepted. Ordinal only indicates there is an order: 10 is better than 9, 9 is better than 8, etc. However ordinal does not imply a multiple or percentage of difference. Ten is not twice as good as 5, or 6 is not 10% better than 5.

On the ordinal scale used, 10 would be for the alternative that has the least cost or least impact to primary nursery areas, and 1 would be for the alternative with the most cost or greatest impact to primary nursery areas. All the other alternatives were distributed proportionally to the nearest whole number between the two extremes on the 1 to 10 scale. A rating of 5 would be a cost or fish passage potential mid-way between the two extremes. Costs are total costs which includes mitigation. Most evaluation factors have a weight of one, but the first two factors have a weight of two (2) since they are the most important considerations. In summary, the matrix indicates that the southern basin (pilot's preferred) is the best alternative. Potential mitigation sites in the vicinity of Wilmington are indicated in figure 3.3b.

3.3.5 Ship simulation. The Wilmington District has proposed that the existing turning basins on the Northeast Cape Fear River be abandoned and a new basin be relocated upstream from the Hilton RR Bridge. The Hilton RR Bridge is the most upstream bridge of the two bridge structures spanning the Northeast Cape Fear River. These alternatives were developed to provide more efficient navigation, enhance safety, and reduce maintenance costs. Four turning basin alternatives have been identified. The expansion of the existing turn alternative has been designated as alternative B1. The three additional turning basin

alternatives, located above the Hilton RR Bridge in a downstream direction, were assigned designations of C1, C2, and C3. The Wilmington District requested that the US Army Engineer Research and Development Center (ERDC) conduct a desktop study of the proposed turning basin alternatives to assess navigational issues that need to be addressed by conducting additional ship simulation studies. A MEMORANDUM FOR Commander, US Army Engineer District, Wilmington, dated 10 April 07 (Appendix 3.3.5), presents ERDC recommendations and methodology utilized to determine the necessity of developing additional ship simulation studies. Based on their review of the hydrodynamic model studies included in the Taylor Engineering shoaling analysis report (Appendix 3.8.3) and the ship simulator studies conducted for the authorization of the existing turning basin, ERDC concurs that a ship simulation study is not necessary to evaluate the safety of the turning basin alternatives proposed by the Wilmington District.

3.4 Mitigation for new turning basin. Mitigation will be required for the proposed relocated turning basin in the Northeast Cape Fear River above the Hilton Railroad Bridge. The impact associated with mitigation is the loss of primary nursery area (PNA) that is within the footprint of the relocated basin. PNA is defined as tidal saltwaters which provide essential habitat for the early development of commercially and recreationally important fish and shellfish. These areas are designated by the NC Division of Marine Fisheries. In the vicinity of the potential basin relocation, PNA is located in all portions of the river except within the authorized federal channel and associated side slopes. The basin locations are indicated in Figures 3.1a and b, and the area of PNA impacted for each basin is indicated in table 3.4a. The nursery areas of most concern are those that are converted from shallow areas (< 10 feet deep) to a depth greater than 10 feet deep since the shallow habitat is of greater PNA value. The mitigation required for each turning basin alternative is indicated in table 3.4a which is based on the analysis in appendix 3.4. The cost for construction of the turning basins and mitigation alternatives are indicated in table 3.4b along with fish passage discussed below. Appendix 3.4 also includes the results of the incremental analysis (Institute for Water Resources, http://www.pmcl.com/iwrplan) performed for the mitigations options discussed below.

The Mitigation PDT for the Wilmington Harbor GRR involved in the development of appendix 3.4 is an interagency team lead by the Wilmington District Ecosystem Restoration Coordinator. Interagency team members include Federal and state resource specialist including; US Fish and Wildlife Service, NOAA Fisheries, NC Division of Marine Fisheries and the NC Wildlife Resources Commission. The team leader and agency representatives have specific knowledge and experience the Cape Fear River ecosystem. This team is responsible for the mitigation assessment for the GRR. The mitigation assessment procedure is a derivation of two standard environmental assessment methods, combining elements of Wetland Evaluation Technique (WET) and Habitat Evaluation Procedure (HEP). This method has been successfully used

for prior Wilmington Harbor mitigation analysis. Like WET this analysis is based on evaluation of functions and values and like HEP output is measured in Habitat Units generated by multiplying Habitat Value X Area. Functions pertinent to the specific impact site (Cape Fear River, mid river primary nursery) were selected from a comprehensive list that includes general wetland functions as identified in WET with additional estuarine functions and values added by the Mitigation PDT to assure inclusion of site specific concerns. Initial selection of functions to be considered, and assignment of weighting factors and relative values were made by the team leader using best professional judgment, based on site specific observations during team site inspections and review of aerial photographs, survey data and other information gathered from the impact area and alternative mitigation sites. Area estimates were based on geospatial analysis using GIS. While the values are subjective, they were derived from experience in restoration ecology, impact assessment and mitigation analysis within the Cape Fear River ecosystem. Scores were verified by the District Lead Biologist for the GRR and the preliminary analysis was reviewed by the Mitigation PTD and coordinated with the GRR PDT to confirm that values, weighting factors and findings were realistic and would pass a test of ecological reasonability. Team comments were addressed and requested revisions made. The GRR Mitigation PDT reviewed the revised mitigation analysis and a general team consensus has provided tentative concurrence with scores, methods and findings, appropriate for this stage of the planning process. The revised analysis has also been discussed with the GRR PDT and all comments have been considered.

Table 3.4a Acres of Primary Nursery Habitat Impacted for the Turning Basin Alternatives and Associated Required Mitigation

	Area Impacted (Acres)								
Alternative/Basin	Surface to -5ft	-5 to -10ft	**Subtotal shallower than -10ft**	-10ft to -15ft	-15ft to -20ft	-20ft and deeper	**Subtotal deeper than -10ft**	**Total**	**Mitigation Required (HUs)[1]**
Pilot's Preferred Alternative (C3)	2.96	1.83	4.79	2.28	5.70	5.44	13.43	**18.22**	9.8
Middle Turning Basin Alternative (C2)	3.50	2.20	5.70	4.53	4.04	4.60	13.17	**18.87**	10.1
Upper Turning Basin Alternative (C1)	1.98	5.43	7.41	4.80	3.57	3.75	12.12	**19.53**	10.7
Southern States Turning Basin Alternative (B1)	0.29	1.25	1.54	2.93	2.81	1.24	6.98	**8.52**	4.2

1. Based on habitat units (HUs) developed in appendix 3.4.

Table 3.4b Summary of Cost Estimates of Fish Passage Alternatives, Turning Basins, and Primary Nursery Area Mitigation.

Fish Passage Alternatives	Lock and Dam #1	Lock and Dam #2	Lock and Dam #3
Remove Lock and Dam[3]	$ 3,270,278	$ 2,040,649	$ 2,535,833
Rehab Lock and Dam	$ 5,546,719	$ 4,811,429	$ 1,203,603
Rehab Lock and Dam, Do not Fill Scour Hole	$ 2,652,319	$ 1,012,529	
Rock rapids with a Lower Dam with Reduced Water Supply Capacity	$ 2,486,112	$ 1,799,674	$ 1,860,725
Lower Dam with no Fish Passage Needed	$ 2,805,461	$ 2,237,240	$ 2,058,888
Rapids Downstream of Dam with filling scour hole[1]	$ 11,328,187	$ 10,706,330	$ 4,253,588
Rapids Downstream of Dam without filling scour hole[2]	$ 8,433,787	$ 6,907,000	
Rapids in Lock Chamber	$ 1,322,636	$ 1,457,053	$ 1,221,975
Rapids Downstream and in Lock Chamber	$ 13,579,245	$ 15,992,774	$ 5,363,362
Rapids Downstream of Dam with Lock and Dam # 2 Removed			$ 6,992,640
Rapids Downstream of Dam and in Lock with Lock and Dam # 2 Removed			$ 7,925,358
Short Diversion Channel	$ 2,577,757	$ 3,815,296	$ 15,957,351
Natural Like By-pass	$ 4,203,625		
Rock Ramp Upstream of the Dam	$ 16,197,301	$ 18,573,115	

Turning Basin Alternatives	without mitigation
Pilots Preferred Basin (C3)	$13,033,000
Middle Basin (C2)	$14,513,000
New Northern Basin (C1)	$15,244,000
Southern States Basin (B1)	$25,971,000

Turning Basin Alternatives	with mitigation[4]
Pilots Preferred Basin	$20,690,000
Middle Basin	$22,170,000
New Northern Basin	$22,901,000
Southern States Basin	$28,539,000

Primary Nursery Area Mitigation	
Island 12	
9.4 acres	$2,568,000
12.6 acres	$3,666,000
18.5 acres	$10,758,000
Alligator Creek	$5,632,716

1. Lock and Dam # 3 does not have a scour hole downstream
2. Rock to stabilize the scour hole is an O&M cost and would be required if the dam is left in place regardless of which fish passage alternative is selected. If the scour hole is stabilized first and a rock ramp constructed downstream of the dam, the cost of the ramp will be reduced.
3. If Lock and Dam # 2 is removed, then the costs impacts at Lock and Dam # 3 needs to be included. This additional cost is uncertain at this point but would be related to lower water levels downstream of Lock and Dam #3. For example, impacts could include reduced locking efficiency for fish passage due to lower water levels in the lock chamber, and Lock and Dam # 3 stability issues. Interim fish passage fixes could include a rock ramp in the lock chamber, and potential dam instability issues addressed by placing rock on the downstream face of the dam.
4. Mitigation costs for the first 3 alternatives assumes a rock ramp at Lock and Dam # 2 and $750,000 for three years of fish passage monitoring. For the last alternative, the mitigation cost is for the 9.4 acre Island 12 restoration.

Table 3.4c. Decision Matrix for Relocated Turning Basin, Northeast Cape Fear River, NC

	Turning Basins			
	Chemserve Area			
Evaluation Factors	Southern	Middle	Northern	Southern States
Mitigation Habitat Units Required[1] (10 least impact,1 worst)	2	2	1	10
Subtotal (Weight X 2)	**4**	**4**	**2**	**20**
Costs: Dredging (Construction & O&M) and Mitigation[2]	10	7	1	1
Subtotal (Weight X 2)	**20**	**14**	**2**	**2**
Pilot's Preference (10 Best, 1 worst)	10	5	5	1
Impact of Potential Need for Congressional Approval and Additional Time Requirement (5 Best, 1 worst)	5	5	5	1
Total[3]	**41**	**30**	**15**	**34**

1. Mitigation Habitat Units Required. Chemserve (Southern =9.8 acres; Middle =10.1 acres; Northern =10.7 acres) Southern States =4.2 acres. The ranking of the alternatives are inversely proportional to the habitat units (appendix 3.4) impacted.

2. Chemserve (Southern~$21 million (m); Middle~$22 m; Northern~$24) Southern States~$24 m
The ranking of the alternatives are inversely proportional to the costs and include mitigation.

3. Alternative with the highest total is the best choice.

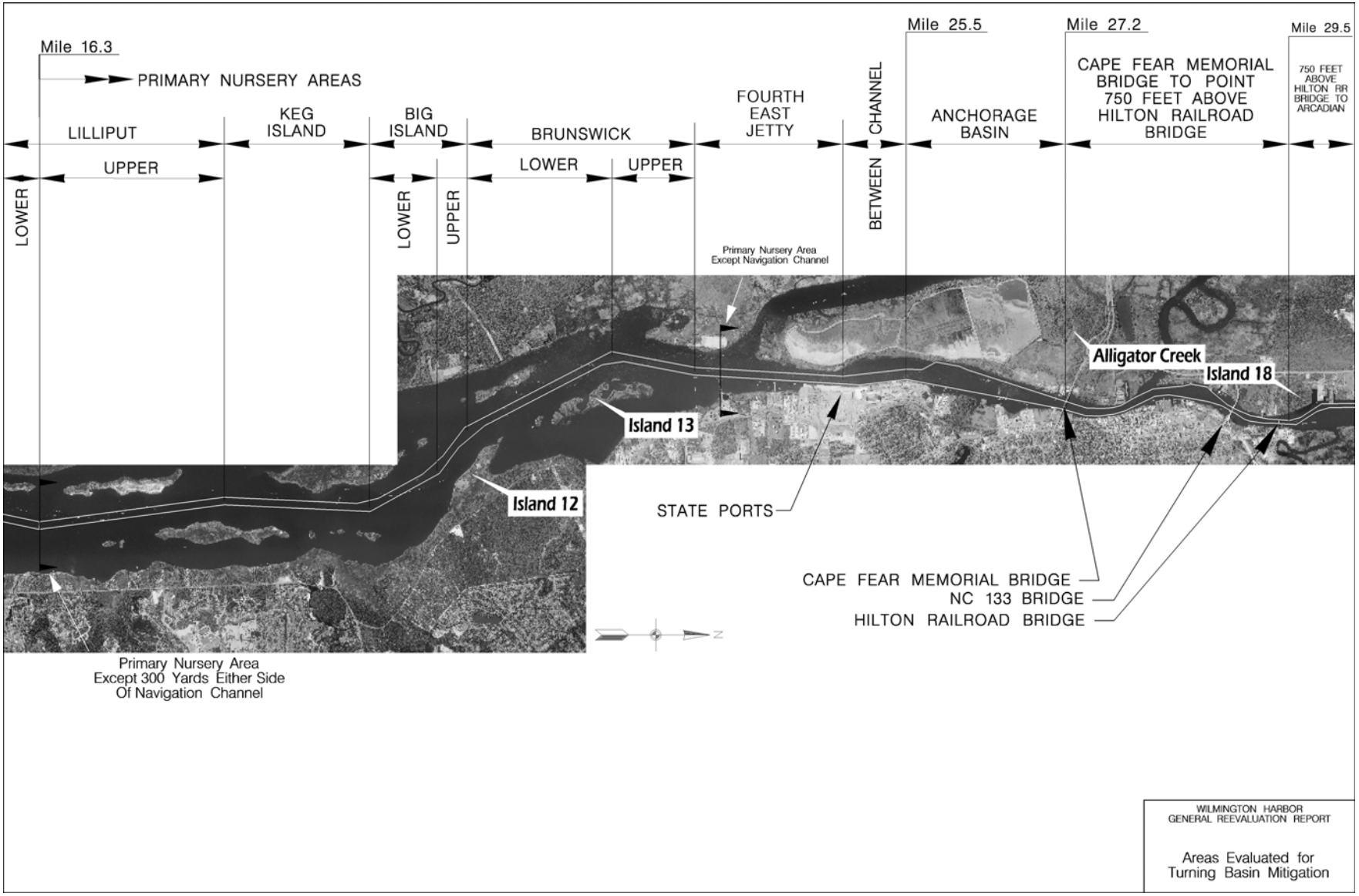


Figure 3.4a Areas Evaluated for Turning Basin Mitigation
Alligator Creek and Islands 12, 13, and 18

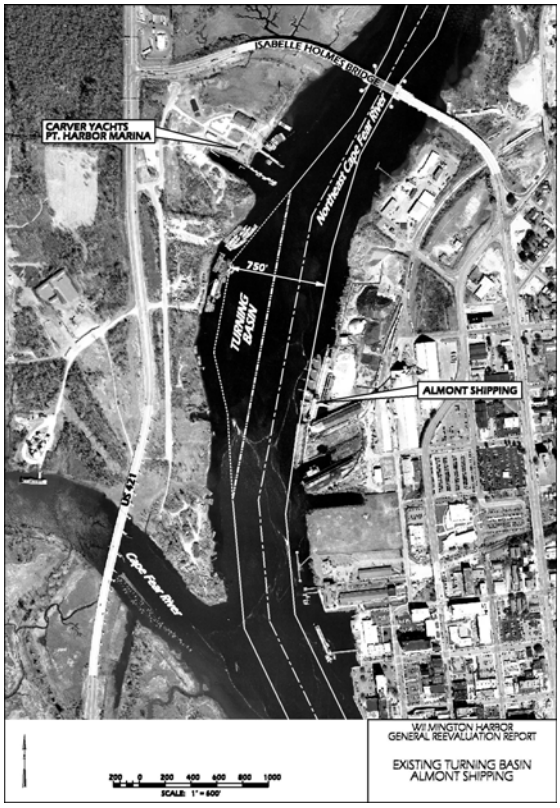


Figure 3.4b Existing Almont turning basin, potentially abandoned

Figure 3.4c Existing Southern States turning basin, potentially abandoned

3.4.1 Fill-in abandoned basins. If one of the three basin alternatives near Chemserve is chosen then the other two existing basins (Almont and Southern States) could be recommended for deauthorization and subsequently abandoned. The authorized federal project for these two basins combined is 19.3 acres. Considering the natural shoaling rates for the basins, the Almont Basin will equilibrate in about 19 years with 4.4 acres less than 10 feet deep, and the Southern States Basin will equilibrate in about 17 years with 1.0 acre less than 10 feet deep. The shoaling process would take many years to provide any measurable benefit, and the sediment surface will not be as consolidated as natural substrate. Thus the habitat value for benthos and fisheries in the abandoned basin will probably not be as high as that for natural substrate. Since this shoaling will occur naturally without expenditure of funds, this option was not considered a mitigation alternative (appendix 3.4)

Enhancing the natural shoaling rate by placing dredged material in the abandoned basin from construction of the selected basin or from maintenance activities were evaluated. Using dredged material from construction of the selected basin would not be feasible because the basin to be abandoned would need to remain in operation until construction of the relocated basin is completed. Using maintenance material to fill the abandoned basin once the relocated basin is complete is an option, but this maintenance material has a high percent of silts and clays (>40%). This would create extensive turbidity and suspended solids problems, plus confining the material to the abandoned basin would be difficult.

For these reasons, filling the abandoned basins with dredged material was eliminated as a feasible alternative.

One of the basin alternatives is to expand the Southern States basin, and abandon the Almont basin. Placing dredged material in the Almont basin would have the same issues as indicated above.

3.4.2 Restoration of Alligator Creek on Eagle Island. Prior to the development of Eagle Island as a confined disposal facility (CDF) for dredged material, there were several tidal creeks that traversed the site (figure 3.4.2a). Many of these creeks are now covered by the CDF, but the portion of Alligator Creek that runs northwest to southeast is outside the dike alignment that surrounds the CDF. This creek has closed up over the decades since the access road to Eagle Island blocked tidal flow in the creek.

Figure 3.4.2c Existing access road blocking Alligator Creek

The mitigation action would restore approximately 10,000 feet of creek, and place large box culverts through the access road or a bridge to raise the access the creek. Conceptual drawings are indicated on figure 3.4.2b. All excavated material (about 161,700 cubic yards) would be pumped to the Eagle Island confined disposal area. The proposed restoration action would restore about 25 acres of intertidal and subtidal habitat. This would include 6.8 acres of intertidal marsh (saltmarsh cordgrass, *Spartina alterniflora*) to be planted adjacent to the restored creek. The connections to Redmond Creek and Cape Fear River would

Figure 3.4.2d Alligator Creek restoration: Redmond Creek connection area

Figure 3.4.2e Alligator Creek restoration: Cape Fear River connection area

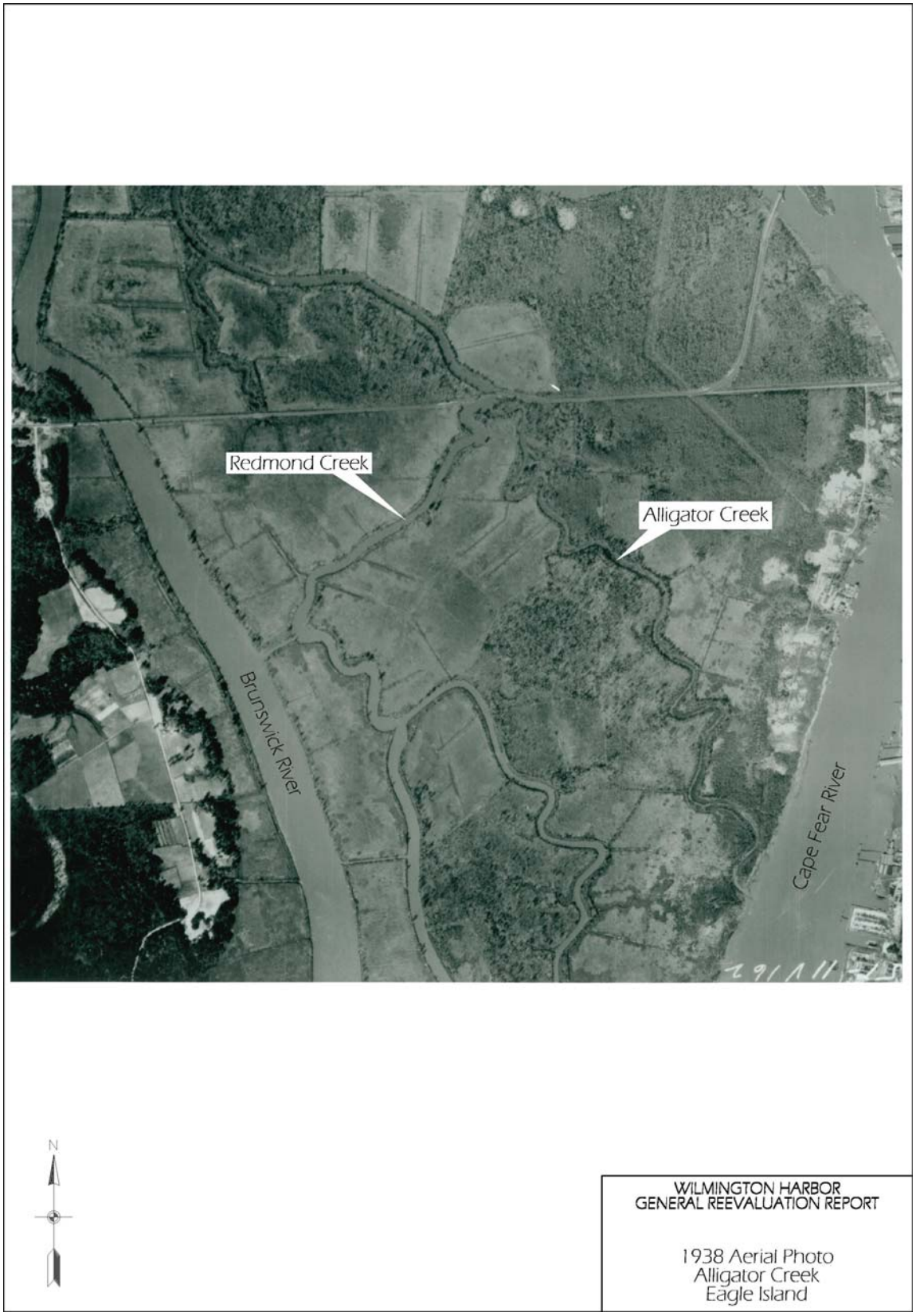


Figure 3.4.2a Alligator Creek, Eagle Island
1938 aerial photograph

Figure 3.4.2b Alligator Creek restoration corridor
Cape Fear River to Redmond Creek

be about 10,000 feet apart which would provide enough tidal head difference to allow about 3.2 - 5.1 exchanges of tidal water through the restoration area per 25 hour tidal cycle. The restored creek would have similar meanders as the old creek based on historical aerial photography. This restoration would also allow regular tidal flushing to the interior marshes that has not occurred in years. The existing marsh along and adjacent to the restored creek alignment consist primarily of common reed (*Phragmites australis*).

3.4.3 Island restorations. Islands 12, 13, and 18 discussed below were all created by dredged material from the adjacent federal channel. They were all created and primarily used before the passage of the National Environmental Policy Act in 1969. Restoration basically involves the removal of the dredged material created domes down to intertidal marsh and open water habitat.

3.4.3.1 Island 12 restoration. Island 12 was created by dredged material from the adjacent federal channel prior to the 1970's. The island was created by disposal of dredged material from a pipeline since several open sandy/marl covered domes are scattered throughout the site. The island is surrounded by water on all sides (figure 3.4.3.1). The federal channel is on the west side with intermittent narrow patches of saltmarsh cordgrass located along the shoreline. Extensive areas of marsh surround the rest of the island. Most of the marsh is saltmarsh cordgrass with patches of narrow leaf cattails (*Typha angustifolia*). The back (east) side of the island is adjacent to PNA.

Figures 3.4.3.1a:
Island 12 Stream Creation:
Connection area

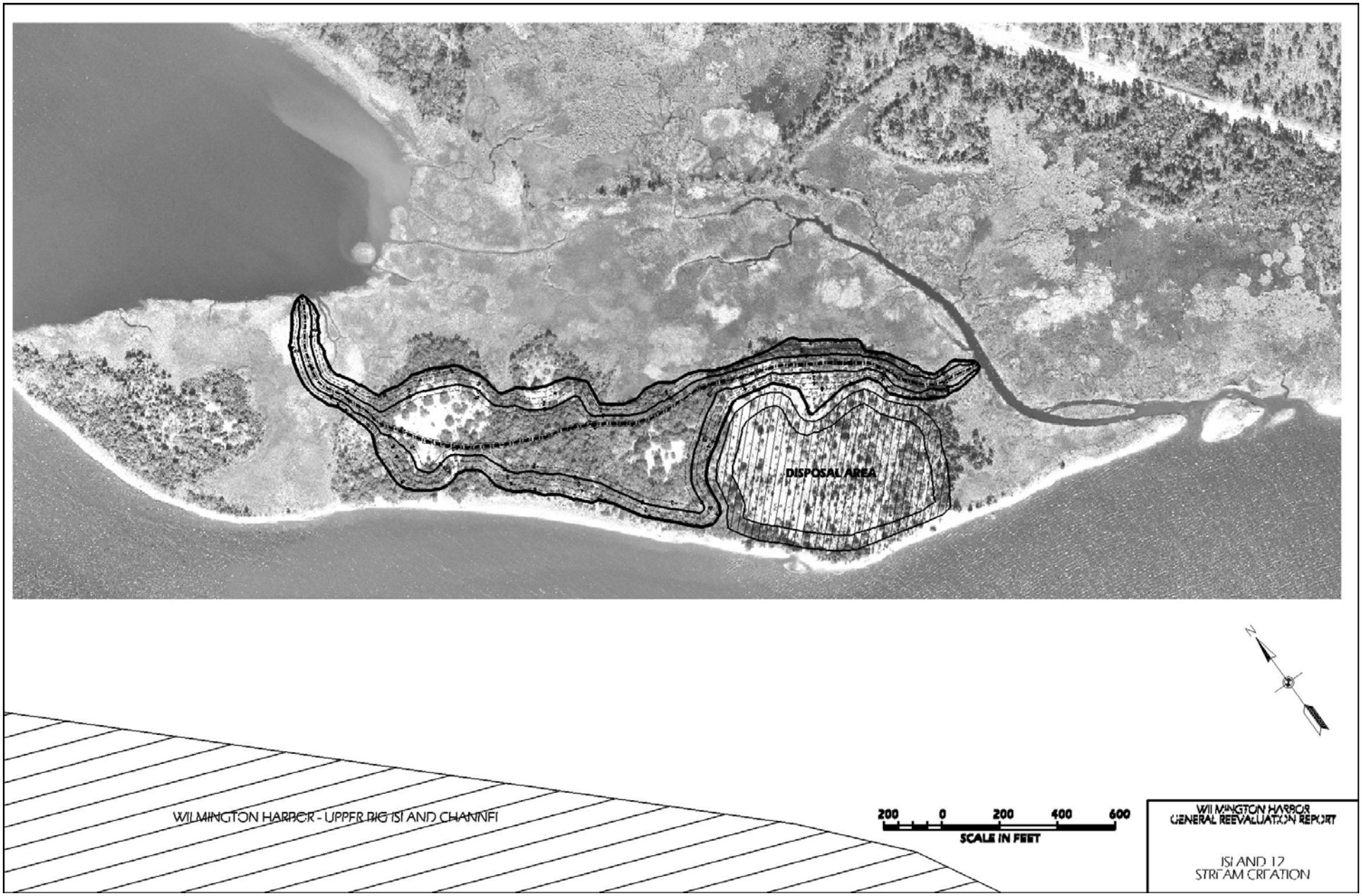


Figure 3.4.3.1b Island 12 Stream Creation

The proposed restoration action would involve three alternatives (table 3.4b). These three alternatives would create about 9.4, 12.6, and 18.5 acres of intertidal and subtidal habitat that is connected to PNA. Figure 3.4.3.1 illustrates the middle alternative. The restoration would include about 3 acres of intertidal marsh (saltmarsh cordgrass, *Spartina alterniflora*) to be planted along the interior shoreline. The connections will be primarily through narrow leaf cattail marsh and wetland shrubs (*Baccharis halimifolia*) and extend to the -4 foot contour of the nursery area. The connections will be about 2,700 feet apart which will provide enough tidal head difference to allow about 1.4 – 1.6 exchanges of tidal water through the restoration area per 25 hour tidal cycle. Most of this restoration would be between the 5 and 8 foot contours of the island except where the restored area would be connected to the primary nursery. The work would generally stay below the 8 foot contour to reduce the amount of material to be removed, and stay above the 5 foot contour to avoid existing wetlands. All excavated material (for the smaller two alternatives) would be spread over the remaining sandy/marl covered domes on the island. The 18.5 acres alternative would pump the excavated material to the Eagle Island confined disposal facility (CDF). On Island 12, sapling live oak trees and other vegetation would be planted on the domes where dredged material was placed in order to enhance the terrestrial value of the site.

Figure 3.4.3.1c Island 12 Stream Creation: connection area at southeastern creek

3.4.3.1d: West shoreline of Island 12 Federal channel in offshore waters

3.4.3.2 Island 13 amplification. Approximately 30 acres of intertidal habitat were created in 2001 as mitigation for Wilmington Harbor deepening. The upland was obviously created by disposal of dredged material from a pipeline since several sandy/marl covered domes were scattered throughout the site. Several of these domes were removed to produce the intertidal habitat, and the material was pumped to island 14, upland CDF just south of Eagle Island. Several small vegetated domes (< 5 acres) remain on the island that could be removed to create similar intertidal habitat. However, the resource agencies prefer to leave these domes undisturbed because of the diversity of habitat they provide. Also restoring these small domes alone would not provide the mitigation acreage needed, and the unit costs for these small areas would be more

expensive that constructing larger restoration areas at Island 12 or restoring Alligator Creek.

3.4.3.3 Island 18 restoration. Island 18 is surrounded by the river on the east, railroad tracks on the south and west, and a boat basin on the north. Approximately 22 acres of mixed marsh surround the upland portion of the site. This marsh consists of cattails, sawgrass (*Cladium jamaicense*), and common reed. The upland portion on the site is about 28 acres and is dominated by live, willow and water oaks (*Quercus virginiana, phellos, and nigra*, respectively) with an understory of the same species plus privet (*Ligustrum sinense*). Several low swales are present between the dredged material domes.

Figure 3.4.3.2 Island 13 – view south

Restoration opportunities include excavating creeks through the marsh to increase tidal flushing along with lowering the swales to intertidal elevations and connecting them to the constructed tidal creeks. These constructed creeks would be connected to the NE Cape Fear River which is designated as PNA adjacent to the site. Also, the uplands could be preserved as unique upland. However, the resource agencies were not receptive to this potential mitigation component due to the proposed excavation of a creek in marsh where a creek was not evident previously—and due to the preservation of upland as a component of mitigation for loss of PNA.

Figure 3.4.3.3 Island 18 – view south from northeast point

3.4.4 Improved fish passage at the locks and dams. Under the NMFS Biological Opinion (BO) dated August 2000; the Corps was required to construct fish passage at Lock and Dam No. 1, and study fish passage options at Locks and Dams 2 & 3 (NMFS 2000). See Figures 3.4.4 aa – cc. During recent mitigation discussions with the resource agencies regarding the

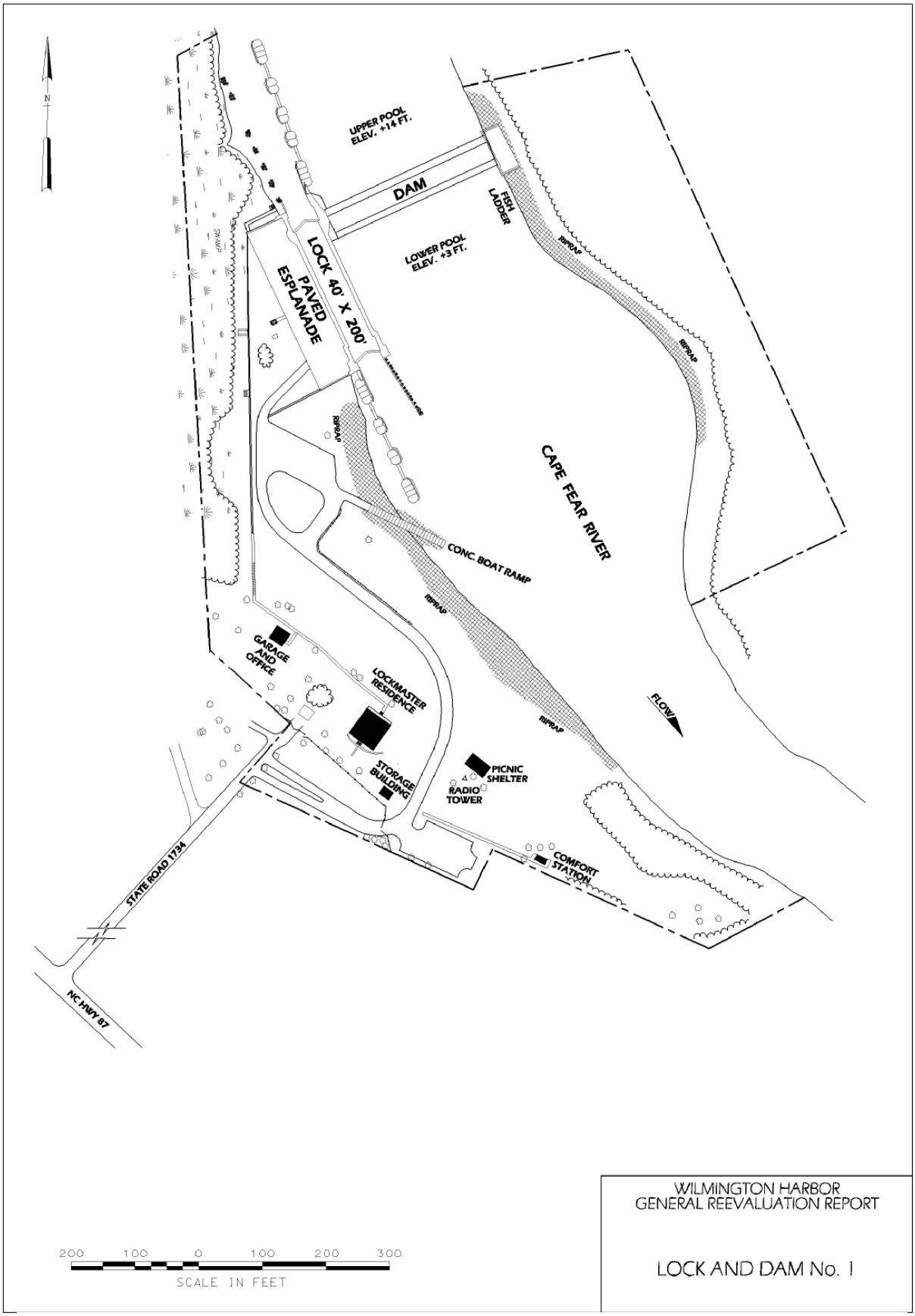


Figure 3.4.4 aa – Existing Lock and Dam No. 1

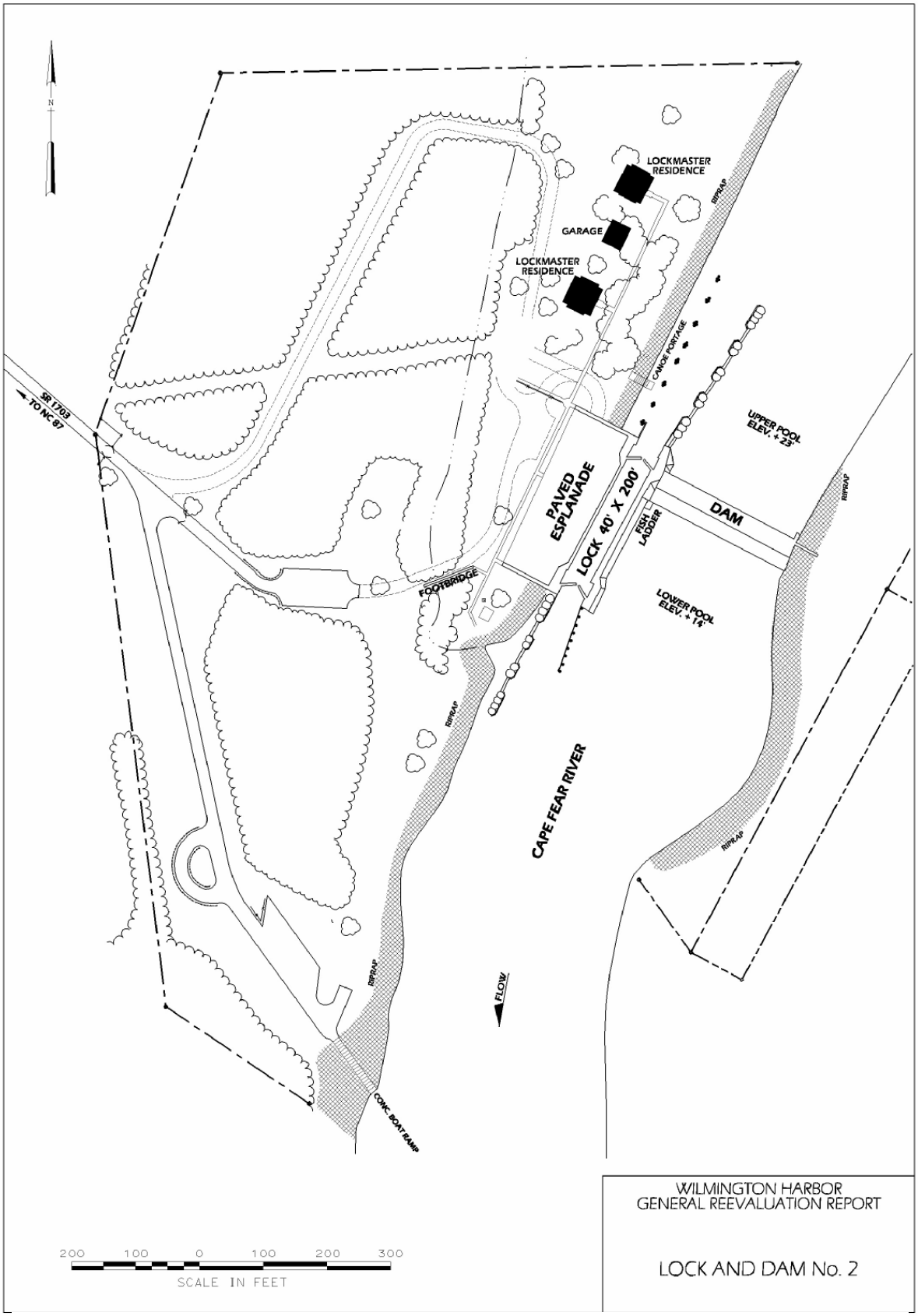


Figure 3.4.4 ab – Existing Lock and Dam No. 2

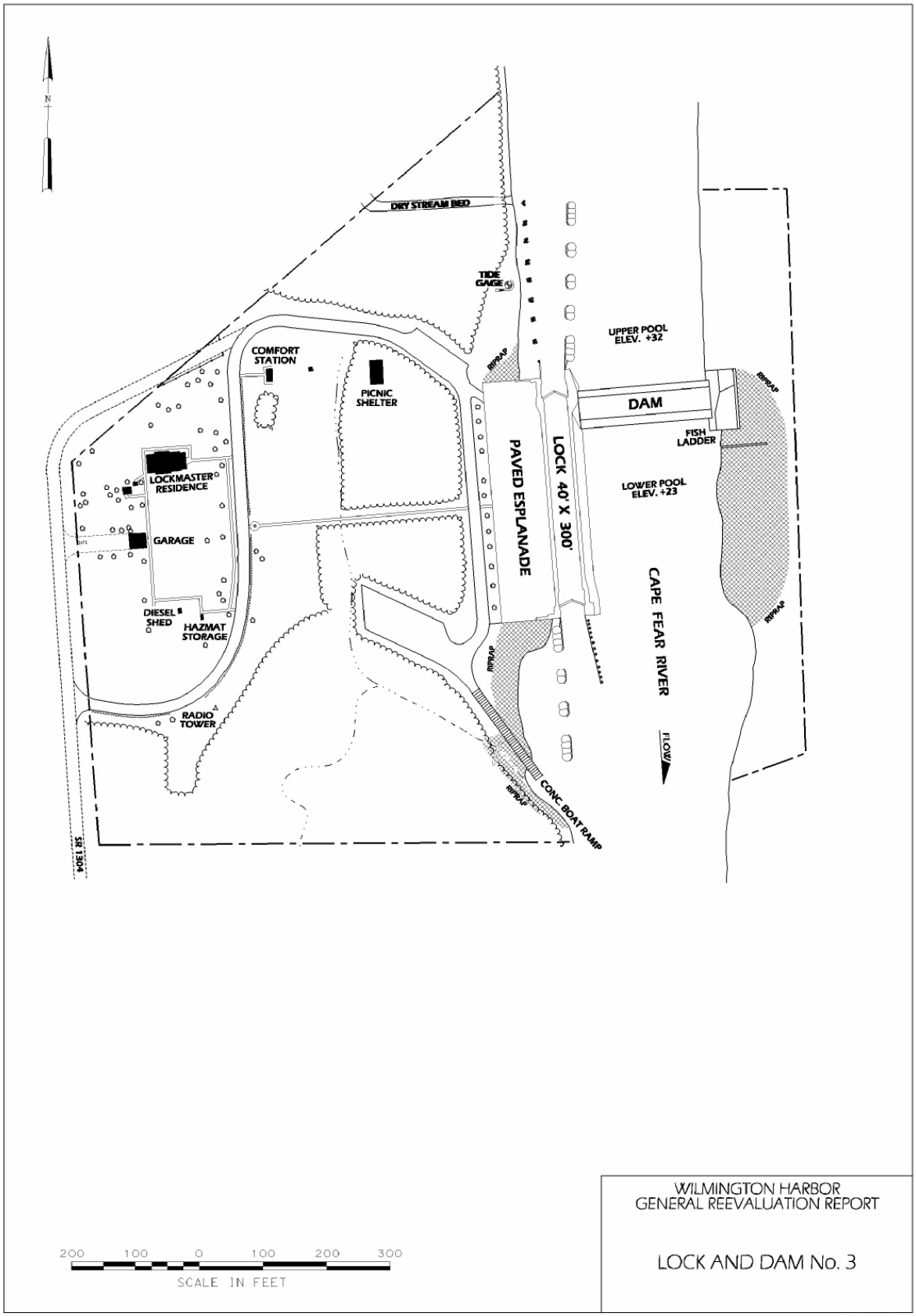


Figure 3.4.4 ac – Existing Lock and Dam No. 3

relocated turning basin, going beyond the BO requirements was considered, by constructing fish passage at Locks and Dams 2 and/or 3 as mitigation for the relocated turning basin. The resource agencies were very interested in fish passage at all Locks and Dams. Fish passage options are discussed below.

3.5 Fish passage.

3.5.1 Requirements. To date, the Wilmington Harbor deepening project has resulted in two environmental impacts that required mitigation. First, construction has and will result in the loss of wetlands and PNAs. To mitigate for this impact, a 30-acre embayment on Disposal Island 13 was altered to create intertidal wetlands and PNA (see paragraph 3.4.3.2 above). Another component of this mitigation was acquisition by the non-Federal sponsor, the State of North Carolina, of about 700 acres of prevention of degradation (POD) lands on the Northeast Cape Fear River. This acquisition would protect about 29 acres of existing PNA. The acquisition procedure for this component is ongoing.

The second aspect of mitigation is for potential impacts to the endangered shortnose sturgeon resulting from blasting to remove rock. Through discussions with National Marine Fisheries Service (NMFS), condition number 8 of the August 3, 2000 Biological Opinion (BO) indicated that the COE must construct a fish passage structure at Lock and Dam No. 1 on the Cape Fear River, and must study the best fish passage design for Locks and Dams 2 & 3. The type of fish passage structure was not specified in the BO. Without this condition, NMFS would have given the Corps a take of zero which would have effectively precluded blasting and the successful completion of the project. With this condition, the Corps was given a take of (2) shortnose sturgeon. Blasting to date has not resulted in a documented take of a sturgeon.

Fish passage alternatives could include recommendations to modify or remove one, two, or all three of the locks and dams on the Cape Fear River for mitigation purposes. This is a potential recommendation since the locks and dams were justified on commercial traffic, but there is no documented commercial traffic using the locks and dams since 1997, and no change in the commercial traffic regime is expected. Because of the lack of traffic, the navigation channel upstream of Lock and Dam # 1 has not been maintained since 1992.

Before any such removal or modification can be recommended, the potential effects on water supply safe yield and associated reliability of the water users upstream of the locks and dams must be determined. And other factors must be evaluated such as future navigation, recreational and commercial fishing, public use of the parks associated with the locks and dams, wetland impacts identified, etc. If a recommendation is made to modify the locks and dams that would preclude future commercial navigation, congressional approval would be required to deauthorize the project before the recommendation could be implemented.

Tables 3.5.1a - 3.5.1c are decision matrices used to assist the PDT in selecting the best fish passage alternative at each of the three locks and dams. These

matrixes were developed during four PDT/IPT (interagency project team) meetings in April and May 2007. The fish passage alternatives are listed along the top of each matrix and evaluation factors are listed along the left side. The rating is an ordinal scale from 1 to 10: 1 is the worst and 10 is the best. The scale is very common and widely accepted. Ordinal only indicates there is an order: 10 is better than 9, 9 is better than 8, etc. However ordinal does not imply a multiple or percentage of difference. Ten is not twice as good as 5, or 6 is not 10% better than 5.

On the ordinal scale used, 10 would be for the alternative that has the least cost or best fish passage, and 1 would be for the alternative with the most cost or lowest potential for fish passage. All the other alternatives were distributed proportionally to the nearest whole number between the two extremes on the 1 to 10 scale. A rating of 5 would be a cost or fish passage potential mid-way between the two extremes. Costs are total costs: fish passage construction and all other costs including impact to water supply intake facilities.

Most evaluation factors have a weight of one, but the first two factors have a weight of two (2) since they are the most important considerations. If a column is shaded, that alternative was unacceptable because it would not adequately pass fish (fish passage factor less than 8). The "fish passage factor" was a consensus of the project delivery team (PDT) which consists of the Corps and federal and state resource agencies. Basically it is a risk factor. A 10 would indicate that all anadromous fish attempting to pass could (e.g. dam removed) and a low factor like 2 would indicated that probably not enough fish would pass to improve current population conditions. Thus an 8-10 should greatly exceed current fish passage and population conditions and a 3 may only slightly improve current conditions. For example the fish passage alternative of a rock ramp in the lock chamber was given a fish passage factor of “2” because it is believed fish could not find it better than current locking procedure since the entrance is the same.

Also a column is shaded if it would have a severe impact on water supply (rating value less than 3). For example, the fish passage alternatives of removing and lowering the dams (Lock and Dam Nos. 1 & 3) were given a “Potential Effects on Water Supply” rating of “1” because of the major impact on water supply infrastructure and loss of storage.

In these matrixes, the fish passage alternative with the highest total is the best. Summarizing the discussion below, the best alternative for Lock and Dams # 1&3 is a rock ramp downstream of the dam. For # 2, the best fish passage alternative is either dam removal or a rock ramp downstream of the dam. Before a final decision can be made more information is needed regarding dam removal, such as potential impact to a planned water supply intake upstream of the dam, and stability issues at Lock and Dam # 3 due to a lowered pool downstream.

None of the alternatives discussed below are anticipated to have high maintenance costs.

Table 3.5.1a Decision Matrix for Fish Passage at Lock and Dam # 1, Cape Fear River, NC

	Fish Passage Alternatives											
		Lower Dams		Rock Ramp								
Evaluation Factors	Remove Dams	Rock Ramp with Reduced Water Supply Capacity	No Fish Passage Needed[2]	Downstream of Dam Only	Downstream of Dam & Lock	Upstream of Dam[7]	Lock Only	Diversion Channel	Diversion Channel & Rock Ramp in Lock	Natural-like Bypass	Nature-Like Bypass & Rock Ramp in Lock	Improve Locking[1]
Pass Sturgeon & other Anadromous Fish[3] (Fish Passage Factor, 10 best passage, 1 worst)	10	8	9	8	7	8	2	3	5	3	5	
Subtotal (Weight X 2)	**20**	**16**	**18**	**16**	**14**	**16**	**4**	**6**	**10**	**6**	**10**	
Potential Effects on Water Supply Safe Yield and Associated Reliability[4] (10 least impact, 1 worst)	1	2	1	10	10	10	10	8	8	8	8	
Subtotal (Weight X 2)	**2**	**4**	**2**	**20**	**20**	**20**	**20**	**16**	**16**	**16**	**16**	
Construction and O&M Cost[5] (10 lowest, 1 highest)	1	4	1	5	3	1	10	9	9	8	8	
Commercial & Recreational Navigation Impacted (10 least impact, 1 worst)	1	4	1	10	5	5	5	10	5	10	5	
Real Estate Acquisition (5 least impact, 1 worst)	5	5	5	5	5	4	5	4	4	1	1	
Deauthorization Required	Yes	Yes	Yes	No	Yes	Yes	Yes	No	Yes	No	Yes	No
Total[6]	**29**	**33**	**27**	**56**	**47**	**46**	**44**	**45**	**44**	**41**	**40**	

1. Not feasible since the 2000 BO from NMFS required better fish passage than locking.

2. Dam lowered enough so that anadromous fish could pass over the top during spring flows, and a minimal pool would exist during drought conditions.

3. Includes increased recreational and commercial fishing benefits. If the fish passage factor is less than 8, this form of fish passage is not likely to receive concurrence from NMFS and other resource agencies. See section 3.5.2.2 for explanation of the fish passage factor.

4. Includes loss of storage. If value is less than 3, this form of fish passage is likely to have a major impact and would be unacceptable to water supply users.

5. Costs including impacts to water supply infrastructure. The ranking of alternatiaves are inversely proportional to costs. Costs range from about $1 million (10) to $18 million (1).

6. Alternative with the highest total is the best choice.

7. The dam for the upstream alternative would have the save crest elevation as the existing dam. See section 3.5.3.5 for a description of this alternative.

Unacceptable alternative that will either not adequately pass anadromous fish, more expensive than a similar alternative, preclude access by the navy to Lock and Dam #2, or have severe an impact on water supply users.

Table 3.5.1b Decision Matrix for Fish Passage at Lock and Dam # 2, Cape Fear River, NC

	Fish Passage Alternatives											
		Lower Dams		Rock Ramp								
Evaluation Factors	Remove Dams	Rock Ramp with Reduced Water Supply Capacity	No Fish Passage Needed[2]	Downstream of Dam Only	Downstream of Dam & Lock	Upstream of Dam[8]	Lock Only	Diversion Channel	Diversion Channel & Rock Ramp in Lock	Natural-like Bypass	Nature-Like Bypass & Rock Ramp in Lock	Improve Locking[1]
Pass Sturgeon & other Anadromous Fish[3] (Fish Passage Factor, 10 best passage, 1 worst)	10	8	9	8	7	8	2	3	5	3	5	
Subtotal (Weight X 2)	**20**	**16**	**18**	**16**	**14**	**16**	**4**	**6**	**10**	**6**	**10**	
Potential Effects on Water Supply Safe Yield and Associated Reliability[4] (10 least impact, 1 worst)	5	9	8	10	10	10	10	8	8	8	8	
Subtotal (Weight X 2)	**10**	**18**	**16**	**20**	**20**	**20**	**20**	**16**	**16**	**16**	**16**	
Construction and O&M Cost[5] (10 lowest, 1 highest)	8	10	10	5	2	1	10	6	5	0	0	
Commercial & Recreational Navigation Impacted (10 least impact, 1 worst)	5	5	1	10	5	5	5	10	5	10	5	
Funding potential including public and private[6] (10 best potential, 1 worst potential)	8	8	7	6	5	7	2	5	6	4	5	
Deauthorization Required	Yes	Yes	Yes	No	Yes	Yes	Yes	No	Yes	No	Yes	No
Total[7]	**51**	**57**	**52**	**57**	**46**	**49**	**41**	**43**	**42**	**36**	**36**	

1. Not feasible since the 2000 BO from NMFS required better fish passage than locking.

2. Dam lowered enough so that anadromous fish could pass over the top during spring flows, and a minimal pool would exist during drought conditions. This would preclude full use of the site by the Navy.

3. Includes increased recreational and commercial fishing benefits. If the fish passage factor is less than 8, this form of fish passage is not likely to receive concurrence from NMFS and other resource agencies. See section 3.5.2.2 for explanation of the fish passage factor.

4. Includes loss of storage

5. The ranking of alternatiaves are inversely proportional to costs. Costs range from about $1 million (10) to $19 million (1).

6. Funding potential would be the greatest for projects that provide the maximum potential to protect water supply and greatest potential to pass anadromous fish.

7. Alternative with the highest total is the best choice.

8. The dam for the upstream alternative would have the save crest elevation as the existing dam. See section 3.5.3.5 for a description of this alternative.

Unacceptable alternative that will either not adequately pass anadromous fish, more expensive than a similar alternative, have severe an impact on water supply users, preclude the navy from passing the site, or nature-like bypass not feasible since floodplain is absent.

Table 3.5.1c Decision Matrix for Fish Passage at Lock and Dam # 3, Cape Fear River, NC

	Fish Passage Alternatives														
		Lower Dams			Rock Ramp										
Evaluation Factors	Remove Dams	Rock Ramp with Reduced Water Supply Capacity	Rock Ramp with Reduced Water Supply Capacity, L&D#2 Out	No Fish Passage Needed[2]	Downstream of Dam Only	Downstream of Dam & Lock	Upstream of Dam[8]	L&D #2 Out Dam Only	L&D #2 Out Dam & Lock	Lock Only	Diversion Channel	Diversion Channel & Rock Ramp in Lock	Natural-like Bypass	Nature-Like Bypass & Rock Ramp in Lock	Improve Locking[1]
Pass Sturgeon & other Anadromous Fish[3] (Fish Passage Factor, 10 best passage, 1 worst)	10	8	8	9	8	7		8	7	1	3	5	3	5	
Subtotal (Weight X 2)	**20**	**16**	**16**	**18**	**16**	**14**		**16**	**14**	**2**	**6**	**10**	**6**	**10**	
Potential Effects on Water Supply Safe Yield and Associated Reliability[4] (10 least impact, 1 worst)	1	2	2	1	10	10		10	10	10	8	8	8	8	
Subtotal (Weight X 2)	**2**	**4**	**4**	**2**	**20**	**20**		**20**	**20**	**20**	**16**	**16**	**16**	**16**	
Construction and O&M Cost[5] (10 lowest, 1 highest)	8	10	7	9	8	8		8	8	10	2	1	0	0	
Commercial & Recreational Navigation Impacted (10 least impact, 1 worst)	5	5	3	1	10	5		5	5	5	10	5	10	5	
Funding potential including public and private[6] (10 best potential, 1 worst potential)	5	6	7	5	10	5		10	5	2	2	3	2	3	
Deauthorization Required	Yes	Yes	Yes	Yes	No	Yes	Yes	Yes	Yes	Yes	No	Yes	No	Yes	No
Total[7]	**40**	**41**	**37**	**35**	**64**	**52**		**59**	**52**	**39**	**36**	**35**	**34**	**34**	

1. Not feasible since the 2000 BO from NMFS required better fish passage than locking.

2. Dam lowered enough so that anadromous fish could pass over the top during spring flows, and a minimal pool would exist during drought conditions.

3. Includes increased recreational and commercial fishing benefits. If fish passage factor is less than 8, this form of fish passage is not likely to receive concurrence from NMFS and other resource agencies. See section 3.5.2.2 for explanation of the fish passage factor.

4. Includes loss of storage

5. Costs including impacts to water supply infrastructure. The ranking of alternatiaves are inversely proportional to costs. Costs range from about $1 million (10) to $16 million (1).

6. Funding potential would be the greatest for projects that provide the maximum potential to protect water supply and greatest potential to pass anadromous fish.

7. Alternative with the highest total is the best choice.

8. The dam for the upstream alternative was not evaluated in detail because there is no scour hole downstream of Lock and Dam # 3.

Unacceptable alternative that will either not adequately pass anadromous fish, more expensive than a similar alternative, have severe an impact on water supply users, or nature-like bypass not feasible since floodplain is absent.

3.5.2 Fish Passage Alternatives:

Deauthorization NOT REQUIRED. The locks and dams were authorized based on commercial navigation using the locks. The following alternatives do not affect potential commercial or recreational navigation because they do not involve lowering the navigation pool behind the dam or blocking the lock. Therefore deauthorization of the locks and dams would not be required in order to implement one of these alternatives.

Figure 3.5.2.1a Lock gate in operation

3.5.2.1 Improve locking at all three locks and dams. This is the fish passage method currently being conducted at all three locks and dams. Locking is being conducted each spring from about mid-March through mid-May which includes the peak of the anadromous fish migration period. The locking starts at Lock and Dam No. 1 first then, moving upstream, a week later at Lock and Dam No. 2, followed by a week later at Lock and Dam No. 3. The locking stops in the same sequence with a week lag between each facility. Monitoring performed using sonic tags indicate that about 50 percent of the American shad (*Alosa sapidissima*) and striped bass (*Morone saxatilis)* approaching the dams pass upstream (Moser 2000; CZR 2003, 2003, 2004). However if 50 percent of the fish pass the first lock and only 50 percent of those fish pass the second lock and only 50 percent of those fish pass the third lock, then only about 12.5 percent (0.5 x 0.5 x 0.5 =0.125) of the fish make it upstream of Lock and Dam No. 3.

Fish passage using locking has been previously maximized via former studies (Hall 1998, Moser 2000), and by removing shoals each spring that form in front of the lock chamber. Removing the shoals facilitates the anadromous fish in finding the lock chamber. Also since the 2000 NMFS BO required construction of a fish passage structure at Lock and Dam No. 1, locking by itself is not a viable option.

3.5.2.1b Fish passage locking at Lock and Dam No. 3

3.5.2.2 Nature-like fish bypass at Lock and Dam No. 1. This alternative was selected as the proposed fish passage method in the Environmental Assessment /Finding Of No Significant Impact (EA/FONSI) for fish passage at Lock and Dam # 1 (USACE 2002, 2003). This EA/FONSI was prepared in response to the August 2000 NMFS BO. This project was not constructed due to funding constraints and real estate acquisition issues.

The nature-like fish alternative would involve construction of a 3,800' fish passage channel around Lock & Dam #1 in the floodplain along the east side of the Cape Fear River (figure 15A, Appendix 8). This length of construction is required to have the velocities low enough (2-5 feet/sec) so that anadromous fish can successfully pass. As with any bypass channel, there are two other key factors (other than velocity) that dictate if the fish find and use the fishway. First, the fish entrance needs to be located as close to the dam as possible without compromising the integrity of the structure. Second, there needs to be sufficient attraction flow so that the fish respond to it. Most studies indicate that at least 5 percent of the river flow needs to be routed through the bypass. To assure success, the bypass was designed to pass 10 percent of the river flow.

When considering the fish passage alternatives under the EA/FONSI, modifying or removing the lock and dam was not considered since that would have effectively deauthorized the project purpose by eliminating commercial navigation at Lock & Dam No. 1. By affecting the pool level at Lock and Dam No. 1, this would have also resulted in eliminating the commercial navigation capability at the other two upstream locks and dams.

A nature-like bypass is only feasible at Lock and Dam No. 1 because it is the only location that has an extensive floodplain next to the dam. The other projects have extensive high ground adjacent to the projects which would make excavation costs of a long channel prohibitive.

The bottom of the nature-like bypass channel exit would be about 3 feet lower than the dam. This is to assure water would be present in the channel at all flow conditions. The water users at Lock and Dam No. 1(City of Wilmington and the Lower Cape Fear Water and Sewer Authority) were concerned that during low river flow conditions the channel might lower the pool level and thereby impact water supply yield. To preclude this potential, the exit was designed to be blocked by stop logs. This should not preclude anadromous fish passage, since such low flow conditions normally occur in the summer or fall, not during the spring when anadromous fish are migrating on the spawning run.

This alternative was eliminated because it has a fish passage factor of less than 8 (3).

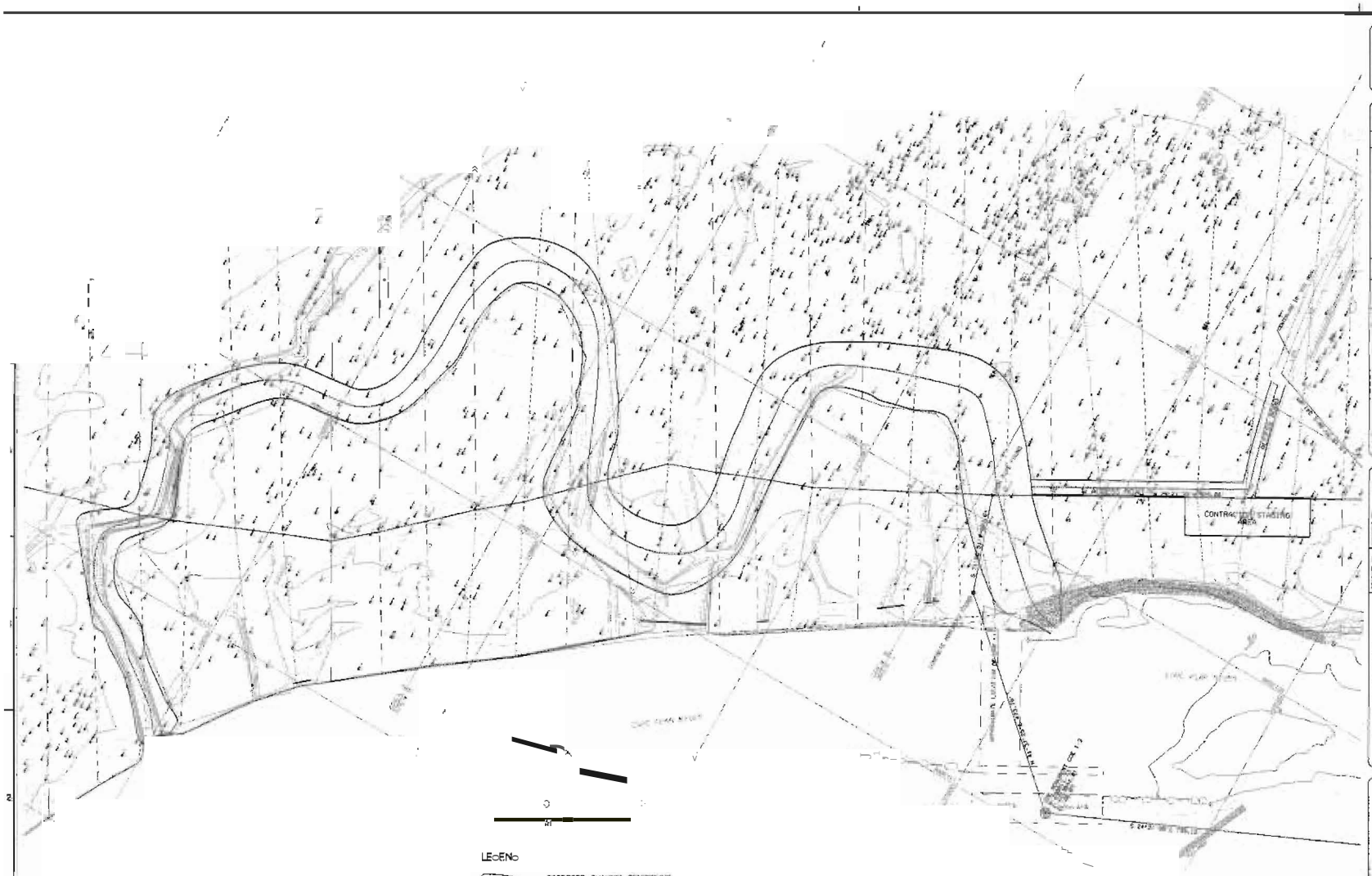


FIGURE 15A

REFERENCE: SOURCE AND BORROW EASEMENT SURVEY AT LOCK AND DAM No.1, BLADEN COUNTY, NC, BY McKIM & CREED ENGINEERS
TOPOGRAPHIC SURVEY AT LOCK AND DAM No.1, BLADEN COUNTY, NC, BY McKIM & CREED ENGINEERS, P.A. JULY 12, 2001.
SURVEY OF SRCAL MILE No.38.8 NEAR LOCK AND DAM No.1, BLADEN COUNTY, NC, BY W.W.W. AUGUST 16, 2000.

3.5.2.3 Diversion rock rapids at all three locks and dams. This fish passage would be designed similar to the proposed fish diversion channel at new Savannah Bluff Lock and Dam near Augusta, Georgia (USACE, Savannah 2002). The diversion rock ramp alternative would be slightly different at each lock and dam due to the different topography at each site (Figures 15-18, Appendix 8). At Locks and Dams 1 and 2, they would be most similar because they both would be on the east side of the river (opposite side from the lock). A diversion channel at Lock and Dam No, 3 would need to be located on the lock (west) side because of the high bluff on the east side. This would also be the most expensive (Table 3.4b) diversion channel because of its length around the lock chamber, volume of material to be excavated, and bridge required to provide access to the lock and dam.

As with the nature-like bypass, these diversion channels would divert about 10 percent of the river flow to provide fish attraction flow, and the fish entrance would be as close to the dam as possible so that the fish are more likely to find the entrance. The slope of the channel is 5 percent or flatter and rock placement is designed so that bypass would appear as a natural rapids to the migrating fish. Large rocks (generally 3-4 feet in diameter) are placed in parallel veins about 20 feet apart, which results in veins about ever foot drop in elevation. The surface rocks are sized such that they will not be moved by water velocity or debris such as downed trees. The rocks are placed so that they slow the water velocity and provide “flow shadows” which allows the fish to rest. Generally the rocks in the veins are contiguous, but occasionally there are gaps in the veins to allow the fish to pass in low flow conditions. However, most of the time during the spring migration period, the rocks are submerged and rapids just appears to be an area of rough water. The center of the rapids is about 1 foot lower than the sides so that during low flow the water is concentrated in the middle so that the fish can still pass.

As with the nature-like bypass channel, the exit of the diversion channel would be about 3 feet lower than the dam. This is to assure water would be present in the channel at all flow conditions. To preclude the water users concerns of lowering the pool level during low flow conditions, the exit will be designed to be blocked by stop logs or similar structure if necessary. This should not preclude anadromous fish passage, since such low flow conditions normally occur in the summer or fall, not during the spring when anadromous fish are migrating on the spawning run.

This alternative was eliminated because it has a fish passage factor of less than 8 (3).

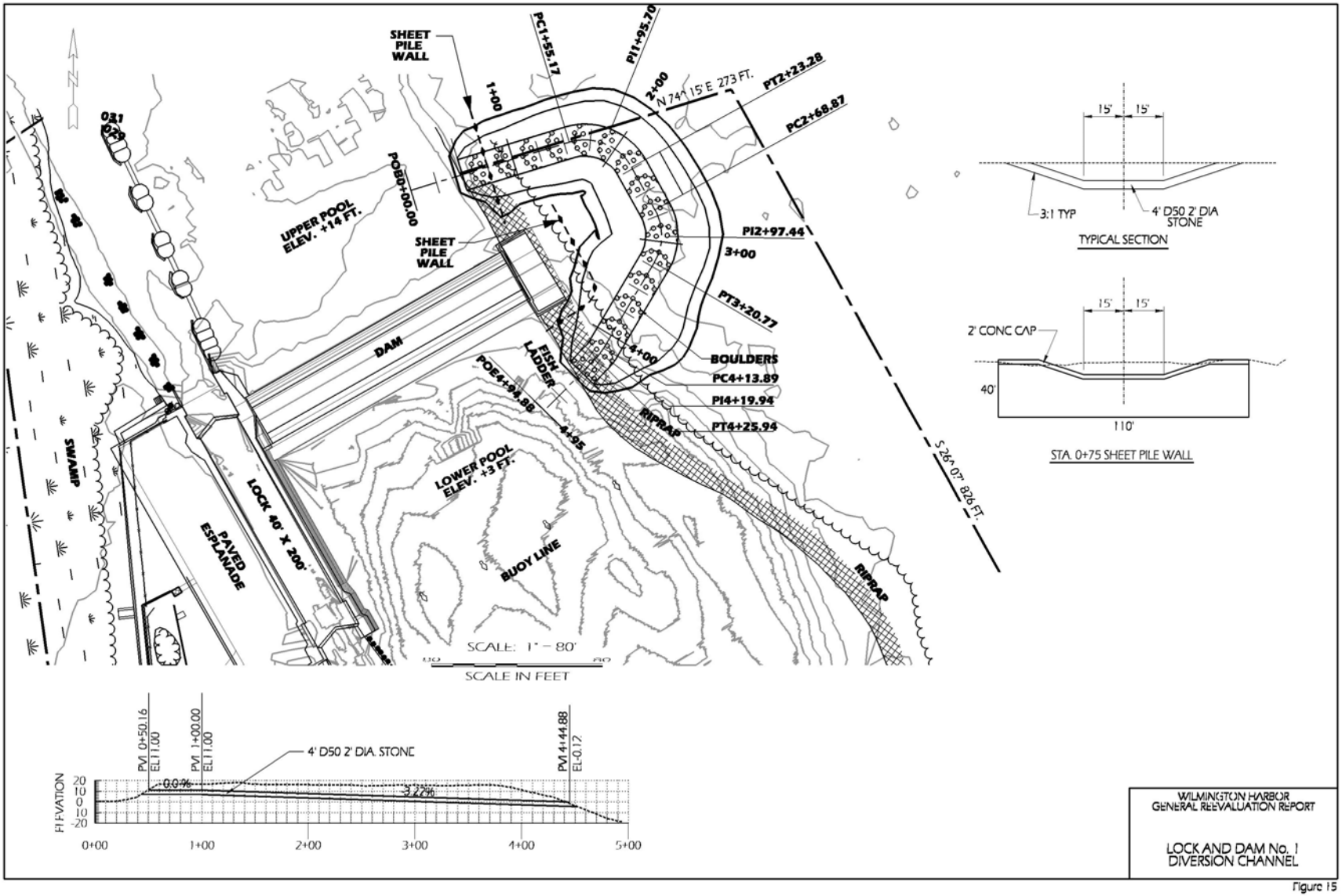


Figure 3.5.2.3 Diversion Rock Rapids (Lock and Dam No. 1)

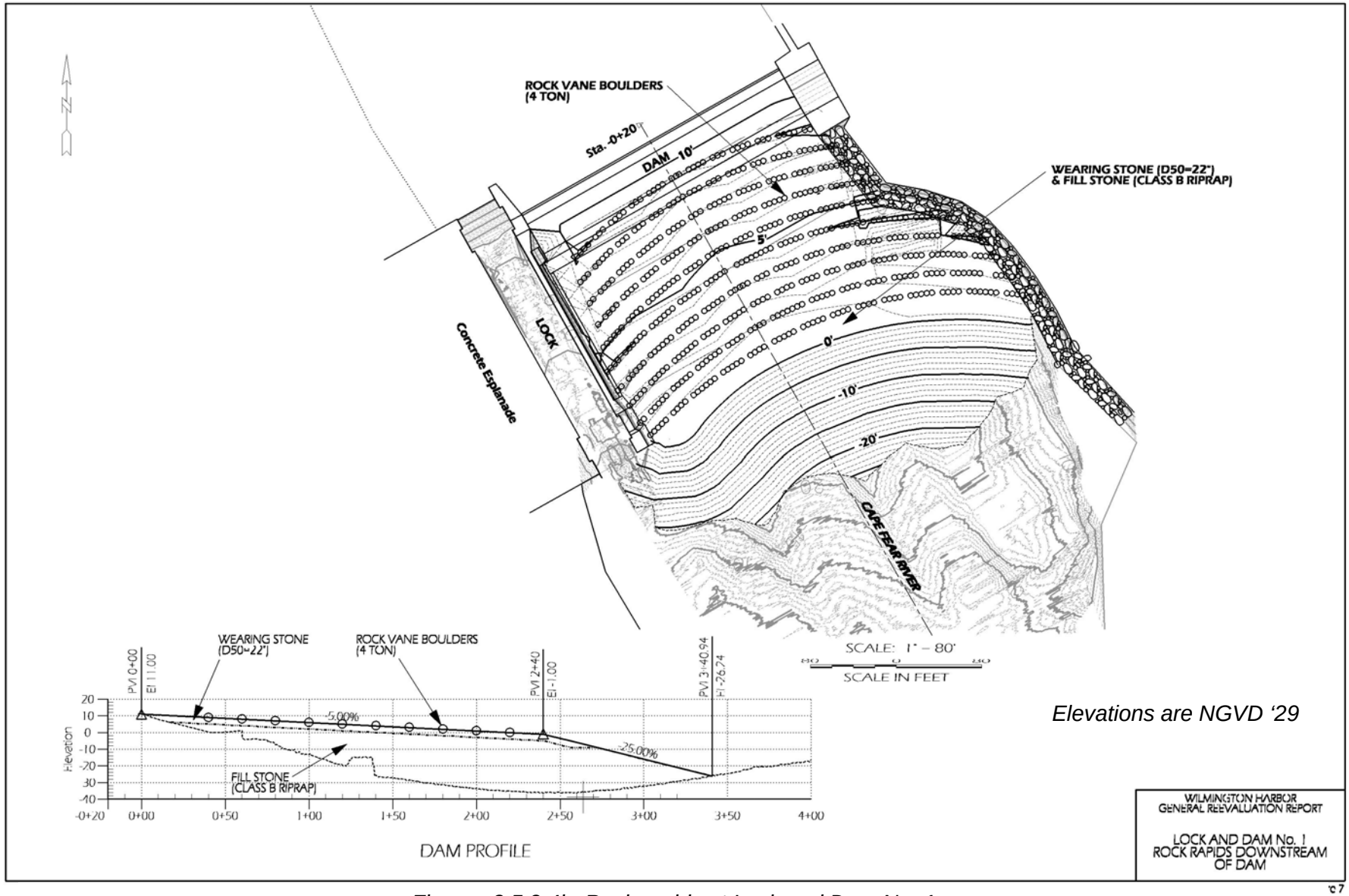
3.5.2.4 Rock rapids across entire downstream face of dam at all three locks and dams. This fish passage method has been used on several dams in the upper Midwest and was developed by Luther Aadland of the Minnesota Department of Natural Resources. The Corps has also constructed rock ramps. An example is the Riverside Dam on the Red River in Fargo, ND. This structure has been shown to pass all types of fish including sturgeon.

Figures 3.5.2.4a Riverside Dam, Red River: Fargo, ND

Construction is similar to the rock rapids in a diversion channel except the rapids are constructed along the entire downstream face of the dam (Figures 7, 10, and 13, Appendix 8). The slope of the rapids is generally at 5 percent or flatter and rock placement is designed so that bypass would appear as a natural rapids to the migrating fish. Large rocks (generally 3-4 feet in diameter) are placed in parallel veins about 20 feet apart, which is results in veins about every foot drop in elevation. The surface rocks are sized such that they will not be moved by water velocity or debris such as downed trees. The rocks are placed so that they slow the water velocity and provide “flow shadows” which allows the fish to rest. Generally the rocks in the veins are contiguous, but occasionally there are gaps in the veins to allow the fish to pass in low flow conditions. However, most of the time during the spring migration period, the rocks are submerged and rapids just appear to be an area of rough water. The center of the rapids is about 1 foot lower than the sides so that during low flow the water is concentrated in the middle so that the fish can still pass. Another advantage of the lower center is that the flow is concentrated toward the middle of the stream which helps preclude erosion of the adjacent shorelines.

There are two problems with this alternative at the Cape Fear River Locks and Dams. First, below Locks and Dams # 1 and 2 there are scour holes (about 40 feet deep) caused by water plunging over the dams. Filling the scour holes is a significant portion of the cost of this alternative. There is not a scour hole at Lock and Dam No. 3 because a rock covered timber mat was placed downstream of the dam during construction. This mat is still intact, but no such mat was constructed at the other two dams. Second, locks have not been present when

rock rapids were constructed at other dams. If rocks were placed adjacent to the lock wall up to the crest on the dam, the lock wall may experience instability. Therefore, the PDT recommends a gap in the rapids be left adjacent to the lock wall (Figures 7, 10, 13, Appendix 8). This should not compromise fish passage. This fish passage alternative is viable at all three locks and dams since it has a fish passage factor of 8.

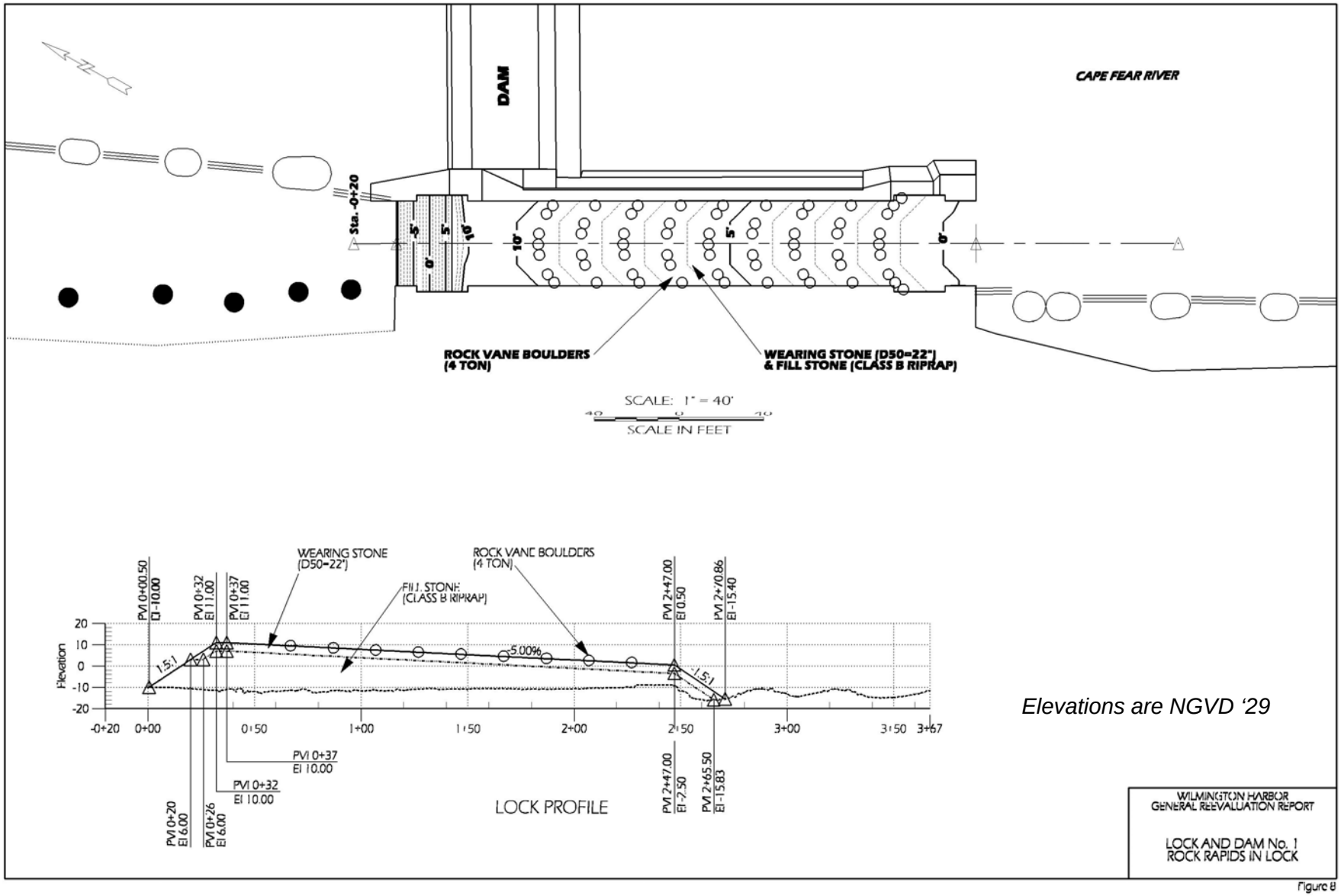


Figures 3.5.2.4b Rock rapids at Lock and Dam No. 1

3.5.3 Fish Passage Alternatives at All Three Locks and Dams: Deauthorization REQUIRED.

As indicated above, the locks and dams were authorized based on commercial navigation using the locks. The following alternatives affect potential commercial and recreational navigation because they either involve lowering the navigation pool behind the dam and/or blocking the lock. Therefore deauthorization of the locks and dams would be required in order to implement these alternatives.

3.5.3.1 Rock rapids in lock chamber(s). This alternative is very similar to diversion rock rapids, but instead of placing the rapids in an excavated diversion channel, the rapids are place in the lock chamber (Figures 8, 11, 14, Appendix 8). The design features are generally the same including sizing the rocks, slope, attraction flow, and placement of the rock veins. However there are two disadvantages of this alternative compared to the diversion channel. First the fish entrance (lock opening) is 200 feet downstream of the dam (300 feet for Lock and Dam No. 3), verses adjacent to the dam for the diversion channel. Thus fish would have more difficulty finding the rapids. Second with a rock ramp in the lock chamber, commercial and recreational traffic would be precluded. Also, this alternative has only a fish passage factor of 2. For these reasons, this alternative was dropped from further consideration.



Figures 3.5.3.1 Rock rapids in lock (Lock and Dam No. 1)

3.5.3.2 Lower the dam(s) with reduced water supply capacity. This alternative would only be applicable to Lock and Dam 1 and 3 since there is currently no water supply intake upstream of Lock and Dam No. 2. A water supply intake is proposed near Tar Heel, NC by the Lower Cape Fear Water and Sewer Authority, but the Corps has requested the Authority to evaluate placing the intake low enough so that if Lock and Dam No. 2 was lowered or removed, the intake would not be affected. Locking may not be feasible if any of the dams are lowered.

Neither Lock or Dam No. 1 or 3 can be lowered much without affecting the existing intakes during low flow conditions. Lock and Dam No. 1 can only be lowered about 1-foot before affecting the City of Wilmington's intake and Lock and Dam No. 3 can only be lowered about 2 feet before affecting Fayetteville's intakes. This would result in a loss of dependable storage during drought conditions. Replacement of this storage with an offline reservoir or other form of storage would be prohibitively expensive. In addition to reducing dependable storage, the lower elevation would reduce the pumping capability of the facilities.

These issues would also apply to the alternative of lowering Lock and Dam No. 3 with taking out Lock and Dam No. 2. The only advantage this alternative has is better fish passage at No. 2. The impacts on water users upstream of No. 3 are the same.

The indicated lowering would not result in appreciable savings for a rock rapids constructed at this elevation versus the existing crest elevation. The length of a diversion channel or nature-like bypass would be shorter because of lowering of the head difference between upstream and downstream of the dam. However this cost savings would be offset by having to excavate these channels deeper to reach the elevation of the lowered upstream pool. For the reasons given above, this alternative was eliminated from further consideration.

3.5.3.3 Lower the dam(s) such that no fish passage is needed. This alternative involves lowering the dam to the point that anadromous fish could pass over the dam during normal spring river discharges, but during drought conditions a low pool would be backed up behind the lowered dam. This alternative would result in lowering Lock and Dam No. 1 by about 9 feet and the other two locks and dams by about 6 feet each. Locking may not be feasible with this alternative.

The main disadvantages of this alternative are to the water users. The water users would have a minimal pool elevation, but lose basically all of their existing drought condition storage. In addition, the water users would need to bear a considerable capital expense to modify their facilities. This would be about $14.6 million total at Lock and Dam No. 1 for the City of Wilmington and Lower Cape Fear Water and Sewer Authority combined and about $1.6 million at Lock and Dam No. 3 for the City of Fayetteville (USACE Mobile 2007). However, the City of Fayetteville indicated that their cost would be higher ($8-9 million)

(Malcolm Pirnie 2007). The intakes would be located closer to the river bottom which could increase sediment problems in the intake structure, plus the water supply facilities may need to be taken offline for a period of time while the modifications are being implemented.

A water supply intake is proposed near Tar Heel, NC by the Lower Cape Fear Water and Sewer Authority, but the Corps has requested the Authority to evaluate placing the intake low enough so that if Lock and Dam No. 2 was lowered or removed, the intake would not be affected.

The Navy/Marine Corps has been using Lock and Dam No. 2 for training for about 3 1/2 years and plan to continue the training operations for the foreseeable future. They indicate the Cape Fear River along with the locks and dams provide an ideal riverine training area for Iraq and other mission areas around the world. When the Iraq mission is complete, that will not diminish the need for riverine training. To date the Navy/Marine Corps have deployed about 100 troops at Lock and Dam No. 2 for about 2 weeks 4 times per year. This will probably increase to 8-10 times per year in the near future. Continued locking operations at Lock and Dam No. 1 are essential to access Lock and Dam No. 2, but leaving Lock and Dam No. 2 in place is desired but not essential. However vessels need to pass up and downstream on Lock and Dam No. 2, so the lock needs to remain operational or the dam removed entirely.

For these reasons, this alternative was eliminated from further consideration.

3.5.3.4 Remove the dam(s). Removing all portions of the locks and dams would be the best alternative for fish passage since the river would be returned to its natural condition with no obstructions to passage during any flow conditions. However, it would have the greatest adverse impact on the water users. This alternative would have the impacts of the previous alternative (lower the dam for fish passage with minimal pool), but there would be no guaranteed minimal pool because no dam related structure would be left in the river.

For these reasons, this alternative was eliminated from further consideration at Lock and Dams 1 & 3. It is still feasible at Lock and Dam # 2 because currently there is not an intake upstream of that dam. However if Lock and Dam No. 2 was removed, then the upstream pool would be lowered up to Lock and Dam No. 3. This would increase the costs of a rock ramp or other alternative at Lock and Dam No.3 (Table 3.4b).

If Lock and Dam #2 is removed with #1 still in place, #1 would back up water to the vicinity of the proposed intake near Tar Heel. This would provide about 5-6 feet of water depth in the river at extreme low flow conditions (Appendix 3.8.3).

3.5.3.5 Place rock rapids upstream of Lock and Dams Nos. 1 & 2. Most of the expense of constructing a rock ramp at Locks and Dams 1 & 2 is the cost of filling the scour holes downstream of the dams. There is no scour hole upstream of the dams. A new steel or concrete sheet pile dam (at the same elevation of

the existing dam) could be constructed about 300 feet upstream of the existing dams and a rock ramp placed on it. Then the old dam would be demolished and the shoreline restored, but the park and associated features including the boat ramp could be preserved.

However as Table 3.4b indicates, this would be more expensive than placing a rock ramp on the old dam, and navigation would be precluded because the dam would span the entire river. Therefore, this alternative was eliminated from further consideration.

This alternative was not investigated for Lock and Dam #3 since there is not a scour hole downstream.

3.5.4 Combination of Fish Passage Alternatives.

3.5.4.1 Rock rapids across the entire downstream face of dam and in the lock chamber. This would preclude use of the lock by vessels and require deauthorization of the project. Also this would make the rock ramp about 40 feet wider (width of the lock chamber). Under normal flow conditions this would not be a problem for fish passage, but during low flow conditions it would cause the depth of water going over the dam to be slightly lower which could impact fish passage. Also this combination alternative would be more expensive since it would require more rock (Table 3.4b). As indicated in paragraph 3.5.2.4, there is only one advantage of this alternative. If a rock ramp is placed across the entire downstream face of the dam, rock would be placed adjacent to the lock wall. This may cause unacceptable stress on the lock wall which would be offset by placing a rock ramp in the lock chamber.

Because of increased cost and possible reduced potential fish passage compared to a rock rapids across the entire downstream face of the dam, this alternative was eliminated from further consideration.

3.5.4.2 Diversion channel and rock ramp in the lock chamber. This would also preclude use of the lock by vessels and require deauthorization of the project. If 10 percent of the river flow was passed through each fish passage structure (total 20 percent), the combination of the two structures would probably be better than just one. This combination would also be cheaper than a rock ramp, but also would probably not be as efficient in passing fish (i.e. rock ramp uses 100 percent of the river flow and the fish don't need to find an entrance).

This alternative was eliminated because it has a fish passage factor (5) less than 8.

3.5.4.3 Nature-like bypass and rock ramp in the lock chamber. Nature-like bypass and rock ramp in the lock chamber: The assessment of this alternative is the same as for "Diversion channel and rock ramp in the lock chamber" except that nature-like combination could only be used at Lock and Dam No. 1 because it is the only project that has a floodplain next to the dam.

3.6 Alternatives not considered in detail. There are many other fish passage alternatives that are not evaluated in detail for this project. They are either too expensive and require high maintenance costs (e.g. fish lifts), or would be quickly clogged by the abundant woody debris in the river (e.g. slot or denil pass).

3.7 General environmental considerations. For the environmental conditions below, the discussion will generally be divided into 2 parts: One for the Locks and Dams, and the other for the Relocated Turning Basin. They are separate ecosystems: the locks and dams system is in fresh water dominated by river flow, and the relocated turning basin system is in brackish water dominated by lunar tides.

3.7.1 Estuarine and Riverine Resources.

3.7.1.1 Locks and Dams. The Cape Fear River below Lock and Dam No. 1 is essentially fresh water, but because the river is still tidally influenced in the area, there is the potential for a diverse assemblage of fishes to occur. Fishery resources in this part of the Cape Fear River can be classified into three categories: permanent resident species, anadromous species, and estuarine dependent species. Upstream of Lock and Dam No. 1, there are essentially only permanent resident species and anadromous species. Anadromous species are those that spend most of their life in the estuarine or marine environment, but migrate upstream into freshwater rivers to spawn.

Resident fishes include members of the herring, minnow, sucker, catfish, sunfish, and perch families. Both gizzard shad (*Dorosoma cepedianum*) and threadfin shad (*D. petenense*) commonly occur in the river. The minnows include common carp (*Cyprinus carpio*), whitefin shiner (*Cyprinella nivea*), and spottail shiner (*Notropis hudsonius*). Several species of suckers are present but they are uncommon. While not many species are present, the catfishes dominate the biomass of fishes. Two large introduced species, blue catfish (*Ictalurus furcatus)* and flathead catfish (*Pylodictis olivaris*) predominate. At least eight species of sunfish occur in this portion of the Cape Fear River. Bluegill *(Lepomis macrochirus*), redear sunfish (*L. microlophus*), and largemouth bass (*Micropterus salmoides*) are the most common. A recreational fishery exists here for sunfishes and catfishes.

At least six species of anadromous/catadromous fishes occur in this part of the Cape Fear River. American shad (*Alosa sapidissima*) is the dominant member of this group. Spawning runs occur each spring in the Cape Fear River and a significant recreational and commercial fishery has developed below the dam. Blueback herring (*Alosa aestivalis*) and hickory shad (*Alosa mediocris*) also make spawning runs but they occur in much fewer numbers than American shad. A resident population of the striped bass (*Morone saxatilis*) is present and may migrate as far upstream as Buckhorn Dam. Its population is low and it has to compete with introduced hybrid bass, which have escaped from Jordan Lake.

Both the Atlantic sturgeon (*Acipenser oxyrhynchus*) and the endangered shortnose sturgeon (*Acipenser brevirostrum*) occur in the Cape Fear River and make spawning runs up tributaries of the Cape Fear (Black and Northeast Cape Fear rivers) and possibly the mainstem Cape Fear River also. The endangered shortnose sturgeon is discussed in more detail in appendix B. Elvers of the catadromous American eel (*Anguilla rostrata*) ascend the river each year to spend their early lives in the freshwater tributaries.

A number of estuarine-dependent fishes are known to occur near Lock and Dam No. 1. Dominant in this group are striped mullet (*Mugil cephalus*), freshwater goby (*Gobionellus shufeldti*), and hogchoker (*Trinectes maculatus*). Others noted from the area are Atlantic menhaden (*Brevoortia tyrannus*), ladyfish (*Elops saurus*), Atlantic needlefish (*Strongylura marina*), and southern flounder (*Paralichthys lethostigma*). Others likely occur there since they have been found further downstream. These include spot (*Leisotomus xanthurus*), Atlantic croaker (*Micropogonias undulatus*), and an invertebrate, the blue crab (*Callinecetes sapidus*).

3.7.1.2 Relocated turning basin and PNA mitigation sites. The most abundant fishes of these waters are the estuarine dependent species which inhabit the estuary as larvae and the ocean as juveniles or adults. This group includes species which spawn offshore, such as the Atlantic croaker (*Micropogon undulatus*), spot (*Leiostomus xanthurus*), Atlantic menhaden (*Brevoortia tyrannus*), star drum (*Stellifer lanceolatus*), southern kingfish (*Menticirrhus americanus*), flounders (*Paralichthys* spp.), mullets (*Mugil* spp.), anchovies (*Anchoa* spp.), blue crab (*Callinectes sapidus*), and penaeid shrimp (*Penaeus* spp.), as well as species which spawn in the estuary, such as red drum (*Sciaenops ocellatus*) and weakfish (*Cynoscion regalis*).

Anadromous species such as blueback herring (*Alosa aestivalis*), American shad *(Alosa sapidissima*), hickory shad (*Alosa mediocris*), alewife (*Alosa pseudoharengus*), striped bass (*Morone saxatilis*), and Atlantic sturgeon (*Acipenser oxyrhynchus*) pass through the Cape Fear estuary en route to upper river spawning and nursery areas (Walburg and Nichols, 1967; Nichols and Louder, 1970). Anadromous fish use is highest from mid-winter to mid-spring. The shortnose sturgeon *(Acipenser brevirostrum*), which is federally listed as endangered, is an anadromous species known to occur in the Cape Fear River. The catadromous American eel (*Anguilla rostrata*), is widely distributed in the Cape Fear River estuary and migrates through the area of the bar channel (Schwartz et al., 1981).

The State of North Carolina defines Primary Nursery Areas (PNAs) as tidal saltwaters which provide essential habitat for the early development of commercially important fish and shellfish. It is in these estuarine areas that many fish species undergo initial post-larval development. PNAs are designated by the North Carolina Marine Fisheries Commission. In the vicinity of the relocated turning basin and the Alligator Creek restoration area at Eagle Island, PNAs extend from bank to bank except the federal channel and associated side slopes.

In the vicinity of the Island 12 mitigation area, PNAs are present either side of the federal channel beginning 300 yards from the bottom toe of the channel (where the channel and side slopes intersect). Table 3.4a indicates the acres of PNAs that would be affected by each alternative, and the mitigation HUs that would be required for each alternative.

The fishery resources are less likely to be impacted by potential blasting during construction of one of the relocated turning basin alternatives than construction of the two the basins that are currently authorized (Almont and Southern States). These authorized basins are covered under the August 2000 BO from National Marine Fisheries Service (NMFS 2000), and the 2000 EA (USACE 2000). The volume of rock to be removed for the currently authorized basins is about 207,000 cubic yards; it is about the same as that to be removed from the pilots preferred alternative (224,000 cubic yards). However, the rock is harder in the Almont basin than in the pilots' preferred alternative. If the pilots preferred alternative is constructed the other two basins would be abandoned and no dredging performed there. With the volume of rock being about the same, and the rock not as hard for the pilots preferred alternative, the number of blasts, if any, is likely to be fewer and thus the impacts to fishery resources are likely to be less.

Impacts of blasting on fish was discussed in detail in USACE 2000, and included the channel adjacent to all the relocated turning basin alternatives

3.7.1.3 Essential Fish Habitat. The 1996 Congressional amendments to the Magnuson-Stevens Fishery Conservation and Management Act (MSFCMA) (PL 94-265) set forth requirements for the National Marine Fisheries Service (NMFS), regional fishery management councils (FMC), and other Federal agencies to identify and protect important marine and anadromous fish habitat. These amendments established procedures for the identification of Essential Fish Habitat (EFH) and a requirement for interagency coordination to further the conservation of federally managed fisheries.

Impacts on habitat categories potentially present in the vicinity of the turning basin alternatives or restoration areas at Alligator Creek or Island 12 are discussed below. Essential Fish Habitat is not applicable to the locks and dams since according to National Marine Fisheries Service no essential fish habitat is located upstream of Lock and Dam #1 or in the potential construction area for fish passage alternatives downstream of that dam (NMFS 2007).

Impacts on Aquatic Beds and Wetlands: Except for emergent marsh at the potential restoration sites (Alligator Creek and Island 12) none of the other habitats are located in the project area or will not be impacted. The marsh to be impacted is primarily common reed (which is considered a nuisance species) at Alligator Creek and cattails at Island 12.

Impacts on Intertidal Flats, Oyster Reefs, and Shell Banks: Only intertidal flats are located in the project area. They are where the potential restoration sites (Alligator Creek and Island 12) would be connected to the adjacent existing PNA. These

areas would only be deepened to about -4 feet mean low water to provide access for tidal waters to the site and adequate tidal exchange. This shallow depth will not significantly alter the function of the flats.

Impacts on State-designated Areas Important for Managed Species: Primary Nursery Areas are designated by the NC Marine Fisheries Commission and are defined by the State of North Carolina as tidal saltwaters which provide essential habitat for the early development of commercially important fish and shellfish (15 NC Administrative Code 3B .1405). Many fish species undergo initial post-larval development in these areas. PNAs in the Wilmington Harbor Project are located along 15 miles of the Cape Fear River from about river mile 16 (Upper Lilliput Channel) upstream to the project limit near river mile 31 (Figure 3.4.2). They extend from the shoreline to an imaginary line 300 yards from the channel centerline from Upper Lilliput Channel to Upper Brunswick Channel (about river mile 24), inclusive. Island 12 is located in this reach with the back (east) being adjacent to PNA and to which to restored area would connect. Upstream from the Upper Brunswick Channel, PNAs extend from the river shoreline to the edge of the harbor channel. Alligator Creek and the relocated turning basin alternatives are located in this area. Table 3.4a indicates the acres of PNA impacted by the turning basin alternatives. This loss has been minimized to the extent feasible, and the mitigation available from potential restoration of Island 12 and Alligator Creek or fish passage will adequately mitigate for this loss.

Impacts on the Marine and Estuarine Water Column: The only work in the estuarine environment proposed is limited to the constructed of a relocated turning basin and potential PNA mitigation. Dredging operations conducted during construction and maintenance of the turning basin or connection of the mitigation sites to PNAs may create impacts in the estuarine water column in the immediate vicinity of the activity. These impacts may include minor and short-term suspended sediment plumes and related turbidity, as well as the release of soluble trace constituents from the sediment. During dredging, turbidity increases outside the dredging area should be less than 25 NTUs and are, therefore, considered insignificant. If a hopper dredge or scow is used, dredge/scow overflow will not be allowed. Overall water quality impacts of the proposed action are expected to be short-term and minor. Living estuarine resources dependent upon good water quality are not expected to experience significant adverse impacts due to water quality changes.

Impact Summary for Essential Fish Habitat: The proposed action is not expected to cause any significant adverse impacts to Essential Fish Habitat or EFH species.

3.7.2 Terrestrial resources.

3.7.2.1 Locks and Dams. On the west side at each lock and dam is a park area with picnic tables, shelter, toilet facilities, boat ramps, locks and associated facilities. There are also houses where the lockmasters lived when commercial

traffic was frequent on the river. Due to the lack of traffic, these houses are currently only used as an office for the lockmaster during the day.

On the east side of the river across from the parks, the land is vegetated with mature forest. The areas at Lock and Dams 2 & 3 are high bluffs rarely flooded by river events and are dominated by oak/hickory forests. Across from Lock and Dam # 1 is a wooded floodplain. There is an intermittent natural levee along the river that is 2 to 4 feet higher in elevation than the interior floodplain. The over story of the floodplain is mostly a closed canopy dominated by red maple (*Acer rubrum*), black gum (*Nyssa sylvatica*), bald cypress (*Taxodium distichum*) American elm (*Ulmus americana*), and various ash (*Fraxinus* spp.), oaks (*Quercus* spp.) and hickory (Carya spp.) species. Downed trees were present probably due to recent hurricanes. The under story is fairly open containing scattered privet (*Ligustrum sinense*), green briar (*Smilax* spp.) and saplings of over story species.

There are no prime and unique agricultural lands on any of the lands where a fish passage structure may be constructed.

3.7.2.2 Relocated turning basin. No terrestrial resources would be affected by any of the relocated turning basin alternatives. However if Island 12 was restored, terrestrial habitat would be converted to intertidal marsh or subtidal habitat. Most of these terrestrial habitats are open sandy/marl covered domes created by dredged material or low shrub thickets which provide minimal habitat value.

3.7.3 Protected species. An updated list of endangered and threatened (E&T) species for the project area was obtained from National Marine Fisheries Service (NMFS) (USACE 2007 & http://sero.nmfs.noaa.gov/pr/pdf/North%20Carolina.pdf) and the U.S. Fish and Wildlife Service (USFWS) (http://www.fws.gov/nc-es/es/countyfr.html) in June 2007. For the locks and dams and the relocated turning basin, the only species likely to occur in the project area is the endangered shortnose sturgeon (*Acipenser brevirostrum*). Fish passage at the locks and dams would be designed to benefit this species by enabling it to reach its traditional spawning grounds.

The shortnose sturgeon is less likely to be impacted by construction of one of the relocated turning basin alternatives than construction of the two basins that are currently authorized (Almont and Southern States). These authorized basins are covered under the August 2000 BO from National Marine Fisheries Service (NMFS 2000), and the 2000 EA (USACE 2000). The volume of rock to be removed for the currently authorized basins is about 207,000 cubic yards; it is about the same as that to be removed from the pilots preferred alternative (224,000 cubic yards). However, the rock is harder in the Almont basin than in the pilots' preferred alternative. If the pilots preferred alternative is constructed the other two basins would be abandoned and no dredging performed there. With the volume of rock being about the same, and the rock not as hard for the pilots preferred alternative, the number of blasts, if any, is likely to be fewer and thus the impacts to sturgeon are likely to be less.

Impacts of blasting on endangered and threatened species was discussed in detail in USACE 2000, and included the channel adjacent to all the relocated turning basin alternatives.

A biological assessment under the Endangered Species Act will be prepared and included in the draft Feasibility Report/EIS which will be provided to both the US Fish and Wildlife Service and the National Marine Fisheries Service.

3.7.4 Sediment and contaminants. Sediments in rivers and waterways often contain contaminants from sources within the watershed they drain. The Cape Fear River system drains a large area entirely within the state's boundaries. It flows from the north central piedmont region near Greensboro to the Atlantic Ocean. There are more than 100 municipalities and all or portions of 27 counties located in the basin (NCDEM 1995). The most densely populated areas are near Greensboro, Durham, Chapel Hill, Fayetteville, and Wilmington. Major industries in the basin include chemical manufacturing, paper and fiber, agriculture, and silviculture. The basin includes concentrated hog and turkey producing regions in Sampson and Duplin Counties. The Cape Fear River contains a concentration of industries along the lower portion of the Cape Fear River and the Cape Fear River estuary. Thus the watershed drains rural, urban, and industrial areas. Sediments are a sink for contaminants from various sources including controlled and uncontrolled releases. Sediments sequester point and non-point pollutants or contaminants contributed by activities in the watershed. Organic and inorganic contaminants have a strong affinity for silt (fine grained sediment) and the organic fraction of sediments. Sediments accumulate in areas where environmental conditions allow deposition. This may be areas such as a quiescent area in impounded reaches behind some dams such those on the Cape Fear River or in manmade channels and basins.

Assessment of the sediment contamination has been or will be conducted at all project sites. There are no regulations or sediment standards that dictate the approach used in evaluating potential sediment contamination at dam sites. However, there are well-established procedures for evaluating contaminant-related impacts from sediments proposed for dredging. These effects-based procedures are useful in evaluating potential sediment contaminant pathways and impacts. Three U.S. Army Corps of Engineers (USACE) and the Environmental Protection Agency (EPA) guidance documents were utilized:

a. "Evaluation of Dredged Material Proposed for Discharge in Waters of the U.S. – Testing Manual" (EPA/USACE 1998), commonly referred to as the Inland Testing Manual.
b. "Environmental Effects of Dredged Material Management Alternatives - A Technical Framework" (Technical Framework – EPA/USACE 2004).
c. "Evaluation of Dredged Material Proposed for Ocean Disposal – Testing Manual" (EPA/USACE 1991), commonly referred to as the Ocean Testing Manual.

3.7.4.1 Locks and Dams

Tier I Evaluation.

A Tier I (EPA/USACE 1998) assessment using readily available existing information to assess the potential sediment contaminant concerns was conducted and reported in USFWS (2006). The tier 1 reviewed databases and files maintained by State and Federal natural resource agencies. A geographic information systems (GIS) map was made which notes the proximity of pollutant sources to areas which would accumulate sediment including impoundments behinds the dams. Environmental studies for this portion of the Cape Fear River prepared by others with an emphasis on water and sediment chemistry were reviewed. A site reconnaissance on August 21, 2006 was conducted to determine general composition and identify depositional areas where pollutants may accumulate. People with knowledge of the dams, the river, and local pollution sources were interviewed.

The Tier I data indicate no known significant organic or inorganic pollutant problems in a one-mile assessment area surrounding the impounded reaches of the locks and dams. Because of identified contaminant sources, the potential of the dams to trap sediments in the low gradient inner coastal plain and the overall paucity of sediment data for the assessment area, additional physical, chemical and toxicity testing of sediments from depositional areas was recommended.

Tier II and III Evaluations - Sediment Sampling, Physical, Chemical, and Biological Evaluations

A Tier II and III (EPA/USACE 1998) assessment of sediments from within the impounded reaches of three locks and dams on the Cape Fear River was conducted and reported in USFWS (2007). Sediment sampling and analyses were conducted to obtain data on the physical and chemical characteristics of the sediments behind the locks and dams to compare to sediment screening values and assess the toxicity of sediments and sediment elutriates to sensitive aquatic organisms.

Twelve sediment samples from within impounded reaches of the three Cape Fear River Locks and Dams were sampled. Whole sediment toxicity tests with midges and amphipods showed no adverse effects of the sediments on test organism survival and growth. Sediment elutriate toxicity tests with sensitive aquatic organisms (cladocerans) resulted in statistically significant reductions in survival in samples from four sites. Manganese had among the strongest correlation and the most plausible biological association with the reduced survival observed.

Based on the results of sediment chemistry and toxicity tests, contamination in surface sediments behind the Cape Fear River Locks and Dams is unlikely to be a concern in-place. Mobilization of sediments may be a water column concern. However, extrapolation of the elutriate test conditions to those conditions which

may be expected during proposed actions which may cause resuspension of sediments. Information is needed to assess the significance of this issue.

3.7.4.2 Ship Turning Basin in Northeast Cape Fear River

Tier II and III Evaluations - Sediment Sampling, Physical, Chemical, and Biological Evaluations

A Tier II and Tier III (EPA/USACE 1991) assessment of sediments from the two existing turning basins in the Northeast Cape Fear River, just above the mouth of the Northeast Cape Fear River at the upstream limits of the project were conducted and reported in USACE (2005). As a part of this assessment a Tier I - Evaluation of Existing Information specifically reviewing the results of previous chemical and biological evaluations of Wilmington Harbor Federal navigation project dredged material dating back to 1978. The Tier II and Tier III testing design and list of contaminants of concern were closely coordinated with EPA, Region 4.

These sediment assessments were conducted for the purpose of evaluating the sediment for the purpose dredging from the proposed maintenance and authorized improvements of the Wilmington Harbor Federal Navigation Project and disposal in the New Wilmington Ocean Dredged Material Disposal Site (ODMDS). The U.S. Environmental Protection Agency's (EPA) Ocean Dumping Regulations and Criteria (40 CFR 220-228) require in Part 225 that authorizations under Section 103 of the Marine Protection, Research, and Sanctuaries Act of 1972, as amended, for transportation of dredged material for the purpose of dumping it in ocean waters will be evaluated by the U.S. Army Corps of Engineers in accordance with criteria set forth in Part 227.

Acute 10-day whole sediment acute toxicity tests were performed on Northeast Cape Fear River test sediments using an infaunal amphipod and a polychaete (marine worm). The polychaetes exhibited survivorship of 97% and above and the amphipod exhibited 99-100% survivorship in all test sediments. These results show that the whole sediment does not cause significant acute toxicity.

The potential for bioaccumulation of contaminants was evaluated through 28-day solid phase tests using bivalves and polychaetes. The tissue contaminants tested for included metals and PAHs and dioxins/furans. The list of contaminants of concern, in tissues, was determined after review of sediment analyses and following coordination with EPA, Region 4.

The bioaccumulation potential was evaluated two ways. First the concentrations in the tissues exposed to test treatments were statistically compared to tissues exposed to a reference. Significant differences occurred between the test and reference exposed tissues. Secondly, the magnitude and number of contaminants observed in the test tissues were assessed using available FDA Action Limits, Ecological Non-Specific Effects Thresholds or the EPA Region 4

background tissue concentrations. Based on these evaluations, the sediments tested in the Northeast Cape Fear River do not contribute to unacceptable levels of bioaccumulation in test organisms. These sediments meet the Limiting Permissible Concentration (LPC) for bioaccumulation and complies with the bioaccumulation aspects of the benthic criteria of Part 227.13(c)(3).

Sediment elutriate toxicity tests using sediments collected from the Northeast Cape Fear River existing turning basins. Organisms for these tests included a fish, the inland silverside, a mysid shrimp, and the larvae of a bivalve. These sediment elutriates did not exhibit significant toxicity. Survivorship was greater than 90% for the fish and the shrimp. The average normal mussel larvae development rate in the elutriate treatments ranged from 78% to 96% with no statistically significant differences between control seawater and the elutriates.

Based on this review and the results of testing of sediments from the Northeast Cape Fear River, the sediments do not contain contaminants in concentrations would preclude dredging and appropriate disposal including ocean disposal. EPA Region 4 has concurred with this evaluation.

Alternative Turning Basins

Construction of a new turning basin located in the middle of the Northeast Cape Fear River portion of the Wilmington Harbor Project is an alternative being considered. Evaluations of sediments from specific alternative locations being considered (Alternatives B1, C1, C2, and C3, section 3.3) have not been performed. The industrial areas adjacent to the alternative turning basins warrant site-specific evaluation of sediments which may be dredged during basin construction. Testing procedures for evaluating contaminant-related impacts from sediments proposed for dredging as previously discussed will be used. These procedures will be required to evaluate potential sediment contaminant pathways and impacts for these specific alternatives. Results will be included in the draft EIS.

3.7.5 Wetlands.

3.7.5.1 Locks and Dams. Most of the wetlands adjacent to the Cape Fear River are located downstream of Lock and Dam # 2. These are wooded wetlands dominated by bald cypress, black gum, green ash and similar species, and are generally flooded by rising river water on at least an annual frequency. None of the fish passage alternatives would be constructed in wetlands, but if Lock and Dam # 1 was lowered or removed, the flooding of some wetlands could be reduced to a biannual frequency (Appendix 3.8.3). However since the hydric soils and vegetation would not be altered, those areas would still be classified as wetlands.

3.7.5.2 Relocated turning basin. No wetlands will be involved in the construction of any of the relocated turning basins alternatives. However if Alligator Creek or Island 12, are restored for mitigation of turning basin impacts,

wetlands will be involved. The Alligator Creek alignment is dominated by common reed. Upon restoration, approximately 25 acres of intertidal and subtidal habitat would be restored. This would include 6.8 acres of intertidal marsh (saltmarsh cordgrass, *Spartina alterniflora*) to be planted adjacent to the restored creek. The existing wetlands at island 12 that will be altered consist primarily of narrow leaf cattails. The marsh is primarily at the northern end of the island where the restored area is connected to existing PNA. Based on the alternative potentially selected, about 9.4, 12.6, or 18.5 acres of intertidal and subtidal habitat will be restored. This would include about 3 acres of intertidal marsh (saltmarsh cordgrass, *Spartina alterniflora*) to be planted along the shoreline.

3.7.6 Cultural Resources

3.7.6.1 Turning Basin alternatives. The Cape Fear and Northeast Cape Fear Rivers have a long and active history as one of the most significant waterways in North Carolina. Spanish explorers sighted the Cape Fear River at least as early as the first quarter of the 16th century and European settlement began in 1664 with the establishment of Charles Town near the mouth of Town Creek. The settlement of Charles Town was accompanied by the rise of scattered plantations and farms spreading out for 60 miles along the rivers. The most significant pre-Colonial settlement was Brunswick Town, established in 1725. Although never a large town, Brunswick Town survived for 60 years as the administrative center for North Carolina's five ports of entry. By 1733, the town of New Carthage, later renamed Wilmington, had been laid out, and in a few decades it would begin to outstrip Brunswick Town as a cultural and maritime center.

During the years leading up to the Revolution, both the Cape Fear and Northeast Cape Fear Rivers thrived, with plantations extending up the Northeast and its tributary, Smith Creek. This growth continued into the 19th century, when up to 40 ships per month were visiting Wilmington's harbor. In 1840, Wilmington obtained the status of North Carolina’s largest town. In the following two decades, it doubled in size becoming a magnet for New England entrepreneurs and businessmen. By the mid-19th century there were over 140 named landings located along the 115 miles of river between Wilmington and Fayetteville and, by 1905, channel improvements made the Northeast Cape Fear River navigable for pole boats all the way to Kornegay's Bridge, 103 miles above the river's mouth. Soon, ship building, fertilizer and brick factories, shipping terminals, and other capital intensive industries began to replace commercial fishing, hunting, forestry and agriculture as economically dominant businesses. The 1940s and the Second World War brought an influx of newcomers and a renewed energy to the city. The North Carolina Shipbuilding Company employed thousands of workers, who delivered 243 new ships for the cause. In 1945, the North Carolina Legislature approved the State Port Authority, which provided support for transforming the World War II shipyard into a first class port facility. During the last half of the 20th century, the Cape Fear and Northeast Cape Fear Rivers began to transition from waterfront industry and commerce to residential and light

commercial enterprises. The popularity of recreational boating and waterfront recreation are again changing the face of both rivers.

The importance of the shifting economic base, from agriculture, to industry, and commerce, is reflected along the banks of the rivers and underwater by the remains of shipyards, Liberty Ships, marine railways and dry docks, commercial docks, and the hulks of vessels of virtually every variety. These archaeological remains document the importance of the area as a maritime center. Thirty-seven historic shipwrecks are listed on the 1985 National Register of Historic Places Registration addendum for the Wilmington Historic District prepared by the North Carolina Division of Archives and History. In addition, over 130 shipwrecks are known from the lower Cape Fear - Northeast Cape Fear River vicinity. The Northeast Cape Fear alone has spurned over 40 research papers addressing various shipwreck investigations ranging from the Revolutionary War Period to the present day.

Within the area of effect of the proposed basins, five known shipwreck sites have received some level of professional attention, ranging from basic location and identification survey to full-blown on-site mapping and historic documentation. The presence of these cultural resources within the project area indicates that there could be a high potential for adverse impacts from basin construction and operation, depending in part on which basins are chosen and the method of construction. Further details on these cultural resources and potential impacts are addresses in Section 5.2 of this report.

3.7.6.2 Lock and Dam Alternatives. Of all the rivers that empty into the Atlantic Ocean along the coastline of North Carolina, the Cape Fear River is the only one that has a system of locks and dams along its course. Consequently, it is the only “slack water” navigational stream in all of eastern North Carolina - a notable fact given that the state has few significant navigable waterways. Outside of the middle Cape Fear valley, however, few people in North Carolina are even aware that such a system exists.

Figure 3.7.6.2a Early days of the Cape Fear Locks and Dams

The navigation system along the Cape Fear consists of three sets of locks and dams built to ensure a safe and reliable river channel for barge traffic between Wilmington, near the mouth of the Cape Fear, and Fayetteville, located

115 miles upstream. All three locks and dams are located in Bladen County, North Carolina. Lock and Dam No. 1 and No. 2 were planned together and built sequentially, with some overlap. Workers at No. 1, located the furthest downstream, began construction in 1913 and finished in 1915; the construction of No. 2 commenced in 1914 and was completed in 1917. Lock and Dam No. 3, located the furthest upstream, was built much later, between 1933 and 1935. In 1965, No. 3 was renamed the William O. Huske Lock and Dam for its chief promoter, a prominent Fayetteville businessman. At all three sites, the locks are located on the west or southwest side of the river, with the dams and abutments located on the east or northeast side.

Figure 3.7.6.2b Cape Fear Locks and Dams today

Congress authorized the first survey for a possible lock and dam project on the Cape Fear on June 1, 1900 and the Rivers and Harbors Act of June 13, 1902 provided $1,350,000 for three locks and dams, which would provide an eight-foot deep channel between Wilmington and Fayetteville. Engineering evaluations led to the selection of the sites in early 1906. The three sites selected were: Kings Bluff, about 40 miles above Wilmington; Browns Landing, 72 miles above Wilmington; and Tolars Landing, 95 miles above Wilmington and less than 20 miles below Fayetteville. The plans for Lock and Dam No. 1 were begun in early 1911 and finalized and approved on May 14, 1913. The final blueprints for the lockmasters' houses date to 1912.

The construction of Lock and Dam Nos. 1 and 2 began with No. 1 at Kings Bluff. When certain key mileposts were passed in that construction, work began on No. 2 at Browns Landing. As a result, there was a calculated lag between the work at No. 1 and No. 2. Initial work at No. 1 began in 1912 and was completed in July of 1915. Work began at No. 2 in 1914, with the opening of that lock occurring in August of 1917. Whenever possible, the work was staggered to make the most efficient use of the work crews and the available materials, much of which was shifted from No. 1 to No. 2 as the work allowed.

Figure 3.7.6.2c Dam No. 1

The locks and dams on the Cape Fear River are unique partly because of the river itself. In North Carolina, only the Cape Fear could offer transportation from

the Piedmont to a seaport without intervening shallow sounds. It was also the only river that could accommodate “slack water” navigation in eastern North Carolina. It is no wonder that the drive to open the channel between Fayetteville and Wilmington was so unyielding. Given the historical importance of these locks and dams in providing navigation to the inland sections of the state, the three properties are considered eligible to the NRHP under Criterion A; they possess significance on a state and local level for their role in river transportation in North Carolina.

Figure 3.7.6.2d Dam No. 2

Figure 3.7.6.2e Dam No. 3

The locks and dams and their environs are also significant for their engineering and architectural design and therefore, are also eligible to the NRHP under Criterion C. These structures, particularly Lock and Dam Nos. 1 and 2, were essentially built on a bed of sand that posed some complicated engineering dilemmas unique to the sites. The lock floors at Nos. 1 and 2 were laid underwater using a tremie—an uncommon practice typically employed on pier and bridge construction. This technique garnered national attention in two contemporary journals—*Engineering News Record* (Vol. 76, September 21, 1916) and *Professional Memoirs* (Vol. 8, 1916 and Vol. 9, 1917). While the timber crib dam at Dam No. 1 was a common design, the fact that it remained in fair condition for three decades before receiving a concrete apron for stabilization is notable. The dam was then repeatedly altered and improved in order to sustain its life span. The dam at No. 2 is quite unique in its design. Not a standard rock-fill dam with an impervious central earth core or concrete face, it was simply constructed of rows of steel sheet piles driven in the riverbed on the upstream and downstream sides. Riprap and capstone was then loosely dumped between the sheeting. Dam No. 3 appears to be a typical concrete gravity structure similar to those constructed across the country during this period.

It has been recommended that the three locks and dams and their associated historic buildings be considered eligible to the NRHP as a multiple property submission of three historic districts that comprise the Cape Fear Navigation System. The Districts are thematically related.

3.7.7 Spawning areas for anadromous fish.

3.7.7.1 Locks and Dams. Historic records indicate that anadromous fish spawned up to the fall line on the Cape Fear River which is upstream of the Jordan Lake Dam near Moncure, NC. A study performed by Dial Cordy and Associates under contract to the Wilmington District Corps of Engineers in the spring of 2006 indicated that spawning of American shad and striped bass (only anadromous species where significant eggs and larvae were collected) occurred primarily upstream of Lock and Dams 2 & 3 (DC 2006). North Carolina State University is currently under contract to the Wilmington District Corps of Engineers to determine the anadromous fish spawning areas during the spring of 2007 and 2008. The results of these investigations will be included in the draft and final GRR/EIS when available.

3.7.7.2 Relocated turning basin. Anadromous fish use the harbor in the vicinity of the turning basins and mitigation sites for feeding, maturation, and migration, but do not use the area for spawning.

3.7.8 Saltwater intrusion. A concern of the PDT was the potential of saltwater intrusion into areas above Lock and Dam No. 1, and resulting adverse effects on freshwater supply for intakes above the dam. A saltwater intrusion investigation was done to determine the furthest upstream limit of the saltwater front on the Cape Fear River, and model analysis did not show salinity reaching Lock and Dam No. 1. In addition, since there is no proposal to remove Lock and Dam No. 1, saltwater intrusion would not become an issue. See Appendix 3.7.8 for the saltwater intrusion report.

3.7.9 Recreation. Recreational boaters enjoy fishing in the Cape Fear River in the vicinity of the locks and dams. In addition, recreational areas have been available at the three locks and dams since 1965. The maintenance of sufficient depth for these recreational uses is dependent on the proper function and operation of the lock and dam, especially during drought and low flow conditions.

Since 1965, the Corps of Engineers has maintained and upgraded many of the recreational facilities at the CFRL&D to provide safe and sanitary accommodations for boaters, fishermen, picnickers, and sightseers. These facilities include picnic areas, restrooms, parking areas, fishing areas, boat launching ramps, and open grassed areas. Throughout the 1990s, the annual visitation for the three locks and dams has averaged nearly 200,000 visitors. The land dedicated to recreation activities at each lock and dam is as follows:

Lock and Dam Number 1	6 Acres
Lock and Dam Number 2	21 Acres
William O. Huske Lock and Dam	12 Acres
Total	39 Acres

3.7.10 Geotechnical and Ground Water.

3.7.10.1 Geotechnical. This GRR evaluates a variety of alternatives involving changes to the Wilmington Harbor Project on the Northeast Cape Fear River and the three lock and dam projects on the Cape Fear River. Refer to Appendix 3.7.10.1, "Wilmington Harbor GRR Geotechnical Appendix", included in this GRR, for details of the geotechnical engineering and geology aspects and considerations for these proposals.

3.7.10.2 Ground Water. Refer to Appendix 3.7.10.2 for review of ground water studies which have been performed for the study area on the Cape Fear River.

3.7.11 Shoaling. Navigation improvements associated with Northeast Cape Fear River portion of Wilmington Harbor consist of deepening and maintaining the navigation channel and the development of turning basins to accommodate the design vessel associated with this portion of the harbor. In addition to the authorized plan, four alternative turning basin locations and configurations are proposed to improve navigational operations and safety. The alternative turning basins were assigned designations of B1, C1, C2, and C3. A shoaling analysis was conducted to determine annual shoaling associated with these proposed plans. A shoaling analysis of the Northeast Cape Fear turning basin alternatives conducted by Taylor Engineering (Appendix 3.7.11), long term maintenance dredging records, and hydrographic surveys were used to determine annual shoaling quantities for the turning basin alternatives and reaches within Northeast Cape Fear River navigation project. The Taylor Engineering Study evaluated shoaling in the upper 8000 feet of the project. Historic dredging records for the Northeast Cape Fear River are compiled for two segments of the navigation channel. The lower segment with an authorized depth of 39 feet (mllw) starts at the Cape Fear Memorial Bridge and ends at the Hilton RR Bridge, a distance of approximately 11,000 feet. The upper segment with an authorized depth of 35 feet (mllw) starts at the Hilton RR Bridge and ends approximately 700 feet above Southern States Chemical, a distance of approximately 7,300 feet. Annual shoaling estimates for the upper segment of the river utilized hydrographic surveys conducted from 1995 through 2003 to establish annual channel shoaling rates. These channel shoaling rates were combined with turning basin shoaling predictions from the Taylor sediment transport analysis to establish a total annual shoaling value in the upper segment of the river for each turning basin alternative. A ratio of the turning basin shoaling relative to the total annual shoaling was established for each alternative. The ratio was applied to the historic annual shoaling volumes for the upper river segment to determine the annual shoaling for each turning basin. These annual turning basin shoaling rates were combined with the annual channel shoaling rates to establish the total annual shoaling associated with the upper segment of the Northeast Cape River. Annual shoaling rates for the lower segment of the Northeast Cape Fear River were developed from historic dredging records. The Almont turning basin is located in the lower segment of the river. Hydrographic surveys were used to determine annual shoaling associated with the Almont turning basin. With the development of one of the turning basin alternatives for the upper segment of the river, the Almont turning basin will not be required for navigation. Annual shoaling

associated with the Almont turning basin was subtracted from the historic annual shoaling in the lower segment of the Northeast Cape Fear River. Average annual shoaling rates for the Northeast Cape Fear River are summarized in Table 3.7.11.

Table 3.7.11

NECFR shoaling totals for alternative turning basin plans

River Segment > alternatives	lower channel Cape Fear Memorial Bridge to Hilton RR	lower Almont turning basin	upper channel from Hilton RR to Southern States	upper Southern States turning basin authorized	upper turning basin shoaling for alternatives B1,C1,C2 or C3	total shoaling
depth - authorized	39 ft	39ft	35ft	35ft		
shoaling-cy/yr	16,770	8,500	17,651	3,600		46,521
depth w/ B1	39 ft	abandon	39ft	39ft		
shoaling-cy/yr	16,770	0	17,651	0	4,300	38,721
depth w/C1	39 ft	abandon	35ft	abandon		
shoaling-cy/yr	16,770	0	17,651	0	10,700	45,121
depth w/C2	39 ft	abandon	35ft	abandon		
shoaling-cy/yr	16,770	0	17,651	0	10,000	44,421
depth w/C3	39 ft	abandon	35ft	abandon		
shoaling-cy/yr	16,770	0	17,651	0	2,000	36,421

In 2005, Taylor Engineering conducted a shoaling study for the alternative turning basin locations/configurations. The chief tasks Wilmington District assigned to Taylor Engineering included:

- Determine historic shoaling rates in the proposed turning basin's vicinity.
- Define potential shoaling rates for the four turning basin alternatives including identification of potential shoaling trends and problems.
- Recommend best alternative based on least potential of shoaling and maintenance requirements.

The resulting report is included in Appendix 3.7.11, “Wilmington Harbor - 96 Act Project Northeast Cape Fear River Turning Basin(s) Shoaling Analysis”. Based on the DMS and hydrodynamic modeling analysis performed, option *C3* is the preferred alternative for the NE Cape Fear River turning basin in terms of least potential of shoaling and maintenance requirements.

3.8 Major design/operational considerations.

3.8.1 Water supply. There are five existing water supply intakes on the Cape Fear River upstream of the locks and dams: Two upstream of Lock and Dam # 1, and three upstream of Lock and Dam # 3. One of the two intakes upstream of Lock and Dam # 1 is operated by the City of Wilmington, and the other is operated by the Lower Cape Fear Water and Sewer Authority. If the dam is lowered or removed, these facilities would be adversely impacted (paragraph 3.5.3.3). The Authority proposes to add an additional intake upstream of the dam, parallel to their existing line. The Corps has requested that the Authority evaluate installing this intake so that it would not be adversely impacted if the dam was lowered or removed.

Dupont Fayetteville operates one of the three intakes upstream of Lock and Dam # 3. This intake is located about a mile upstream of the dam, but is located low enough in the river such that if would not be adversely affected if the dam was lowered or removed. The two additional intakes are about 20 miles upstream of the dam and are operated by the City of Fayetteville. Even though these intakes are 20 miles upstream of the dam, they are located in the pool created by the dam. If the dam is lowered or removed, these utilities would be adversely impacted (paragraph 3.5.3.3 and 3.5.3.4).

Currently there is not an intake upstream of Lock and Dam # 2, but one is proposed by the Lower Cape Fear Water and Sewer Authority near Tar Heel, NC. The primary initial user, Smithfield Foods, has bee required by the State of North Carolina to move from groundwater to surface water. This has been required because a groundwater cone of depression over 100 feet deep has been created around the Smithfield Foods facility. The Corps has requested that the Authority evaluate installing this intake so that it would not be adversely impacted if the dam was lowered or removed.

See table 3.8.1 for information on the recent and projected demands for these water supply facilities. This information was obtained from the water users and the NC Division of Water Resources website (http://www.ncwater.org).

Table 3.8.1. Water Supply Demand in Million Gallons per day (MGD) by the Surface Water Supply Users from Lock and Dam #1 Through the Headwaters of Lock and Dam # 3.

		Demand (MGD) by Year	
Location and Water Users		**2002**	**2030**
Upstream of Lock and Dam # 1			
	City of Wilmington	16.6	30.7
	Lower Cape Fear Water and Sewer Authority[1]	43.2	96.1
Upstream of Lock and Dam # 2			
	Proposed Lower Cape Fear Water and Sewer Authority	0	12
Upstream of Lock and Dam # 3			
	Dupont Fayetteville	10.8	19.6
	City of Fayetteville	25.2	82.6
	Total	**95.8**	**241**

1. Includes proposed additional intake in year 2030 column

Source: Information obtained from the users, their consultants, or the NC Division of Water Resources website (http://www.ncwater.org).

See Appendix 3.8.1 for an evaluation of water supply intakes by Mobile District.

3.8.2 Potential for commercial use of the locks.

3.8.2.1 Small boat navigation. Presently, recreational boats comprise the largest category of boats being locked on the Cape Fear River. As commercial traffic has virtually ceased since 1994, recreational traffic has been on the rise. Since 1994, recreational boats have averaged 45, 20, and 16 lockages each year at Lock and Dam Number 1, 2, and 3, respectively. Most of these lockages were for small recreational fishing craft.

3.8.2.2 Commercial navigation. The Waterborne Commerce of the United States Statistics for commerce transported along the Cape Fear River Above Wilmington are listed below. These statistics include the three CFRL&D. The majority of this reported cargo moves were from Wilmington to industrial sites downstream of Lock and Dam Number 1 (river mile 30) and did not require lockage. In other words, these statistics include the tons of commodities locked along the Cape Fear River in addition to downstream commodity movements.

TABLE 3.8.2.2
Cape Fear River Above Wilmington
Short Tons of Cargo

YEAR	TONS
1984	110,356
1985	95,082
1986	182,465
1987	277,110
1988	279,805
1989	180,237
1990	307,982
1991	396,509
1992	458,229
1993	373,832
1994	407,000
1995	348,000
1996	N/A

3.8.2.3 Commercial terminal facilities. There are twelve waterfront facilities on the river with a total frontage of 1,190 feet. Some of the facilities in the Fayetteville area are deteriorating from lack of use, but could be restored if traffic on the Cape Fear River from Wilmington to Fayetteville warranted the investment.

3.8.2.4 Commercial cargo trends. Traffic on the Cape Fear River from Wilmington to Fayetteville was brisk from the turn of the century to the early 1970's. Until recently, three river terminals in Fayetteville received barged commodities that had been locked through the CFRL&D. Liquid nitrogen was the primary product being barged up the river, while logs were barged down the river. In the early 1990s, more than 60 barges carrying a total of over 60,000 tons of products were locked through the CFRL&D annually both upriver and downriver. By 1993, with the closing of International Paper's operations in Elizabethtown, North Carolina (near Lock and Dam Number 2), the commercial lockages were cut in half. However, this decrease in locked cargo was offset by increases in barge traffic below Lock and Dam Number 1. In 1995, only one commercial barge was locked going both up and down river at Lock and Dam Numbers 1 and 2 (see below).

Table 3.8.2.4
Commercial Traffic to Fayetteville

Year	Barge Tows	Short Tons
1989	45	56,000
1990	54	62,000
1991	64	40,000
1992	57	65,000
1993	39	36,000
1994	28	26,000
1995	2	1,000
1996	1	500
1997	0	0

3.8.2.5 Alternative commercial transportation. Today, the railroad has captured the liquid nitrogen shipments between Wilmington and Fayetteville. The transfer from barges to rail coincided with the closing of the CFRL&D for major rehabilitation in 1994. The operation of the locks was interrupted for about four months while new seals and other work was accomplished. When the locks were reopened in the fall of 1994, the railroad had captured the liquid nitrogen business.

The highway system is the other transportation mode within the study area providing competing services to commercial navigation along the Cape Fear River. North Carolina Highway 87 follows the Cape Fear River valley and, although it is not a limited access highway, it carries a significant amount of truck traffic. The travel time by truck between Fayetteville and Wilmington is approximately 2-1/2 hours as compared to about 26-1/2 hours by tug and barge.

3.8.2.6 Analysis of alternative transportation. Shippers of bulk commodities in Southeastern North Carolina have a choice of truck or rail. If they are located along the Cape Fear River between Wilmington and Fayetteville, they also have a choice of barge. Rules of thumb for choosing between the three methods of transportation say trucks have the advantage up to 200 miles, rail the advantage from 200 to 500 miles, and barges for longer hauls where the barge route can have a competitive routing. Most of the traffic between Wilmington and Fayetteville is by truck, with some also by rail. The last commodity to use the CFRL&D, Liquid Nitrogen, is now moved by rail between Wilmington and Fayetteville.

One product that seems to use barging economically was the timber industry. Timber was harvested along the Cape Fear River and brought to the river for shipment. It could be hauled to paper mills at Charleston and Georgetown in South Carolina and Plymouth, New Bern, Riegelwood and Roanoke Rapids in North Carolina. These barge runs were sufficiently long to provide an advantage over truck or rail. But difficulties in operation and efficiency caused the barging of logs to cease on the CFRL&D, and other commodities have had similar problems.

Typical rate studies show that trucking costs about 20 cents per ton mile, rail about 10 cents per ton mile, and barging only 4 cents per ton mile. Barging is especially economical on high volume routes like the transportation of high volume fertilizer between Aurora and Morehead City, fuel oil between Wilmington and Riegelwood, and chemicals barged on the Northeast Cape Fear River. While there have been recent interest and inquiries about barging on the Cape Fear Above Wilmington, no firm prospects are currently identified.

3.8.2.7 Military uses. Section 3.5.3.3.

3.8.2.8 Evaluation of Commercial Traffic on the Cape Fear River above Wilmington.

3.8.2.8.1 Commercial navigation. Refer to section 3.8.2.2 for commercial navigation statistics.

3.8.2.8.2 Commercial commodities. The remaining commercial traffic on the Cape Fear River above Wilmington does not proceed as far up river as Lock and Dam 1. The traffic shown for 2004 included 164 tons of residual fuel oil and 570,000 tons of coal and lignite. These commodities were delivered to factories and power plants upstream of Wilmington but downstream of Lock and Dam 1.

3.8.2.8.3 Commercial cargo trends. Refer to 3.8.2.4 for data on commercial cargo trends.

3.8.3 Water level modeling for dam alteration or removal. The determination of the effects on the water surface elevations and flood plains was accomplished using the HEC-RAS program. The input data was developed using the HEC-2 backwater model developed in the 1970's for the Cape Fear River from NC 11, just below Lock and Dam 1, to the confluence of the Haw and Deep Rivers. The HEC-2 model that was developed for the Cumberland County Flood Insurance study in the 80's was inserted into the basic HEC-2 model. The reach lengths and the stations of the cross sections were adjusted to fit the aerial photographs that were used for this project. Since the project limits were from Lock and Dam 1 to the Fayetteville water supply intakes the stations all the way to the end of the model were not adjusted. The various discharges that were used came from evaluation of the historical data of the gages at Lock and Dam 1, Lock and Dam 3, Fayetteville, and Lillington.

The vertical datum of the HEC-2 models were based on the National Geodetic Vertical datum of 1929 (NGVD), when they were converted to the HEC-RAS format the vertical datum was not changed. The datum of the mapping and the topography is based on North American Horizontal Datum of 1983 in North Carolina State Plane Feet and the North American Vertical Datum of 1988, (NAVD). For the Study area the conversion is NGVD '29 minus ≈1 foot equals NAVD '88. The backwater summaries and profile plots elevations are NGVD '29. The flood plains are mapped on NAVD '88 topographic maps with the elevations

adjusted to NAVD ’88. The elevations of the Locks and Dams were taken from the Project Map books and adjusted from Baldhead Datum to NGVD ’29 datum.

The historical records of the gages in the study reach of the Cape Fear River were used to establish the minimum potential discharge. The records were searched for the minimum discharge that covered a period of 3 continuous days or more. This turned out to be approximately 300 cubic feet per second (CFS). This discharge was used to evaluate the effects of the various alternatives on the water supply intakes. The effects of the alternatives on the adjacent flood plains and wetlands was evaluated using the 1 year and 2 year frequency floods. A comparison of the 1 year with existing conditions to the 2 year with the dams removed was made and it was determined that there would be no loss of wet lands. See Section 3.7.5 Wetlands for additional information. Using a discharge of 5,000 CFS for the spawning season flow the alternatives were evaluated as to the impact on fish passage that each alternative produced and was also used to set the maximum height the dams could be and not require a rock ramp for passage. The HEC-RAS model was extended downstream of NC 11 approximately 5 miles in order to run an unsteady flow conditions to determine the starting conditions for the steady state flow models.

There are four alternatives being considered that would alter the dams and associated pool levels: (1) remove the locks and dams, (2) drop the top of the dams to the lowest elevation possible and not require modifications to intake structures, but with reduced water supply capacity, (3) lower the tops of the dams so that no rock ramps are needed for fish passage, (4) leave Locks and Dams 1 and 3 as-is and remove Lock and Dam No. 2. Table 3.8.3 provides a summary of the water surface elevations at key points along the river for each alternative. The elevation of the top of the dams was chosen using the following criteria. Alternative 1, the dams were removed down to the natural ground level. Alternative 2, each pumping facility was evaluated and the minimum elevation required for proper operation was determined and based on the profile for 300 cfs the lowest top of the dam was determined. Refer to Appendix 3.8.1, *Wilmington Harbor GRR Lock and Dam Study Intake Evaluation*, prepared by the U.S. Army Corps of Engineers Mobile District, dated March 2007 for additional information. Alternative 3, using the spawning season discharge of approximately 5,000 cfs, the top of the dam elevations were selected that gave enough flow over the dam and had the downstream water surface elevation at least level with the top of the dam, so as to provide unimpeded fish passage.

Appendix 3.8.3 “Hydraulics” has more detailed information on these subjects.

WILMINGTON HARBOR GRR - LOCKS AND DAMS STUDY

Water Surface Elevations @ 300 CFS

July 27, 2007

LOCATION DESCRIPTION	River Station Feet from Mouth	Existing conditions FEET NGVD Dam Elevation	Existing conditions FEET NGVD Water Surface Elevation	Alternative 1 Locks and Dams Removed FEET NGVD Dam Elevation	Alternative 1 Locks and Dams Removed FEET NGVD Water Surface Elevation	Alternative 2 L&D's lowered to elevation of reduced water supply capacity FEET NGVD Dam Elevation	Alternative 2 L&D's lowered to elevation of reduced water supply capacity FEET NGVD Water Surface Elevation	Alternative 3 Locks and Dams lowered so that no rock ramps are required FEET NGVD Dam Elevation	Alternative 3 Locks and Dams lowered so that no rock ramps are required FEET NGVD Water Surface Elevation	Alternative 4 L&D's 1&3 as is L&D 2 removed FEET NGVD Dam Elevation	Alternative 4 L&D's 1&3 as is L&D 2 removed FEET NGVD Water Surface Elevation
Lock & Dam 1	353000	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50	10.70	11.20
City of Wilmington Intake	353440	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50	10.70	11.20
LCFWASA Intake	353760	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50	10.70	11.20
Elwell's Ferry	399290	10.70	11.20	-5.5*	1.60	8.00	8.56	2.00	2.70	10.70	11.20
Lock & Dam 2	523670	20.10	20.50	2*	3.20	2*	8.60	14.00	14.60	2*	11.2
Proposed Smithfield Foods Intake	619290	20.10	20.70	2*	13.20	2*	12.90	14.00	14.90	2*	13.1
Lock & Dam 3	645300	29.10	29.40	9.9*	13.30	27.20	27.60	23.00	23.60	29.10	29.40
Dupont Intake	650700	29.10	29.60	9.9*	14.00	27.20	27.80	23.00	23.60	29.10	29.60
Fayetteville Glenville Intake	764605	29.10	29.70	9.9*	23.00	27.20	28.00	23.00	24.10	29.10	29.70
Fayetteville Hoffer Intake	764605	29.10	29.70	9.9*	23.00	27.20	28.00	23.00	24.10	29.10	29.70

NOTE: * indicates that the dam has been removed and that is natural bottom at the site

Table 3.8.3: Water Surface Elevations at 300 CFS

3.8.4 Future ownership of locks and dams, and recreation areas. [TO BE ADDED WHEN AVAILABLE.]

3.8.5 Pilots' preferred turning basin alternative.

3.8.5.1 Descriptions of turning basins on the Northeast Cape Fear River. The Almont turning basin has been the only turning basin available in the NECF 32-foot authorized project. As the deepening studies progressed, discussions were requested by the river pilots regarding alternative locations for turning basins that would not require them to back stern-first through the bridges.

The preferred choice by the pilots was based upon their current needs in the river. The Chemserve terminal became the preferred location for a turning basin since it has the highest current use. The Almont Terminal has been closed and commercial shipping activities moved to other locations in the harbor, principally the NCSPA docks. Other permutations of turning basins in this area were set up for evaluation as well as an alternative developed around the existing turning basin at the upper end of the current 25-foot process. Four alternatives emerged, all of which would serve the needs of the pilots. All alternatives would have construction methods of some combination of rock blasting, mechanical excavating and cutter-suction hydraulic dredging. The alternatives are described in detail in section 3.3.

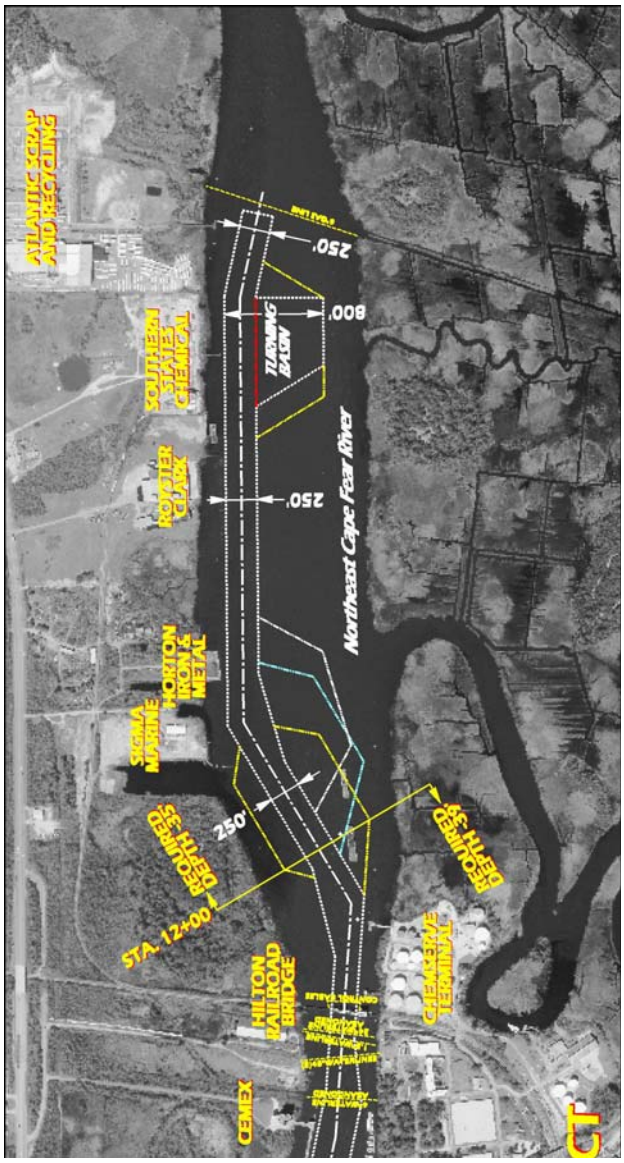


Figure 3.8.5.1 Turning basin alternative locations

3.8.5.1.1 Alternative B1 – modify existing North Turning Basin

Locate the NECFR primary turning basin at site of existing North turning basin (-25' project) at junction of reaches 1 and 2. This basin is near the site of Southern States Chemical terminal. The basin is lengthened 250' to the south and flared into the adjacent channel at the north and south. The existing turning basin at Reach 7 (Almont) will be abandoned.

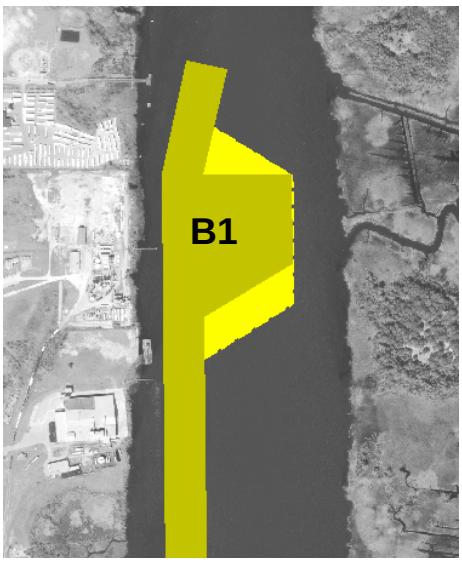


Figure 3.8.5.1.1 Turning basin Alternative B1

First Costs	$28,539,172
Annualized (over 50 yrs)	$1,533,189
Avg Annual O&M for years 1-50 (see below)	$266,551
Average Annual Costs	$1,799,741

3.8.5.1.2 Alternative C3 – new turning basin (Chemserve), South Option

Locate the NECFR primary turning basin just north of Chemserve in Reach 3 – this is the southern most of the three options in this area. This has been described as the "pilots preferred alternative" because of its proximity to the Chemserve terminal and its functional geometry relating to adjacent reaches. Turning basin at Reach 7 (Almont) and at Reach 2 (Southern States) would be abandoned.

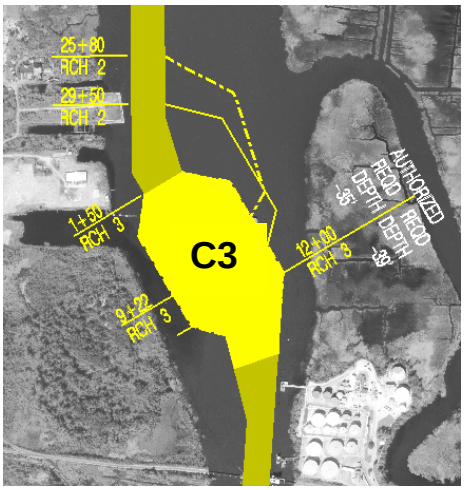
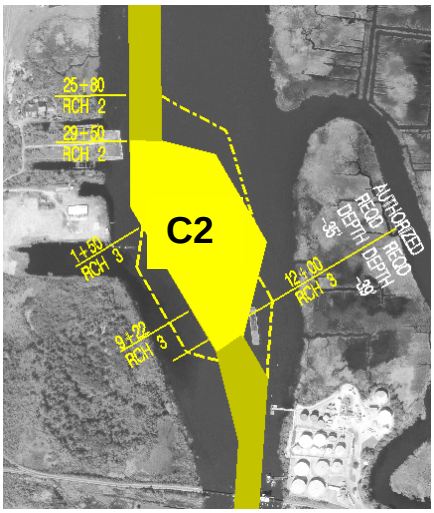


Figure 3.8.5.1.2 Turning basin Alternative C3

First Costs	$20,690,366
Annualized (over 50 yrs)	$1,111,534
Avg Annual O&M for years 1-50 (see below)	$249,120
Average Annual Costs	$1,360,653

3.8.5.1.3 Alternative C2 – new turning basin (Chemserve), Middle Option

Locate the NECFR primary turning basin just north of Chemserve in Reaches 3 and 2 – this is the middle of the three options in this area. Turning basin at Reach 7 (Almont) and at Reach 2 (Southern States) would be abandoned.

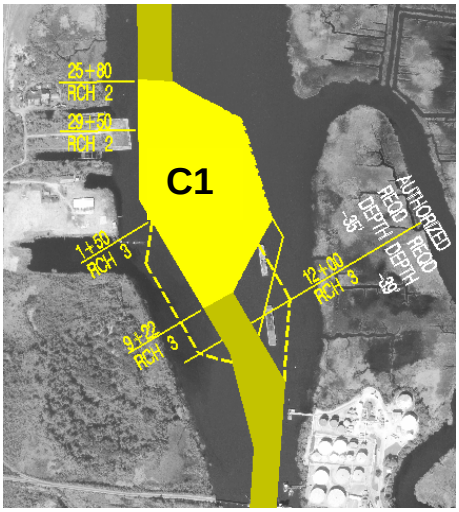


First Costs	$22,169,697
Annualized (over 50 yrs)	$1,191,007
Avg Annual O&M for years 1-50 (see below)	$303,840
Average Annual Costs	$1,494,846

Figure 3.8.5.1.3 Turning basin Alternative C2

3.8.5.1.4 Alternative C1 – New Turning Basin (Chemserve), North Option

Locate the NECFR primary turning basin just north of Chemserve in Reaches 3 and 2 – this is the upper northern most of the three options in this area. Turning basin at Reach 7 (Almont) and at Reach 2 (Southern States) would be abandoned.



First Costs	$22,901,350
Annualized (over 50 yrs)	$1,230,313
Avg Annual O&M for years 1-50 (see below)	$308,628
Average Annual Costs	$1,538,940

Figure 3.8.5.1.4 Turning basin Alternative C1

3.8.5.1.5 Build the Two Authorized Turning Basins (Almont and Southern States)

Build the authorized project, which includes 2 turning basins, one at Almont at a depth of 38 feet plus overdepth and one at Southern States at 34 feet plus overdepth. This

would require the construction and maintenance of 2 turning basins, neither of which would be optimally located for the vessel traffic in the area. This alternative is included in the authorized project and does not require additional mitigation. The costs of the authorized project alternative are given below.

First Costs	$23,089,555
Annualized (over 50 yrs)	$1,240,424
Avg Annual O&M	
for years 1-50 (see below)	$318,204
Average Annual Costs	$1,558,628

3.8.5.1.6 Comparison of Economic Costs. The basin alternatives have been presented previously and discussed. The Authorized Project would require no additional mitigation, but would require the excavation and regular maintenance of two basins. The Alternative of deepening the channel up to the Southern States basin and enlarging that basin is the most expensive Alternative and the least acceptable to the Pilots. The South Basin at Chemserve is the least cost Alternative and most preferred by the Pilots. Economic costs for each of the Alternatives are presented in the table below. Total average annual economic costs include interest and amortization of first costs over a 50-year period at an interest rate of 4 7/8 percent (factor of .053722) and average annual operation and maintenance costs.

Alternative	First Costs	Interest & Amortization	Operation & Maintenance	Total Average Annual Costs
Authorized Project	23,089,555	1,240,424	318,204	1,558,628
Alt B1 - Southern States Basin	28,539,172	1,533,189	266,551	1,799,741
Alt C3 - Chemserve South Basin	20,690,366	1,111,534	249,120	1,360,653
Alt C2 - Chemserve Middle Basin	22,169,697	1,191,007	303,840	1,494,846
Alt C1 - Chemserve North Basin	22,901,350	1,230,313	308,628	1,538,940

3.8.5.2 Evaluation of Alternative Turning Basins. Each of the above turning basin alternatives will serve the needs of the expected vessel traffic in the 38-foot and 34-foot reaches of the Northeast Cape Fear River. The existing traffic in these reaches calls at two facilities, Chemserve Terminal and Cemex. The Chemserve Terminal has about 110 to 130 vessel calls a year, and the Cemex facility about 10 to 20 vessel calls a year. Alternative turning basin locations are all designed to allow the vessels to proceed into the Northeast Cape Fear River, turn using tug assistance, and proceed to the dock. Because vessels cannot be expected to turn only when light loaded, the basins are designed to turn fully loaded vessels up to a loaded draft of 36 to 38 feet using tide assistance. If the upstream most turning basin is constructed, the entire NECFR channel will need to be deepened from 34 to 38 feet to serve the vessel traffic.

We have evaluated all the basins at the various locations and are recommending locating the Turning basin in the vicinity of Chemserve Terminal. This basin would best

serve the Chemserve and Cemex facilities, and would not require any vessel to be backed through a bridge. Future vessels calling facilities upstream of Chemserve would be required to back up or down the river channel a distance of up to 1 mile in order to turn in the proposed Basin at Chemserve. The docking pilots believe this is feasible, especially considering the infrequent traffic in the area. We received the following information from the docking pilots in relation to the recommended South Chemserve Turning Basin:

The South site offers:

1. An extended area well beyond and in line with the ChemServ Terminal. This added area will allow heavily laden vessels the much needed additional room to stop and safely dock.

2. Vessels departing ChemServ, after completing their turn to go down stream, would be positioned nearly in alignment with the Hilton RR, having little or no way (stopped).

3. After turning, the South site also offers a comparatively wide body of water in which to maneuver and align an out bound vessel with the Hilton RR Bridge.

4. From the time a vessel has all lines clear of the dock and is ready to leave the berth until the vessel is turned and aligned with the Hilton RR Bridge should average approximately twenty minutes.

While the center and north sites provide nearby turning options for both Chemserve and Cemex, the docking pilots do not believe they would be as functional and provide the following information:

The Center & North sites:

1. Do not offer any allowance to over extend a vessel's approach to the ChemServ Terminal.

2. Requires the outbound vessel, after turning, to proceed into a restrictive and narrow channel adjacent to the ChemServ Terminal while making a turn of approximately 30 degrees to alien with the Hilton RR Bridge.

3. Though the amount of time it will take to sail and turn a vessel will not be much greater, maneuvering to alien with the Hilton RR Bridge will be much more hazardous.

#s 1, 2 & 3 are based on reduced safety.

The docking pilots do not believe the northern most turning basin located at Southern States dock would be very functional. They provide the following assessment of that option:

If the costs of construction of the Southern State site was half that of the South, Center or North sites, I can not find a reason to endorse its use.

Besides being the preferred turning basin for the docking pilots, the South Turning basin at Chemserve would be the least cost alternative for the Northeast Cape Fear River portion of the Wilmington harbor project. While the 3 basins in the vicinity of

Chemserve would have similar time requirements, the upstream most basin near Southern States would require additional travel time. Much of the time in turning a vessel is in slowing and stopping the ship in order to turn it with tugs and/or thrusters. Typical turning times for ships would be approximately 1/2 hour, include slowing, turning, and re-acceleration. The upstream-most basin would take the greatest time to turn and dock. It is approximately 1 mile upstream of the basin at Chemserve, and would require 2 additional miles of travel. At a typical speed of dead slow (1 to 3 knots) in this part of the river, the time differential would be approximately one hour. At typical ship costs of about $1,000 per hour at sea, the costs for the 150 ship calls would be about $150,000 annually. Since the proposed Turning Basin is located very near the Chemserve and Cemex terminals, a least cost operation in the NECFR portion of the harbor would be expected.

These basins were evaluated using “desk-top Ship Simulation” and findings of the simulation model confirmed the location of the proposed turning basin as functional and efficient. See Appendix 3.3.5 for Ship Simulation details.

3.9 Costs of the alternative plans.

3.9.1 Cost Estimates for turning basins. Following are concept-level cost estimates for the turning basin alternatives.

Table 3.9.1a Turning Basin Costs

SOUTHERN STATES TURNING BASIN COST RANGE - - REACH 2

ALTERNATIVE 1

	CY	UNIT PRICE		AVG RX DEPTH 2.4-FT **with 20% contingency**
ROCK & SEDIMENT	907,829	23.84	**$21,642,643**	**$25,971,172**

NEW SOUTHERN TURN BASIN COST RANGE - REACH 3

ALTERNATIVE 2

	CY	UNIT PRICE		AVG RX DEPTH 7-FT **with 20% contingency**
ROCK & SEDIMENT	633,672	17.14	**$10,861,138**	**$13,033,366**

NEW MIDDLE TURN BASIN COST RANGE - REACH 3

ALTERNATIVE 3

	CY	UNIT PRICE		AVG RX DEPTH 4.3-FT **with 20% contingency**
ROCK & SEDIMENT	759,189	15.93	**$12,093,881**	**$14,512,657**

NEW NORTH TURN BASIN COST RANGE - REACH 3

ALTERNATIVE 4

	CY	UNIT PRICE		AVG RX DEPTH 4-FT **with 20% contingency**
ROCK & SEDIMENT	837,970	15.16	**$12,703,625**	**$15,244,350**

ALMONT TURNING BASIN COST RANGE - - REACH 7

ALTERNATIVE 5

	CY	UNIT PRICE		AVG RX DEPTH 7-FT **with 20% contingency**
ROCK & SEDIMENT	270,394	27	**$7,300,638**	**$8,760,766**

SHOALING RATES AND COST PER CY

	CY	UNIT PRICE		**with 20% contingency**
O&M yardage	37,021	6	**$222,126**	**$266,551**
O&M yardage	36,421	5.7	**$207,600**	**$249,120**
O&M yardage	44,421	5.7	**$253,200**	**$303,840**
O&M yardage	45,121	5.7	**$257,190**	**$308,628**
O&M yardage	46,521	5.7	**$265,170**	**$318,204**

Table 3.9.1 a – Turning Basin Costs

#REF!

ALMONT TURNING BASIN COST RANGE - - REACH 7

ALTERNATE 5

BLASTING & CLAMSHELL Removal to ODMDS

AVG RX DEPTH 7-FT
limited by vibration to 4,000sf/day

	CY	UNIT PRICE		
ROCK	172,855	67.7	$11,702,284	blast + bucket/barge
SEDIMENT	97,539	5	$487,695	pipeline
	270,394			
TOTAL PRICE			**$12,189,979**	
AVG UNIT PRICE			$45	

USE **WITH 20%contincy**
$36 **$43**

HYDRAULIC DREDGE OF ROCK WITHOUT BLASTING

	CY	UNIT PRICE		
ROCK	172,855	40	$6,914,200	
SEDIMENT	97,539	5	$487,695	
	270,394			
TOTAL PRICE			**$7,401,895**	
AVG UNIT PRICE			$27	

SOUTHERN STATES TURNING BASIN COST RANGE - - REACH 2

ALTERNATE 1

BLASTING & CLAMSHELL Removal to ODMDS

AVG RX DEPTH 2.4-FT

	CY	UNIT PRICE		
ROCK	303,739	47	$14,275,733	blast + bucket/barge
SEDIMENT	604,090	6	$3,624,540	pipeline
	907,829			
TOTAL PRICE			**$17,900,273**	
AVG UNIT PRICE			$20	

USE **WITH 20%contincy**
$20 **$24**

HYDRAULIC DREDGE OF ROCK WITHOUT BLASTING

	CY	UNIT PRICE		
ROCK	303,739	50	$15,186,950	pipeline
SEDIMENT	604,090	6	$3,624,540	pipeline
	907,829			
TOTAL PRICE			**$18,811,490**	
AVG UNIT PRICE			$21	

TOTAL RANGE OF ORIGINAL CONTRACT FOR 2-TURNING BASINS

NEW OPTIONS BELOW FOR ALTERNATE TURNING BASINS AND COSTS

OPTION 1 - NEW NORTHERN TURN BASIN COST RANGE

ALTERNATE 4

BLASTING & CLAMSHELL Removal to ODMDS

AVG RX DEPTH 4-FT

	CY	UNIT PRICE		
ROCK	160,101	73.3	$11,735,403	blast + bucket/barge
SEDIMENT	677,869	5.7	$3,863,853	pipeline
	837,970			
TOTAL PRICE			**$15,599,257**	
AVG UNIT PRICE			$19	

USE **WITH 20%contincy**
$16 **$19**

WITHOUT SHOALING & MITIGATION

HYDRAULIC DREDGE OF ROCK WITHOUT BLASTING

	CY	UNIT PRICE		
ROCK	160,101	45	$7,204,545	
SEDIMENT	677,869	5.7	$3,863,853	
TOTAL PRICE			$11,068,398	
AVG UNIT PRICE			$13	

WITHOUT SHOALING & MITIGATION

OPTION 2 - NEW MIDDLE TURN BASIN COST RANGE

ALTERNATE 3

BLASTING & CLAMSHELL Removal to ODMDS

AVG RX DEPTH 4.3-FT

	CY	UNIT PRICE		
ROCK	149,343	73.3	$10,946,842	blast + bucket/barge
SEDIMENT	597,741	5.7	$3,407,124	pipeline
	747,084			
TOTAL PRICE			**$14,353,966**	
AVG UNIT PRICE			$19	

USE **WITH 20%contincy**
$16 **$20**

WITHOUT SHOALING & MITIGATION

HYDRAULIC DREDGE OF ROCK WITHOUT BLASTING

	CY	UNIT PRICE		
ROCK	149,343	45	$6,720,435	
SEDIMENT	597,741	5.7	$3,407,124	
TOTAL PRICE			$10,127,559	
AVG UNIT PRICE			$14	

WITHOUT SHOALING & MITIGATION

OPTION 3 - NEW SOUTHERN TURN BASIN COST RANGE

ALTERNATE 2

BLASTING & CLAMSHELL Removal to ODMDS

AVG RX DEPTH 7-FT

	CY	UNIT PRICE		
ROCK	178,881	50.3	$8,997,714	blast + bucket/barge
SEDIMENT	454,791	5.7	$2,592,309	pipeline
	633,672			
TOTAL PRICE			**$11,590,023**	
AVG UNIT PRICE			$18	

USE **WITH 20%contincy**
$18 **$21**

WITHOUT SHOALING & MITIGATION

HYDRAULIC DREDGE OF ROCK WITHOUT BLASTING

	CY	UNIT PRICE		
ROCK	178,881	45	$8,049,645	
SEDIMENT	454,791	5.7	$2,592,309	
TOTAL PRICE			$10,641,954	
AVG UNIT PRICE			$17	

WITHOUT SHOALING & MITIGATION

Table 3.9.1 b – Turning Basin Costs

3.9.2 Cost Estimates for fish passage at the Locks and Dams. The estimates for the three Locks and Dams were produced with the aid of MCACES Gold Estimating Software. The data bases used were the following: Models – SAS95E – SAS models June 2004, Unit Prices – SAS95D – SAS District UPB Sept 2005, Crews – SAS95C – SAS Crews July 2004, Labor – BR0106 – Ft. Bragg 2006, and Equipment – SAS95H – SAS Equipment Sept 2005. These local data bases are the most recently updated by Cost Engineering Section of the Savannah District.

The actual cost of the rock and placement were derived from the bid opening for the construction of the Bird Island in the Savannah Harbor, which was a 2002 competitively bid construction project with many similarities to projects involving the three dams. These similarities include but are not limited to the purchase, loading and placement of filter stone, riprap, and large protective stone boulders from barges and the removal of obstacles, sheet pile, and debris from aquatic areas and last the hauling of all these materials.

Crews were formed with expected staffing and equipment, and production numbers were backed into using the average bids from the contractors, and escalating these numbers to 2007 prices. Then, production was determined from the bid prices divided by the known quantities of rock that were placed. It must be reiterated that the unit price book for MCACES is outdated, so we relied heavily upon the verifiable numbers of equipment costs, labor costs, production, and bid prices.

These estimates are not construction level detailed estimates but are order of magnitude estimates relying upon known prices and quantities instead of detailed plans and specs. Since the plans for the dams follow three distinct lines of remediation: rehab the dams, remove the dams, provide fish passage, or a combination of these features, the estimates are remarkably similar and the repetitiveness of the designs and estimates will provide a good consistent comparison, if not accurate construction dollar amounts. A table showing a matrix of the plans and their respective costs is summarized in Table 3.4b. It must be stressed that a construction estimate will be needed once the desired plan or plans have been selected.

3.9.3 Mitigation cost estimates for new turning basin.

3.9.3.1 Island 13 amplification. This alternative was eliminated.
3.9.3.2 Alligator Creek restoration. See Table 3.4b.
3.9.3.3 Island 18 restoration. This alternative was eliminated.
3.9.3.4 Island 12 restoration. See Table 3.4b.
3.9.3.5 Improved fish passage at Lock and Dam # 1 due to potential blasting. See table 3.4b.

3.10 Benefits of the alternative plans.

3.10.1 Fish passage. The Corps of Engineers is required to provide Fish Passage and Lock and Dam 1 under the Biological Opinion for the Wilmington Harbor Deepening Project. In addition, the Corps must study fish passage options at Lock and Dam 2 and Lock and Dam 3 (William O. Huske), This report will examine a number of ways to provide acceptable fish passage, and look at ancillary benefits for each method of providing passage.

The U.S. Fish and Wildlife Service (Service) has realized for several years that the Cape Fear River's striped bass, shad, sturgeon and river herring have been denied access to their spawning grounds in the Cape Fear River.
There are two main sections of spawning grounds in the Cape Fear River. The primary spawning ground is in the Cape Fear River between Erwin and Buckhorn Dam and the second section is in the Deep River above Buckhorn Dam. Both of these sections are above all three of the Cape Fear River Lock and Dams. Currently sturgeon are completely denied access to this area by the lock and dams because they will not pass through the lock chambers. A percentage of the striped bass, shad, and herring will pass through each lock but by the time they go through all three and reach the primary spawning area only a small fraction remain (around 12 percent). For example with about half the fish passing through the each lock 50 percent would pass the first dam, 25 percent the second dam and 12.5 percent the third (½ x ½ x ½). This multiplier effect can only be addressed by removing the dams or by the installation of very effective fish passage. Full rock ramp would solve that problem. Full rock ramps reduce flow velocity over dams so effectively that fish can swim over the dam. Full rock ramps are based on the very simple concept of making the dams mimic a natural rapids that fish are well adapted to pass by placing rocks downstream of the dam in a series of steps.

The Service believes that if migratory fish are allowed to easily reach the primary spawning grounds the populations of striped bass, sturgeon, shad and river herring would begin recovery. Once the fish are allowed to reach their primary spawning grounds the Service will begin to work with the owner of Buckhorn Dam to provide fish passage to allow access to the Deep River. The Service has estimated the size of recovered migratory fish populations for the Cape Fear River. These estimates were made after reviewing the historical literature and the life history requirements of these fish and are applicable for either dam removal or installation of full rock ramps at all three lock and dams.

The Cape Fear River should support a population of approximately 2,000,000 striped bass, 5,000 Atlantic sturgeon, 25,000 shortnose sturgeon, 3,000,000 pounds of American shad, and 12,000,000 pounds of river herring. The recovered populations of shad and river herring would provide an important food resource to coastal fish such as flounder, bluefish, speckled trout, and red drum because these fish would be expected to feed heavily on the small fingerling river herring and shad as they migrate back through the river and into the ocean. For

the several years the river herring and shad are in the ocean before returning as adults to reproduce in the river, these fish are important food for larger fish such as tuna and marine mammals such as whales. The Cape Fear River is North Carolina's largest river basin and the only river in North Carolina that flows directly into the ocean. If its migratory fish populations were allowed to recover by allowing the fish to reach their primary spawning grounds, the economic return to the state is expected to be large. According to the 2001 National Survey of Fishing, Hunting, and Wildlife-Associated Recreation (http://www.census.gov/prod/2003pubs/01fhw/fhw01-nc.pdf , see page 9) fishing expenditures for 2001 in North Carolina totaled 1.1 billion dollars. The Service believes that the Cape Fear River has the potential to become one of the best places to fish on the east coast if the migratory fish are allowed to reach their spawning habitat.

The numbers in the letter below were calculated by Mike Wicker with the US Fish and Wildlife Service using the amount of habitat that would be available and comparing that with known populations in other systems. The estimates were adjusted to fit using simple algebra.

The Service estimates that the Cape Fear River currently supports 10,000 Striped bass (30,000 pounds), 300 Atlantic sturgeon, 50 shortnose sturgeon, 200,000 pounds of American shad, and 1,000 pounds of river herring.

The Service recommends that the sturgeon not be included in the economic analysis so no conversion from numbers to weight should be necessary. The restoration of sturgeon is extremely important to the recovery of the endangered shortnose sturgeon and to the status of Atlantic sturgeon but neither of these supports recreational or commercial use now and may not in the future even with restored numbers.

The other species are economically valuable (both recreationally and commercially) and increases in the numbers would be expected to have a very large benefit to the economy. To convert striped bass numbers to pounds use an average weight of 3 pounds. Assume that 1/4th of the poundage of each species is harvested each year. These economic benefits will occur every year.

(Restored weight - Current weight) divided by 4 will provide annual harvestable benefit for each species.

Mike Wicker of the US Fish and Wildlife service provided the following information to help compute a value for the additional fishery resources that would be produced by the provision of fish passage at the three locks and dams.

`Probably the easiest thing to do is use price per pound simply from DMF landings data value and poundage figures from most recent year and then to note that recreational value would be substantially higher.

So for striped bass:

2,000,000 (restored population) - 10,000 (current population) = 1990000 striped bass benefit

1990000 striped bass x 3 pounds (average size) = 5970000 pounds striped bass benefit

5970000 x .25 (harvestable proportion each year) = 1492500 pounds harvestable striped bass benefit each year

1492500 x 1.94 ($ per pound based on 2005 DMF commercial data) = 2,895,450 dollars per year benefit from striped bass

for American shad:

3,000,000 (restored population in pounds) - 200,000 (current population in pounds) = 2,800,000 pounds American shad benefit

2,800,000 pounds American shad benefit x .25 (harvestable proportion each year)

700,000 pounds x 1.07 ($ per pound based on 2005 DMF commercial data) = 749,000 dollars per year benefit from American shad

for river herring:

12,000,000 pounds (restored population in pounds) - 1,000 (current population in pounds) = 11,999,000 pounds benefit

11,999,000 pounds benefit x .25 (harvestable proportion each year) 2,999,750 pounds x .52 ($ per pound based on 2005 DMF commercial data) = 1,559,870 dollars per year benefit from river herring

total per year benefit:

$2,895,450 (striped bass) + $749,000 (America shad) + $1,559,870 (river herring) = $5,204,320 per year benefit from restoring the river's anadromous fish populations

This value is conservative because recreational valuation would be much higher and this approach does not consider fish which will benefit from foraging on young herring and shad in route through the Cape Fear Estuary to the ocean such as flounder, speckled trout, red drum and bluefish which are all valuable species. However this is the best approach I think because it is the simplest with the least assumptions.

Valuing the Recreational Fishery that would be created by the increased production of this much fish stocks, including endangered species, would not be easy. Just valuing the current recreational fishing value in North Carolina is not a simple matter. From a recent wildlife article:

Estimates of recreational expenditures are many and vary wildly. Perhaps the safest estimate comes from the U.S. Fish and Wildlife Service's 2001 National Survey of Fishing, Hunting and Wildlife-Associated Recreation. That survey showed close to 700,000 recreational saltwater anglers spent $246,155,000 that year in North Carolina. For the year 1999, however, the National Oceanic and Atmospheric Administration had estimated direct expenditures of $622,641.It should come as no surprise that all those

recreational anglers also can have a big impact on fisheries. Sport fishermen have higher harvests than commercial fishermen on more than a dozen finfish species. Sometimes the harvest is dramatically higher, as in the case of striped bass, yellowfin tuna and dolphin.

Recreational Valuation (see table B4-7 in http://www.epa.gov/waterscience/316b/phase2/casestudy/chb4.pdf)

Since the recreational fishing value of landing increased fish stocks is much higher than the commercial value, using the increase in commercial value of the stocks would be a minimum value of increased production to the Nation. And increasing the abundance of the endangered short-nosed sturgeon would not be quantifiable, and there are few river basins on the Southeast Coast that would have the opportunity to produce this amount of species recovery at any price.

Removal of the Locks and Dams. The Cape Fear River has been improved for Navigation since the first Lock and Dam (Number 1) was constructed following authorization in 1910. To determine the expected conditions in the Cape Fear River following removal of the dams, old Reports of the Chief on Engineers were examined for descriptions of river conditions. To quote from the 1872 Report, "during the summer months, navigation is much interrupted by sandbars and shoals. …The bed of the river, being composed almost entirely of sand, is constantly changing from the action of freshets. During summer months, the volume of water is insufficient to give a good channel, when spread, as it is, over the whole river-bed; consequently, there is no channel, the river presenting a succession of sand-bars and shoals for nearly the whole distance of sixty-six miles." Returning to this condition by removal of the three locks and dams would have a detrimental impact on Water Supply, Recreation, Commercial Navigation and neighboring communities.

If the locks and dams were modified to provide fish passage via rock ramps, the locks and dams could be operated for Commercial and Recreational use, and water supply protected.

3.10.2 Relocation of ship turning basin in Northeast Cape Fear River.

Each of the turning basin alternatives will serve the needs of the expected vessel traffic in the 38-foot and 34-foot reaches of the Northeast Cape Fear River. The existing traffic in these reaches calls at two facilities, Chemserve Terminal and Cemex. The Chemserve Terminal has about 110 to 130 vessel calls a year, and the Cemex facility about 10 to 20 vessel calls a year. Alternative turning basin locations are all designed to allow the vessels to proceed into the Northeast Cape Fear River, turn using tug assistance, and proceed to the dock. Because vessels cannot be expected to turn only when light loaded, the basins are designed to turn fully loaded vessels up to a loaded draft of 36 to 38 feet using tide assistance. If the upstream most turning basin is constructed, the entire NECFR channel will need to be deepened from 34 to 38 feet to serve the vessel traffic.

We have evaluated all the basins at the various locations and are recommending locating the Turning basin in the vicinity of Chemserve Terminal. This basin would best serve the Chemserve and Cemex facilities, and would not require any vessel to be backed through a bridge. Future vessels calling facilities upstream of Chemserve would be required to back up or down the river channel a distance of up to 1 mile in order to turn in the proposed Basin at Chemserve. The docking pilots believe this is feasible, especially considering the infrequent traffic in the area.

The alternative turning basins considered in the vicinity of Chemserve do not have any great time or cost savings over any other basin. Much of the time in turning a vessel is in slowing and stopping the ship in order to turn it with tugs and/or thrusters. Typical turning times for ships would be approximately 1 hour, include slowing, turning, and re-acceleration. The upstream-most basin may take the greatest time to turn and dock, but since it is only 1 mile upstream of the basin at Chemserve, it would require 2 additional miles of travel. At a typical speed of dead slow (1 to 3 knots) in this part of the river, the time differential would be approximately one hour. At typical ship costs of about $1,000 per hour at sea, the costs for the 150 ship calls would be about $150,000 annually. Since the proposed Turning Basin is located very near the Chemserve and Cemex terminals, a least cost operation in the NECFR portion of the harbor can be expected.

These basins were evaluated using “desk-top Ship Simulation” and findings of the simulation model confirmed the location of the proposed turning basin as functional and efficient.

3.11 Identification of: NED Plan for new Turning Basin and NER Plan for Locks and Dams.

3.11.1 Description of NED Plan for Turning Basin. The recommended NED Plan for a new turning basin is Alternative C3, creation of middle turning basin, south, as described in 3.3.2. This is the river pilots’ preferred alternative for a new turning basin. Mitigation, as determined by the PDT, would be construction of a rock ramp below the face of Dam No. 2, or alternatively, dam removal, for fish passage.

3.11.2 Description of NER Plan for fish passage at Locks and Dams. The recommended NER Plan for the Locks and Dams is construction of rock rapids across the entire downstream faces of dams (leaving the locks operational) at all three locks and dams, as described in 3.5.2.4. Alternatively, Lock and Dam No. 2 could be removed.

3.11.3 Risk and Uncertainty. For the NED and NER Plans, there are no significant elements of risk and uncertainty related to direct endangerment of

human life or property damage. Minor elements of risk and uncertainty for alternatives other than the NED and NER Plans are as follows:

3.11.3.1 Narrow openings for fish passage at the Locks and Dams. There is a factor of risk and uncertainty related to fish passage performance of some of the alternatives, where a relatively narrow passage is provided, for example, a diversion channel or nature-like bypass. The concern would be that the percentage of fish finding and negotiating a narrow opening would be unacceptably low. However, for the NER Plan—rock ramps downstream of faces of the dams, or removal of Lock and Dam No. 2—the fish passage width would be the total width of the dams, and this concern would not apply. The ramp design would be based upon proven rock ramps that have been built in other areas of the U.S., which successfully provide a high percentage of fish passage over dams. The rock ramps would be constructed using configurations known to accommodate passage of the fish species of concern. For the NER Plan there are no significant elements of risk or uncertainty.

3.11.3.2 Effects on water supply/storage pools above the dams. Risk and uncertainty would apply to available water supply/storage above the dam(s) if the top elevations were lowered, or Locks and Dams No. 1 and 3 were removed. However, for the NER Plan, construction of rock ramps below the faces of these dams, the top elevations would remain unchanged. The NER Plan therefore would carry no significant elements of risk or uncertainty.

3.11.4 Allocation and apportionment of costs and responsibilities. [TO BE ADDED WHEN AVAILABLE.]

4.0 POLICTY ISSUES AND QUESTIONS (TO BE ADDED WHEN AVAILABLE)

4.1 Congressional Deauthorization
4.2 Disposal of Government Property
4.3 902 Limit

5 STATUS OF ENVIRONMENTAL COMPLIANCE ACTIONS, COORDINATION, DOCUMENTATION

5.1 Coordination. Extensive coordination has been conducted on this project since the agency scoping meeting held on June 30, 2005. Details of this coordination are indicated in paragraph 10 below.

5.2 Cultural resources

5.2.1 Turning basin alternatives. All project planning activities are being coordinated with the NC State Historic Preservation Officer per the terms of the National Historic Preservation Act (NHPA) and the Abandoned Shipwreck Act (ASA). NHPA is implemented per requirements specified in Section 106 as codified at 36 CFR 800, *Protection of Historic Properties*. The State's primary staff for review of underwater and maritime cultural resources is located at the Underwater Archaeology Branch at Fort Fisher Historic Site located in Kure Beach, NC. Formal coordination of selected GRR alternatives will be documented in coordination letters and/or a MOA.

Cultural resources surveys (magnetometer, side-scan sonar, and scuba) have been conducted over the existing navigation channels and areas of future channel improvements. In addition, a number of independent studies have addressed resources in nearby portions of the GRR project area. The studies accomplished to date have established the existence of the shipwrecks noted in Table 5.2.1. Each is listed with reference to the nearest basin alternative and any required documentation, should that alternative be selected. The basin alternatives are numbered as follows:

Basin C3 (west) is on the west side of the channel just upstream of Hilton Railroad Bridge and adjacent to Sigma Marine.

Basins C3 (east), C2, and C1 are on the east side of the channel, with basin C3 being the southernmost of three overlapping basins that begin upstream of Chemserve Terminal.

Basin B1 is the northernmost turning basin across from Southern States Chemical.

Table 5.2.1 KNOWN CULTURAL RESOURCES SITES		
Shipwreck	Basin	Documentation Needed
WILM3-C	C2, C1	None; under study as part of WH-NeCFR Project.
0047NER	C3 (east), C2, C1	Yes. Site within potential construction-operation impact area.
0014NER	C3 (east)	None. Site isolated from impacts by mooring dolphins and old pilings.
0046NER	C3 (east)	None. Site isolated from impacts by mooring dolphins and old pilings.
0009NER	C3 (west)	None. Site can no longer be found and is assumed destroyed.

In addition to these known sites, there are areas adjacent to some basin alternatives that have not been studied sufficiently to allow for clearance under S. 106 NHPA. Within these areas, impacts may occur as a result of construction and/or operation, and remote sensing survey is required to identify any potential cultural resources. Basins C3 (west) and B1 may not require further survey, or will require only minimal supplemental remote sensing data. Basins C3 east, C2, and C1 have considerable portions of the basin footprint surveyed, but construction impacts may result outside of the surveyed area. Therefore, a detailed review of the most recent data for these areas will be examined and further survey will be implemented to assure that areas that may be impacted by construction staging, swing anchors, or dredging.

It should be noted that, even taken together, the areas requiring further survey are small, and the remote sensing survey can be accomplished in only 2 or 3 days. However, the small size of the survey areas in no way guarantees that resources will not be discovered. If potential shipwreck sites are discovered, they will require initial diver investigation to verify the nature of the resource and its condition. For resources that warrant further attention data recovery may be required to mitigate potential adverse impacts.

5.2.2 Lock and Dam alternatives. The Cape Fear River Locks and Dams have been documented and evaluated per National Register of Historic Places criteria and requirements of the Historic American Engineering Record. The following summary is based upon studies conducted by New South Associates for the Wilmington District. These include *Phase I Archaeological Survey Cape Fear Locks and Dams and*

Figure 5.2.2a Lock and Dam No. 1

Proposed New Fish Channel (2002) and *Documentation and Assessment: Cape Fear River Locks and Dams, Bladen County, NC* (2003). The data recovery activities called for in the summary have been reviewed by the State Historic Preservation Officer and meet the intent and guidance of Sections 106 and 110 of the National Historic Preservation Act and 36 CFR 800, *Protection of Historic Properties*. Formal coordination of selected GRR alternatives will be documented in coordination letters and/or a MOA.

Figure 5.2.2b Lock and Dam No. 2

Lock and Dam 1. Five and seven-tenths acres of L&D-1 are considered eligible for inclusion on the National Register of Historic Places (NRHP). Contributing elements include the lock and dam, the Lockmaster Residence, and one original storage shed. Consequences of adverse effect: coordinate and implement MOA and Recordation Plan with the SHPO for documentation of the Residence and storage building.

Lock and Dam 1 also contains one prehistoric site that will require Phase II documentation to determine if it is eligible for inclusion on the NRHP.

Lock and Dam 2. Five and one-half acres of L&D-2 are considered eligible for inclusion on the NRHP. Contributing elements include the lock and dam, the East Lockmaster Residence, and the West Lockmaster Residence. Consequences of adverse effect: none. All coordination and documentation have been completed.

Figure 5.2.2c Lock and Dam No. 3

Lock and Dam 3. Two and three-quarters acres of L&D-3 are considered eligible for inclusion on the NRHP. Contributing elements include the lock and dam, Lockmaster Residence, tool house, pump house, hydrant house, and garage. Consequences of adverse effect: coordinate and implement MOA and recordation plan with the SHPO for documentation of the Residence and other buildings.

5.3 Estuarine and riverine resources. As indicated in paragraphs 3.7.1 and 3.7.2 a diverse aquatic resource is present in the Cape Fear River system. This resource will be enhanced by improved anadromous fish passage at Lock and Dam # 1. PNA will be impacted by a relocated turning basin, but a sound mitigation plan will compensate for this loss.

5.4 Terrestrial resources. As indicated in paragraph 3.7.3, no significant terrestrial resources will be impacted. If Island 12 is restored, several sandy/marl covered domes will be altered and a low shrub ticket removed to restore PNA. If diversion channels for fish passage are constructed at the locks and dams, or a nature-like bypass constructed at Lock and Dam # 1, mature forest will be removed. None of this habitat is rare and the impact is considered minor compared to enhancing anadromous fish spawning success.

5.5 Endangered and Threatened species. As indicated in paragraph 3.7.4, the only listed species in the project area is the endangered shortnose sturgeon. Fish passage is being required under the Endangered Species Act to increase the passage of the shortnose sturgeon. Therefore the fish passage method chosen should benefit the species.

Construction of the relocated turning basin may require blasting which could impact the shortnose sturgeon. However, the rock in the selected southern basin is less dense than the currently authorized basins (Almont and Southern States). Therefore the new basin should have less need for blasting which would decrease the potential impact on the species compared to the currently authorized project.

5.6 Recreation and aesthetic resources. When fish passage is constructed at Lock and Dam # 1, more anadromous fish will go further upstream and have greater spawning success. This would eventually result in greater fisheries resources and greater recreational fishing activities. If the locks and dams are not altered by fish passage (e.g. rock ramp, diversion channel, nature-like bypass) recreational boating activities should not be adversely impacted. If the dams are lowered or removed, traffic in the river could be restricted during low flow conditions due to low river levels. If a rock ramp was placed in the lock chamber, vessels could no longer be locked to the upstream pool.

Aesthetic conditions would not be significantly altered. If the dams were lowered or removed, the river would quickly return to its historic conditions. Any mud banks exposed along the edge of the river should quickly become vegetated. A rock rapids or a diversion channel would appear to most individual as natural rapids, and a nature-like bypass as a natural stream.

If the Cape Fear Locks and Dams are deauthorized, we anticipate that the parks and associated lands would be divested to a public entity such as the State of North Carolina and that the parks would remain operational.

There should be no recreational or aesthetic impacts associated with the construction of the turning basin. Since the turning basin would be underwater and the water clarity is low, the new turning basin would not be noticed. Shallow draft vessels already turn near Chemserve Terminals so vessel turning there are not an unusual site. Most people will not observe the construction at the mitigations sites on Eagle Island or Island 12 since they are remote. Also once construction is complete, the restored areas will look natural.

5.7 Sediment and Contaminants. Assessment of the sediment contamination has been or will be conducted at all project sites. There are no regulations or sediment standards that dictate the approach used in evaluating potential sediment contamination at dam sites. However, there are well-established procedures for evaluating contaminant-related impacts from sediments. These effects-based procedures are useful in evaluating potential sediment contaminant pathways and impacts. U.S. Army Corps of Engineers (USACE) and the Environmental Protection Agency (EPA) guidance documents were utilized.

Based on the results of testing of sediments from the existing Northeast Cape Fear River basin (section 3.7.4.2), those sediments do not contain contaminants in concentrations that would preclude dredging and appropriate disposal including ocean disposal. EPA concurred by letter dated September 7, 2005 that the sediments tested in the existing Northeast Cape Fear River portion of the Wilmington Harbor Federal Navigation project meet the EPA's Ocean Dumping Regulations and Criteria (40 CFR 220-228).

Evaluation of sediments from the proposed alternative turning basin location has not been performed but is required (section 3.7.4.2). The industrial areas adjacent to the alternative turning basins warrant site-specific evaluation of sediments which may be dredged during basin construction. Testing procedures for evaluating contaminant-related impacts from specific turning basin sediments proposed for dredging as previously discussed will be used and reported in the DEIS (section 3.7.4.2).

Based on the results of sediment chemistry and toxicity tests, contamination in surface sediments behind the Cape Fear River Locks and Dams is unlikely to be a concern.

5.8 Dredging window. For the locks and dams, the only excavation that may be required for fish passage would be if a diversion channel or nature-like bypass was constructed. All of this work would be upland except where the fish exit and entrance would connect to the river. Connection may cause turbid water to enter the river and excavation at the connection may increase turbidity and suspended solids. Therefore, excavation would be done in the upland first and connection to the river last. The connection would not be made during the anadromous fish spawning season (February – mid-June). If a rock ramp is constructed, placement of rock in the river and associated activities would also avoid the spawning season.

The dredging window in the vicinity of the relocated turning basin and PNA location (Island 12 and Alligator Creek) is from August 1 through January 31. In water activities would be avoided outside that period. However restoration work on the interior sections of Island 12 and Alligator Creek could be conducted year-round. Connection may cause turbid water to enter the river and excavation at the connection may increase turbidity and suspended solids. Therefore, connection to the primary nursery areas will only occur during the window.

5.9 Environmental mitigation. The August 2000 NMFS BO under the Endangered Species Act required that fish passage be constructed at Lock and Dam # 1 and be studied at Lock and Dams 2 & 3 (see paragraph 3.5.1). This was required due to the potential impacts of blasting rock to deepen the authorized Wilmington Harbor Project. Mitigation is also required for the relocated turning basin in the Northeast Cape Fear River (paragraph 3.4).

5.10 USFWS Recommendations. Draft Coordination Act is expected in October 2007.

5.11 NEPA Documentation. A draft and final environmental impact statement will be integrated in the feasibility report required for the GRR. The draft report should be published in spring 2008, the final in summer 2008, and the Record of Decision signed in spring 2009.

6 INDEPENDENT REVIEWS

6.1 Peer Review Work Plan. To be added when available.

6.2 Independent Technical Review. To be added when available.

6.3 Value Engineering. The Value Engineering Study Summary Report is presented in Appendix 6.3.

7 LEGAL REVIEW DOCUMENTATION. To be added when available.

8 STATUS OF ONGOING TECHNICAL ACTIVITIES. (See Appendix 8 for overall engineering considerations.)

8.1 Relocation of Ship Turning Basin in Northeast Cape Fear River. One of the considered options for turning basin mitigation, the Eagle Island stream restoration concept design, is based upon aerial LIDAR data and should require some field verification of the data. This concept design as well as Island 12 restoration will continue to be refined as better data is acquired or verified. Refinements will also occur as the design is coordinated with the appropriate environmental agencies.

8.2 Fish Passage.

8.2.1 Lock wall stability at all three locks and dams. The river walls of the three Locks will have a stability analysis completed to determine the effects of the weight of the rock from the rock rapids. The lock wall stability analysis will be conducted in accordance with the requirements of EM 1110-2-2602 “Planning and Design of Navigation Lock Walls and Appurtenances”, EM 1110-2-2502 “Retaining Wall”, and EM 1110-2-2406 “Design of Pile Structures and Foundations”.

8.2.2 Rock rapids across entire downstream face of dam at all three locks and dams. The three dams: Dam No. 1, Dam No. 2, and the William O. Huske Dam (#3) will have a stability analysis completed to determine if they are stable in their present condition. The three dams will also have stability analysis completed, which will include conditions with rock rapids. An additional analysis will be preformed for Lock and Dam # 3 which will assume that #2 is removed. The dam stability analysis will be conducted in accordance with the requirements of EM 110-2-1902 “Slope Stability” and will utilize the U-Texas 4 Stability Analysis Program.

8.3 Municipal water supply intake evaluation. A study of five existing municipal water supply intake structures and pump stations was conducted by the Corps of Engineers. The study consists of evaluating the impacts to the existing pump stations if: 1) the dam were removed, 2) the dam were lowered so that there would be reduced water supply capacity, and 3) the dam were lowered to an elevation that would allow fish passage without construction of rock rapids. See Appendix 3.8.1 for details of the Corps intake evaluation.

Also see section 3.5.3.3.

8.4 Planning Models. Weighted spreadsheet matrices have been used as tools for PDT discussions, evaluations, and development of recommendations. Although these spreadsheets are best understood by PDT members as a whole, IWR Plan is the certified planning model being used for final adaptation of the team’s analyses and recommendations.

9 REAL ESTATE. To be added when available.

9.1 Real Estate Overview
9.2 LERRDs
9.3 Sponsor responsibilities
9.4 Easement Revision
9.5 Real Estate Estimate
9.6 Remaining Real Estate Work
9.6.1 Gross Appraisal Revision
9.6.2 Staging Area Identification
9.6.3 Sponsors Capability
9.6.4 Code of Accounts

10 ENVIRONMENTAL COMPLIANCE, COORDINATION, AND RESOURCE AGENCY VIEWS.

The project is designed to be fully compliant with all environmental requirements including NEPA, the Endangered Species Act, Sections 404 and 401 of the Clean Water Act, National Historic Preservation Act, etc. Coordination for this project was initiated with an agency scoping meeting in June 2005, and continued with at least monthly PDT meetings beginning in December 2005. In addition to the Corps, the PDT members are from state and federal agencies, water users, and citizen groups. These meetings are opened to the public and usually at every meeting there are several environmental and water user representatives present. In addition, we hold quarterly progress meetings to inform all interested parties on the status of the project. Besides Corps personnel, these meeting generally have 15-20 additional attendees. We also have conference lines available for those who can not attend but desire to participate. Minutes from all these meeting are posted on the Wilmington District website and other appropriate information is also posted there such as surveys, reports, and PowerPoint presentations given at the quarterly meetings (http://www.saw.usace.army.mil/wilmington-harbor/GRR/GRR.htm). Also numerous presentations have been made to local governments, organizations and citizen groups.

All monthly meetings are held the third Wednesday of every month, and the quarterly meetings are also generally held on the third Wednesday. Reminder notices of these meeting are emailed to about 100 individuals.

In addition to these monthly and quarterly meetings, weekly meetings were held from mid-April through mid-May 2007 to tentatively select the best fish passage and turning basin alternatives, and to select the mitigation plan for impacts related to the selected turning basin.

The Corps has received several resolutions from local governments and organizations regarding the GRR project. These resolutions fall into three basic categories:

1. Support fish passage at the locks and dams, but indicated opposition to removal of any locks and dams (Cape Fear River Assembly, Town of Elizabethtown, Lower Cape Fear Water and Sewer Authority);
2. Request thorough coordination and evaluation of all pertinent factors including water supply, economic and financial impacts to citizens (City of Wilmington);
3. Support construction of a rock ramp for fish passage at Lock and Dam #1 (Lower Cape Fear Water and Sewer Authority, City of Wilmington).

All the resolutions are included in appendix 10.

11 PROJECT MILESTONES AND COMPLETION DATES.

Milestone	Completion Date
Scoping Meeting	June 30, 2005
Alternative Formulation Briefing	September 2007
Circulated Draft Feasibility Report/EIS	March 2008
Circulated Final Feasibility Report/EIS	August 2008
Record of Decision Signed	May 2009

12 STATUS OF MCACES ESTIMATE. Refer to Appendix 12 for the current version of the MCACES Estimate.

13 HEADQUARTERS GUIDANCE MEMORANDUM. To be added when available.

14 HEADQUARTERS GUIDANCE COMPLIANCE MEMORANDUM. To be added when available.

15 STATUS OF NON-FEDERAL SPONSOR SUPPORT. The State of North Carolina, Department of Environment and Natural Resources, Division of Water Resources, is the non-Federal sponsor of this GRR. Following is a letter from this agency expressing their interest, and willingness to provide 25 percent of the cost of implementing the recommendations of the GRR.

NCDENR
North Carolina Department of Environment and Natural Resources
Division of Water Resources

Michael F. Easley, Governor

William G. Ross Jr., Secretary
John Morris, Director

May 9,2007

Colonel John P. Pulliam
District Engineer
US Army Corps of Engineers
PO Box 1890
Wilmington, NC 28402-1890

Dear Colonel Pulliam:

The State of North Carolina is pleased with the progress of the General Re-evaluation Report (GRR) for Wilmington Harbor, North Carolina The GRR is addressing essential issues of turning basin construction and of fish passage at the Cape Fear River locks and dams.

The State of North Carolina, as the non-federal sponsor of the Wilmington Harbor Project, intends to provide 25 percent of the cost of implementing the recommendations of the GRR through general fund appropriations by the North Carolina General Assembly, subject to North Carolina's statutory and constitutional procedures for such commitments. The State of North Carolina has statutory authority under the Federal Water Resources Development Law of 1969 (G.S. 143-215.38 et. seq.) to make binding commitments to carry out the non-federal responsibilities related to Corps of Engineers projects, including making cash contributions to projects.

We appreciate the opportunity to work closely with the Wilmington District toward the implementation of these most important elements of the Wilmington Harbor Project.

Sincerely,

John N. Morris

10hn N. Morris

lM!k.rn

1611 Mail Service Center, Raleigh, North Carolina 27699·1611
Phone: 91 9-733-4064 \ FAX: 919-733-3558 \ Internet: www.ncwater.org

An Equal Opportunity / Affirmative Action Employer - 50% Recycled \ 10% Post Consumer Paper

One North Carolina Naturally

16.0 REFERENCES.

CZR. 2002. Preconstruction monitoring of American Shad, Atlantic Sturgeon and Striped Bass Passage at Lock and Dam # 1 on the Cape Fear River, NC. Year 1. Prepared for the Wilmington District Corps of Engineers. http://www.saw.usace.army.mil/wilmington-harbor/FINALbrunswick2002.pdf

CZR. 2003. Preconstruction monitoring of American Shad, Atlantic Sturgeon and Striped Bass Passage at Lock and Dam # 1 on the Cape Fear River, NC. Year 2. Prepared for the Wilmington District Corps of Engineers. http://www.saw.usace.army.mil/wilmington-harbor/Final_Report_2003.pdf

CZR. 2004. Preconstruction monitoring of American Shad, Atlantic Sturgeon and Striped Bass Passage at Lock and Dam # 1 on the Cape Fear River, NC. Year 3. Prepared for the Wilmington District Corps of Engineers. http://www.saw.usace.army.mil/wilmington-harbor/Report%202004%20no-apps.pdf

Dial Cordy and Associates. 2006. Cape Fear River Anadromous Fish Larvae and Egg Survey. Prepared for the Wilmington District Corps of Engineers. http://www.saw.usace.army.mil/wilmington-harbor/GRR/GRR_files/Cape%20Fear%20River%20Anadramous%20Fish%20Larvae%20and%20Egg%20Survey%20Final.pdf

Giese, G.L., Wilder, H.B., and Parker, G.G. 1985, Hydrology of Major Estuaries and Sounds of North Carolina: U.S. Geological Survey Water-Supply Paper 2221.

Hall, J. R., Moser, M. L., A. M. Darazsdi. 1998. Improving passage efficiency of adult American shad at low-elevation dams with navigation locks. Completion report to the NC Sea Grant Fisheries Resource Grant Program.

Malcolm Pirnie. 2007. Technical Memorandum, US Army Corps of Engineers General Reevaluation Report for the Wilmington Harbor Deepening Project, Review of the Corps' Water Supply Intake Evaluation. Project No. 3141-001. Prepared for Mick Noland Fayetteville Public Works Commission.

Moser, M. L., A. M. Darazsdi, and J. R. Hall. 2000. Improving passage efficiency of adult American shad at low-elevation dams with navigation locks. North American Journal of Fisheries Management 20:376-385.

Nichols, P.R. and E.D. Louder. 1970. Upstream passage of anadromous fish through navigation locks and use of the stream for spawning and nursery habitat, Cape Fear River, North Carolina, 1962-66., U.S. Fish and Wildlife Service, Circ. 352.

DRAFT

NMFS, Southeast Region. 2000. Biological Opinion, Preconstruction Modifications of Authorized Improvements (Involving Blasting and Dredging) in Wilmington Harbor, North Carolina.

NMFS. 2007. Personal communication, Ron Sechler National Marine Fisheries Service June 26, 2007.

Schwartz, F.J., W.T. Hogarth, and M.P. Weinstein. 1981. Marine and freshwater fishes of the Cape Fear River estuary, North Carolina, and their distribution in relation to environmental factors. Brimleyana No. 7:17-37. July 1981.

US Army Corps of Engineers, Mobile District. 2007. Wilmington Harbor GRR Lock and Dam Study Intake Evaluation. http://www.saw.usace.army.mil/wilmington-harbor/GRR/GRR_files/Intake%20EvaluationFinaApr1607l.pdf

US Army Corps of Engineers, Savannah District. 2002. New Savannah Bluff Lock and Dam Project Savannah River Georgia and South Carolina Fish Passage Facility Engineering Report, prepared by Framatone ANP DE&S, Inc. http://www.saw.usace.army.mil/wilmington-harbor/GRR/GRR_files/New%20Savannah%20L&D%20Fish%20Passage_Dec%2002.pdf

US Army Corps of Engineers, Wilmington District. 2000. Environmental Assessment, Preconstruction Modifications of Authorized Improvements, Wilmington Harbor, North Carolina.

US Army Corps of Engineers, Wilmington District. 2002. Environmental Assessment, Fish Bypass at Lock and Dam #1, Cape Fear River, Bladen County, North Carolina.

US Army Corps of Engineers, Wilmington District. 2003. Finding of No Significant Impact, Fish Bypass at Lock and Dam #1, Cape Fear River, Bladen County, North Carolina.

US Army Corps of Engineers, Wilmington District. 2007. National Marine Fisheries Service, Threatened and Endangered Species likely to occur in the project area. Email June 26, 2007.

Walburg, C.H. and P.R. Nichols. 1967. Biology and management of the American shad and status of the fisheries, Atlantic coast of the United States, 1960. U.S. Fish and Wildlife Service, Spec. Sci. Rpt.-Fish. No. 550. pp. 35-38.

DRAFT

APPENDICES

(See separate “Appendices” volume for text of appendices)

1.1	HQUSACE Authority Concurrence Email
3.3.5	Ship Simulation
3.4	Environmental Appendix – Mitigation Analysis
3.7.10.1	Geotechnical
3.7.10.2	Review of Ground Water Studies
3.7.11	Turning Basins Shoaling Analysis
3.8.1	Intake Evaluation
3.8.3	Hydraulics
6.3	Value Engineering Study Summary Report
8	Engineering Considerations
10	Resolutions
12	MCACES Estimate

DRAFT

Wilmington Harbor

96 Act General Re-evaluation Report (GRR)
for Relocated Turning Basin and Remaining Mitigation Features

Alternative Formulation Briefing
Preconference Materials: **APPENDICES**
August 2007

Chemserve turning basin site, Northeast Cape Fear River

US Army Corps of Engineers
Wilmington District

APPENDICES

1.1	HQUSACE Authority Concurrence Email
3.3.5	Ship Simulation
3.4	Environmental Appendix – Mitigation Analysis
3.7.8	Saltwater Intrusion
3.7.10.1	Geotechnical
3.7.10.2	Review of Ground Water Studies
3.7.11	Turning Basins Shoaling Analysis
3.8.1	Intake Evaluation
3.8.3	Hydraulics
6.3	Value Engineering Study Summary Report
8	Engineering Considerations
10	Resolutions
12	MCACES Estimate

Appendix 1.1

HQUSACE AUTHORITY CONCURRENCE EMAIL

Concur with one clarification. The statement " the appropriate requests and approvals would be required to deauthorize L&D #1 and any effected upstream navigation" is too vague. What is required to deauthorize navigation is for Congress to enact legislation.

From: Haggett, Sharon F SAW
Sent: Tuesday, June 28, 2005 5:50 PM
To: Fach, George N JR HQ02
Cc: Osborne, William H SAD; Paynes, Wilbert V SAD; Long, Coleman SAW; Haggett, Sharon F SAW; Colella, Samuel J SAW; Medlock, James M SAW; Wood, Ben SAW
Subject: Wilmington Harbor- 96 Act GRR on Mitigation Features

Skip -

Per multiple e-mails and our phone conversations on Thursday, 16 June and today, June 28; HQUSACE, CESAD, and CESAW supports moving forward with a General Re-evaluation Report (GRR) on the remaining mitigation features to be implemented on the Wilmington Harbor - 96 Act Navigation project. It is understood that this is in accordance with policy guidance letter recently issued for GRRs and also in accord with the Planning Guidance Notebook (ER -1105-2-100). Appendix G - Planning Reports and Programs.

This GRR will be cost shared with the non-Federal sponsor, the State of North Carolina, in accordance with the currently executed Project Cooperation Agreement. It is anticipated that the GRR effort will take approximately 36 months to complete at a cost of between $1.2 to $1.5 million. The GRR will encompass the full efforts of a feasibility level document in a re-evaluation of the scope of the mitigation in the authorized project. In approximately 18 months from initiation (July 2005) of the GRR , an Alternative Formulation Briefing will be conducted with HQUSACE and SAD PDT members.

As the GRR alternatives are developed and a recommendation is sent forward for approval, CESAW understands the level of approval required will be dependent on the recommended alternative. In particular, should the GRR recommend a mitigation feature for the required fish passage at Lock & Dam #1 be a feature that would negate the authorized purpose of navigation at L&D 1 then the appropriate requests and approvals would be required to deauthorize L&D #1 and any effected upstream navigation features (L&D 2 and/or L&D 3) prior to implementation of the recommended mitigation alternative.

Phone conversation participants were:

HQUSACE - Skip Fach, CECW-SAD

SAD - Bill Osborne, CESAD-CM-C (SAW DST), Wilbert Paynes, CESAD-CM-P

SAW - Coleman Long, CESAW-TS-P, Frank Yelverton, CESAW-TS-PE, Sharon Haggett, CESAW-PM-C

Request you provide concurrence with this memorandum.

Thanks,
Sharon

Sharon F. Haggett, P.E.
Senior Project Manager
U.S. Army Corps of Engineers,
Wilmington District
P.O. Box 1890
Wilmington, North Carolina 28402
Phone (910) 251-4441
Fax (910) 251-4653

****
****

As background, the following indicates what was authorized, what has been done, what has not been done, and what will be evaluated in the GRR and why.

The original study authority for Wilmington Harbor was adopted 8 September 1988 by the House, and in summary indicates that the study should determine whether any modifications are advisable "with particular reference to the commercial navigation needs from the Atlantic Ocean to the upper ends of navigation on the Cape Fear River above Wilmington to Fayetteville, North Carolina, and to Kornegays Bridge on the Northeast (Cape Fear River) River." The final feasibility report for the Wilmington Harbor study was published in June 1996, and WRDA 1996 authorized the Wilmington Harbor - 96 Act construction.

The authorized project consists of deepening the ocean bar and entrance channels from the authorized depth of 40 feet to 44 feet; deepening the authorized 38-foot project to 42 feet up to and including the anchorage basin immediately upriver from the State Ports Authority dock, and extending the anchorage basin northward by 300 feet; the existing 400-foot wide channel will be widen to 600 feet over a total length of 6.2 miles including Lower and Upper Midnight and Lower Lilliput reaches; widen five turns and bends by 100 to 200 feet providing a total average channel width of 500 to 675 feet; widening the Fourth East Jetty Channel to 500 feet over a total length of 1.5 miles; deepening the 32-foot channel between Castle Street and the Hilton Railroad Bridge, the 32-foot turning basin just above the mouth of the Northeast Cape Fear River on the west side to 38 feet, and the 25-foot channel from the Hilton Railroad Bridge to 750 feet upstream all to a depth of 38 feet; deepening the 25-foot channel from 750 feet upstream of the Hilton Railroad Bridge to the turning basin near the upstream limits of the project to 34 feet, along with widening of the channel from 200 to 250 feet; and widening the turning basin from 700 to 800 feet; and mitigation.

To date, the project has seen completion of approximately 33 of the authorized 37 miles of authorized channel deepening (or up to the N.C. State Port Authority facility that is located south of downtown Wilmington), and the passing lane is under construction. The remaining construction contract is referred to as the Northeast Cape Fear River portion, which begins just north of the N.C. State Port Authority turning basin and goes through the densely developed area of downtown Wilmington. It is scheduled for contract award in April 2006 to accommodate structural inventory requirements prior to commencement of dredging within the required environmental window which begins in August and continues for six months or through January of the following calendar year. It is anticipated this portion of the deepening work will require at least three environmental windows due to the significant amount of rock removal required.

The project resulted in two environmental impacts that required mitigation.

First, construction would result in the loss of wetlands and primary fish nursery areas. To mitigate for this impact, a 30 acre portion of a disposal island was altered to create intertidal wetlands and primary nursery area. Another component of this mitigation would be acquisition by the non-Federal sponsor, State of North Carolina, of 700 acres (+/-) of prevention of degradation (POD) lands on the Northeast Cape Fear River. This acquisition has not occurred to date.

The second aspect of mitigation is for potential impacts to the endangered shortnose sturgeon resulting from blasting to remove rock. Through discussions with National Marine Fisheries Service (NMFS), condition number 8 of the 3 August 2000 biological opinion (BO) indicated that "The COE must construct a fish passage structure at Lock and Dam #1 on the Cape Fear River. The COE and all other interested agencies and experts must be involved in developing the best design for the fish passage structure." With this condition, NMFS gave the Wilmington District a take limit of two shortnose sturgeon. Without this condition, NMFS would have given us a take of zero which would have effectively precluded blasting and the successful completion of the project. Blasting to date has not resulted in the take of a sturgeon. The type of fish passage structure was not specified in the BO, and an Environmental Assessment /Finding Of No Significant Impact (EA/FONSI) was prepared to fully coordinate alternatives. The alternative selected was the construction of a 3,800' (+/-) fish passage channel around Lock & Dam #1 on the Cape Fear River at a cost of approximately $3,000,000. This project has not been constructed to date due to funding constraints and real estate acquisition not being completed by the non-Federal sponsor. When selecting the fish passage alternative to be constructed, removing or altering the lock and dam was not considered since that would have effectively caused the authorized purpose of navigation to be eliminated at Lock & Dam #1 Structure thus eliminating the requirement for navigation at the other two upstream locks and dams.

The final mitigation action will be for a relocated turning basin in the Northeast Cape Fear River. This mitigation has not been coordinated yet since this is a recent request by the Docking Pilots and supported by the non-Federal project sponsor, State of North Carolina, and the major stakeholders, the city of Wilmington and the users. Also, the resource agencies have been hesitant to meet to discuss the relocated turning basin alternatives due to what is perceived to be lack of progress on the Corps' commitments for fish passage. However at this point, the agencies have indicated that they would prefer a study such as this GRR that would consider other alternatives to fish passage, such as modification or removal of one, two, or all three locks and dams.

SAW believes the GRR is the correct mechanism to re-study mitigation opportunities because it has been approximately 9 years since the project was authorized under the Wilmington Harbor - 96 Act. There is great potential for changes in the technical solutions for the required mitigation that may have a significant cost savings to the project and will meet the intent of the Corps' commitments to provide the required mitigation. This may include recommendations to modify or remove some or all of the locks and dams on the Cape Fear River. This is a potential recommendation because minimal commercial traffic has used the locks and dams since 1996 and no commercial traffic is expected. Because of the lack of traffic, the navigation channel to the lock and dam has not been maintained.

END

Appendix 3.3.5

SHIP SIMULATION

MEMORANDUM FOR Commander, US Army Engineer District, Wilmington (CESAW-TS-EC/Mr. Greg Williams), PO Box 1890, Wilmington, NC 28402-1890

SUBJECT: Assessment for Conducting Ship Simulation Study for Cape Fear River Channel

1. The US Army Engineer District, Wilmington (SAW) has requested the US Army Engineer Research and Development Center (ERDC) to assess the need for conducting an additional ship simulation study. This study is to explore combining and relocating the two existing turning basins on the NE Cape Fear River channel to a location above the two bridge structures which span the navigation channel that is more proximal to channel users. This would allow a greater degree of safety as the pilots would be able to turn the ships prior to traversing outbound through these structures.

2. In conjunction with this proposal, SAW contracted with Taylor Engineering North Carolina to conduct a shoaling analysis to determine possible alternative locations for the turning basin. The purpose of the report is to identify potential areas for a replacement turning basin which would be less susceptible to shoaling and thus prove more economical by reducing the effort required to maintain project depth via maintenance dredging. Taylor Engineering (Albada and Carvalho, 2005) published a report on behalf of SAW outlining a potential location of a new turning basin downstream (south) of the existing turning basin. Figure 1 is an aerial photo of the project area and shows the general area of the existing turning basin and proposed turning basin locations. The proposed turning basin has three alternatives, each at the same general location with an 800-ft area but slightly different geometries. The configurations investigated are shown in Figure 2-1 and 2-2, taken from Albada and Carvalho. Based on the conclusions in the shoaling analysis, Option C3 as shown in Figure 2-2 was recommended as the new turning basin location in the report.

3. Engineering Regulation 1110-2-1403. Engineering and Design: Studies by Corps Hydraulic and Hydrologic Facilities and Others – Paragraph 6b states that "Hydraulic design studies associated with the planning, design, construction, operation, and maintenance of navigation channels will include a ship-simulation investigation unless omission of such an investigation is approved by HQUSACE. This policy does not pertain to the design of small-craft harbor channels." Simulator studies are required to ensure that proposed improvements adequately address navigation concerns in the project areas and to ensure optimization of design. The SAW has not implemented the authorized project in the existing turning basin and has concluded that the proposed relocation of the authorized basin will improve navigation conditions by allowing vessels to be turned prior to departure, allowing them to be turned under either their own power, or with tug assistance. Presently, they require tug assistance. The Cape Fear Docking Pilots,

Inc. and the Wilmington Docking Pilots Association have also indicated their preference to this location in a letter to SAW. Therefore, SAW has requested a waiver of the requirements to conduct a simulation study. The ERDC conducted a desktop study to evaluate SAW's request.

4. On behalf of SAW, ERDC has reviewed the report written based on the original simulation study conducted in 1999 at the STAR Center (Webb, 1999). This simulator study was conducted under supervision of Mr. Dennis Webb, P.E. of ERDC, Coastal and Hydraulics Laboratory. During this simulation study, simulations were conducted in different reaches of the navigation channel using several design vessels. One of the reaches tested was the uppermost portion of the navigation channel which included the then proposed turning basin which has since been authorized. The design vessel used in the simulation was a 606-ft x 88-ft chemical tanker drafting 28-ft at high tide. Figure 3, taken from Webb, 1999, shows the track plot of that vessel including a turn in the authorized turning basin. While it is evident that the vessel left the channel during the turning maneuver, it is also readily evident that the basin provided sufficient area to complete the turning maneuver. The channel exit in the STAR Center simulation was likely due to inability of the pilot to determine the exact vessel location with respect to the dredged turning basin. This issue is easily addressed by locating additional aids-to-navigation (ATONS) in the turning basin in order to assist pilots in determining the exact location of the vessel with respect to the channel lines. In order to compare the authorized turning basin with the proposed turning basin, the track plots in the authorized turning basin from the STAR Center simulation were translated to the proposed turning basins, C1, C2, and C3 as shown in Figure 2.2. These translations are shown in Figures 4-1, 4-2, and 4-3, respectively. Figures 4-1 through 4-3 clearly indicate that the design vessel can conduct the turning maneuver in any of the proposed turning basins given current magnitudes are similar.

5. Additionally, ERDC reviewed the shoaling analysis report prepared by Taylor Engineering for SAW. As part of the shoaling analysis, a RMA2 hydrodynamic investigation was conducted for each of the alternatives proposed by SAW. Based on the information in the shoaling analysis, the magnitude of the velocities in the area of the proposed turning basin location for all plans are less than the velocity vectors in the authorized turning basin. There are some slightly increased velocities noted in areas outside the proposed basins. However, inside the proposed basins only decreases in velocities occur.

6. Based on the results of the hydrodynamic model included in the shoaling analysis report and a review of the simulator study conducted during the existing authorizing of the turning basin, ERDC concurs with SAW's conclusion that the proposed relocation of the Cape Fear River turning basin will likely improve navigation in the area and a simulation study is not necessary to evaluate the proposal's safety based on the following justification.

a. The proposed turning basin will maintain the size of the authorized basin, which is larger than the existing turning basin, and has been proven adequate in size based on the 1999 STAR Center simulation study.

b. Velocity magnitudes inside the proposed turning basin are significantly lower than those in the authorized turning basin which should result in improved tug effectiveness during turning maneuvers.

c. The Cape Fear Docking Pilots, Inc. and the Wilmington Pilots Association both support relocation of the turning basin to the proposed location.

d. Navigation conditions during turning maneuvers in the proposed basins should be very similar and potentially more easily executed than those in the existing turning basin.

7. If you have any questions, please contact Mr. Tim Shelton, P.E. at (601) 634-3012 or Mr. Dennis Webb, P.E., Chief, Navigation Branch at (601) 634-2455 regarding the desktop simulator study.

William D. Martin, PhD for
THOMAS W. RICHARDSON
Director

7 Encls

Figure 1. Location of Cape Fear River existing turning basin.

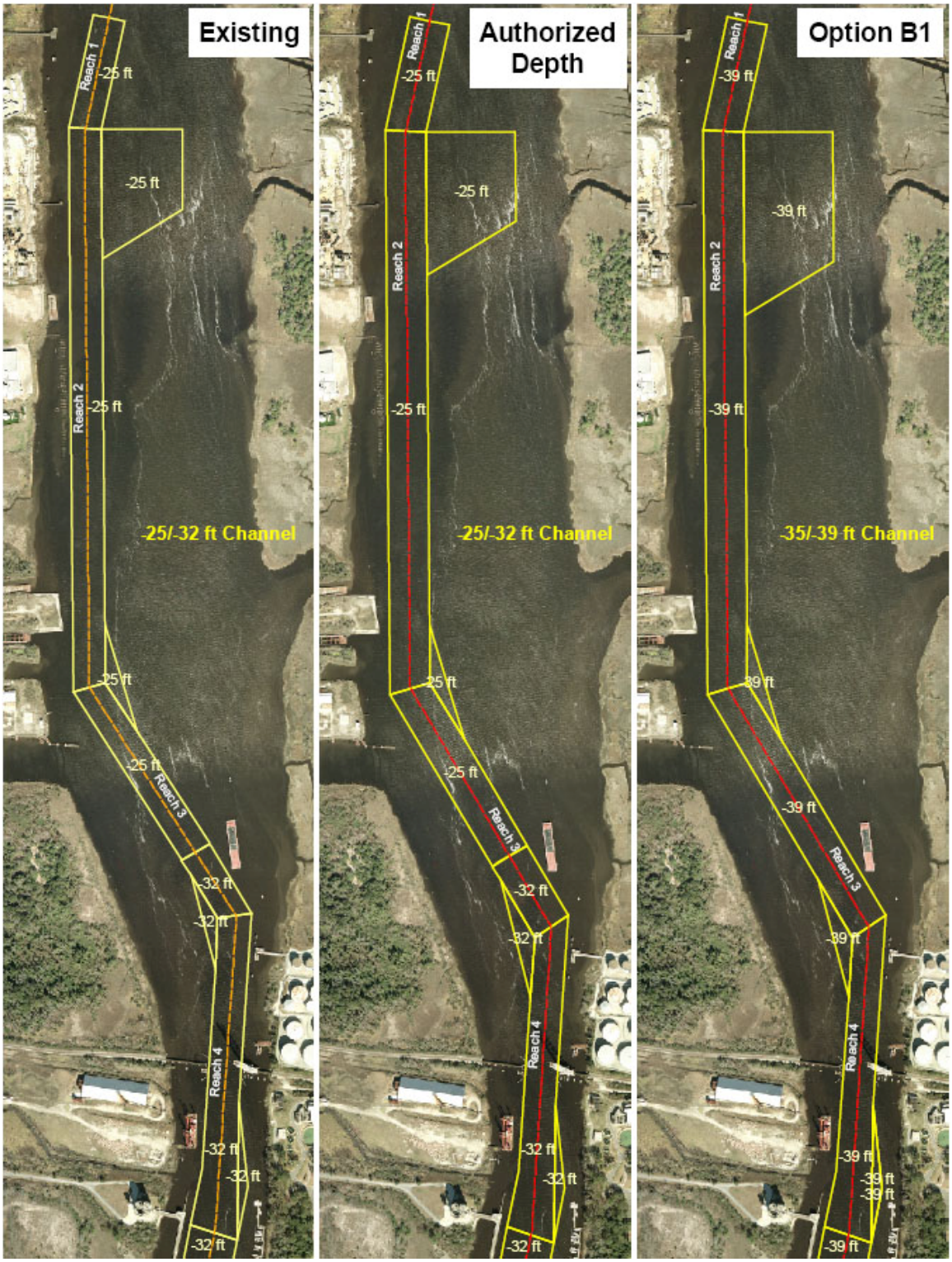


Figure 2-1. NE Cape Fear River Turning Basin alternatives.

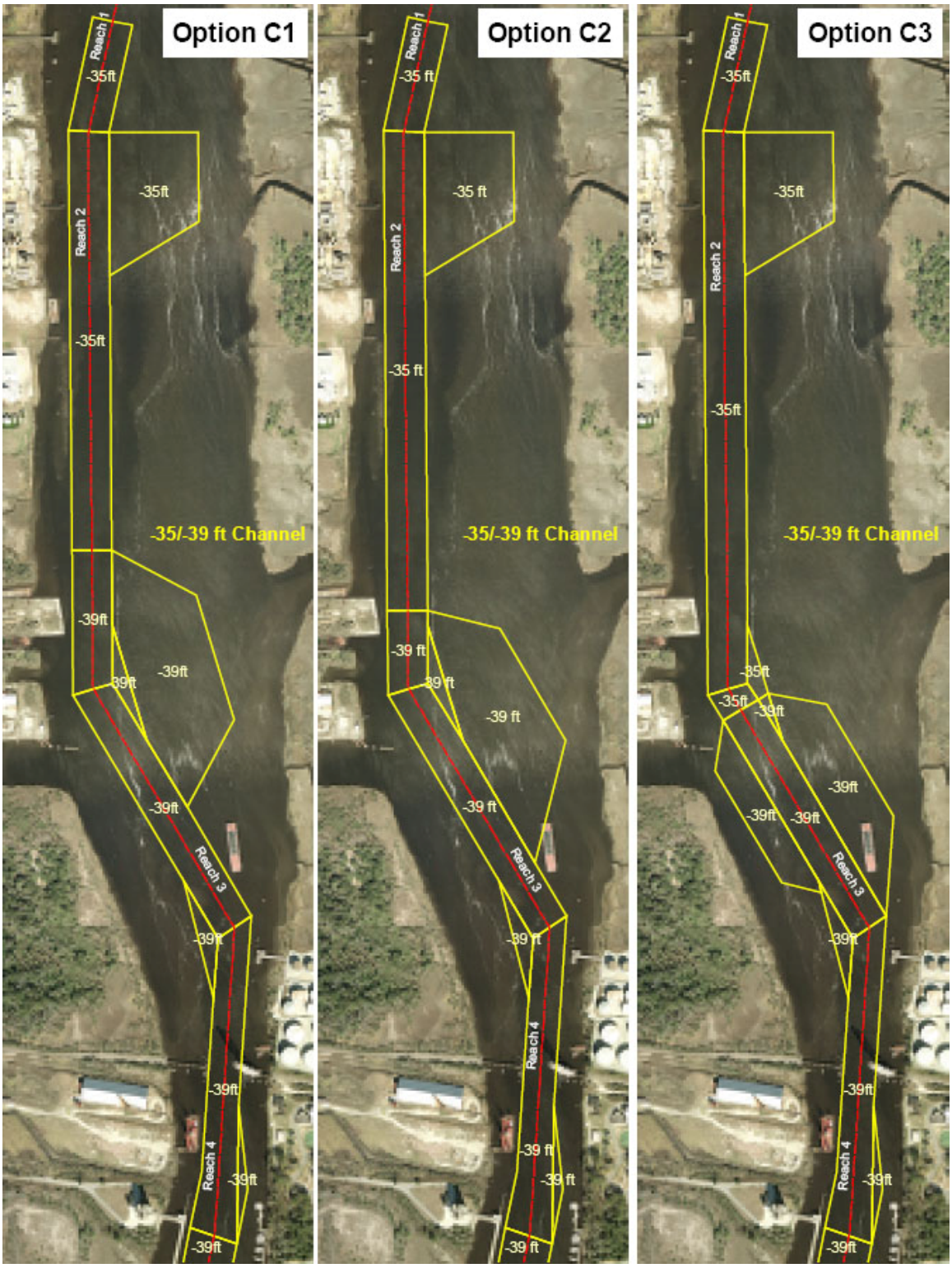
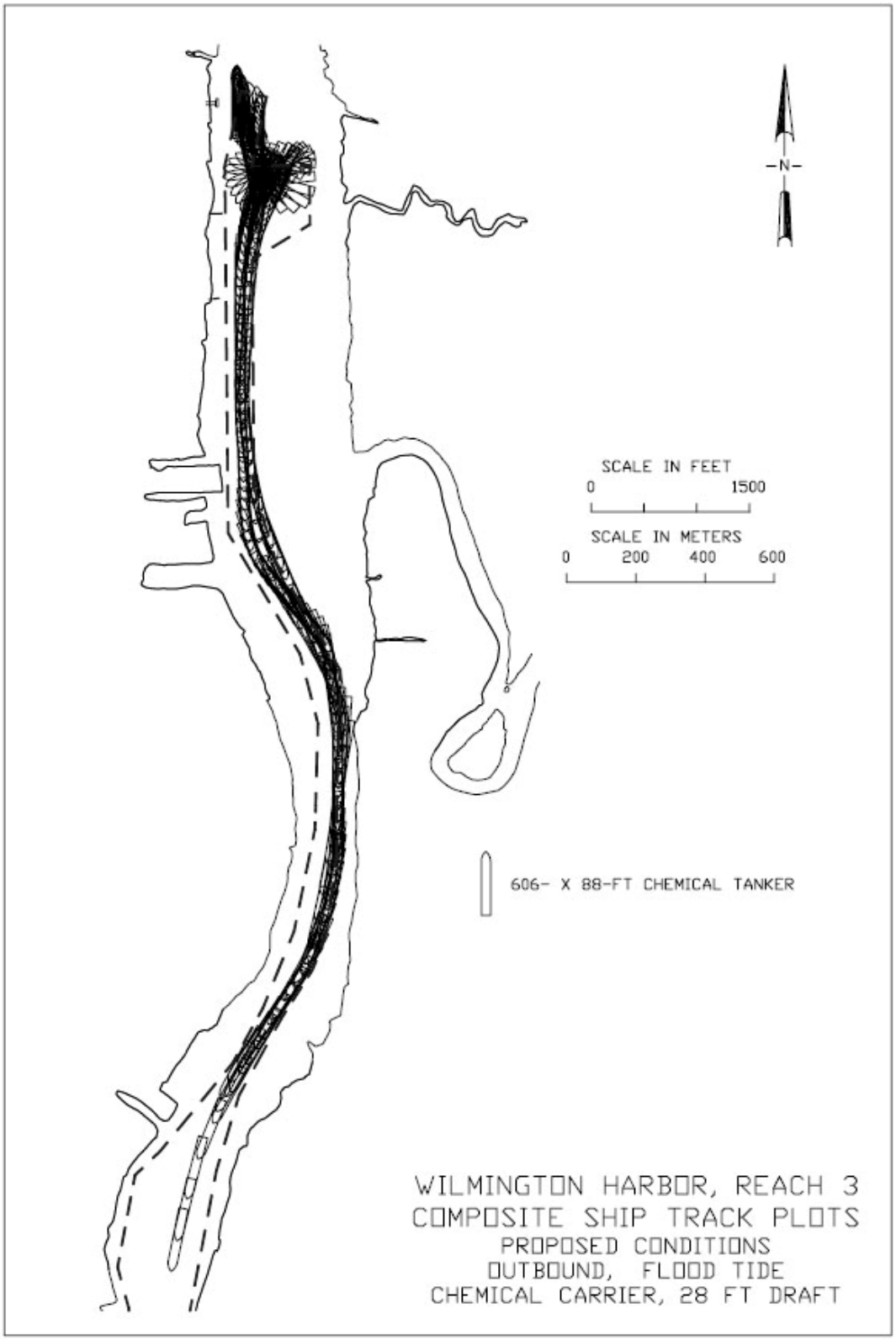


Figure 2-2. NE Cape Fear River Turning Basin alternatives.



ENCLOSURE 106

Figure 3. Track plot of design vessel from 1999 STAR Center report.

Figure 4-1. Turning maneuver from STAR Center report superimposed on TB C1.

Figure 4-2. Turning maneuver from STAR Center report superimposed on TB C2.

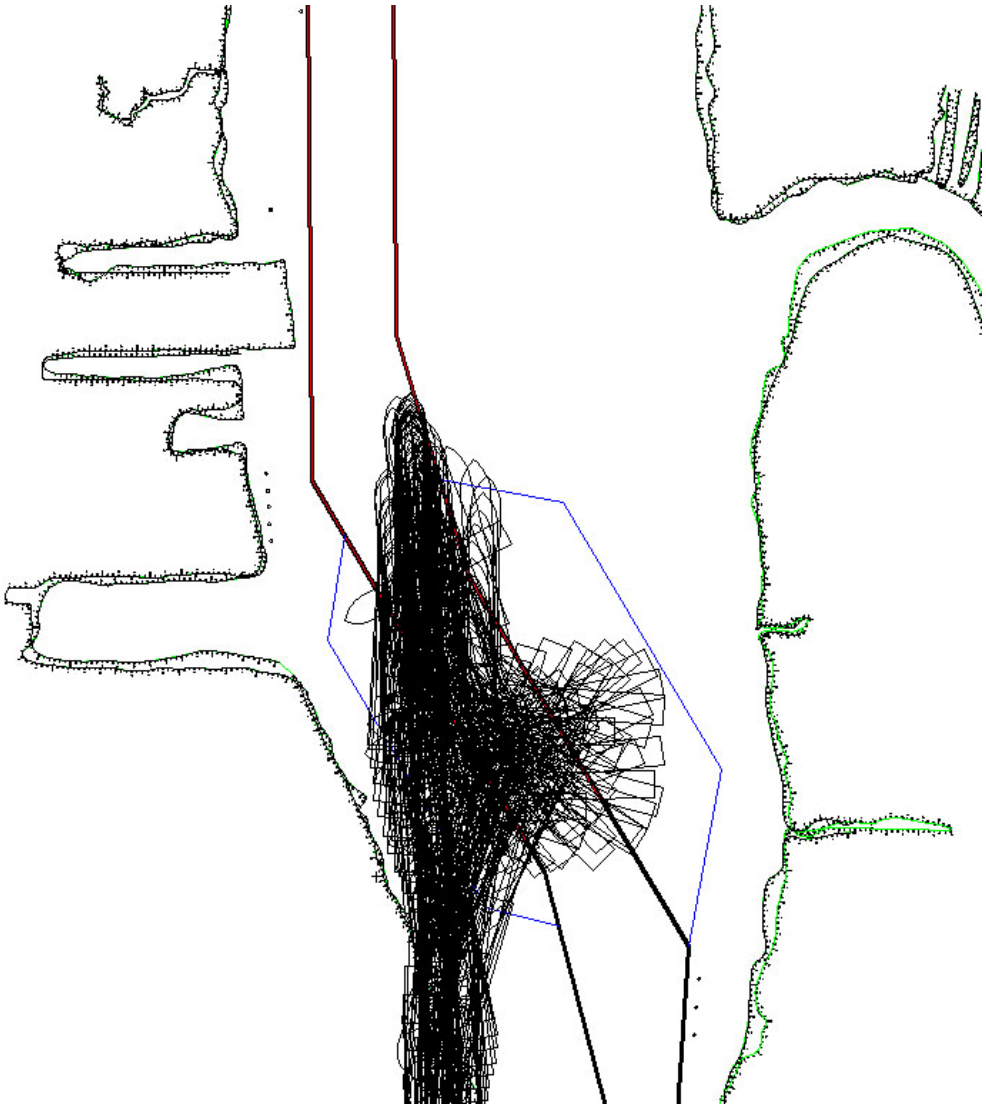


Figure 4-3. Turning maneuver from STAR Center report superimposed on TB C3.

END

Appendix 3.4

ENVIRONMENTAL APPENDIX
MITIGATION ANALYSIS

APPENDIX 3.4
FOR THE
WILMINGTON HARBOR GRR

1.00 FUNCTIONAL ANALYSIS OF PROJECT IMPACTS

A qualitative functional analysis of relative estuarine values within the project impact area (with and without the proposed project) is shown in Table 1. Estuarine functions considered to have high relative importance (5.0) in the mitigation analysis include fish spawning and nursery areas, anadromous fish habitat, and water quality. Wildlife and waterfowl feeding was also considered but were judged to be of only low to moderate importance (2.0).

As shown in Table 1, the habitat value for undredged bottom was scored as moderate to high ranging from 0.6 to 0.8 in relative habitat valve. Shallow areas (less than 10 feet deep) were determined to be of higher value than deep water areas. Dredged turning basins were rated as low value habitats (0.1) due to their depth and frequent disturbance from maintenance dredging and prop wash from ships maneuvering in the basin.

Under the proposed plan two existing turning basins totaling 18.9 acres (Almont and Southern States) will be abandoned. Upon abandonment these site will begin to fill in and undergo natural restoration. However, based on a shoaling analysis (see attachments 1&2), only a portion of the area (5.4 acres) will recover to shallow water habitat within the next 20 years. The habitat value for recovering turning basins (0.2) would be greater than the dredged condition (0.1) but, due to the slow in-filling process and expected unconsolidated nature the incoming material, the increase in average annual value is expected to be minor.

As shown at the bottom of Table 1, 4.8 acres of shallow primary nursery habitat and 13.9 acres of deep water primary nursery habitat within the project impact area had Relative Habitat Values of 0.8 and 0.6 respectively under the existing (without project) condition, with a possible range of 0.0 (no value) to 1.0 (high value). This provides 13.7 Habitat Units (HUs), based on a modified Habitat Evaluation Procedure (HEP) where Habitat Units = Relative Habitat Value X Area. The new turning basin (18.2 acres) would continue to provide some of the same functions, but the relative value would be significantly reduced (0.1). The portion of the abandoned turning basin that is not expected to shoal (13.5 acres) would have a similar low relative value. This provides (3.9) Habitat Units under the with project condition, a net loss of 9.8 Habitat Units.

Table 1

Table 1, Impacts

RELATIVE ESTUARINE VALUES

Project Name: Pilots Alternative: New Turning Basin - Wilmington Harbor GRR
HabitatType: Primary Nursery NE Capr Fear River
Location: NE Cape Fear River
Evaluator: Corps of Engineers -Wilmington

#	Estuarine Functions	Relative Import. 0-5	Undredged <10 feet		Undredged >10 feet		Turning Basin		Recovering TB	
			Rating	Weighted	Rating	Weighted	Rating	Weighted	Rating	Weighted
1	Flood Conveyence	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
2	Waves and Erosion	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
3	Flood Storage	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
4	Sediment Control	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
5	Fish Habitat									
	a. Spawning	5	0.7	3.5	0.6	3.0	0.1	0.4	0.2	1.0
	b. Nursery	5	0.9	4.5	0.8	4.0	0.1	0.5	0.2	1.0
	c. Anadromous fish	5	0.8	4.0	0.8	4.0	0.2	0.8	0.3	1.5
6	Shellfish Habitat	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
7	Waterfowl Habitat									
	a. Nesting	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2	0.5	1.0	0.0	0.0	0.0	0.0	0.0	0.0
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
8	Wildlife Habitat									
	a. Nesting/Breeding	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2	0.8	1.6	0.1	0.2	0.0	0.0	0.0	0.0
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
9	Recreation	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
10	Water Supply	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
11	Food Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
12	Timber Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
13	Historical Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
14	Education & Research	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
15	Aesthetic Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
16	Water Quality	5	0.9	4.5	0.6	3.0	0.1	0.5	0.3	1.5
	Total	24		19		14		2		5
	Estuarine Value =			0.8		0.6		0.1		0.2

Habitat Condition	Existing Condition		
	Rel. Value	Ac.	HUs*
Undredged <10 feet	0.8	4.8	3.8
Undredged >10 feet	0.6	13.9	8.2
Turning Basins	0.1	18.9	1.6
Recovering Dredged Bottom	0.2	0.0	0.0
		37.6	13.7

Habitat Condition	With Project Conditions		
	Rel. Value	Ac.	HUs*
Undredged <10 feet	0.8	0.0	0.0
Undredged >10 feet	0.6	0.0	0.0
Turning Basins	0.1	32.2	2.8
Recovering Dredged Bottom	0.2	5.4	1.1
		37.6	3.9

	Habitat Units (HUs)	Mitigation Goal (HUs)
Impact Acres	9.8	9.8

NOTES: Relative Wetland Values: High = 1.0, Moderate = 0.5, Low = 0.1, No Value = 0.0
Relative Importance: High =5, Moderate=3, Low = 1, Not Applicable =0
Habitat Units (HUs)

2.00 FUNCTIONAL EVALUATION OF MITIGATION REQUIREMENTS FOR ALTERNATIVE MITIGATON PLANS

A qualitative evaluation of relative functional values that would be provided by alternative restoration plans is described below. The estuarine functions that were considered and the weighting of various functions are consistent with those used for the impact analysis. For Primary nursery restoration plans (Alligator Creek and Island 12), initial post construction values are expected to be low since habitats will be established on presently non-aquatic substrates or sites infested with *Phragmites.* Stream restoration by dam removal would be expected to have higher initial post construction values since dewatering would be immediate the river is an existing aquatic site. This analysis recognizes the need for maturation of newly restored systems before they will fully function as well as the original habitats impacted by the project and assumes that replacement habitats to be restored will take 10 years to fully achieve the values associated with habitats to be impacted. The benefits of enhancement by rock ramps are expected immediately upon construction and would remain constant throughout the analysis period.

3.00 MITIGATION ALTERNATIVES

Mitigation alternatives that were evaluated include; various scale restoration projects at Dredged Material Disposal Island 12 (where sandy uplands would be converted to primary nursery), Restoration of Alligator Creek (a 10,000 foot long silted estuarine creek infested with *Pragmites),* Restoration of a segment of the Cape Fear River by Dam Removal and Anadromous Fish Passage by rock ramp construction.

Restore Primary Nursery at Disposal Island 12 (Table 2). Three incremental size scenarios were evaluated including; a 9.4 acre, small-scale restoration with on-site material disposal; a 12.6 acre restoration covering the maximum available area with onsite disposal and an 18.5 acre restoration that included the maximum available area with off site disposal. Since all of these plans would restore existing uplands no estuarine Habitat Value was assigned to the existing condition. It is expected that the habitat value at these site would be relatively low (0.1) just after construction (year 0), moderate (0.5) at year 5 and relatively high (0.8) at year 10. The Average annual habitat value for the 10 year period of analysis would be 0.5. Therefore 21.1 acres of this mitigation type would be needed to meet the mitigation goal of 9.8 HUs. Even the largest plan (18.5 acres) at Island 12 would not fully meet the mitigation goal by itself.

Table 2, Island 12

RELATIVE ESTUARINE VALUES

Project Name: Pilots Alternative: New Turning Basin - Wilmington Harbor GRR
Habitat Type: Restored Primary Nursery on Dredged Material
Location: Dredged Disposal Island 12 (9.4 acre plan onsite disposal)
Evaluator: Corps of Engineers -Wilmington

#	Estuarine Functions	Relative Import. 0-5	Year 0 Rating	Year 0 Weighted	Year 5 Rating	Year 5 Weighted	Year 10 Rating	Year 10 Weighted
1	Flood Conveyence	0	n/a	n/a	n/a	n/a	n/a	n/a
2	Waves and Erosion	0	n/a	n/a	n/a	n/a	n/a	n/a
3	Flood Storage	0	n/a	n/a	n/a	n/a	n/a	n/a
4	Sediment Control	0	n/a	n/a	n/a	n/a	n/a	n/a
5	Fish Habitat							
	a. Spawning	5	0.1	0.5	0.6	3.0	0.8	4.0
	b. Nursery	5	0.1	0.5	0.4	2.0	1.0	5.0
	c. Anadromous Fish	5	0.1	0.5	0.4	2.0	0.6	3.0
6	Shellfish Habitat	0	n/a	n/a	n/a	n/a	n/a	n/a
7	Waterfowl Habitat							
	a. Nesting	0	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2	0.2	0.4	0.6	1.2	0.8	1.6
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a
8	Wildlife Habitat							
	a. Nesting/Breeding	0	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2	0.1	0.2	0.6	1.2	0.8	1.6
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a
9	Recreation	0	n/a	n/a	n/a	n/a	n/a	n/a
10	Water Supply	0	n/a	n/a	n/a	n/a	n/a	n/a
11	Food Production	0	n/a	n/a	n/a	n/a	n/a	n/a
12	Timber Production	0	n/a	n/a	n/a	n/a	n/a	n/a
13	Historical Values	0	n/a	n/a	n/a	n/a	n/a	n/a
14	Education & Research	0	n/a	n/a	n/a	n/a	n/a	n/a
15	Aesthetic Values	0	n/a	n/a	n/a	n/a	n/a	n/a
16	Water Quality	5	0.1	0.5	0.4	2.0	0.8	4.0
	Total	24		3		11		19
	Wetland Value =			0.1		0.5		0.8

Mitigation Plan

Restore PN at Disposal Island 12	Year 0	Year 5	Year 10	Ave. Ann. HUs	Mitigation Goal	Acres Needed	Acres Available
9.4 Acre Plan	0.1	0.5	0.8	0.5	9.8	21.1	9.4
12.6 Acre Plan	0.1	0.5	0.8	0.5	9.8	21.1	12.6
18.5 Acre Plan	0.1	0.5	0.8	0.5	9.8	21.1	18.5

NOTES: Relative Wetland Values: High = 1.0, Moderate = 0.5, Low = 0.1, No Value = 0.0
Relative Importance: High =5, Moderate=3, Low = 1, Not Applicable =0
Habitat Units (HUs)

Restore Alligator Creek (Table 3). The remnant Alligator Creek (existing condition) is highly degraded by siltation and *Phragmites;* however, it is a wetland with some limited tidal influence and is considered to provide a habitat value of 0.2 under the existing condition. Immediately upon construction, values would be expected to increase (0.3) and continue to increase over time with a moderate value (0.5) at year five and relatively high value (0.8) at year 10. Average annual habitat value would be 0.4 and therefore 28.2 acres of this habitat type would be needed to meet the mitigation goal. This plan was developed to restore (to the degree practical) the stream conditions that existed prior to its degradation. Partial construction would not provide the needed habitat output so this alternative was not evaluated in increments. As shown in Table 3, 28.2 acres of this mitigation type would be needed to meet the mitigation goal of 9.8 HUs. Only 25.2 acres this mitigation type is available and therefore it would not fully meet the mitigation goal by itself.

Table 3 Alligator Creek

RELATIVE ESTUARINE VALUES

Project Name: Pilots Alternative: New Turning Basin - Wilmington Harbor GRR
Wetland Type: Restored Tidal Creek
Location: North of Eagle Island Confined Disposal Facility
Evaluator: Corps of Engineers -Wilmington

#	Estuarine Functions	Relative Import.	Existing Condition		Year 0		Year 5		Year 10	
		0-5	Rating	Weighted	Rating	Weighted	Rating	Weighted	Rating	Weighted
1	Flood Conveyence	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
2	Waves and Erosion	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
3	Flood Storage	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
4	Sediment Control	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
5	Fish Habitat									
	a. Spawning	5	0.2	1.0	0.3	1.5	0.5	2.5	0.8	4.0
	b. Nursery	5	0.2	1.0	0.3	1.5	0.6	3.0	0.9	4.5
	c. Anadromous Fish	5	0.2	1.0	0.3	1.5	0.5	2.5	0.7	3.5
6	Shellfish Habitat	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
7	Waterfowl Habitat									
	a. Nesting	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2	0.2	0.4	0.2	0.4	0.5	1.0	0.9	1.8
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
8	Wildlife Habitat									0.0
	a. Nesting/Breeding	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2	0.3	0.6	0.3	0.6	0.6	1.2	0.8	1.6
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
9	Recreation	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
10	Water Supply	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
11	Food Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
12	Timber Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
13	Historical Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
14	Education & Research	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
15	Aesthetic Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
16	Water Quality	5	0.1	0.5	0.1	0.5	0.5	2.5	0.9	4.5
	Total	24		5		6		13		20
Restored Value			0.2		0.3		0.5		0.8	
Existing Condition					0.2		0.2		0.2	
	Net Habitat Value				0.1		0.3		0.6	

Mitigation Plan	Net Habitat Units/ Ac.			Ave. Ann.	Mitigation	Acres	Acres
Restore Alligator Creek	Year 0	Year 5	Year 10	HUs/Ac.	Goal	Needed	Available
	0.1	0.3	0.6	0.3	9.8	28.2	25.2

NOTES: Relative Wetland Values: High = 1.0, Moderate = 0.5, Low = 0.1, No Value = 0.0
Relative Importance: High =5, Moderate=3, Low = 1, Not Applicable =0
Habitat Units (HUs)

Rock Ramp at Cape Fear River Lock and Dam 2 (Table 4). The existing conditions in the Cape Fear River above lock and dam 2 was rated 0.4. This score acknowledges recent improvements in fish locking procedures at the locks and dams and assumes proposed fish bypass facilities at Lock and Dam 1 are in place; however, this reach is generally impounded up to Lock and Dam #3 and is considered to be in a degraded condition. The primary benefit of the rock ramps would be anadromous fish passage; however, minor increases in water quality and other habitat functions would also be expected. The net average annual HUs for this enhancement is 0.1 requiring 156.6 acres to meet the mitigation goal. This action will improve fish passage to a significant stream segment (about 24 miles X about 200 feet wide); however, it is expected that only a portion of this area would actually be utilized by anadromous fish, so an efficiency factor of 33% was applied. It is estimated that 192 acres of enhanced habitat would be provided by this mitigation plan. Therefore implementation of this plan would exceed the area requirements and the mitigation goal.

Table 4, Rock Ramp @L&D 2

RELATIVE ESTUARINE VALUES

Project Name: Pilots Alternative: New Turning Basin - Wilmington Harbor GRR
Wetland Type: Primary Nursery - Fish Access Blocked by Dam
Location: Cape Fear River above Lock and Dam #2
Evaluator: Corps of Engineers -Wilmington

#	Estuarine Functions	Relative Import.	Existing		Year 0		Year 5		Year 10	
		0-5	Rating	Weighted	Rating	Weighted	Rating	Weighted	Rating	Weighted
1	Flood Conveyence	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
2	Waves and Erosion	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
3	Flood Storage	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
4	Sediment Control	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
5	Fish Habitat									
	a. Spawning	5.0	0.4	2.0	0.4	2.0	0.4	2.0	0.4	2.0
	b. Nursery	5.0	0.4	2.0	0.4	2.0	0.4	2.0	0.4	2.0
	c. Anadromous Fish	5.0	0.3	1.5	0.5	2.5	0.5	2.5	0.5	2.5
6	Shellfish Habitat	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
7	Waterfowl Habitat									
	a. Nesting	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2.0	0.4	0.8	0.4	0.8	0.4	0.8	0.4	0.8
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
8	Wildlife Habitat									
	a. Nesting/Breeding	0.0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2.0	0.4	0.8	0.4	0.8	0.4	0.8	0.4	0.8
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
9	Recreation	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
10	Water Supply	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
11	Food Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
12	Timber Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
13	Historical Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
14	Education & Research	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
15	Aesthetic Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
16	Water Quality	5.0	0.5	2.5	0.6	3.0	0.6	3.0	0.6	3.0
	Total	24.0		9.6		11.1		11.1		11.1
	Restored Value		0.4		0.5		0.5		0.5	
	Existing Condition				0.4		0.4		0.4	
	Net Habitat Value				0.1		0.1		0.1	

Area Benefited	length (mi.)	width (ft)	efficiency %
	24.0	200.0	33.0%

Mitigation Plan	Net Habitat Units/ Ac.			Ave. Ann.	Mitigation	Acres	Acres
Rock ramp @ L&D#2	Year 0	Year 5	Year 10	HUs/Ac.	Goal	Needed	Available
	0.1	0.1	0.1	0.1	9.8	156.5	192.0

NOTES: Relative Wetland Values: High = 1.0, Moderate = 0.5, Low = 0.1, No Value = 0.0
Relative Importance: High =5, Moderate=3, Low = 1, Not Applicable =0
Habitat Units (HUs)

Removal of Cape Fear River Lock and Dam 2 (Tables 5). The existing conditions in the Cape Fear River above lock and dam 2 was rated 0.4 as described above. The benefits of the dam removal would be restoration of 24 miles of stream between Lock and Dam # 2 and Lock and Dam #3. The net average annual HUs/ac for this enhancement is 0.4 requiring 22.1 acres to meet the mitigation goal. This action will restore a significant stream segment of about 24 miles. An efficiency factor of 80% was applied considering continued backwater effect from Lock and Dam #1 that is not proposed for removal. It is estimated that 465.5 acres of enhanced habitat would be provided by this mitigation plan. Therefore implementation of this plan would exceed the area requirements and the mitigation goal.

Table 5, Remove L&D #2

RELATIVE ESTUARINE VALUES

Project Name: Pilots Alternative: New Turning Basin - Wilmington Harbor GRR
Wetland Type: Primary Nursery - Fish Access Blocked by Dam
Location: Cape Fear River above Lock and Dam #2
Evaluator: Corps of Engineers -Wilmington

#	Estuarine Functions	Relative Import.	Existing		Year 0		Year 5		Year 10	
		0-5	Rating	Weighted	Rating	Weighted	Rating	Weighted	Rating	Weighted
1	Flood Conveyence	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
2	Waves and Erosion	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
3	Flood Storage	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
4	Sediment Control	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
5	Fish Habitat									
	a. Spawning	5.0	0.4	2.0	0.8	4.0	0.8	4.0	0.8	4.0
	b. Nursery	5.0	0.4	2.0	0.9	4.5	0.9	4.5	0.9	4.5
	c. Anadromous Fish	5.0	0.3	1.5	0.7	3.5	0.8	4.0	0.8	4.0
6	Shellfish Habitat	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
7	Waterfowl Habitat									
	a. Nesting	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2.0	0.4	0.8	0.8	1.6	0.8	1.6	0.8	1.6
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
8	Wildlife Habitat									
	a. Nesting/Breeding	0.0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2.0	0.4	0.8	0.8	1.6	0.9	1.8	0.9	1.8
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
9	Recreation	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
10	Water Supply	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
11	Food Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
12	Timber Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
13	Historical Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
14	Education & Research	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
15	Aesthetic Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
16	Water Quality	5.0	0.5	2.5	0.9	4.5	0.9	4.5	0.9	4.5
	Total	24.0		9.6		19.7		20.4		20.40
Restored Value			0.4		0.8		0.9		0.9	
Existing Condition					0.4		0.4		0.4	
	Net Habitat Value				0.4		0.5		0.5	

Area Benefited	length (mi.)	width (ft)	effeciency %
	24.0	200.0	80.0%

Mitigation Plan	Net Habitat Units/ Ac.			Ave. Ann.	Mitigation	Acres	Acres
Remove L&D #2	Year 0	Year 5	Year 10	HUs/Ac.	Goal	Needed	Available
	0.4	0.5	0.5	0.4	9.8	22.1	465.5

NOTES: Relative Wetland Values: High = 1.0, Moderate = 0.5, Low = 0.1, No Value = 0.0
Relative Importance: High =5, Moderate=3, Low = 1, Not Applicable =0

Rock Ramp at Cape Fear River Lock and Dam 3 (Table 6). A rock ramp at 3 would not be effective without fish passage at L&D #2, so it would not be implemented alone. The existing conditions in the Cape Fear River above Lock and Dam 3 were scored 0.6. This assumes fish bypass facilities are in place at Lock and Dam 1 and Lock and Dam #2. The net average annual HUs/ac. provided by this enhancement is 0.2. Althought this enhancement value is small, this action would extend improved fish passage benefits to an additional 81 river miles. Assuming a 33% efficiency factor, the addition of this plan increment to plans involving removal or a rock ramp at Lock and Dam #2 would generate an excess 141.8 HUs.

Table 6, Rock Ramp @ L&D 3

RELATIVE ESTUARINE VALUES

Project Name: Pilots Alternative: New Turning Basin - Wilmington Harbor GRR
Wetland Type: Primary Nursery - Fish Access Blocked by Dam
Location: Cape Fear River above Lock and Dam #2
Evaluator: Corps of Engineers -Wilmington

#	Estuarine Functions	Relative Import.	Existing		Year 0		Year 5		Year 10	
		0-5	Rating	Weighted	Rating	Weighted	Rating	Weighted	Rating	Weighted
1	Flood Conveyance	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
2	Waves and Erosion	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
3	Flood Storage	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
4	Sediment Control	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
5	Fish Habitat									
	a. Spawning	5.0	0.6	3.0	0.8	4.0	0.8	4.0	0.8	4.0
	b. Nursery	5.0	0.6	3.0	0.9	4.5	0.9	4.5	0.9	4.5
	c. Anadromous Fish	5.0	0.2	1.0	0.5	2.5	0.5	2.5	0.5	2.5
6	Shellfish Habitat	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
7	Waterfowl Habitat									
	a. Nesting	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2.0	0.6	1.2	0.7	1.4	0.7	1.4	0.7	1.4
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
8	Wildlife Habitat									
	a. Nesting/Breeding	0.0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
	b. Feeding	2.0	0.6	1.2	0.8	1.6	0.9	1.8	0.9	1.8
	c. Cover	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
9	Recreation	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
10	Water Supply	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
11	Food Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
12	Timber Production	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
13	Historical Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
14	Education & Research	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
15	Aesthetic Values	0	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a
16	Water Quality	5.0	0.8	4.0	0.9	4.5	0.9	4.5	0.9	4.5
	Total	24.0		13.4		18.5		18.7		18.70
Value			0.6		0.8		0.8		0.8	
Existing Condition					0.6		0.6		0.6	
	Net Habitat Value				0.2		0.2		0.2	

Area Benefited	length (mi.)	width (ft)	efficiency %
	81.0	200.0	33.0%

Mitigation Plan	Net Habitat Units/ Ac.			Ave. Ann.	Mitigation	Acres	Acres	HUs
Rock Ramp @ L&D 3	Year 0	Year 5	Year 10	HUs/Ac.	Goal	Needed	Available	Generated
	0.2	0.2	0.2	0.2	9.8	N/A	648.0	141.8

NOTES: Relative Wetland Values: High = 1.0, Moderate = 0.5, Low = 0.1, No Value = 0.0
Relative Importance: High =5, Moderate=3, Low = 1, Not Applicable =0
Habitat Units (HUs)

Summary. Table 7 provides a summary of project impacts, mitigation goals and the output of various mitigation alternatives.

PROJECT IMPACTS AND MITIGATION GOAL - ACRES & HABITAT UNIT

Habitat Condition	Existing Conditior			With Project		Net Loss		Goal
	Rel. Value	Acres Present	Habitat Units	Acres Present	Habitat Units	Acres	Habitat Units	Habitat Units
Undredged <10 feet	0.8	4.8	3.8	0.0	0.0	4.8	3.8	3.8
Undredged >10 feet	0.6	13.9	8.2	0.0	0.0	13.9	8.2	8.2
Turning Basins	0.1	18.9	1.6	32.2	2.8	-13.3	-1.1	-1.1
Recovering Dredged Bottom	0.2	0.0	0.0	5.4	1.1	-5.4	-1.1	-1.1
								9.8

MITIGATION ACRES REQUIREC

Mitigation Plans	Habitat Units /Acre @ Given Interva			Av. An. HUs/Acre		Mit. Goal		Acres Needed	Acres Available
	Yr. 0	Yr. 5	Yr. 10						
Restore 9.4 ac.Primary Nursery at Disposal Island 12	0.1	0.5	0.8	0.5	HUs	9.8	HUs	21.1	9.4
Restore 12.6 ac.Primary Nursery at Disposal Island 12	0.1	0.5	0.8	0.5	HUs	9.8	HUs	21.1	12.6
Restore 18.5 ac. Primary Nursery at Disposal Island 12	0.1	0.5	0.8	0.5	HUs	9.8	HUs	21.1	18.5
Restore Alligator Creek	0.1	0.3	0.6	0.3	HUs	9.8	HUs	28.2	25.2
Remove L&D #2	0.4	0.5	0.5	0.4	HUs	9.8	HUs	22.1	465.5
Rock Ram at L&D # 2	0.1	0.1	0.1	0.1	HUs	9.8	HUs	156.5	192.0
Rock Ramp at L&D # 3 (only incombination w/ #2)	0.2	0.2	0.2	0.2	HUs	9.8	HUs	N/A	648.0

Notes: The mitigation goal of 11.9 HUs remains the same for all alternatives, acreage requirements increase or decraese depending the habitat value (Av. Ann. HUs/Ac.) provided by a given plan component

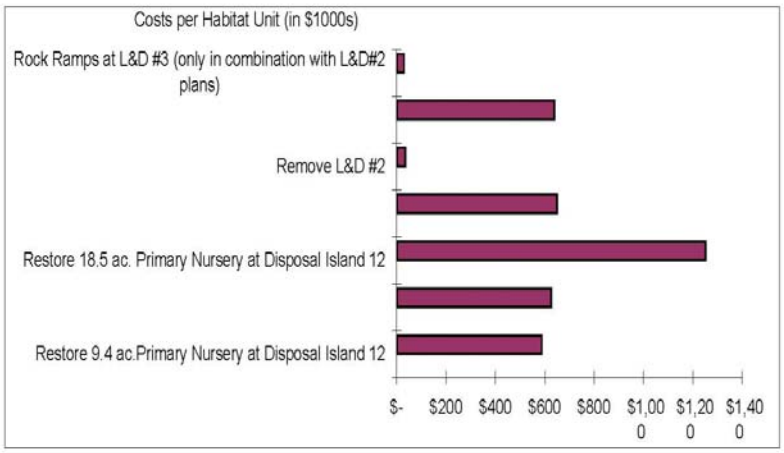
4.00 INCREMENTAL COST ANALYSIS (Table 8)

An incremental cost analysis is shown in Table 8. This compares costs per HU for alternative means of mitigation. With the exception of a rock ramp at Lock and Dam #3 that must be combined with a project at Lock and Dan #2 to be effective, removal of Lock and Dam #2 is the most cost effective plan. The least cost effective plan is the largest scale restoration of Island 12. This plan is incrementally more costly than the smaller scale projects at Island 12, because of the additional work and costs needed to dispose of the material offsite. The cost per HU for restoration of primary nursery at Island 12 with onsite disposal is greater than dam removal or either rock ramp plans, but, less than restoration of Alligator Creek.

Table 8, Incremental Analysis

MITIGATION COST (in Thousands)

Mitigation Plans	Const. Cost	Monitor	Acres Avail	HUs	Total Cost
Restore 9.4 ac.Primary Nursery at Disposal Island 12	$ 2,518	$ 50	9.4	4.4	$ 2,568
Restore 12.6 ac.Primary Nursery at Disposal Island 12	$ 3,616	$ 50	12.6	5.9	$ 3,666
Restore 18.5 ac. Primary Nursery at Disposal Island 12	$ 10,708	$ 50	18.5	8.6	$ 10,758
Restore Alligator Creek	$ 5,633	$ 50	25.2	8.7	$ 5,683
Remove L&D #2	$ 6,494	$ 750	465.5	206.1	$ 7,244
Rock Ram at L&D # 2	$ 6,907	$ 750	192.0	12.0	$ 7,657
Rock Ramps at L&D #3 (only in combination with L&D#2 plans)	$ 4,250	-	840.0	141.8	$ 4,250



Plan Combinations (Table 9 and Figure 1).

The only plans that provide enough HUs to meet the mitigation goal without being combined with other plans are removal of Lock and Dam #2 and a rock ramp at Lock and Dam #2. All of the other restoration options require combination with another plan to meet the goal. A short list of plan combinations is shown in Table 9. All potential effective plans and plan combination identified by IWR Plan are shown on Figure 1.

Table 9, Potential Plan Combos

POTENTIAL MITIGATION PLANS

Mitigation Method	Habitat Units	Cost
Restore 9.4 ac.Primary Nursery at Disposal Island 12	4.4	2568.0
Restore Alligator Creek	8.7	5683.0
Total	13.1	8251.0

Mitigation Method	Habitat Units	Cost
Restore 12.6 ac.Primary Nursery at Disposal Island 12	5.9	3666.0
Restore Alligator Creek	8.7	5683.0
Total	14.6	9349.0

Mitigation Method	Habitat Units	Cost
Restore 18.5 ac. Primary Nursery at Disposal Island 12	8.6	10758.0
Restore Alligator Creek	8.7	5683.0
Total	17.3	16441.0

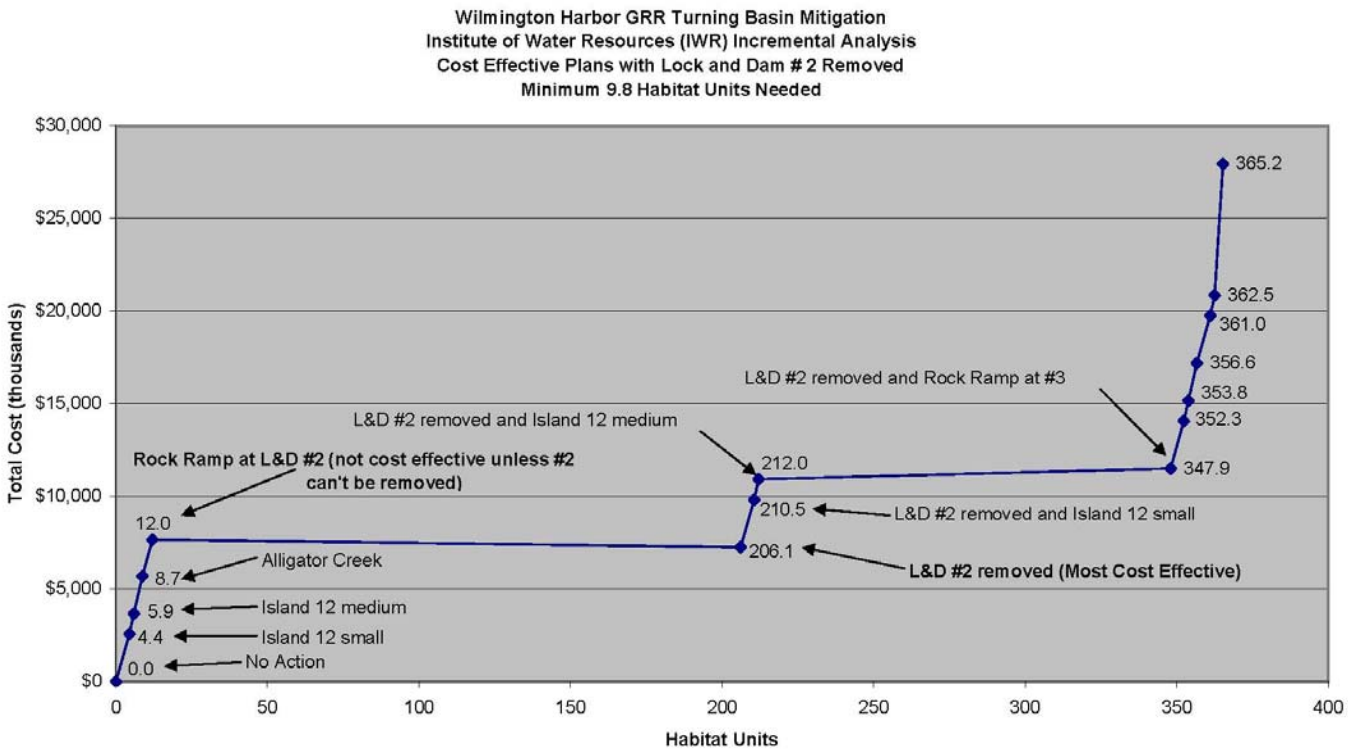
Mitigation Method	Habitat Units	Cost
Remove L&D #2	206.1	7244.0
Total	206.1	7244.0

Mitigation Method	Habitat Units	Cost
Rock Ram at L&D # 2	12.0	7657.0
Total	12.0	7657.0

Mitigation Method	Habitat Units	Cost
Remove L&D #2 w/ Rock Ram at # 3	347.8	11494.0
Total	347.8	11494.0

Mitigation Method	Habitat Units	Cost
Rock Ramp L&D #2 w/ Rock Ramp L&D #3	153.8	11907.0
Total	153.8	11907.0

Figure 1, Effective Plans Graph



ATTACHMENT 1 Shoaling Analysis Almont Shipping Facility

Subject:
Shoaling associated with the abandonment of the turning basin adjacent to the abandoned Almont Shipping facility

Introduction and problem description:
The existing turning basin located adjacent to the abandoned Almont Shipping facility is 750 feet wide. The 400 ft. wide navigation channel runs through the eastern portion of the basin. Proposed plans for navigational improvements for the Northeast Cape River include deepening the navigation channel to elevation -39 ft. (mllw) and abandonment of the remaining 350 ft. wide western portion of the turning basin. The western portion of the abandoned turning basin will collect sediment resulting from natural shoaling processes with shoaling progressively building from the western side of the abandoned basin at a non-uniform rate in an eastward direction toward the navigation channel. Environmental classification for shallow water habitat is limited to water depths less than -10 feet (mllw). Determination of the portion of the turning basin that will eventually meet this environmental condition requires an analysis of the expected areal extent and time associated with the establishment of the required condition.

Methodology:
The expected areal extent of the portion of the abandoned turning basin that will shoal to a depth of -10 feet (mllw) was determined by analyzing bank to bank hydrographic surveys of the Northeast Cape Fear River from a position located approximately 900 feet upstream from the Isabel Holmes Bridge to a position located approximately 950 feet below the southern terminus of the turning basin, a distance of 5250 feet. The -10 ft. contour through the surveyed area was compared to the adjacent limit of the navigation channel to establish a reasonable distance from the navigation channel that would shoal to -10 feet (mllw), given similar flow and sedimentation conditions. This evaluation indicated that a depth of -10feet could be expected to develop approximately 170 feet from the limit of the navigation channel. Since the total width of the navigation channel and the turning basin is 750 feet and the navigation channel is 400 feet wide, the turning basin area expected to shoal to depths of -10 feet (mllw) is located in the western 180 feet. Shoaling within this area will not occur uniformly, so the area was divided into three 60 ft. segments. These three segments were designated as TB-1, TB-2 and TB-3 in a west to east direction. After dredging and before dredging surveys conducted in 1999 and 2005, respectively, were compared to establish shoaling volumes and rates for each segment. Average depth based on the 2005 after dredging survey and the computed shoaling rates were used to compute the time required for each segment to shoal to a depth of -10 feet (mllw).

Results and Conclusions:
The western portion of the turning basin will shoal to a depth of -10 feet (mllw). The total shoal area for TB-1, TB-2, and TB-3 is 192,930s.f. (4.4acres). Shoaling rates associated with TB-1, TB-2, and TB-3 are 2 ft/yr, 1.2 ft/yr, and 0.7 ft/yr, respectively. The time for TB-1, TB-2, and TB-3 to shoal to -10 feet (mllw) was computed to be 9.7 years, 16 years, and 27.5 years, respectively, with an overall areal weighted average of 19years. Further

refinement of this analysis could be accomplished through the evaluation of additional historic dredging cycles depending on the availability of hydrographic surveys.

ATTACHMENT 2 Southern States Turning Basin Shoaling Analysis

Subject:
Shoaling associated with the abandonment of the turning basin adjacent to the Southern States Chemical facility.

Introduction and problem description:
The existing turning basin located adjacent to the Southern States Chemical facility is 800 ft. wide on the north side, 550 ft. long on the east side and tapers to 885 ft. along the eastern side of the 250 ft. wide navigation channel which runs along the western portion of the basin. Proposed navigational improvements for the Northeast Cape Fear River may include the development of alternative turning basin locations and abandonment of the Southern States turning basin. Since environmental classification for shallow water habitat is limited to water depths of less than -10 feet (mllw), a shoaling analysis is required to determine the areal extent and time associated with the establishment of a shallow water habitat should the turning basin be abandoned.

Methodology:
Shoaling trends and the areal extent of areas within the turning basin which may reach shoal depths of -10 feet (mllw) were determined by reviewing historic dredging records dating back to 1954 and analyzing hydrographic surveys conducted for the area from 1994 through 2006. Historic dredging requirements for the Northeast Cape Fear Project above the Hilton R.R. Bridge are minimal and reflect a historic average annual volume of approximately 19,000cubic yards. An areal weighted proportion of the historic annual volume associated with turning basin is 3600 cubic yards per year. Given an annual uniform shoaling distribution of 3600 cubic yards, the turning basin would shoal at a rate of .25 feet /year. Hydrographic surveys indicate that shoal within the turning basin is not uniform and the majority of the shoaling occurs along 300 feet of the northern boundary. This shoal area starts adjacent to the navigation channel and builds in southerly direction for approximately 150 feet. Hydrographic surveys and mapping of the Northeast Cape Fear River indicate flow patterns that bring the thalweg of the river along the eastern section of the channel where it enters the northeastern boundary of the turning basin and bends westward within the basin and exits in the vicinity of the southwest corner of the basin. This flow pattern through the turning basin tends to minimize shoaling within the basin and maintain water depths at or near the current project depths of -25 feet (mllw).

Results and Conclusions:
A portion of the turning basin located adjacent to the northern boundary will shoal to a depth of -10 feet (mllw). The estimated total shoal area of 1 acre represents approximately 10 percent of the total turning basin area. The time required for this area to shoal to -10 feet will average 17 years and range from 5 to 27 years.

END

Appendix 3.7.8

SALTWATER INTRUSION

Saltwater intrusion in a riverine system refers to the movement of saltwater from the ocean moving landward or upstream into areas of freshwater. The movement of this saltwater front is chiefly driven by tides, freshwater discharge, density currents and wind. For the Cape Fear River the location of the saltwater front is dictated mainly by River discharge and tides.

This investigation into saltwater intrusion focuses on the maximum distance upstream the saltwater front reaches up the Cape Fear River. The investigation involved reviewing results of existing numerical models and review of water quality data in the Cape Fear Estuary. Salinity is usually determined in the field by measuring the conductivity and temperature of the water. Conductivity is proportional to the amount of dissolved solids in the water and for seawater the chief constituent of concern is chloride.

The National Secondary Drinking Water Regulations list chloride levels of 250 mg/L as the upper limit for drinking water. A chloride level of 250 mg/L roughly equates to a salinity of 0.45 ppt and a specific conductance of 1,000 umho/cm. 250 mg/L is commonly used as the limit for the presence of seawater and the location of this concentration is referred to as the saltwater front.

Review of water quality data from the Cape Fear River focused on the drought conditions from 2001 to 2003. During this period, observed 7-day low flows at L&D #1 reached 350 cfs. The long-term daily average discharge is 5,500 cfs.

A.) Water Quality Modeling

As part of the proposed deepening of the Cape Fear River ship channel, the Wilmington District initiated the development of a 3-dimensional numerical model of the Cape Fear River system. The purpose of the model was to determine the effects of various channel alterations on the hydrodynamic and salinity behavior of the system. The work included a field data collection effort between August 12, 1993 and October 18, 1993. Water surface elevation, channel velocity and salinity were recorded at ten stations and were used to calibrate the model. See *Figure 3.7.9.3 Lower Cape Fear Monitoring Stations*. The field work and numerical modeling was performed by the COE Engineer Research and Development Center and are summarized separately by Benson (1995) and McAdory (2000), respectively. The model assumes existing conditions to be the 1999 38-ft deep River channel.

The field data collected at NC Highway 11 (Station 11) did not record any measurable level of salinity during the two month deployment. The water surface elevation at this station is dictated by river discharge but includes an average tidal range of 3.3-ft. This tide range is dampened to less than 1.0-ft during high flow. Salinity values of 0.0 to 12.0 ppt were recorded at Navassa (Station 7).

The numerical model indicated that with the 44-ft channel deepening the average maximum high tide in the downtown Wilmington area increase by about 2.0 inches above existing conditions. The model indicated that salinity of the Cape

Fear system from Ft. Caswell to above Navassa decreased in salinity with the 44-ft deepening compared to existing conditions. This analysis used an average freshwater discharge of about 2,000 cfs, combined flows of the Cape Fear, Northeast and Black Rivers. With low flow conditions of 950 cfs the 44-ft deepening continued to show lower salinity than existing conditions.

The counter-intuitive decrease in salinity with channel deepening is the result of two competing salinity transport mechanisms at work in the estuary (McAdory, 2000). Channel deepening encourages salinity stratification which increases salinity intrusion into the estuary by density driven currents. Channel deepening also increases the tide range which increases mixing of the water column within the estuary. The additional mixing with the 44-ft deepening overcomes the additional stratification and density currents resulting in less saltwater intrusion into the Cape Fear system.

A second water quality model of the Cape Fear Estuary was developed by the City of Wilmington and New Hanover County to evaluate total maximum daily loads for discharge permits. The 3-Dimensional EFDC water quality model did not address the maximum upstream limit of the saltwater front. The model grid of the Cape Fear River above Navassa does not have sufficient resolution for detailed analysis of salinity movement.

A third model of the Cape Fear developed by the North Carolina Division of Water Resources is not capable of analyzing water quality and does not extend below L&D #1. The Cape Fear River Basin Model was developed as a tool to analyze water supply issues and is not applicable to saltwater intrusion.

B.) USGS North Carolina Estuary Studies

The US Geological Survey report on hydrology of major estuaries in North Carolina (Giese, 1985) describes the Cape Fear estuarine system as partially mixed. There is a saline gradient with depth but there is not a distinct saltwater wedge as seen in other southeastern estuaries.

The report estimates that the maximum upstream movement of saltwater (~0.4 ppt) in the Cape Fear River likely occurred in October 1953 during Hurricane Hazel. The saltwater front is estimated to have reached 20 miles upstream of Wilmington or 19 miles below Lock and Dam #1. The high storm surge associated with Hazel coincided with very low flows on the Cape Fear (290 ft^3/s at L&D #3) to produce this brief, extreme event.

The USGS also studied the upstream limit of the saltwater front that has a 50 percent chance of being equaled or exceeded in any one year on the Cape Fear. The furthest upstream location of this event is 20 miles upstream of Wilmington near the confluence of the Black River. The report noted that minimum releases from Jordan Dam should considerably reduce the maximum extent of the saltwater intrusion into the Cape Fear Estuary.

C.) Lower Cape Fear River Program

The Lower Cape Fear River Program (LCFRP) includes 35 water quality sampling stations throughout the Cape Fear, Northeast Cape Fear and Black River watersheds and has been in place since 1995. Samples are taken monthly. Seven stations are located along the Cape Fear between Wilmington and L&D #1 and were in operation during the drought of 2002. Specific conductivity was among other water quality parameters measured. The LCFRP data was combined with data in Section D for analysis.

D.) NC Division of Water Quality – Ambient Monitoring System

As part of its Ambient Monitoring System the North Carolina Division of Water Quality (DWQ) has been collecting monthly samples at six stations on the Cape Fear between L&D #1 and Navassa. Monthly samples have been taken since 1974. The DWQ and LCFRP data is stored on the EPA's STORET data retrieval system (http://www.epa.gov/storet/index.html).

Samples are gathered at three depths in the water column at each station. Review of conductivity readings taken at the various depths indicate that the Cape Fear is well mixed at Navassa with little stratification of salinity. No well defined saltwater wedge was noted for stations above Navassa.

DWQ and LCFRP samples gathered at NC11 during the 2002 drought had a peak specific conductivity of 217 umho/cm, chloride of 60 mg/L and salinity of 0.1 ppt. Conductivity at NC11 averages 150 umho/cm during normal flow conditions. At Hale Point (25 miles below L&D #1) the peak conductivity was 334 umho/cm, during normal flow the average is 163 umho/cm. The data is summarized in Table 1.

Location	Distance to L&D #1 (mi)	Peak Chloride (mg/L)	Peak Salinity (ppt)	Specific Conductance (umho/cm)	Date
L&D #2	-32.7	65	0.1	257	Aug. 2002
NC 11	2	60	0.1	217	Sep. 2002
Riegelwood	8.8	72	0.1	275	May 2002
Acme	10.4	100	0.2	385	Aug. 2002
DuPont	22.8	110	0.2	410	Oct. 2002
Hale Pt	25.3	90	0.2	334	Aug. 2002
Indian Crk	30	1,700	3.1	5170	Aug. 2002
Navassa	34.2	9,100	14.4	24230	Dec. 2001

Table 1 – Water Quality from 2002 Drought

E.) Wilmington Harbor Project – Tide and Salinity Monitoring Program

As part of the Wilmington Harbor Widening and Deepening Project a monitoring program was initiated in 1999 to determine if the project impacted the tidal range and salinity in the Cape Fear Estuary (Hackney, 2007). Monitoring stations were established at 12 locations on the Cape Fear, Northeast Cape Fear and Black Rivers to measure tide and salinity. Data is recorded every three minutes. The furthest upstream station on the Cape Fear River is at Dollison Landing (P8), about 25 miles downstream of L&D #1.

The highest salinity recorded at Dollison Landing was 5.7 ppt in August 2002 during drought conditions. This is an estimate chloride level of 3,100 mg/L. A salinity value of 5.4 ppt was recorded September 2005 following Hurricane Ophelia. Salinity averages between 0.1 and 0.2 ppt at Dollison Landing during normal river discharge.

F.) International Paper – Daily Water Quality Sampling

International Paper maintains a water intake structure on the Cape Fear River at its Riegelwood facility. Riegelwood is located approximately 9.0 miles downstream of L&D #1. Water quality samples are taken daily and include test for chloride levels.

International Paper provided the Wilmington District water quality data for 2002 and 2003. The lowest flows at L&D #1 occurred from June to September 2002 with an average of 931 cfs. The average chloride level at Riegelwood during this period was 57.3 mg/L. A peak chloride level of 72 mg/L was reached on September 1, 2002. These values are below the USEPA recommended standard of 250 mg/L for chloride in drinking water. The average chloride level measured at Riegelwood during normal flow (~5,500 cfs) on the Cape Fear is approximately 30 mg/L.

G.) Summary and Conclusions

The purpose of this investigation was to determine the furthest upstream limit of the saltwater front on the Cape Fear River. Over 10,000 water quality measurements on the Cape Fear between Wilmington and L&D #1 were reviewed. The data set covered the time period from 1974 to 2006. The highest salinity levels along the River occurred between December 2001 and September 2002. A chloride value of 250 mg/L or a specific conductance value 1,000 umho/cm was assumed to be the limit of saltwater intrusion. During the drought of 2002 it is estimated that the 250 mg/L chloride value reached an upstream limit between Hale Point and Indian Creek. This location is between 25.3 and 30 miles downstream of L&D #1.

One trend noticed in the data set was during low flow conditions on the Cape Fear River specific conductivity at all stations increased, including stations upstream of L&D #1. Figure 3.7.9.1 is a graph of conductance and flow at L&D #2 versus time and Figure 3.7.9.2 is a graph at NC 11, both show similar trends. The increase in conductance at L&D #2 with low flows indicates that something other than chloride from seawater is influencing conductance. Minerals in groundwater and wastewater treatment discharges have a greater effect on conductivity during low flow. Care should be taken when evaluating the presence of seawater in water samples measured with a specific conductance lower than 1,000 umho/cm.

Review of the numeric model used to evaluate the Wilmington Harbor Deepening Project did not indicate salinity increasing in the Cape Fear Estuary with channel deepening and widening. Model analysis of low flow conditions on the Cape Fear River did not indicate salinity reaching L&D #1. Further model analysis would be costly and the lack of data indicating high salinity values above Navassa does not warrant the additional analysis.

The USGS concluded that the furthest upstream limit of the saltwater front occurred in 1953 during Hurricane Hazel. The storm surge pushed the saltwater front upstream to approximately 19 miles below L&D #1. The saltwater front was flushed back downstream with the heavy storm rainfall.

One deficiency in the analysis of the water quality data set is the lack of continuous data between Dollison Landing and Riegelwood. The NC Division of Water Quality and the Lower Cape Fear River Program gather samples only monthly. The most important data set was the International Paper daily samples and chloride analysis at Riegelwood. The Riegelwood samples demonstrated that during the drought of 2002 the saltwater front did not extend as far upstream as Riegelwood. The saltwater front during the drought was at least 25 miles below L&D #1 and therefore removal of the dam structure would not endanger upstream water intakes with saltwater intrusion.

References:

Benson, H.A. and Parman, J.W., 1995, Field Data Collection Report, Cape Fear River, Wilmington, North Carolina, U.S. Army Corps of Engineers, Waterways Experiment Station, Technical Report HL-95-4.

McAdory, R.T., 2000, Cape Fear-Northeast Cape Fear River, North Carolina, Numerical Model Study, U.S. Army Corps of Engineers, ERDC/CHL TR-00-18.

Giese, G.L., Wilder, H.B., and Parker, G.G. 1985, Hydrology of Major Estuaries and Sounds of North Carolina: U.S. Geological Survey Water-Supply Paper 2221.

Hackney, C.T., 2007, Monitoring Effects of a Potential Increased Tidal Range in the Cape Fear Ecosystem Due to Deepening Wilmington Harbor, North Carolina Year 6: June 1, 2005 – May 31, 2006.
(All Reports available at: http://www.saw.usace.army.mil/wilmington-harbor/main.htm)

Lower Cape Fear River Program
(All Reports available at: http://www.uncwil.edu/cmsr/aquaticecology/LCFRP/)

NCDWR Cape Fear River Model
http://www.ncwater.org/Data_and_Modeling/Cape_Fear_River_Basin_Model/

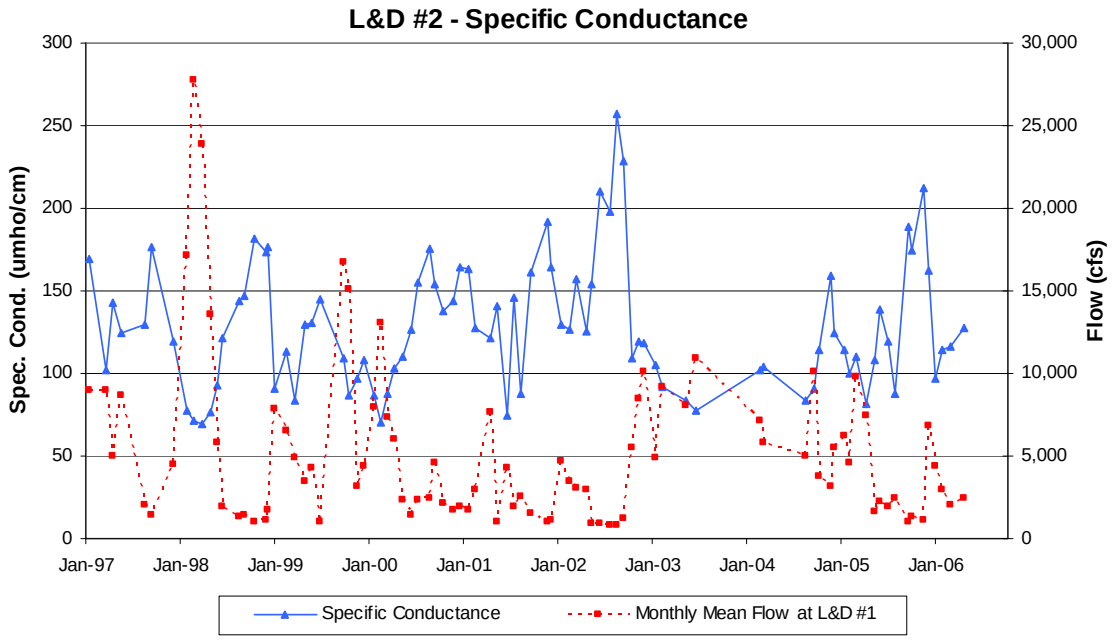


Figure 3.7.9.1 – Specific Conductance and Flow versus Time at L&D #2

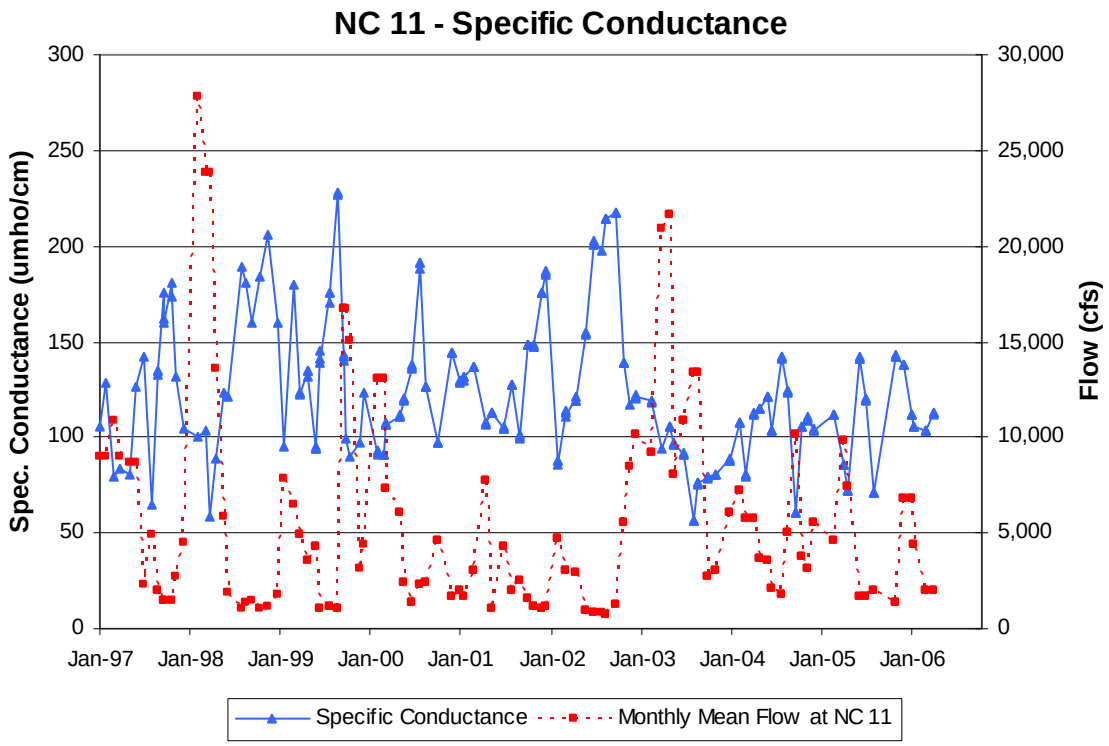


Figure 3.7.9.2 – Specific Conductance and Flow versus Time at NC 11

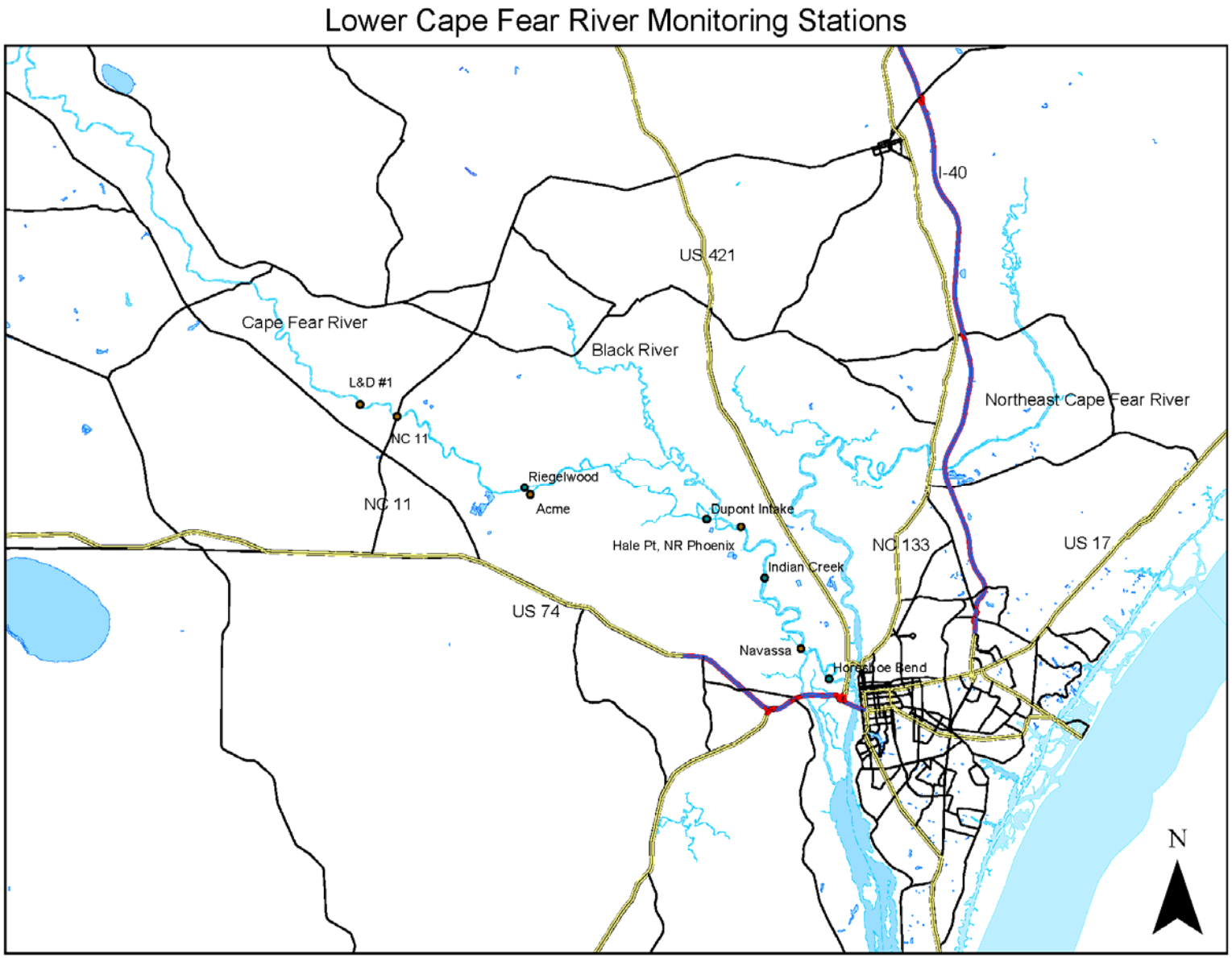


Figure 3.7.9.3 – Lower Cape Fear Monitoring Stations

END

Appendix 3.7.10.1

GEOTECHNICAL

Wilmington Harbor General Re-evaluation Report and Feasibility Study

Geotechnical Appendix

Geotechnical Section
Engineering Branch
Technical Services Division
Wilmington District

August, 2007

TABLE OF CONTENTS

SECTION	PAGE
I. Introduction.	4
II. Location of Study Areas.	4
III. Regional Geology.	4
A. Physiography, Topography, and Relief.	4
B. Geologic Setting.	4
C. Stratigraphy.	5
D. Structural Geology.	5
E. Hydrogeology.	5
IV. Site Geology.	5
A. Northeast Cape Fear River Turning Basin.	5
B. Cape Fear River Locks and Dams.	5
V. Subsurface Investigations.	6
A. Northeast Cape Fear River Turning Basin.	6
1. Geophysical Surveys.	6
2. Drilling.	6
3. Boring Logs.	7
4. Lab Testing.	7
5. 2005 Turning Basin Subsurface Investigation.	7
6. 2005 Investigation Summary of Results.	7

TABLE OF CONTENTS (Continued)

SECTION	PAGE
V. Subsurface Investigations (Continued).	
B. Cape Fear River Locks and Dams.	8
1. 2000 Vibracoring Investigation – L&D No. 1.	8
2. 2006 Boring Investigation.	8
3. Lab Testing.	8
VI. Cape Fear River Locks and Dams.	8
A. Stability Analyses.	8
B. Alternatives.	9
1. Rock Ramps.	9
2. By-passes.	9

I. Introduction. The General Re-evaluation Report (GRR) and Feasibility Report proposes several optional changes to the Wilmington Harbor Project on the Northeast Cape Fear River and the 3 lock and dam projects on the Cape Fear River. This Geotechnical Appendix details the geotechnical engineering and engineering geology aspects and considerations for these proposals.

II. Location of Study Areas. The study areas are located in southeastern North Carolina within the Cape Fear and Northeast Cape Fear River Basins.

The Wilmington Harbor Project includes 34 miles of deep-draft navigation channels and basins in New Hanover and Brunswick counties. The portion of the project studied in the GRR extends from the Anchorage Basin near the State Port Authority facilities, upstream along the Cape Fear River to the confluence with the Northeast Cape Fear River, and then upstream along the Northeast Cape Fear River to the head of deep-draft navigation approximately 1.2 miles above the Hilton Railroad Bridge.

The head of deep-draft navigation along the Cape Fear River is at the confluence with the Northeast Cape Fear River. From here an 8-foot barge channel extends up the Cape Fear River between Brunswick and Pender counties, then between Columbus and Pender counties and into Bladen County to Lock and Dam No. 1 south of Elizabethtown, NC. Lock and Dam No. 2 is located upriver at Elizabethtown. Lock and Dam No. 3 is also located in Bladen County near the Cumberland County line. Its upstream pool extends into Cumberland County to Fayetteville, NC. Together the 3 locks and dams with their accompanying pools extend for approximately 90 river miles along the Cape Fear River.

III. Regional Geology.

A. Physiography, Topography, and Relief. Both study areas are located in the Atlantic Coastal Plain Physiographic Province in what is considered to be the Outer Coastal Plain Sub-province. The rivers and streams are tidal in the Wilmington Harbor Project area. This Tidewater Area of the Outer Coastal Plain extends up the Cape Fear River Valley to Lock and Dam No. 1 in Bladen County. The topography in the Tidewater Area is flat and low-lying with low relief. The incised nature of the Cape Fear River Valley becomes evident with travel upstream towards Lock and Dam No. 1. Above Lock and Dam No. 1, the erosion of valleys by the river and its tributaries has produced topography with a subdued relief.

B. Geologic Setting. The rocks and sediments of the coastal plain are relatively flat-lying and form a gently southeastward-dipping "sedimentary wedge" which also thickens to the southeast. This large sedimentary wedge includes both sedimentary rock units and sediments which have not been cemented or otherwise indurated. The wedge lies upon pre-Mesozoic (older than 248 million years old) crystalline basement rocks. The surface of the basement rocks dips southeast at 40 to 72 feet per mile. The edge of onlap of the wedge onto the crystalline rocks of the Piedmont lies to the northwest of the study areas. The thickness of the wedge varies from 0 feet at the edge of onlap to approximately 1500 feet near Wilmington and it continues to thicken across the continental shelf.

C. Stratigraphy. The North Carolina Coastal Plain has been subjected to repeated transgressions and regressions of the sea since Late Cretaceous time. The sedimentary rocks and sediments of the sedimentary wedge were deposited mainly during the regression portion of the transgression-regression cycle. Some sediments were deposited in a marine environment, while others were deposited as near-marine or non-marine fluvial or deltaic sequences. The preserved record does not represent a continuous record of deposition. Each separate formation typically has a disconformity at its base – a gap in the preserved record caused by non-deposition or erosion. Other disconformities within formations also exist in the record.

D. Structural Geology. The primary structural feature of the subsurface is the Cape Fear Arch. This is an area where uplift of the sediments occurred during the Tertiary Period. The northwest-southeast axis of the Cape Fear Arch is sub-parallel to the Cape Fear River. The uplift along the arch controlled deposition in some cases. For example, the Paleocene Beaufort Formation was not deposited in areas along and near the axis of the arch due to the uplift.

E. Hydrogeology. A review of ground water studies evaluated published modern ground water studies in southeastern North Carolina. The ground water study performed for Wilmington Harbor adequately addresses the area where the new turning basin is to be sited. At this time there is also no need for further ground water studies in the reach of the Cape Fear River containing the locks and dams.

IV. Site Geology.

A. Northeast Cape Fear River Turning Basin. The area(s) in which a turning basin may be constructed is underlain by rock and sediments of the Late Cretaceous Peedee Formation. The Rocky Point Member of the Peedee Formation which occurs at the top of the formation is found in this vicinity, especially near the Hilton Railroad Bridge. (NOTE: The Rocky Point Member has recently been renamed the Scotts Hill Member.) Occasionally, some evidence of the Paleocene Beaufort Formation has also been found.

B. Cape Fear River Locks and Dams. The subsurface in the vicinity of the locks and dams primarily consists of Late Cretaceous formations. These include, in ascending order: the Cape Fear Formation, the Black Creek Group, and the Peedee Formation. A thin veneer of surficial sands covers the formations outside the river valley; alluvial river floodplain deposits overlie the formations in the bottom of the river valley.

Lock & Dam No.1 is underlain by the Peedee Formation and younger deposits including Recent alluvium. The top of rock is approximately 40 feet below mean sea level (msl).

Progressing upriver, the Peedee Formation thins and pinches out in the subsurface just south of the center of Elizabethtown. Lock & Dam No. 2 is underlain by the Black Creek Group which is approximately 200 feet thick in this area. The engineering definition of the top of rock in the form of 100 blow count/foot material is located at approximately 32 to 33 feet below msl. This may be rock or consolidated coarse-grained material including sand, gravel, rock fragments, or a mix of these materials.

Continuing upriver, the Black Creek Group thins until it is only about 100 feet thick at Lock & Dam No. 3. Here the 100 blow count/foot material occurs at 12 to 13 feet below msl. At Lock and Dam No. 3 the historic borings indicate there is a very hard consolidated clay layer at an elevation of 3 feet below msl.

V. Subsurface Investigations.

A. Northeast Cape Fear River Turning Basin.

1. Geophysical Surveys. Ocean Surveys, Inc. (OSI) performed a high-resolution seismic reflection survey in the late 1970's in the area above the Hilton Railroad Bridge. The results were not considered useful. Bottom sediments apparently contained gas bubbles which reflected the seismic signal and distorted the signal return to mask the subsurface. In the 1990's, Geotechnical Branch contracted with OSI again to evaluate whether CHIRP Sonar and/or high-resolution seismic reflection surveys were effective in the project area near the Anchorage Basin. Once again, gas bubbles appeared to mask the signal return making the results unusable.

2. Drilling. Historically, four different types of drilling methods have been used to explore Wilmington Harbor navigation channels and basins. Probes or wash probes have been used to define sediment thickness and/or depth to refusal. Wash probes consist essentially of washing a pipe through sediments until it cannot be advanced without difficulty. These are useful in defining shallow, buried, flat-lying rock surfaces when used in conjunction with standard split spoon drilling (SPT). Probing is usually rapid, with production of 5-10 probes per hour possible depending upon the distance apart. Their drawback is that since no sample of the refusing material is obtained, there is no assurance that refusal is on rock as opposed to on a log, ballast, debris, or a shipwreck.

Vibracores have been used to sample overburden materials. Vibracores are clear plastic tubes which are mechanically vibrated into sediment until refusal. The vibracore equipment the Wilmington District uses is limited to a depth of 20 feet and has less penetrating capability than wash probes. The advantage of using a vibracore is that a continuous sample of the soft soil materials is collected. Since vibracores generally reach refusal in soils, they are not usually used in rock investigations under any significant thickness of soils.

Standard Penetration Test (SPT) or "split-spooning" consists of advancing a standard 1 3/8" I.D. splitspoon sampler in 1.5-foot increments using blows from a 140-pound hammer which is dropped 30 inches. The number of blows per foot relates to the in-place density of the material. A sample is also obtained from within the splitspoon. This method provides the best penetrating ability of the soil drilling methods. It also provides a quantitative method of determining rock. The engineering definition of rock used is consolidated material requiring 100 or more blows per foot.

Rock coring is performed using a rotating carbide or diamond bit and is the common method used to sample rock in the subsurface. When used for dredging site characterization, it is typical and most useful to splitspoon the overburden to the top of rock and then core the rock. Lab testing can then be done on the soil and rock samples

obtained. This provides the maximum information on the subsurface. The primary drawback with both SPT drilling and rock coring is the time and expense required to drill each hole and the cost of drilling holes on the water.

3. Boring Logs. A boring log is prepared for each hole drilled. The boring information from drilling each hole and the method(s) used are documented on the boring log form. Boring logs from this project are available for review in the Geotechnical Section Office. They will be appended to the Geotechnical Appendix in the Draft GRR.

4. Lab Testing. Selected soil samples are chosen to identify soil properties of the recovered samples. The soil laboratory tests typically included gradation, moisture content, and Atterberg limits. Rock samples are typically tested for strength using an unconfined compression test. Other strength tests used on some samples from Wilmington Harbor were shear strength and tensile strength. Results of the laboratory testing will be included in the final GRR document.

5. 2005 Turning Basin Subsurface Investigation. The most recent subsurface investigation for the Turning Basin Study was performed in the summer of 2005. Planning began in the spring with a search and review of all existing subsurface data from nearby channels. Most of the data were wash probe refusals and a few were SPT/core borings. Several anomalous (shallow depth) wash probe refusals were noted. The anomalous data were attributed to the probe encountering submerged and buried logs that typically exist in thick river sediments. These data, the low number of SPT/core borings, and the few rock core samples for examination provided insufficient confidence for adequate definition of the subsurface. SPT/core borings in the three proposed alternative turning basin footprints were necessary to evaluate the sites. Eleven locations in the area of the proposed alternatives which represented gaps in the subsurface data were selected.

Specialized marine plant, self-propelled or with tugboat, and experienced personnel are required to do this work. The CESAS *Explorer* drilling barge was not available to perform the work due to the schedule/time limitations. The District's IDIQ Contractor was awarded a delivery order to engage a subcontractor to take the 11 SPT/core borings. Unfortunately, much of the information from these borings was only marginally adequate as the sub-contracted company possessed minimal marine boring experience. The methodology and results are summarized below. *The subsurface data for the turning basin will need to be supplemented with additional borings during the PED Phase.*

6. 2005 Investigation Summary of Results. Soils were sampled every 5 feet. Some of these samples were sent to the lab for classification to verify field calls. The majority of soils were classified as sands, some poorly-graded and some with silt. There were also layers of silt encountered mostly in the upper part of the holes. Rock samples were generally too fragmented to meet L/D ratios necessary for testing.

B. Cape Fear River Locks and Dams.

1. 2000 Vibracoring Investigation – L&D No.1. In November of 2000, 11 vibracores were performed downstream of the Lock & Dam No. 1 overflow dam. The

purpose was to identify soil types below the dam and record the depth to bottom for a rock ramp design. The results verified that a huge scour hole exists downstream of the dam. The soil materials recovered from the vibracores showed the soils to be sands interlayered with silts. Some wood, rocks, and shells were also encountered. In the deepest area of the scour hole no soils were recovered suggesting either rock or another impenetrable layer exists at this point.

2. 2006 Boring Investigation. Three to four SPT borings were drilled at each lock and dam. These were located on the right bank of the river, landward of the lock and in the general area where excavation of a bypass channel around the lock was proposed. The "nature-like" bypass at Lock and Dam No. 1 is located on the left bank of the river. Drilling for a bypass on the left side of the river at Lock and Dam Nos. 2 and 3 was considered and evaluated. The right bank was considered the more likely side for a bypass at these 2 locks. As a result, no drilling was done on the left bank.

The upper level soils encountered during drilling were sands**;** most had silt and clay mixed in. Further down the boring, fine-grained material, silt and/or clay became predominant with occasional layers of sand. The borings were terminated before encountering rock.

3. Lab Testing. Selected soil samples from the 2006 drilling were tested at a soils laboratory. The tests performed included grain size and Atterberg limits. The test results confirmed that the predominant soil type at depth is a clay and/or silt material.

VI. Cape Fear River Locks and Dams.

A. Stability Analyses. While each of the locks and dams have been in place and functioning satisfactorily for most of the last century, it is unknown if they meet current stability criteria. Stability analyses, which are currently in progress, will be performed to make this determination.

The stability analysis at each dam will be comprised of two parts. The first part will evaluate the stability of the existing dam with present day conditions. The second part will be a stability analysis of the dam with scour holes filled with rock and the fish rock ramps in place. The results of these stability analyses will be included in the Geotechnical Appendix in the Draft GRR.

B. Alternatives.

1. Rock Ramps. The rock ramps at the overflow sections of the dams are expected to improve the stability of the dams. If settlement occurs due to the rock ramps, it is not expected to be of a problematic magnitude.

2. By-passes. Any by-pass alternatives will include excavation of soils. If the by-pass is on the left side of the dam, looking downstream, the soils will be soft, wet, clays or silts. Excavation may require non-conventional equipment and multiple passes to obtain the required cross-section due to the wetness and flowability of the soil. If the by-

pass is on the right side, landside of the lock, the soil can be excavated with conventional equipment. The foundation here should be suitable for a by-pass channel.

END

Appendix 3.7.10.2

REVIEW
OF
GROUND WATER STUDIES

Wilmington Harbor General Re-evaluation Report and Feasibility Study

Cape Fear River Locks and Dams

Review of Ground Water Studies

Ted Zielonka, Geologist
Geotechnical Section
Engineering Branch
Wilmington District

July, 2007

TABLE OF CONTENTS

SECTION	PAGE
1. Introduction	1
1.1 General	1
1.2 Purposes of Review of Ground Water Studies	1
1.3 Scope of Review of Ground Water Studies	1
1.4 Description of Study Area	1
1.5 Physiography, Topography, and Relief	2
1.6 Climate	2
2. Project Geology and Stratigraphy	2
2.1 General	2
2.2 Geologic Setting	2
2.3 Late Cretaceous Period	3
2.3. a. Cape Fear Formation	3
2.3. b. Middendorf Formation	5
2.3. c. Black Creek Group	5
2.3. d. Peedee Formation	6
2.4 Tertiary and Quaternary Periods	6
3. Major Hydrogeologic Units	6
3.1 Lower Cape Fear Aquifer	7
3.2 Lower Cape Fear Aquifer Confining Unit	7
3.3 Upper Cape Fear Aquifer	7

SECTION PAGE

3.4 Upper Cape Fear Aquifer Confining Unit 8

3.5 Black Creek Aquifer 9

3.6 Black Creek Aquifer Confining Unit 9

3.7 Peedee Aquifer 9

3.8 Peedee Aquifer Confining Unit 10

3.9 Surficial Aquifer 10

4. Ground Water Flow 10

4.1 Recharge-Discharge 10

4.2 Pumping and Water Levels 10

5. Aquifer Conditions and Problems 11

5.1 Cones of Depression 11

5.2 Dewatering 11

5.3 Salt Water Intrusion 12

5.4 Capacity Use Area Designation 12

6. Quantitative vs. Qualitative Analysis of Effects of Dam Removal 13

6.1 Confined Aquifers 13

6.2 Unconfined Aquifer 13

7. Ground Water Quality: Manmade Pollutants 14

8. Summary and Conclusions 14

9. Recommendations 15

1. **Introduction**.

1.1. General. The General Re-evaluation Report (GRR) and Feasibility Study proposes several optional changes to the Wilmington Harbor Project on the Northeast Cape Fear River and the 3 lock and dam projects on the Cape Fear River. A ground water study (Lautier, 1996) has been done for the Wilmington Harbor Project. The proposed changes in the harbor navigation channels and basins in the Northeast Cape Fear River are under its purview.

The following review of local and regional ground water studies examines the hydrogeology of the area surrounding the 3 lock and dam projects on the Cape Fear River. The total removal of all the locks and dams was assumed to be the option with the most significant impact on the existing ground water regime.

1.2. Purposes of Review of Ground Water Studies. This report summarizes the review of recently-published ground water studies in the project area. It has multiple purposes:

a. to develop an understanding of the area hydrogeology and ground water issues by performing a review of existing ground water studies and reports;

b. to assess potential impact(s) of the proposed project changes to the existing ground water regime; and

c. to evaluate whether further ground water studies or modeling studies are necessary.

1.3. Scope of Review of Ground Water Studies. The scope of this review included relatively modern - - late 20th century and early 21st century - - published ground water studies, both local and regional. The lock and dam removal option at each of the 3 lock and dam sites would most drastically impact the Cape Fear River pool levels. Therefore, this option was assumed to be the one most capable of affecting/altering the existing ground water regime.

1.4. Description of Study Area. The study area for this ground water review initially included most counties in southeastern North Carolina. The Cape Fear River flows through or along 6 counties in southeastern North Carolina. The Wilmington Harbor Project includes 34 miles of navigation channels and basins in the Cape Fear and Northeast Cape Fear River Basins.

The first conclusion, reached early during this review of ground water studies, was that there is presently no need for further ground water studies in the Wilmington Harbor area. The ground water impact of the Wilmington Harbor Project in New Hanover County and Brunswick County was adequately addressed in the 1996 Wilmington Harbor Ground Water Study by Lautier. Therefore, this study review concentrated on the hydrogeology of the 3 locks and dams and their respective pools.

The study area includes parts of 2 counties, Bladen and Cumberland. It is a narrow corridor which could reasonably be affected by lock and dam removal. Centered on the Cape Fear River, it is 4 miles wide and extends from 2 miles below Lock and Dam No. 1 in Bladen County, upstream into Cumberland County to the upstream limit of the pool from the William O. Huske Lock and Dam (Lock and Dam No. 3) near Fayetteville, NC, a distance of 90 river miles.

1.5. Physiography, Topography, and Relief. The study area is located in the Atlantic Coastal Plain Physiographic Province in the Outer Coastal Plain Sub-province. In North Carolina, the study area is also referred to as being in the Southern Coastal Plain. The topography is mainly the result of the erosional incision of the Cape Fear River and its tributaries into the relatively flat-lying coastal plain sediments. The study area is characterized by slow-moving streams and subdued topographic relief.

1.6. Climate. The climate in the study area is subtropical with long, hot summers and short, mild winters. The average annual precipitation is 47 inches in Cumberland County and 49 inches in Bladen County.

2. **Project Geology and Stratigraphy**.

2.1. General. The relatively flat-lying rocks and sediments of the Atlantic Coastal Plain form a gently southeastward-dipping sedimentary wedge which also thickens to the southeast. This large sedimentary wedge includes both sedimentary rock and sediments which have not been cemented or otherwise indurated. This wedge lies upon pre-Mesozoic crystalline basement rocks. The surface of the basement rocks dips southeast at 40 to 72 feet per mile. The edge of onlap of the sedimentary wedge lies to the northwest of the project study area mostly on the crystalline rocks of the Piedmont. The thickness of the wedge varies from 0 feet at the edge of onlap to more than 1500 feet at the coast near Wilmington, and on to even greater thicknesses offshore on the continental shelf.

2.2. Geologic Setting. The Atlantic Coastal Plain has been subjected to repeated transgressions and regressions of the sea since Late Cretaceous time. These were eustatic sea level changes, primarily caused by the expansion and shrinkage of the glacial ice caps.

During an interglacial period, characterized by a glacial minimum or small to no ice cap, the eustatic sea level would be high and the area which was to become today's Atlantic Coastal Plain would be inundated. Marine sediments would typically be deposited during this transgression of the sea. As the ice caps grew, sea level fell and the sea regressed across the future coastal plain, depositing thicker sequences of sediments. While some were deposited in a marine environment, most beds in the study area were deposited in near-marine environments or as non-marine fluvial or deltaic sequences.

During a glacial maximum, with the sea well offshore, the sediments previously deposited would be subaerially exposed and subject to erosion. Non-deposition or erosion at these times produced a disconformity or gap in the preserved record. Transgressive-regressive cycles are often associated with disconformities. The

transgressive-regressive cycles preserved in coastal plain sediments in the study area are mainly from the Late Cretaceous Period.

2.3. Late Cretaceous Period. The oldest sedimentary rocks in the Coastal Plain sedimentary wedge are from the Mesozoic Era. The only relatively unweathered exposures of these Late Cretaceous units are in the river valleys of the coastal plain between the Roanoke River and the Peedee River. This is partially due to uplift along the Cape Fear Arch during the Tertiary Period which precluded overlap by the deposition of major younger strata.

From oldest to youngest, the major formations are the Cape Fear Formation, the Middendorf Formation, the Black Creek Group, and the Peedee Formation. As recently as the 1960's, these 4 formations together were thought to represent one cycle of deposition. However, numerous deposition cycles are now recognized in the Cretaceous strata of the project area, with the major formations typically separated by a major disconformity. Some of these sedimentary disconformities are regional in extent, and can be noted as far away as in the Cretaceous of Alabama or New Jersey.

A difference of opinion between investigators regarding the stratigraphic interpretation of the Cretaceous has persisted through most of the 20th century. Interbasin areas which are underlain by the Cretaceous units are deeply weathered so separation of the lithologies in road cuts or other shallow excavations is not possible. Some of the lithologies of the Cape Fear Formation, the Middendorf Formation, and the Black Creek Group are very similar, with few distinguishing features. There are also very few fossils to be found in much of the Cretaceous, especially in the Cape Fear and Middendorf formations. Some recent palynological studies have helped to verify correlation of the units. Presently, these pollen assemblage studies are very limited in scope and much work remains to be done.

The following discussion proceeds upward through the sedimentary wedge stratigraphically from the disconformity at the base of the Late Cretaceous sequence. The formations described below are also represented in the stratigraphic chart shown in **Figure 1**. Note that this chart shows the formations found in southeastern North Carolina. Most Tertiary and Quaternary formations do not occur in the study area.

2.3. a. Cape Fear Formation. This is the oldest outcropping sedimentary unit in the North Carolina Coastal Plain. It is only visible in outcrop in the river valleys between the Cape River and the Tar River. Stephenson (1907) named the formation for the beds exposed along the Cape Fear River between Erwin, NC, and the mouth of Rockfish Creek south of Fayetteville. The base of the formation can be seen near Erwin to be lying atop the pre-Mesozoic crystalline basement rocks in an angular unconformity.

GEOLOGIC UNITS			HYDROGEOLOGIC UNITS
SYSTEM	SERIES	FORMATION	AQUIFERS AND CONFINING UNITS
Quarternary	Holocene	Surficial sand deposits	Surficial aquifer
	Pleistocene	Undifferentiated Pleistocene and Pliocene deposits	
Tertiary	Pliocene		
	Oligocene	River Bend Formation[1]	Castle Hayne confining unit
	Eocene	Castle Hayne Formation[2]	Castle Hayne aquifer
	Paleocene	Beaufort Formation[3]	Peedee confining unit
Cretaceous	Upper Cretaceous	Peedee Formation	Peedee aquifer
			Black Creek confining unit
		Black Creek and Middendorf Formations	Black Creek aquifer
			Upper Cape Fear confining unit
		Cape Fear Formation	Upper Cape Fear aquifer
			Lower Cape Fear confining unit
			Lower Cape Fear aquifer
Pre-Cretaceous basement rocks			

[1]Presence limited to southern New Hanover County (Zarra, 1991).
[2]Presence limited to southern and eastern New Hanover County and southeastern Brunswick County (Zarra, 1991).
[3]Presence limited to southern New Hanover County and southeastern Brunswick County (Zarra, 1991).

Figure 1. Relationship of geological formations to hydrogeologic units in southeastern North Carolina.

The basal unit of the Cape Fear Formation contains small boulders and large gravel originating from the underlying pre-Mesozoic rocks. The lithologies of most of the formation consist of thin to thick-bedded clays and sands. The clay beds are typically pale to medium gray and range from thin-bedded to thick-bedded (from one inch to 8 feet thick.) The sand beds also vary widely in thickness. The sands are generally poorly-sorted and massive or cross-bedded. Individual sand grain size ranges from fine to very coarse. Deposition appears to have been primarily in a non-marine, probably upper delta plain environment (Sohl and Owens, 1995).

2.3. b. Middendorf Formation. The type locality for the Middendorf is a weathered exposure along the railroad tracks in Middendorf, South Carolina. Until recently the Middendorf Formation was generally thought to be the basal unit of the Cretaceous in South Carolina, more or less equivalent with the Cape Fear Formation in North Carolina. Today it is recognized as a separate formation occurring between the Cape Fear Formation and the Black Creek Group.

The Middendorf consists of intercalated, lensing, thick-bedded, light-colored sands and clays with local concentrations of clay cast conglomerates (Heron, 1958). Note that this classic description could also be typical of a part of the Cape Fear Formation, illustrating the difficulty of separating the two formations in field exposures.

According to Owens (1988), the Middendorf in a corehole near Conway, SC, consisted mainly of thin interbeds of black clay and light-colored sands. This description is typical of a large part of the Black Creek Group. The lack of faunal assemblages and only scattered reliable subsurface information also contributed to the general misunderstanding of the placement of the Middendorf. Pollen assemblages, though difficult to find, appear to be one of the few reliable ways to differentiate the Cretaceous formations. A recent pollen analysis and the discovery of *Ostrea cretacea* in coreholes in Wilmington, NC, and Charleston, SC, appear to have reliably dated the formation. (Sohl and Owen, 1995.)

The Middendorf Formation was over 300 feet thick in a corehole near Conway, SC. Its thickness in the study area is not well established and it may be absent (either not deposited or eroded) in part of the study area. The similar lithofacies distribution of the Middendorf Formation and the Cape Fear Formation in the Carolinas is probably the result of both being deposited largely in a non-marine deltaic system as delta plain and delta front facies.

2.3. c. Black Creek Group. The Black Creek was identified as a formation for almost all of the 20th century, and was attributed various ages, usually Cretaceous. It was originally termed the Black Creek Marl by Sloan (1907) for exposures along Black Creek, a tributary of the Peedee River, near Darlington in South Carolina.

The characteristic Black Creek thin, interbedded, black clays and light-colored sands can be found in the river valleys north of the Peedee River (Sohl and Owens, 1995). Although the type section is in South Carolina, much work has been done along the Cape Fear River because of the excellent exposures of the Black Creek there.

In 1988, Owens subdivided the Black Creek into 3 distinct formations along the Cape Fear River: the Tar Heel Formation, the Bladen Formation, and the Donoho Creek Formation. Together the 3 formations are considered by Sohl and Owens (1995) as the Black Creek Group and appear to represent the shifting lobes of a deltaic environment.

2.3. c. 1. Tar Heel Formation. The Tar Heel Formation disconformably overlies the Middendorf Formation. The Snow Hill Marl Member present in some areas does not occur in the study area. The formation outcrops from Tar Heel to near Walker's Bluff

along the Cape Fear River. Lithologically, the Tar Heel in the study area consists of thin black carbonaceous clays interbedded with micaceous white-to-buff sands. Silicified and lignitized logs are common.

2.3. c. 2. Bladen Formation. The Bladen Formation outcrops along the Cape Fear River downstream of Elizabethtown. The Bladen Formation consists largely of laminated dark clay with thin interbeds of glauconitic sand.

2.3. c. 3. Donoho Creek Formation. The Donoho Creek Formation is the uppermost member of the Black Creek Group. It is also bounded by disconformities, indicating breaks in the sequence of deposition or intervals of erosion which are common in the Black Creek Group.

2.3. d. Peedee Formation. The Peedee Formation disconformably overlies the Black Creek Group. It consists of gray to light brown silty fine-grained quartz sands and clays, sometimes with shells. Glauconite, pyrite, and phosphorite are also commonly present. It is thought to represent a marine shelf depositional environment.

2.4. Tertiary and Quaternary Periods. Major sedimentary deposits representing most of the Tertiary and Quaternary Periods are absent in the study area. The Paleocene Beaufort Formation, the Eocene Castle Hayne Formation, and the Oligocene River Bend Formation all occur in nearby counties but are not present in the study area because it lies on the axis of the Cape Fear Arch. During the time when these formations were being laid down by multiple transgressions and regressions of the sea, uplift was occurring along the Cape Fear Arch. Current thought is that this uplift kept the study area above the highest stand of the seas and/or any deposits laid down were fully eroded and not preserved in the stratigraphic record.

Some minor deposits of Tertiary and Quaternary materials may be present in the study area. These could include the Pliocene Bear Bluff Formation on uplands outside the river valleys. The Pleistocene Socastee and Penholoway formations may occur in the Cape Fear River Valley on the northeast side of the river. These will be treated as Undifferentiated Pleistocene and Pliocene Deposits in this report.

Quaternary Age sands, silts, and clays form the topmost layer of sediments. These sediments, including any Undifferentiated Pleistocene and Pliocene Deposits, are considered as forming the Surficial Sands Aquifer.

3.0 **Major Hydrogeological Units.**

There are 5 major fresh water aquifers and 4 confining units in the project study area. Some aquifers and confining units include parts of multiple formations or just a part of a formation. The 4 deepest aquifers are confined by the confining units (sometimes referred to as aquicludes or aquitards). The 5th aquifer is the Water Table Aquifer. From oldest to youngest (and deepest to shallowest) the aquifers are: the Lower Cape Fear Aquifer, the Upper Cape Fear Aquifer, the Black Creek Aquifer, the Peedee Aquifer, and the Surficial (or Water Table) Aquifer. The right-hand column of **Figure 1** includes the aquifers and confining units found in southeastern North Carolina. **Figure 2** illustrates

the generalized hydrogeologic cross section in a west-northwest to east-southeast (left to right) direction across the study area approximately 5 miles below Lock and Dam No. 3.

3.1 Lower Cape Fear Aquifer. The deepest aquifer in the project study area is the Lower Cape Fear Aquifer. It consists of hydraulically-connected alternating layers of fine- to coarse-grained poorly-sorted sand, silt, and clay from the lower part of the Cape Fear Formation. The Lower Cape Fear Aquifer is thickest, approximately 80 feet thick (Lautier, 2006), at the southeast end of the study area. It thins progressively northwestward and pinches out at depth against the basement rock several miles northwest of Elizabethtown. Because of this pinchout and the occurrence of the Lower Cape Fear Aquifer only in the deep subsurface (elevation 350 to 700 feet below MSL in the study area) it has the lowest recharge rate of the 5 aquifers (Lautier, 2006).

Outside the project study area, most of the Lower Cape Fear Aquifer contains salt water. Both fresh and salt water occur inside the study area dependent on location. The salt water occurs in the southeastern part of the study area near the Columbus/Pender County line. It extends up the Cape Fear River Valley to a point several miles above Lock and Dam No. 1. Here the fresh water/salt water transition zone begins at the top of the aquifer. The amount of fresh water in the aquifer increases with distance up the valley to the end of the transition zone just south of Elizabethtown. From here northwest to where the aquifer pinches out it contains only fresh water. The fresh water occurs in a northeast-southwest trending zone from the transition zone to the aquifer pinchout at depth several miles northwest of Elizabethtown, about midway between Lock and Dam No. 2 and Lock and Dam No. 3.

The Lower Cape Fear Aquifer is used as a water source by the town of Elizabethtown. While the aquifer occurs at a depth of hundreds of feet in the study area, its ground water is under artesian conditions which cause it to rise to a level between 60 and 80 feet below MSL where it is present in the project area. Fresh water from the Lower Cape Fear Aquifer is of high quality requiring little processing for use.

3.2 Lower Cape Fear Aquifer Confining Unit. This is the deepest aquifer confining unit in the study area. It consists of a relatively impermeable concentration of red, pink, and yellow-brown colored silts, clays, and thin sands within the Cape Fear Formation. Significant hydraulic head differentials exist across these confining beds isolating 2 separate aquifers existing in the Cretaceous Cape Fear Formation. The Lower Cape Fear Confining Unit is about 65 feet thick in the southeastern part of the study area. It also pinches out several miles northwest of Elizabethtown.

3.3 Upper Cape Fear Aquifer. The Upper Cape Fear Aquifer underlies the entire study area. It consists of essentially the same type of alternating lithologic units as are present in the Lower Cape Fear Aquifer, but from the upper part of the Cape Fear Formation. The clays and silts are generally red, pink, and yellowish gray. The sands are feldspathic and are subrounded to subangular. Accessory minerals include pyrite, marcasite, and siderite. The Upper Cape Fear Aquifer is about 40 feet thick from a few miles northwest of Elizabethtown north into southern Cumberland County. It increases to approximately 160 feet thick in the southeastern third of the study area.

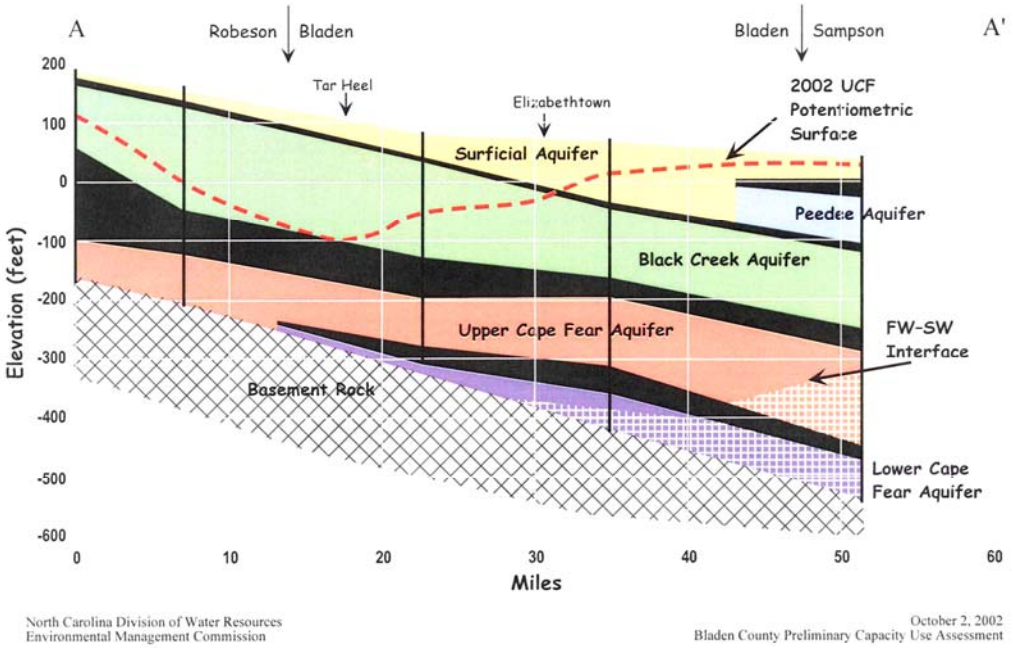


Figure 2. Hydrogeologic cross-section from west-northwest to east-southeast through Bladen County.

Southeast of the study area the Upper Cape Fear aquifer contains salt water. There is a salt water-fresh water transition zone in this aquifer beneath Lock and Dam No. 1. Northwest of the transition zone the aquifer contains fresh water of good quality. Historically the aquifer has been under artesian conditions; overpumping of ground water has altered this in much of the study area.

The Upper Cape Fear Aquifer is used for municipal water supply by Elizabethtown and White Lake, NC. Smithfield Foods near Tar Heel also pumps a large volume of water (approximately 2 mgd) each day.

3.4 Upper Cape Fear Aquifer Confining Unit. This unit consists of clay, silt and thin interbedded sands from the upper part of Cape Fear Formation, the Middendorf Formation where present, and/or the lower part of the Black Creek Group. The unit ranges in thickness from 20 ft. to 120 ft. Unlike most strata, it thickens to the northwest and is thickest near the Cumberland County line.

3.5 Black Creek Aquifer. The Black Creek Aquifer is present under the entire study area. It consists of alternating sands and clays of the Black Creek Group. The sands are generally fine- to medium-grained, gray to olive gray, and poorly-sorted. The clays are generally gray to black and lignitic. Individual sand and clay beds are typically 10 to 20 feet thick. Beds of the Middendorf Formation may be included in this aquifer where they exist, especially in Cumberland County.

The aquifer is generally confined in the uplands adjacent to the river valley. It becomes semi-confined to unconfined where the confining unit has been eroded in the river valleys, especially near Lock and Dam No. 3 in northwest Bladen County and southern Cumberland County. The aquifer is approximately 150 feet thick at each of the 3 locks, and thins to less than 100 feet in Cumberland County.

The Black Creek Aquifer contains fresh water of good quality throughout the study area. Artesian conditions exist throughout most of the study area. Recharge to the Black Creek Aquifer occurs from the downward movement of ground water from overlying aquifers, especially from the Surficial Sands Aquifer where the confining unit is not continuous.

The Black Creek Aquifer is used for municipal water supply by White Lake and Elizabethtown and also is pumped by Smithfield Foods. An aquifer test near White Lake produced a transmissivity of 289 sq. ft./day.

3.6 Black Creek Aquifer Confining Unit. This confining unit is 20-40 feet thick in the lower two-thirds of the study area and thins to less than 20 feet in the upper one-third. It consists of clay and silt with thin sand interbeds that occur in the lower part of the Peedee Formation and the upper part of the Black Creek Group. Beyond the updip limit of the Peedee Formation, lower Quaternary clay and silt beds can make up part of this confining unit. The confining unit is continuous at Lock and Dam No. 1 and Lock and Dam No. 2; the Cape Fear River has eroded through it at Lock and Dam No. 3.

3.7 Peedee Aquifer. The Peedee Aquifer is present in the southern half of the study area, from approximately the town of Elizabethtown south. It dies out just south of the center of Elizabethtown, as does its confining unit. It has a gentle southeast dip of about 5 feet per mile. The top surface of the aquifer is somewhat hummocky in the central part of the study area, apparently due to an erosional cut-and-fill between Quaternary and Peedee age deposits. The aquifer is generally wedge shaped in (NW-SE) profile, thickening toward the southeast to a maximum of approximately 160 feet.

The Peedee Aquifer is often semi-confined to unconfined in the study area due to a lack of confining unit continuity. East of the area, the Peedee commonly contains 2 separate aquifers; however, the requisite confining unit is not present in the study area.

The Peedee Aquifer is also under artesian conditions. Recharge of the aquifer is substantial in areas where its confining layer is discontinuous. Like the Black Creek Aquifer, it contains only fresh water in the study area. Water quality is acceptable but not as good as in the underlying confined aquifers due to dissolved minerals. In Bladen County, the Peedee is used by the towns of Bladenboro and Clarkton for water supply (Lautier, 2006).

3.8 Peedee Aquifer Confining Unit. The Peedee Aquifer confining unit is present in the study area from Lock and Dam No. 1 to near the center of Elizabethtown. It has a maximum thickness of 20 ft. in one well in Bladen County. It may reach a maximum thickness of approximately 30 ft. in the study area. It thins northwestward and dies out near the center of Elizabethtown.

3.9 Surficial Aquifer. The composition of the Surficial or Water Table Aquifer varies widely depending on local subsurface conditions. In general it consists of the veneer of Holocene sands and the underlying permeable materials down to the uppermost confining unit. In the study area this can range from the Peedee Confining Unit in the south down to the Upper Cape Fear Confining Unit in the north. The total thickness of the Surficial Aquifer can be as much as 150 feet. This aquifer is directly recharged by rainfall.

4.0 **Ground Water Flow**.

> *"Ground water flows in a rather complex three dimensional pattern through the subsurface in a multi-layered coastal plain environment. Flow occurs laterally through aquifers from recharge to discharge areas along flowlines which parallel directions of steepest hydraulic gradient. Flow also occurs vertically upward to discharge areas or downward in recharge areas in response to differences in hydraulic head between aquifers." (Lautier, 2006)*

4.1 Recharge-Discharge. Recharge to the aquifer system occurs in several ways, all related, sometimes very indirectly, to rainfall. Rain directly recharges only the Surficial Aquifer throughout the study area. Recharge to the confined aquifers occurs from the Surficial Aquifer and also occurs in areas updip (northwest of) the study area where the deeper formations are exposed or near the surface. These recharge flows are generally downward. Water also migrates laterally within individual aquifers.

The biggest natural discharge area in the study area is the Cape Fear River. When undisturbed by man, flow within an aquifer away from the river is generally to the southeast. Near where the river intersects an aquifer, lateral flow lines within that aquifer angle toward the river; flow angles upward from aquifers below the river. The Cape Fear River and some tributaries have downcut into the Black Creek Aquifer in southern Cumberland County and northern Bladen County. In these areas the Black Creek Aquifer may discharge directly to the river or stream.

4.2 Pumping and Water Levels. Withdrawals of water from wells distort the natural recharge-discharge balance. The pumping well becomes a discharge point and flow lines re-orient towards the well. Significant (industrial strength) pumping can lower the potentiometric head in the vicinity of the well.

5.0 **Aquifer Conditions and Problems**.

5.1 Cones of Depression. A very large cone of depression has formed in the potentiometric surface of the Upper Cape Fear Aquifer. It is centered on Tar Heel and underlies much of Bladen, Cumberland, and Robeson counties **(Figure 3).** Smithfield Foods has been pumping approximately 1.9 mgd from the Upper Cape Fear Aquifer and the Black Creek Aquifer since 1992. Water levels in the Upper Cape Fear Aquifer have been lowered about 4-5 feet per year and are down 175 feet to the top of the aquifer. The potentiometric surface of the Black Creek Aquifer has been lowered about 45 feet and displays a smaller cone of depression.

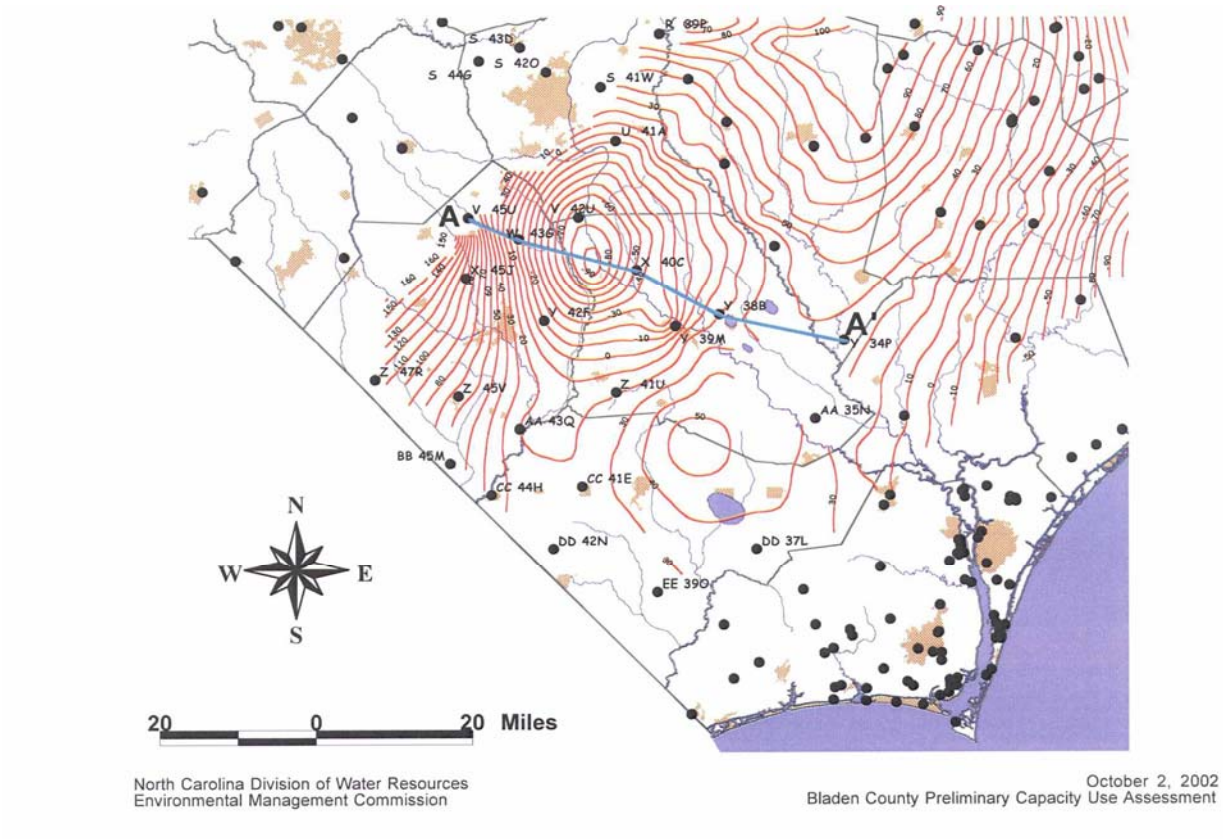


Figure 3. Upper Cape Fear Aquifer Potentiometric Surface.

The unusually large impact of this high volume pumping on the Upper Cape Fear Aquifer appears to be exacerbated by a low rate of recharge, low aquifer transmissivity, effective isolation by thick impermeable confining units, and poor well field design (Lautier, 2006).

5.2 Aquifer Dewatering. Overpumping which leads to lowering of the water level below the top of the aquifer initiates aquifer dewatering. Dewatering induces compaction of the aquifer with the resulting loss of pore space. Dewatering also introduces air into the aquifer and causes a *permanent* loss of yield.

Dewatering causes other problems for water wells. As the water level declines, pump intakes must be lowered. Pump types must be changed. Well yield drops and additional wells must be drilled to supply the same amount of water. Neighboring wells appear to "dry up" and need to be drilled deeper or replaced. Overpumping the additional wells continues to lower the water level, resulting in the loss of the resource for all users.

The Upper Cape Fear Aquifer near Tar Heel is either at the threshold of being dewatered or actually being dewatered. A report of the water level from one well indicated a water level well down into the aquifer.

5.3 Salt Water Intrusion. Overpumping and poor design of well fields can cause upconing of salt water and contamination of the well or field. One well at Tar Heel was reported pumping salt water although salt water was not previously reported in the area. Suspicion is now on a fractured basement rock aquifer which may contain salt water. Prevention is preferred to the cure in salt water intrusion as treatment of salt water is much more costly than treatment of fresh water.

5.4 Capacity Use Area Designation. When ground water is being used in an unsustainable way, the Water Use Act of 1967 allows the Environmental Management Committee (EMC) to request a Capacity Use Investigation. A preliminary Capacity Use Assessment for Bladen County was conducted in 2002. **Figure 4** shows water levels from wells across the county. Water levels in the Upper Cape Fear Aquifer were declining 3-4 feet per year since 1996. Close monitoring of ground water withdrawals at Tar Heel were recommended. The Water Allocation Committee requested a Capacity Use Investigation which was begun in 2002 and will be conducted over the next several years.

If a Capacity Use Area is established, stringent regulations will be enforced. Users of 10,000 gpd or more will likely be required to be permitted. Ground water withdrawals would likely be cut back. Well intakes will be required to be placed at the top of the aquifer in use to avoid any chance for dewatering. Some ground water users will be encouraged to use other available aquifers. Others will find it necessary to switch to treatment of surface water to replace ground water withdrawals.

The Black Creek Aquifer near Tar Heel is not as severely impacted as the Upper Cape Fear Aquifer. However, substituting Black Creek Aquifer water for Upper Cape Fear Aquifer water could lower water levels enough to disrupt hundreds of domestic use wells. This would make a good case for substituting Cape Fear River water for the ground water currently in use. Alternatively, an Aquifer Storage and Recovery (ASR) scheme could treat river water and inject it into an aquifer when there is an ample supply of river water. During the dry season when the river level is low, the stored water could be pumped from the aquifer to supplement the river water treatment plant.

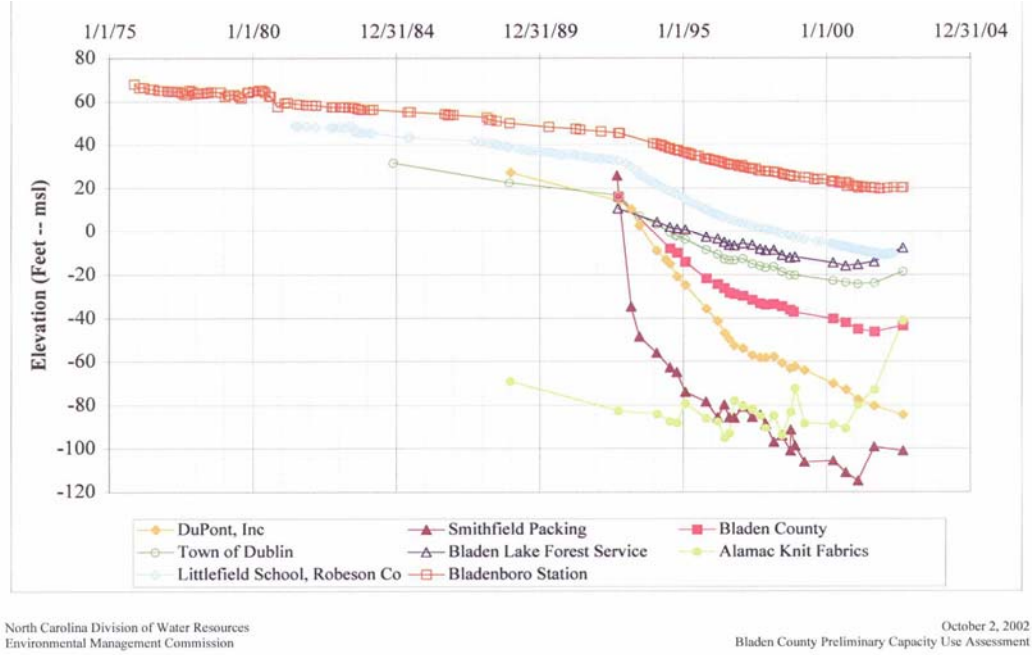


Figure 4. Water levels in wells from 1975 to 2002.

6.0 **<u>Quantitative vs. Qualitative Analysis of Effects of Dam Removal</u>**.
The assumption was made that the river level would be lowered a maximum of 10 feet if all 3 dams were removed. Quantitative analyses are necessary to predict how much the ground water level in each aquifer would drop as a result of the lowered river level. Detailed information is necessary to perform this type of quantitative analysis. A subsurface investigation, aquifer testing program, and extensive modeling would be required to produce such precise results.

Qualitative analysis, however, could give an indication of the effects within an order of magnitude. For example, an aquifer composed of clean 3-6 inch river gravel may have a very high permeability near 1 x 10 cm/sec. Water held in this aquifer would drain freely.

6.1 <u>Confined Aquifers</u>. Waters held in confined aquifers are by definition sequestered by a confining unit of low permeability materials. A typical permeability would be on the order of 1 x 10^{-7} cm/sec. An aquifer of silty sand may have a permeability of 1 x 10^{-3} cm/sec. There would be no lowering of the aquifer water level in this case since the aquifer has a much higher permeability and is under artesian conditions. Any water that leaked through the confining unit would immediately be replaced by water easily flowing through the higher permeability aquifer materials.

6.2 <u>Unconfined Aquifer</u>. (If the confining unit of a typically confined aquifer were breached the aquifer would fall under this category.) Assuming a clean sand for the aquifer medium with a permeability of 1 x 10^{-2} cm/sec. This aquifer could be expected to drain at the face where the water level was lowered. However, there would not be open-channel flow like the gravel above. The sand could be expected to seep due to a resistance to flow through the small pore spaces between sand grains. The seep would try

to equilibrate with the river level but the water table could be expected to rise rapidly with distance away from the bank.

7. **Ground Water Quality: Manmade Pollutants**.
One chemical was noted in research for this study that is present in some water wells near Lock and Dam No. 3. Ammonium Perfluorooctanoate (also known as C8) has been manufactured by the DuPont Company at a "state-of-the-art, $23,000,000 plant" along the Cape Fear River in Bladen County near the Cumberland County line since late in 2002. It is used in the manufacture of *Teflon, Goretex,* and other products such as water-resistant paper coatings. The river pool at this location is controlled by William O. Huske Lock and Dam.

C8 *may be* carcinogenic, mutagenic, and/or teratogenic. It is significant that it is already present in well water since its manufacture at this plant began only 5 years ago. No data was available to indicate which aquifer(s) it has been found in.

8. **Summary and Conclusions.**

8.1 The Wilmington Harbor area is adequately covered in the 1996 ground water study (Lautier).

8.2 This study and report concentrated on the 90 mile reach of the Cape Fear River which includes the locks and dams and their pools. Clarification of the local geology came late to this area, partly due to lack of outcrops and similarity of formations.

8.3 Part of the study area – the upland areas outside the river valley – are likely recharge areas for the Surficial Aquifer. The river and tributary valleys are aquifer discharge areas. This is evidenced by springs near some of the locks and dams. Confined aquifers are recharged updip, from overlying and underlying aquifers, and also laterally.

8.4 Removal of a lock and dam should not cause a change in any confined aquifer. The major aquifers are by definition isolated (confined). The only possible change will be to the Surficial Aquifer. There will be less head inhibiting drainage of water table aquifers after removal of a lock and dam. There will not be a U-tube-like relationship of water table aquifer to river level, but a swifter drainage of the aquifer adjacent to the river should be expected.

8.5 Population growth, agriculture, and commercial and industrial development have demanded an increased water supply in the area. Ground water supply development has largely met that demand.

8.6 Industrial use (overpumping) of the Black Creek and Upper Cape Fear Aquifers may be causing dewatering and damaging the Upper Cape Fear Aquifer at Tar Heel.

8.7 Projected growth will require an increase in water supply in the study area. Encouragement should be provided for the use of surface waters in lieu of ground water especially in the Tar Heel area.

9. **Recommendations.**

9.1 Do not recommend further ground water studies/modeling in the Wilmington Harbor area. The existing Wilmington Harbor Ground Water Study addresses aquifer issues in the harbor area.

9.2 Minimal impact to the surficial aquifer is expected in the study area. No impact to confined aquifers. Recommend no further ground water studies for modifications to the locks and dams.

9.3 Recommend encouraging use of surface water from the Cape Fear River to reduce or prevent potential damage to the Upper Cape Fear Aquifer from the dewatering of the aquifer. Recommend the PDT establish the benefit the locks and dams will provide to the development of the Cape Fear River as a water supply.

END

Appendix 3.7.11

TURNING BASINS SHOALING ANALYSIS

Wilmington Harbor - 96 Act Project
Northeast Cape Fear River Turning Basin(s) Shoaling Analysis

October 2005

Wilmington Harbor - 96 Act Project Northeast Cape Fear River Turning Basin(s) Shoaling Analysis

Prepared for
United States Army Corps of Engineers Wilmington District
By

Edward Albada, P.E.
Taylor Engineering
North Carolina, Inc.
P.O. Box 1809
Carolina Beach, NC 28428
(910)458-8602

Alexandra Carvalho, Ph.D.
Taylor Engineering, Inc.
9000 Cypress Green Dr
Suite 200
Jacksonville, FL 32250
904-731-7040

October 2005
C2005-065

TABLE OF CONTENTS

1.0 INTRODUCTION 1

2.0 STUDY AREA 3

3.0 DMS SHOALING ANALYSIS - NE CAPE FEAR RIVER 6

3.1 DMS-Data Manager Data Consolidation 6

3.1.1 Channel Data 6

3.1.2 Survey Data 10

3.2 DMS-Data Manager Historical Shoaling Analysis - Existing Conditions 11

3.3 DMS-Data Manager Historical Shoaling Analysis - Proposed Conditions 13

3.4 DMS-Manual Shoal Characterizations 17

3.4.1 Northern NE Cape Fear River Shoal 19

3.4.2 East-Central NE Cape Fear River Shoal 19

3.4.3 Southern NE Cape Fear River Shoal 20

3.4.4 NE Cape Fear River Turning Basin Shoal 21

3.5 Summary 21

4.0 DMS-ANALYTICAL TOOLOBOX: HYDRODYNAMIC MODEL 23

4.1 Model Setup 23

4.1.1 Model Schematization 24

4.1.2 Model Boundary Conditions 25

4.1.3 Limited Model Calibration 27

4.2 Model Simulations 28

4.3 Sediment Transport Methodology 31

5.0 DMS-ANALYTICAL TOOLBOX: HYDRODYNAMIC MODEL RESULTS 34

5.1 Existing Simulation 34

5.2 Authorized Simulation 40

5.3 B1 Simulation 43

5.4 C1 Simulation 47

5.5 C2 Simulation 51

5.6 C3 Simulation 55

5.7 Quantitative Analysis 59

5.7.1 Tide Range 59

5.7.2 Flow Rates 59

5.7.3 Average Velocity 60

5.7.4 Shoaling Rates 61

6.0 CONCLUSIONS 64
6.1 Additional Considerations 64
7.0 REFERENCES 65

APPENDICES

LIST OF FIGURES

Figure 2.1 Location Map 3
Figure 2.2 NOAA-predicted Tide Locations 5
Figure 3.1 NE Cape Fear River Channel Design Alternatives (1 of 2) 8
Figure 3.1 (Cont) NE Cape Fear River Channel Design Alternatives (2 of 2) 9
Figure 3.2. NNE Cape Fear River Historical Shoaling Areas, Volumes, and Rates for the Authorized Depth Channel Template (-25/-32 foot project) 12
Figure 3.3 1995 to 2003 Bathymetric Differences within the Surveyed Project Area 14
Figure 3.4 2003 to 2005 Bathymetric Differences within the Surveyed Project Area 15
Figure 3.5 1995 to 2005 Bathymetric Differences within the Surveyed Project Area 16
Figure 3.6 NE Cape Fear River Major Shoaling Areas 18
Figure 4.1 Model Mesh 24
Figure 4.2 Drainage Area vs Average Daily Flows, Coastal Plain, New Hanover County, NC 26
Figure 4.3 Comparison of NOAA-predicted and Simulated Time Series 28
Figure 4.4 Model Simulations 30
Figure 5.1 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *Existing* Simulation 38
Figure 5.2 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *Existing* Simulation 38
Figure 5.3 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *Existing* Simulation 39
Figure 5.4 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *Existing* Simulation 39
Figure 5.5 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow *for Authorized* Simulation 41
Figure 5.6 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *Authorized* Simulation 41
Figure 5.7 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *Authorized* Simulation 42
Figure 5.8 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *Authorized* Simulation 42
Figure 5.9 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *B1* Simulation 44

Figure 5.10 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *B1* Simulation 44
Figure 5.11 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *B1* Simulation 45
Figure 5.12 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak flood flow for *B1* Simulation 45
Figure 5.13 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *B1* Simulation 46
Figure 5.14 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *B1* Simulation 46
Figure 5.15 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C1* Simulation 48
Figure 5.16 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C1* Simulation 48
Figure 5.17 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak ebb flow for C1 Simulation 49
Figure 5.18 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C1* Simulation 49
Figure 5.19 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *C1* Simulation 50
Figure 5.20 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *C1* Simulation 50
Figure 5.21 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C2* Simulation 52
Figure 5.22 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C2* Simulation 52
Figure 5.23 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C2* Simulation 53
Figure 5.24 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C2* Simulation 53
Figure 5.25 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *C2* Simulation 54
Figure 5.26 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *C2* Simulation 54

Figure 5.27 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C3* Simulation 56
Figure 5.28 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C3* Simulation 56
Figure 5.29 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C3* Simulation 57
Figure 5.30 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C3* Simulation 57
Figure 5.31 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *C3* Simulation 58
Figure 5.32 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *C3* Simulation 58

LIST OF TABLES

Table 2.1 NOAA Tide Data 5
Table 3.1 Study Area Condition Survey History Organized by Date (Survey ID=year, month, day)..... 10
Table 3.2 1994 to 2005 Net Shoaling Rates in each Reach of the Navigation Channel..... 11
Table 3.3 Proposed Basin Area Net Change Rates between 1995 and 2005 13
Table 3.4 Total Sediment Volume above the Proposed Channel Template 17
Table 3.5 Northern NE Cape Fear River Shoal Maximum Volumes and Rates 19
Table 3.6 East-Central NE Cape Fear River Shoal Maximum Volumes and Rates 20
Table 3.7 East-Central NE Cape Fear River Shoal Maximum Volumes and Rates 20
Table 4.1 Calibration Summary..... 27
Table 4.2 Average Sediment Properties 33
Table 5.1 Tide Range for *Existing* and Proposed Simulations..... 59
Table 5.2 Flow Rates for *Existing* and Proposed Simulations 60
Table 5.3 Average Velocities for *Existing* and Proposed Simulations..... 61
Table 5.4 Shoaling Rates predicted by the Sediment Transport Model 62

1.0 INTRODUCTION

The U.S. Army Corps of Engineers' Wilmington District (USACE-Wilmington) has proposed to relocate the NE Cape Fear River turning basin downstream of its present position. USACE-Wilmington proposes to stop maintenance dredging of the present basin - located approximately 5,000 ft upstream of the Hilton Railroad Bridge - following the 2005 winter season. As a replacement, the USACE-Wilmington proposed four alternative turning basin locations/configurations. Taylor Engineering conducted this study on behalf of the USACE-Wilmington to recommend a preferred alternative from the proposed turning basins. The chief tasks assigned to Taylor Engineering include:

- Determine historic shoaling rates in the proposed turning basin's vicinity.
- Define potential shoaling rates for the four turning basin alternatives including identification of potential shoaling trends and problems.
- Recommend best alternative based on least potential of shoaling and maintenance requirements.

The Diagnostic Modeling System (DMS) provides a rapid, inexpensive, and reliable capability to develop and evaluate navigation channel operations and maintenance (O&M) alternatives based on limited hydrodynamic and sediment transport information. The DMS provides quick and concise capability to identify, categorize, and evaluate navigation channel sediment deposition hot spots for correction. Jointly developed by the U.S. Army Engineer Research and Development Center and Taylor Engineering, DMS provides an integrated decision support system that provides waterway managers and engineers with a series of data handling and analysis tools to manage current and historical information from navigation channels. Taylor Engineering applied DMS concepts to examine historical shoaling rates in the existing and proposed turning basin based on surveys and dredging records. In addition, Taylor Engineering applied the two-dimensional USACE hydrodynamic model RMA2 to estimate circulation trends and associated sediment transport. Examination of model results led to more detailed analysis of the shoaling patterns in the existing and proposed turning basin configurations.

The tools included with the DMS help engineers identify problem areas of shoaling, characterize the causes of these problems, and develop practical, cost-effective solutions. The methodology relies heavily on the use of established public domain coastal hydrodynamic models such as RMA2 in combination with a suite of engineering analyses and procedures founded upon fundamental principles of fluid dynamics and sediment transport. The DMS is not intended to provide the level of detailed information required for final design or in-depth study. Rather, it is intended to quickly diagnose the problem, categorize it according to its key characteristics, and identify typical corrective actions. Similarly, the DMS can evaluate various project options in an effort to determine a recommended alternative based on basic hydrodynamic and sediment transport principles.

The DMS comprises three components. The DMS-Data Manager, a GIS based computer program, allows the user to organize and view all relevant information within a project area. Examples of data entered to the DMS-Data Manager include digitized bathymetries, aerial photographs, GIS coverages, and shoaling history. This component gives "one-stop" access to all graphical and numerical information associated with a maintained channel. The DMS-Manual, a reference document, contains diagrams and example photographs of different shoal categories. This "web-based field guide" to shoaling problems helps identify the types of shoals by giving the user comparable examples. In addition to providing diagrams and examples, the DMS-Manual provides descriptions of each shoal and potential mechanisms that create it. The DMS-Analytical Toolbox comprises a suite of suggested analytical tools and recommendations for graphical formats for quick diagnosis of shoaling problems and investigation of possible solutions. This toolbox lists recommended software (e.g., hydrodynamic models, wave refraction/diffraction models) and recommended analytical models (e.g., sediment transport, wind generated wave). In addition, the toolbox contains recommended output formats from the models.

The goal of the following DMS analysis is to determine existing shoaling trends and expected shoaling rates in the four proposed turning basins in the Northeast Cape Fear River.

Following this brief introduction, Chapter 2 describes the project area and background. Chapter 3 describes the DMS application. Chapter 4 describes the 2-D modeling effort. Chapter 5 analyzes the results from the two approaches, and Chapter 6 summarizes the report.

2.0 STUDY AREA

Located on the Cape Fear River in New Hanover County, North Carolina (Figure 2.1), the federal navigation project serves as the primary access channel for vessels entering the Port of Wilmington. Thus, given the continually rising cost of maintenance dredging and the potential negative effects of channel shoaling on shipping, this study seeks answers to questions with serious economic consequences. Proper analysis of proposed project changes requires a general understanding of the history and physical characteristics of the NE Cape Fear River turning basin.

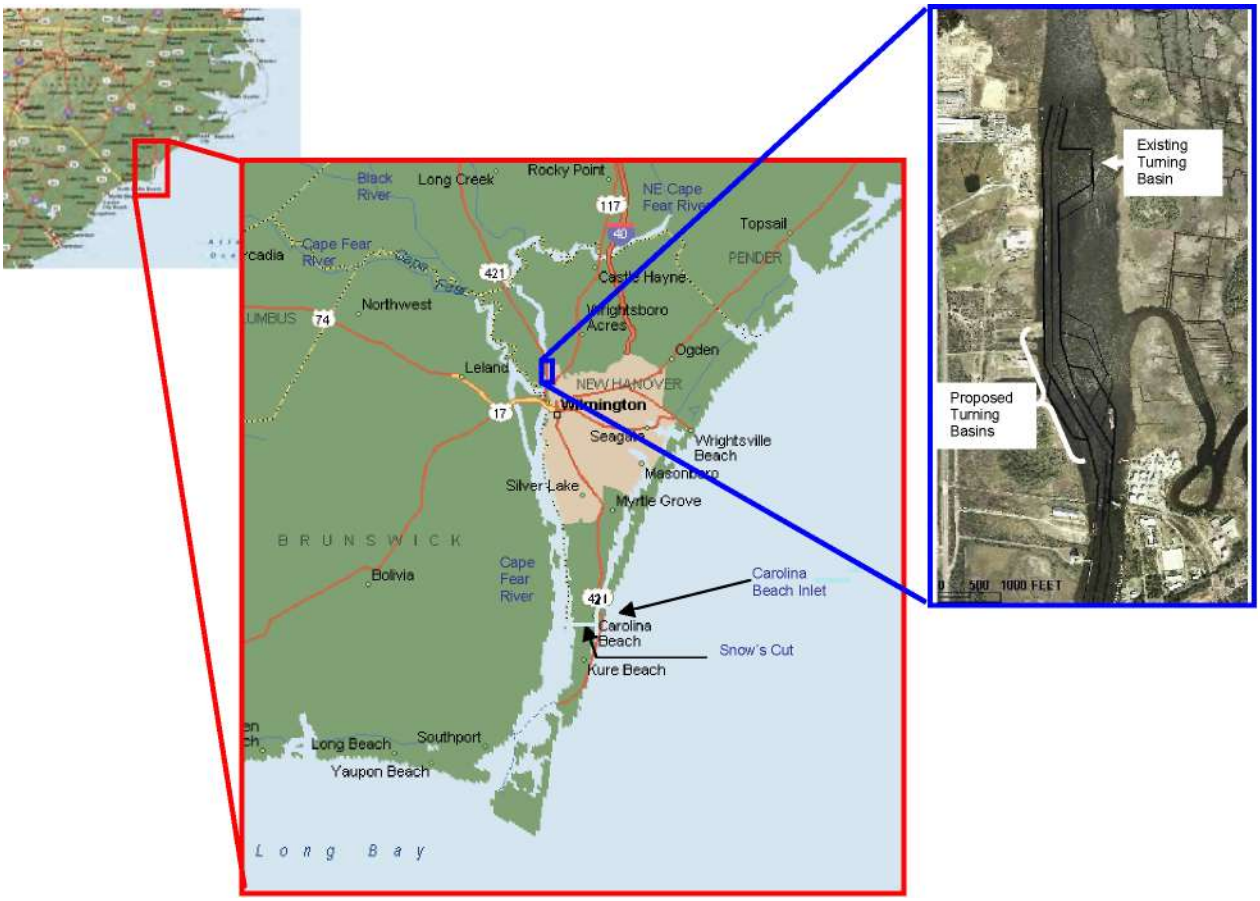


Figure 2.1 Location Map

The Cape Fear River estuarine system - located in the southeastern quadrant of North Carolina - originates in the North Carolina Piedmont region and drains 9,140 square miles (Giese, Wilder, and Parker 1985) (Figure 2.1). The lower Cape Fear River connects to the Atlantic Ocean through both the mouth of the river at Southport/Bald Head Island and through Carolina Beach Inlet (via Snow's Cut). Snow's Cut and Carolina Beach Inlet - both man-made features - were constructed in 1930 and 1952. Snow's Cut lies approximately 13.5 miles from the Cape Fear River mouth while Carolina Beach Inlet

lies approximately two miles further north. The NE Cape Fear River joins the Cape Fear River approximately 27.6 miles upstream of the Cape Fear River mouth.

Tidal influence extends 65 miles upstream of Bald Head Island on the Cape Fear River. Similarly, tidal influence extends 60 and 86 miles upstream of Bald Head Island on the Black River and NE Cape Fear River. (Giese, Wilder, and Parker, 1985).

The major port city of Wilmington is located approximately 26.6 miles upstream of Bald Head Island just downstream of the Cape Fear River/NE Cape Fear River confluence. The existing NE Cape Fear turning basin lies approximately 28.6 miles upstream of Bald Head Island. An urban tributary - Smith Creek - discharges into the NE Cape Fear in the vicinity of the proposed turning basins.

The National Oceanic and Atmospheric Administration (NOAA) publish tidal benchmarks for various locations throughout the Cape Fear basin (Figure 2.2). The mouth of the Cape Fear River at Bald Head Island (#8658901) registers a 4.49 ft semidiurnal tide range. The gage closest to the project area (Wilmington, #8658120) registers a 4.28 ft tide range. Interestingly, tidal benchmarks located between these locations report lower ranges; the Reaves Point station reports the lowest range (4.09 ft). MacAdory (2000) postulates the lower tide ranges between Bald Head Island and Wilmington indicate resonance behavior in the estuary. Table 2.1 presents the tide data, valid for the 1983 - 2001 tidal epoch, for these locations. Notably, NOAA does not provide tidal benchmark information for the Cape Fear gage station; this station is included in Figure 2.2 as a calibration reference discussed in Section 3.1.3.

MacAdory (2000) also demonstrates the effect of channel depth on tide range within the Cape Fear basin. In 1911, tide ranges varied from 4.5 ft at the mouth of the Cape Fear River to 2.7 ft at Wilmington. At that time, the reported channel depth between Bald Head Island and Wilmington was 20 ft (USAED, Wilmington, 1976 as cited in MacAdory, 2000). In 1960, tide ranges varied from 4.2 ft at Southport to 3.4 ft at Wilmington. Depths in the main Cape Fear River channel extended to 34 ft from the mouth upstream to Wilmington and 30 ft to the Hilton Bridge over the NE Cape Fear River channel (USAED, Wilmington, 1960 as cited in MacAdory, 2000).

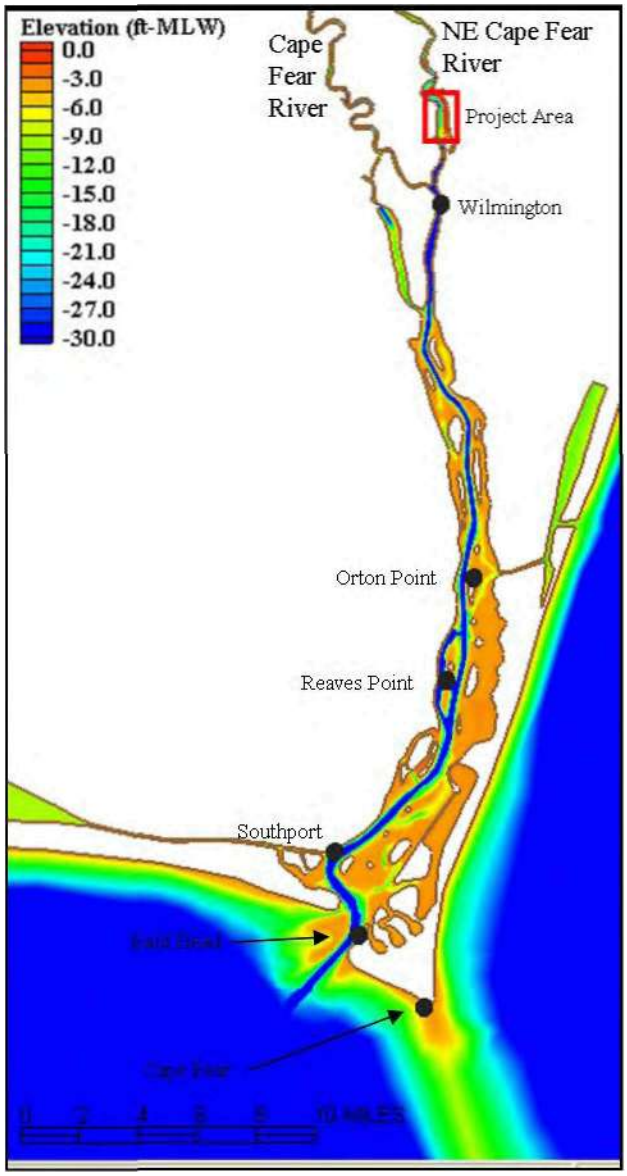


Figure 2.2 NOAA-predicted Tide Locations

Table 2.1 NOAA Tide Data

Station Name NOAA Station ID Location	**Bald Head** 8658901 33° 58.2" N 78° 00.1" W	**Southport** 8659084 33° 59.4" N **78° 01.1" W**	**Reaves Point** 8658622 34° 00.2" N 77° 57.3" W	**Orton Point** 8658501 34° 03.4" N 77° **56.4"** W	**Wilmington** 865850 34° 13.6" N 77° 57.2" W
	ft-MLW				
Mean Higher High	4.85	4.58	4.41	4.46	4.53
Mean High Water	4.49	4.24	4.09	4.17	4.28
Mean Sea Level	2.39	2.16	2.09	2.14	2.28
Mean Tide Level	2.24	2.12	2.05	2.08	2.14
Mean Low Water	0.00	0.00	0.00	0.00	0.00
Mean Lower Low	-0.17	-0.15	-0.16	-0.15	-0.15
Mean Range	4.49	4.24	4.09	4.17	4.28

3.0 DMS SHOALING ANALYSIS - NE CAPE FEAR RIVER

The NE Cape Fear River Turning Basin's DMS-Data Manager application begins with a compilation of available surveys and historical dredging data. Once Incorporated into a geodatabase framework, several DMS-Data Manager tools allow a systematic examination of historic shoaling over the entire navigation project from a single repository. The analyses facilitated by this DMS-Data Manger application section seek to identify the best turning basin alternative with respect to long term shoaling rates and minimum maintenance requirements.

This chapter begins with a review of the inputs into the DMS-Data Manager and continues with an examination of shoaling via the identification of shoaling hot spots, volumes and rates. Next, we analyze changing shoaling rates over the past 10 years within the proposed turning basins. Finally, the DMS-Manual provides a method to categorize shoal types (from the DMS-Data Manager) and identify mechanisms potentially responsible for each of the shoaling categories represented.

3.1 DMS-Data Manager Data Consolidation

3.1.1 Channel Data

The DMS application began with the entry of all relevant data concerning the channel into the DMS-Data Manager. These data, provided by the USACE-Wilmington, include: aerials, channel geometry, and bathymetric surveys from the past 11 years (1994 - 2005). For this application, the DMS data consolidation process included creation of the channel templates (channel footprints, channel centerlines, channel stations, and three dimensional channel surfaces) and upload of past channel surveys (bathymetric soundings, survey boundaries, and shoaling areas). Appendix A describes the DMS Data Manager geodatabase structure. Data entry began with a definition of the required channel templates and subsequent upload of the template designs into the DMS geodatabase. Figure 3.1 shows the six channel templates loaded into the geodatabase with channel depths referenced to MLLW. The templates include:

- *Existing Channel* (a. 200 ft wide, -25 ft-MLLW from Reach 1 Station 1+50 to Reach 3 Station 12+00; b. variable width, -32 ft-MLLW from Reach 3 Station 12+00 south to the Anchorage Basin Station 8+00). This template serves as a historical reference only and not included in the shoaling analysis.
- *Authorized Channel Depths* (a. 250 ft wide, -25 ft-MLLW from Reach 1 Station 1+50 to Reach 3 Station 12+00; b. variable width, -32 ft-MLLW from Reach 3 Station 12+00 south to the Anchorage Basin Station 8+00). This template includes the proposed channel widths and constitutes the baseline template for the historical shoaling investigation.

- *Turning Basin Option B1* (a. 250 ft wide, -39 ft-MLLW from Reach 1 Station 1+50 to the Anchorage Basin Station 8+00). This template includes widening and deepening of the existing turning basin in Reach 2 to -39 ft-MLLW.
- *Turning Basin Option C1* (a. 250 ft wide, -35 ft-MLLW from Reach 1 Station 1+50 to Reach 2 Station 26+00; b. variable width, -39 ft-MLLW from Reach 2 Station 26+00 south to the Anchorage Basin Station 8+00). This template includes the creation of a -39 ft-MLLW turning basin on the eastern side of the channel between Reach 2 Station 26+00 and Reach 3 Station 9+00.
- *Turning Basin Option C2* (a. 250 ft wide, -35 ft-MLLW from Reach 1 Station 1+50 to Reach 2 Station 29+50; b. variable width, -39 ft-MLLW from Reach 2 Station 29+50 south to the Anchorage Basin Station 8+00). This template includes the creation of a -39 ft-MLLW turning basin on the eastern side of the channel between Reach 2 Station 29+50 and Reach 3 Station 13+00.
- *Turning Basin Option C3* (a. 250 ft wide, -35 ft-MLLW from Reach 1 Station 1+50 to Reach 3 Station 2+50; b. variable width, -39 ft-MLLW from Reach 3 Station 2+50 south to the Anchorage Basin Station 8+00). This template includes the creation of -39 ft-MLLW turning basins on the eastern and western sides of the channel along Reach 3. Both turning basins start at Reach 3 Station 2+50. The western basin ends at Reach 3 Station 14+00 while the eastern basin ends at Reach 4 Station 0+00.

Figure 3.1. NE Cape Fear River Channel Design Alternatives (1 of 2)

Figure 3.1. (Cont.) NE Cape Fear River Channel Design Alternatives (2 of 2)

3.1.2 Survey Data

Table 3.1 summarizes the survey data available for the study area organized by date (Survey ID= year, month, day).

Table 3.1 Study Area Condition Survey History Organized by Date (Survey ID=year, month, day)

Survey ID	Survey Date	Elapsed Time	Reach	Description
NECF_1994103110/31	10/31/1994	n/a	1 - 4	Channel turning basin and vicinity
NECF_19950118	1/18/1995	3	1 - 4	Channel turning basin and vicinity
NECF_19950526	5/26/1995	4	1 - 4	Channel turning basin and vicinity
NECF_19950801	8/1/1995	3	1 - 4	Channel turning basin and vicinity
NECF_1 9960404	4/4/1996	8	1	Upper 800 feet of the channel
NECF_1 9960715	7/15/1996	11	1 - 4	Main channel and turning basin
NECF_19961115	11/15/1996	4	1 - 4	Main channel
NECF_19970801	8/1/1997	9	1 - 4	Main channel
NECF_1 9980421	4/21/1998	8	1	Channel section above turning basin
NECF_1 9980429	4/29/1998	8	1 - 4	Main channel
NECF_1 9981006	10/6/1998	5	1 - 4	Main channel
NECF_20021114	11/14/2002	50	1 - 4	Main channel above Hilton Railroad Bridge
NECF_20030218	2/18/2003	3	1 - 4	Main channel above Hilton Railroad Bridge
NECF_20050623	6/23/2005	28	2 - 4	Main channel and vicinity

The condition survey history includes elapsed time between survey events. The elapsed time is the basis for the yearly shoaling rate calculation. Thus, each survey serves as a "reference event," to the

next survey. DMS-Data Manager tools allow comparison of the condition surveys to the original channel template which reveals details of individual shoals in the NE Cape Fear River project area.

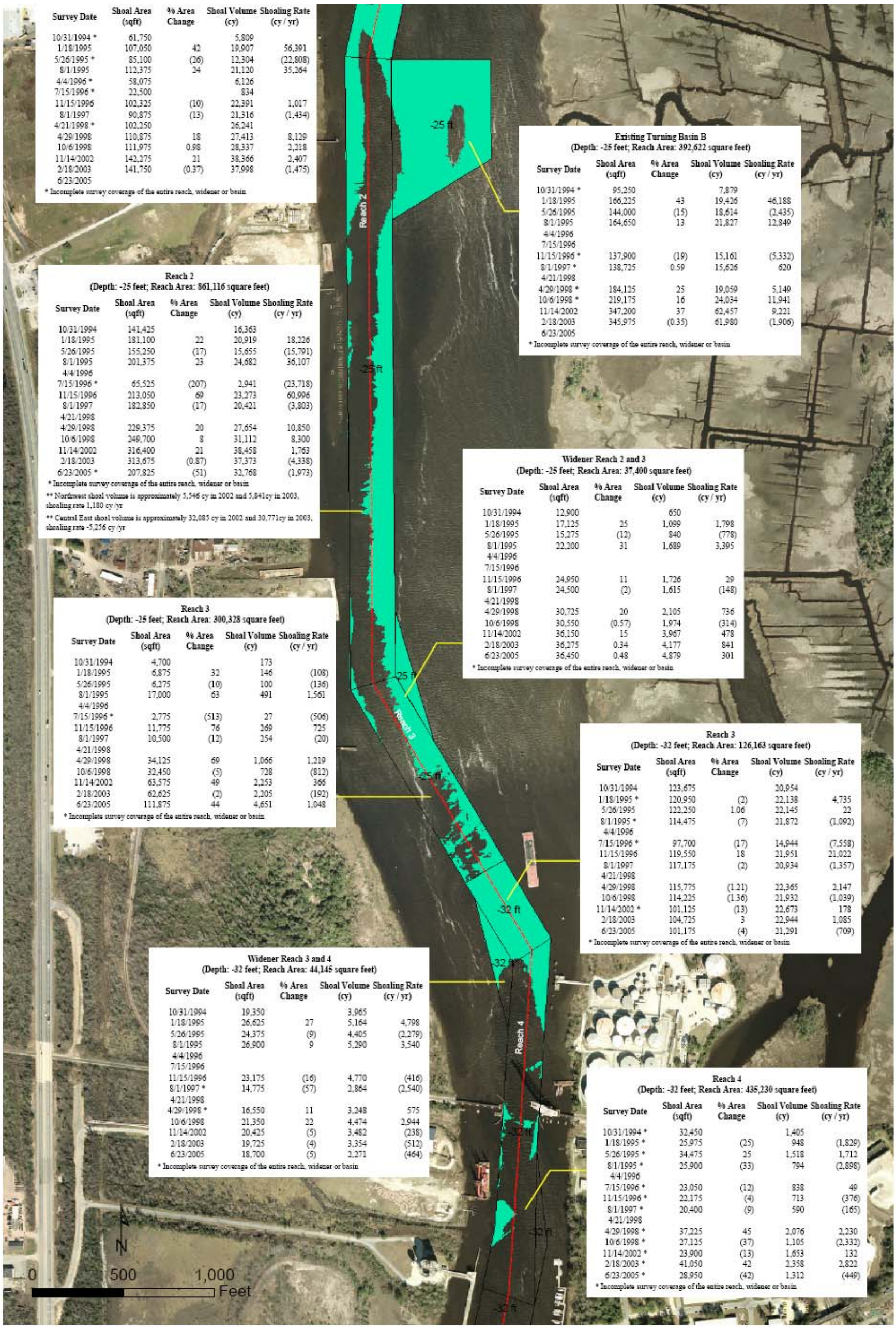
3.2 DMS-Data Manager Historical Shoaling Analysis - Existing Conditions

Each survey reveals a spatial distribution of shoals unique to that survey. The sequence of available surveys provides the means to analyze shoal morphology - spatial movement and volume calculation - for each shoal over time. Figure 3.2 shows the historical shoal volumes and shoaling rates for each reach, widener, and turning basin between 1994 and 2005. Figures 1 to 4 in Appendix B show the shoal area evolution during the same period.

Table 3.2 summarizes the 1994 to 2005 net shoaling rates in each channel reach. This table shows that with the exception of Reach 4, where survey overlap is very poor, the calculated net shoaling rate for the past 11 years is positive in the entire study area — this indicates sediment deposition (or accretion) dominates the sediment dynamics of the area. However, the tables in Figure 3.2 show that the shoaling rates in the navigation channel fluctuated between negative and positive values over these years. This demonstrates the area's dynamic nature in the absence of maintenance dredging.

Table 3.2 1994 to 2005 Net Shoaling Rates in each Reach of the Navigation Channel

Channel Reach	**Net Shoaling Rate** (cy / **yr)**
Reach 1	8,857
Reach 2	7,874
Existing Turning Basin B	8,477
Widener Reach 2 and 3	634
Reach 3 (25 foot Project)	286
Reach 3 (32 foot Project)	1,585
Widener Reach 3 and 4	541
Reach 4	-100



Reach 2
(Depth: -25 feet; Reach Area: 861,116 square feet)

Survey Date	Shoal Area (sqft)	% Area Change	Shoal Volume (cy)	Shoaling Rate (cy / yr)
10/31/1994	141,425		16,363	
1/18/1995	181,100	22	20,919	18,226
5/26/1995	155,250	(17)	15,655	(15,791)
8/1/1995	201,375	23	24,682	36,107
4/4/1996				
7/15/1996 *	65,525	(207)	2,941	(23,718)
11/15/1996	213,050	69	23,273	60,996
8/1/1997	182,850	(17)	20,421	(3,803)
4/21/1998				
4/29/1998	229,375	20	27,654	10,850
10/6/1998	249,700	8	31,112	8,300
11/14/2002	316,400	21	38,458	1,763
2/18/2003	313,675	(0.87)	37,373	(4,338)
6/23/2005 *	207,825	(51)	32,768	(1,973)

* Incomplete survey coverage of the entire reach, widener or basin

** Northwest shoal volume is approximately 5,546 cy in 2002 and 5,841cy in 2003, shoaling rate 1,180 cy /yr

** Central East shoal volume is approximately 32,085 cy in 2002 and 30,771cy in 2003, shoaling rate -5,256 cy /yr

Existing Turning Basin B
(Depth: -25 feet; Reach Area: 392,622 square feet)

Survey Date	Shoal Area (sqft)	% Area Change	Shoal Volume (cy)	Shoaling Rate (cy / yr)
10/31/1994 *	95,250		7,879	
1/18/1995	166,225	43	19,426	46,188
5/26/1995	144,000	(15)	18,614	(2,435)
8/1/1995	164,650	13	21,827	12,849
4/4/1996				
7/15/1996				
11/15/1996 *	137,900	(19)	15,161	(5,332)
8/1/1997 *	138,725	0.59	15,626	620
4/21/1998				
4/29/1998 *	184,125	25	19,059	5,149
10/6/1998 *	219,175	16	24,034	11,941
11/14/2002	347,200	37	62,457	9,221
2/18/2003	345,975	(0.35)	61,980	(1,906)
6/23/2005				

* Incomplete survey coverage of the entire reach, widener or basin

Widener Reach 2 and 3
(Depth: -25 feet; Reach Area: 37,400 square feet)

Survey Date	Shoal Area (sqft)	% Area Change	Shoal Volume (cy)	Shoaling Rate (cy / yr)
10/31/1994	12,900		650	
1/18/1995	17,125	25	1,099	1,798
5/26/1995	15,275	(12)	840	(778)
8/1/1995	22,200	31	1,689	3,395
4/4/1996				
7/15/1996				
11/15/1996	24,950	11	1,726	29
8/1/1997	24,500	(2)	1,615	(148)
4/21/1998				
4/29/1998	30,725	20	2,105	736
10/6/1998	30,550	(0.57)	1,974	(314)
11/14/2002	36,150	15	3,967	478
2/18/2003	36,275	0.34	4,177	841
6/23/2005	36,450	0.48	4,879	301

* Incomplete survey coverage of the entire reach, widener or basin

Reach 3
(Depth: -25 feet; Reach Area: 300,328 square feet)

Survey Date	Shoal Area (sqft)	% Area Change	Shoal Volume (cy)	Shoaling Rate (cy / yr)
10/31/1994	4,700		173	
1/18/1995	6,875	32	146	(108)
5/26/1995	6,275	(10)	100	(136)
8/1/1995	17,000	63	491	1,561
4/4/1996				
7/15/1996 *	2,775	(513)	27	(506)
11/15/1996	11,775	76	269	725
8/1/1997	10,500	(12)	254	(20)
4/21/1998				
4/29/1998	34,125	69	1,066	1,219
10/6/1998	32,450	(5)	728	(812)
11/14/2002	63,575	49	2,253	366
2/18/2003	62,625	(2)	2,205	(192)
6/23/2005	111,875	44	4,651	1,048

* Incomplete survey coverage of the entire reach, widener or basin

Reach 3
(Depth: -32 feet; Reach Area: 126,163 square feet)

Survey Date	Shoal Area (sqft)	% Area Change	Shoal Volume (cy)	Shoaling Rate (cy / yr)
10/31/1994	123,675		20,954	
1/18/1995 *	120,950	(2)	22,138	4,735
5/26/1995	122,250	1.06	22,145	22
8/1/1995 *	114,475	(7)	21,872	(1,092)
4/4/1996				
7/15/1996 *	97,700	(17)	14,944	(7,558)
11/15/1996	119,550	18	21,951	21,022
8/1/1997	117,175	(2)	20,934	(1,357)
4/21/1998				
4/29/1998	115,775	(1.21)	22,365	2,147
10/6/1998	114,225	(1.36)	21,932	(1,039)
11/14/2002 *	101,125	(13)	22,673	178
2/18/2003	104,725	3	22,944	1,085
6/23/2005	101,175	(4)	21,291	(709)

* Incomplete survey coverage of the entire reach, widener or basin

Widener Reach 3 and 4
(Depth: -32 feet; Reach Area: 44,145 square feet)

Survey Date	Shoal Area (sqft)	% Area Change	Shoal Volume (cy)	Shoaling Rate (cy / yr)
10/31/1994	19,350		3,965	
1/18/1995	26,625	27	5,164	4,798
5/26/1995	24,375	(9)	4,405	(2,279)
8/1/1995	26,900	9	5,290	3,540
4/4/1996				
7/15/1996				
11/15/1996	23,175	(16)	4,770	(416)
8/1/1997 *	14,775	(57)	2,864	(2,540)
4/21/1998				
4/29/1998 *	16,550	11	3,248	575
10/6/1998	21,350	22	4,474	2,944
11/14/2002	20,425	(5)	3,482	(238)
2/18/2003	19,725	(4)	3,354	(512)
6/23/2005	18,700	(5)	2,271	(464)

* Incomplete survey coverage of the entire reach, widener or basin

Reach 4
(Depth: -32 feet; Reach Area: 435,230 square feet)

Survey Date	Shoal Area (sqft)	% Area Change	Shoal Volume (cy)	Shoaling Rate (cy / yr)
10/31/1994 *	32,450		1,405	
1/18/1995 *	25,975	(25)	948	(1,829)
5/26/1995 *	34,475	25	1,518	1,712
8/1/1995 *	25,900	(33)	794	(2,898)
4/4/1996				
7/15/1996 *	23,050	(12)	838	49
11/15/1996 *	22,175	(4)	713	(376)
8/1/1997 *	20,400	(9)	590	(165)
4/21/1998				
4/29/1998 *	37,225	45	2,076	2,230
10/6/1998 *	27,125	(37)	1,105	(2,332)
11/14/2002 *	23,900	(13)	1,653	132
2/18/2003 *	41,050	42	2,358	2,822
6/23/2005 *	28,950	(42)	1,312	(449)

* Incomplete survey coverage of the entire reach, widener or basin

Figure 3.2. NNE Cape Fear River Historical Shoaling Areas, Volumes, and Rates for the Authorized Depth Channel Template (-257-32 foot project)

The green shoaling areas in Figure 3.2 indicate shoaling primarily occurs on the eastern side of the channel (See also: Figures 1 through 4 in Appendix B). These areas correspond to changes in channel direction or depth and the existing turning basin.

3.3 DMS-Data Manager Historical Shoaling Analysis - Proposed Conditions

Figures 3.3 through 3.5 show changes in elevation - erosion and deposition patterns - in the study area from 1995 to 2003, 2003 to 2005, and 1995 to 2005. The plots reveal depth changes greater than 6 feet in the proposed turning basin areas to the east and west of the channel. The figures also show erosion (red) and deposition (green) areas for the same period. Sediment deposition is evident on the eastern side of the channel coincident with the proposed *C1, C2* and *C3* turning basins. Erosion appears on the western side of the channel near the proposed *C3* turning basin.

Table 3.3 shows the elevation change rates in the turning basins for the periods 1995 to 2003, 2003 to 2005, and 1995 to 2005 as calculated from the available surveys. Table 3.3 reports change rates for the best available data. However, in certain areas, the surveys did not include sufficient overlap for a meaningful comparison.

Table 3.3 Proposed Basin Area Net Change Rates between 1995 and 2005

Turning Basin	**Net Change Rate in Proposed Turning Basin Area (cy/yr)**		
	1995 and 2003		
Turning Basin Option *B1*	7,211		
	2003 and 2005		
Turning Basin Option *C1*	2,334		
Turning Basin Option *C2*	3,418		
Turning Basin Option *C3*	**1995 and 2005**		
	East	238	5,916
	West	5,678	
	2003 and 2005		
	East	72	-181
	West	-253	

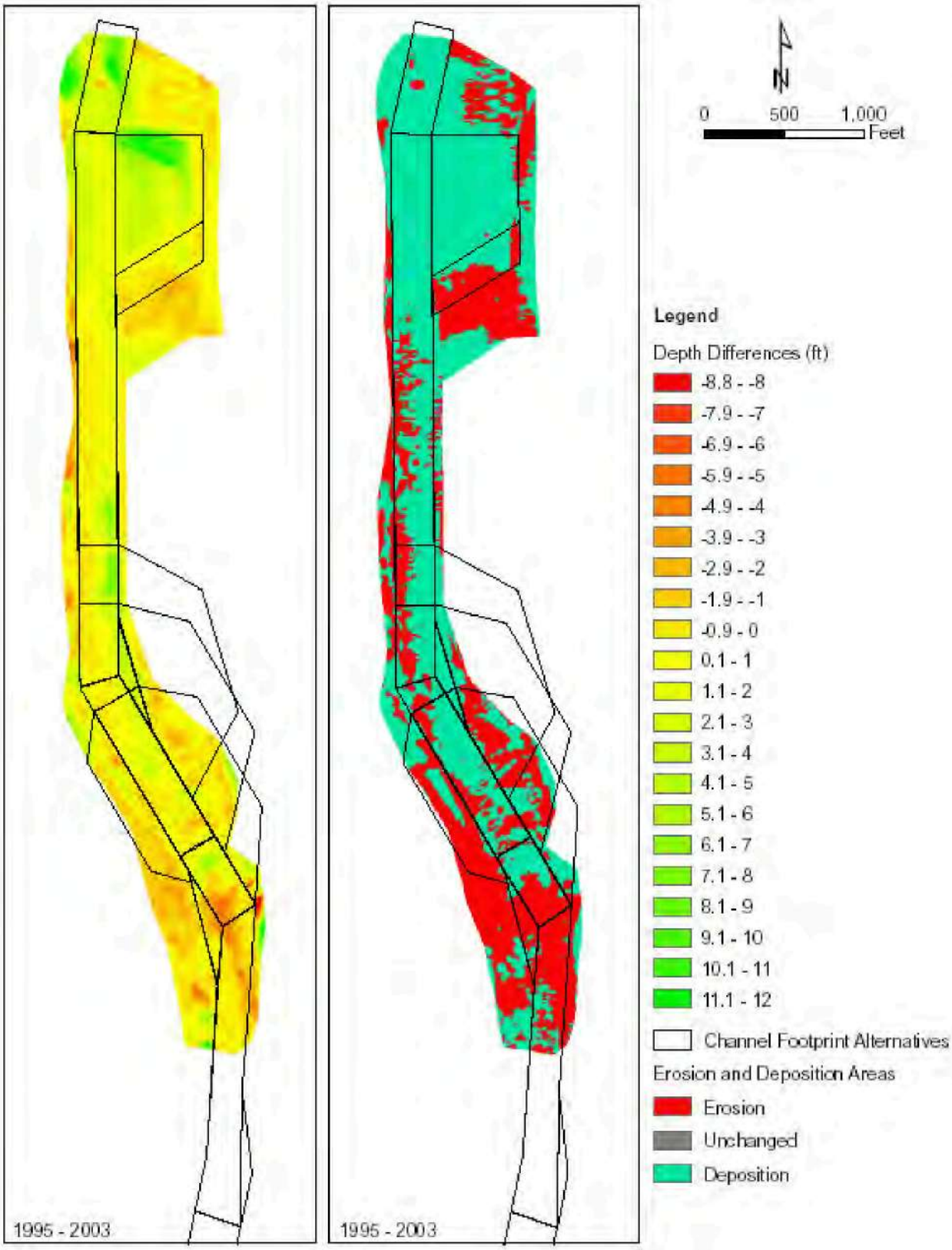


Figure 3.3 1995 to 2003 Bathymetric Differences within the Surveyed Project Area. Orange/Red = Erosion. Green = Deposition.

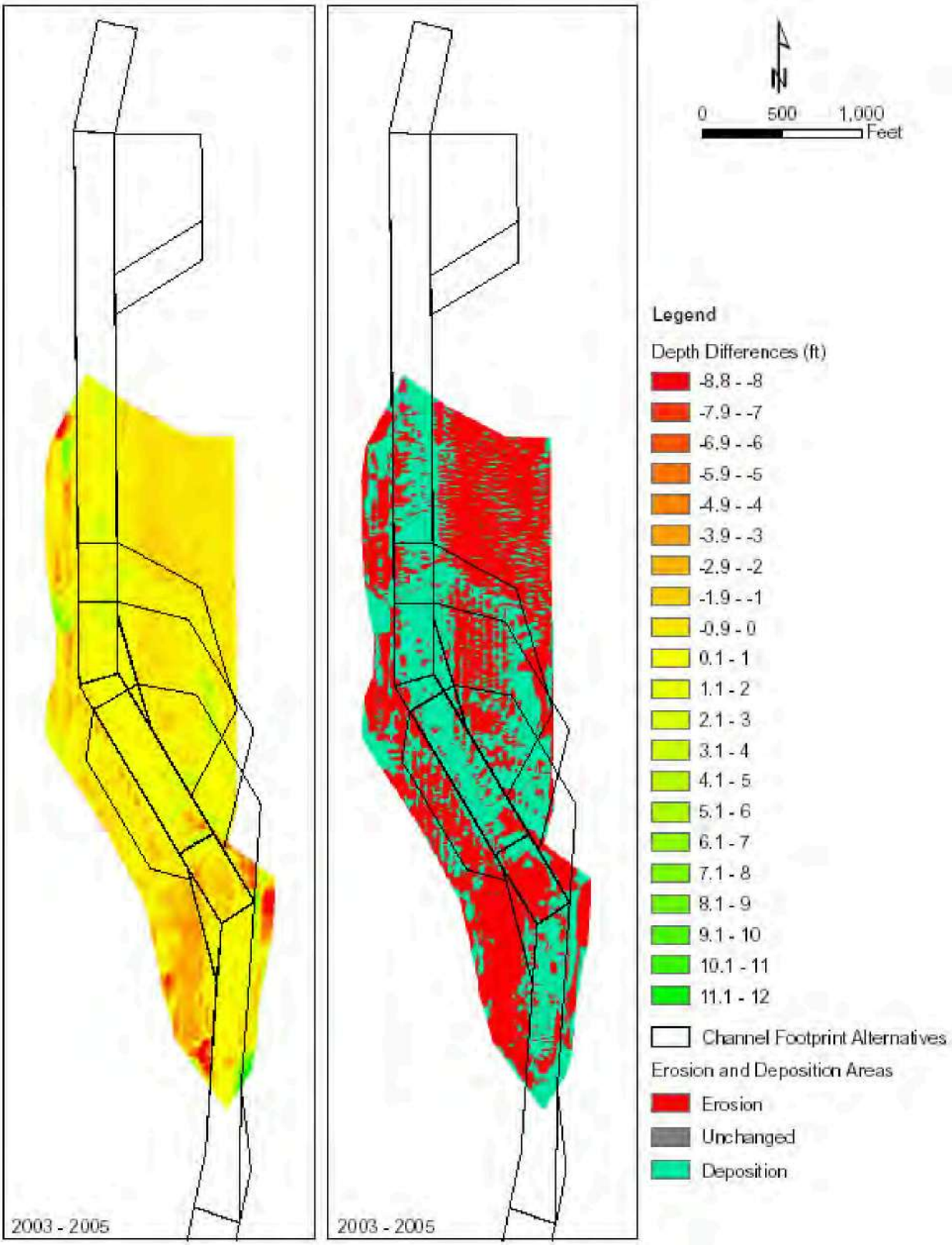


Figure 3.4 2003 to 2005 Bathymetric Differences within the Surveyed Project Area. Orange/Red = Erosion. Green = Deposition.

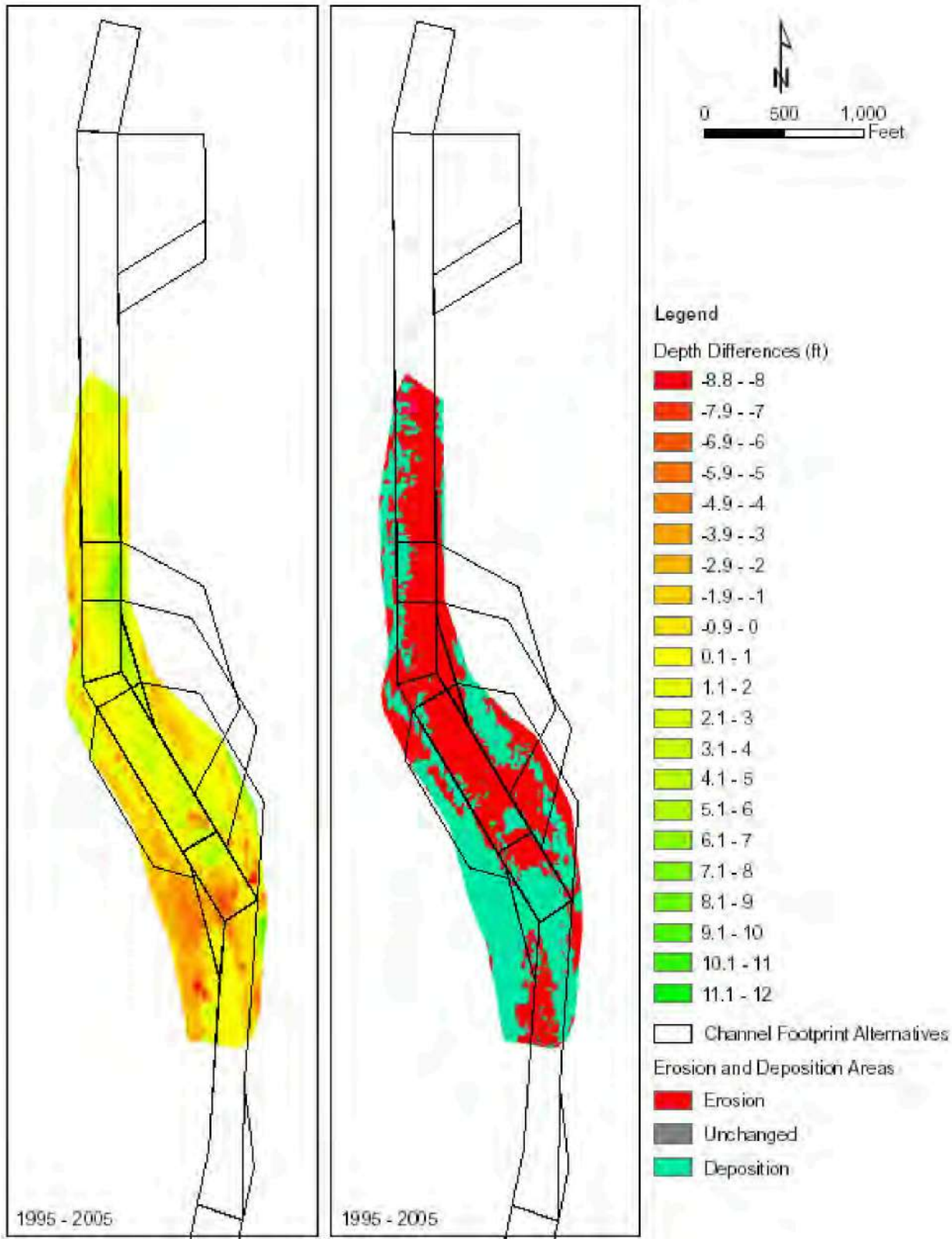


Figure 3.5 1995 to 2005 Bathymetric Differences within the Surveyed Project Area. Orange/Red = Erosion. Green = Deposition.

The variable rates show a dominance of deposition (positive rates) in the proposed turning basin areas to the east of the channel and a mix of deposition (1995 through 2005) and erosion (2003 through 2005) in the proposed turning basin areas west of the channel. Notably, the rate change from positive (deposition) to negative (erosion) may be function of inadequate overlap between the 1995, 2003, and 2005 surveys (Figures 3.3. - 3.5).

Table 3.4 summarizes the volume required to dredge to design elevation for each turning basin option based on the 2003 and 2005 surveys. Figures 1 through 4 in Appendix C show the calculated sediment volume above the proposed channel templates for each individual reach, widener, and turning basin in 2003 and 2005.

Table 3.4 Total Sediment Volume above the Proposed Channel Template

Channel Template	Sediment Volume Above the Proposed Channel Template (cy)
	2003 Survey
Option ***B1***	1,208,501
	2005 Survey
Option ***C1***	1,522,813
Option ***C2***	1,433,012
Option ***C3***	1,307,555

3.4 DMS-Manual Shoal Characterizations

The DMS-Manual categorizes shoals according to geometry and the physical processes that contribute to sediment deposition. For additional information concerning the DMS-Manual, including shoaling categories, access the Web via http://www.taylorengineering.com/DMShome/dmsDefault.htm.

The DMS-Data Manager shoaling analysis described above identified three locations within the authorized channel and two locations outside the currently authorized channel (in areas proposed for turning basins) as significant shoaling areas (Figure 3.6). The following sections describe these areas and describe potential causes of shoaling.

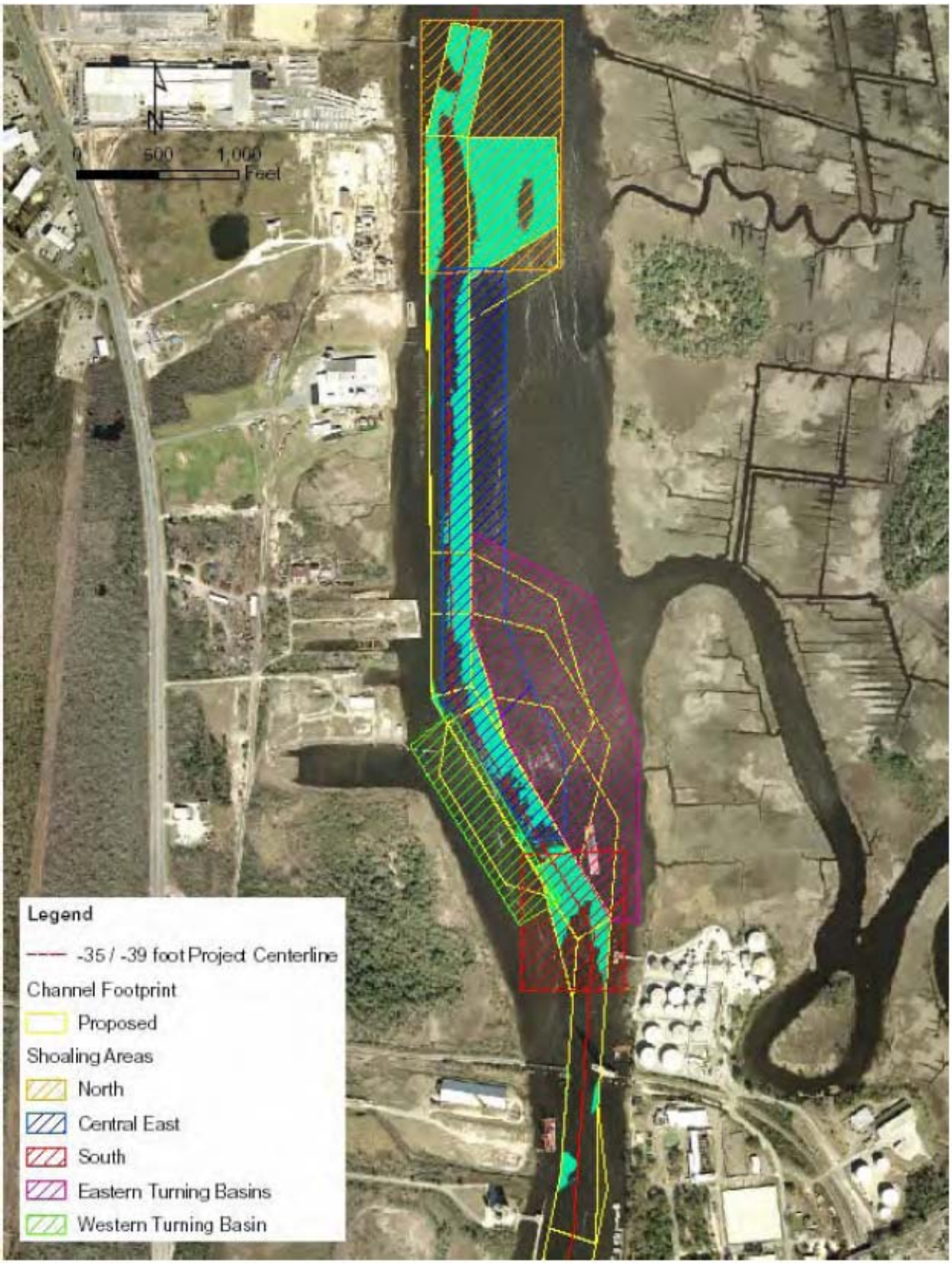


Figure 3.6 NE Cape Fear River Major Shoaling Areas

3.4.1 Northern NE Cape Fear River Shoal

The northern NE Cape Fear River shoal falls between Reach 1 Station 1+50 and Reach 2 Station 11+00 as well as existing turning basin B. Shoaling occurs both to the east and west of the channel and along the sides of turning basin B (Figure 3.2). The tables in Figure 3.2 list shoaling volumes and rates calculated with respect to the design template between 1994 and 2005. The 2003 survey produced the highest shoaling volume above the design depth template (Table 3.5).

Table 3.5 Northern NE Cape Fear River Shoal Maximum Volumes and Rates

Reach	2003 Shoaling Volume (cy)	Shoaling Rate 2002 to 2003 (cy/yr)
Reach 1	37,998	-1,475
Reach 2 Western side	5,841	+1,180
Turning Basin B	61,980	-1,906

The DMS-Manual indicates a combination of *Vertical and Horizontal Channel Expansion* may account for shoaling in this area. Both categories rely on the concept of conservation of mass. Assuming a constant flow, an increase in a channel's effective cross sectional area results in reduced velocity. Increased channel depths from dredging (vertical channel expansion) and/or increased river width (horizontal channel expansion) produce this phenomenon. The reduced velocities can result in increased sediment deposition. In addition, as sand accumulates on the sides of the river and is gradually moved into the channel, shoaling areas within the channel boundaries continue to grow and could, in the absence of dredging, fill the entire channel area.

3.4.2 East-Central NE Cape Fear River Shoal

The east-central NE Cape Fear River shoal falls along the east side of the navigation channel between Reach 2 Station 11+00 and Reach 3 Station 12+00 (Figure 3.2). The tables in Figure 3.2 list shoaling volumes and rates calculated with respect to the design template between 1994 and 2005. Once again, the 2003 survey produced the highest shoaling volumes above the design depth template (Table 3.6). Shoaling rates between 2003 and 2005 show a decrease in the Reach 2/3 widener to 301 cy/yr and an increase to 1,048 cy/yr in the -25 ft-MLLW project area of Reach 3.

The DMS-Manual indicates *Intersection Points - Riverine Sediment Sources* and *Changes in Channel Alignment - Inside of Channel Bends* most likely contribute to shoaling in this area. Smith Creek

Table 3.6 East-Central NE Cape Fear River Shoal Maximum Volumes and Rates

Reach	2003 Shoaling Volume (cy)	Shoaling Rate 2002 to 2003 (cy/yr)
Reach 2 Eastern side	30,771	-5,256
Reach 2/3 Widener	4,177	841
Reach 3 (-25 ft-MLLW)	2,205	-192

enters the Cape Fear River on the eastern side of the channel. As indicated by the shallow depths to the east of the federal channel, the Smith Creek probably serves as a continuous source of sediment. However, this study did not quantify the contribution from Smith Creek. In addition, the channel bend in the southern section of Reach 2 induces flow directed toward the outside (western) edge of the channel and results in higher velocities along that side of the channel. This process typically produces erosion along the outside of the bend and deposition on the inside of the bend. As such, sediment may cross the channel from western to east and contribute to shoal formation in that area.

3.4.3 Southern NE Cape Fear River Shoal

The southern NE Cape Fear River shoal falls between Reach 3 Station 12+00 and Reach 4 Station 5+00. Shoaling occurs along both the east and west sides of the channel in Reach 3 and on the east side of the channel in Reach 4. Figure 3.2 gives shoaling volumes and rates calculated with respect to the design template between 1994 and 2005. The 2005 survey produced the highest shoaling volume above the design depth template (Table 3.7).

Table 3.7 East-Central NE Cape Fear River Shoal Maximum Volumes and Rates

Reach	2003 Shoaling Volume (cy)	Shoaling Rate 2002 to 2003 (cy/yr)
Reach 3	21,291	-709
Reach 3/4 Widener	2,271	-464
Reach 4	1,312	-449

The DMS-Manual indicates *Horizontal Channel Expansion* and *Changes in Channel Alignment - Inside of Channel Bends* most likely contribute to shoaling in this area. In this river section the natural river width increase from south to north as does the width of the maintained channel. An increase in the horizontal channel width slows the flow velocity and induces sediment deposition. As a result, shoaling

occurs near the south end of Reach 3. At the same time, the channel alignment bends towards the northwest. As explained earlier in the previous section, changes in the channel alignment may result in sediment deposition on the inside bend of the channel.

3.4.4 NE Cape Fear River Turning Basin Shoal

Obviously, pre-construction depths within the proposed turning basins are significantly shallower than the design depth (-39 ft-MLLW). The DMS-Manual applies to existing shoals; the proposed turning basins do not technically qualify since they are not actual shoals in existing channels but rather ambient conditions relative to existing flow characteristics. However, an understanding of pre-construction shoaling patterns in these areas can help add to the overall understanding of future impacts once USACE-Wilmington selects and constructs one of the proposed turning basins. As indicated in Table 3.3, a direct survey to survey comparison (2003 to 2005) indicates options *C1* (+2,334 cy/yr) and *C2* (+3,418 cy/yr) tend toward a depositional pattern. Recent results indicate the eastern portion of option *C3* (+72 cy/yr) also tends toward deposition. However, recent surveys indicate the western portion of option *C3* (-253 cy/yr) tends toward an erosional signature. Interestingly, the western portion of option *C3* tends toward deposition when viewed in the longer term (1995 to 2005).

For informational purposes, the DMS-Data Manager calculated the required dredging volume to construct each proposed turning basin relative to the 2005 survey. The required dredging volumes are: approximately 605,808 cy for turning basin Option *C1;* 529,695 cy for turning basin Option *C2,* and 300,696 cy for turning basin Option *C3.*

3.5 DMS-Shoaling Analysis Summary

The preceding analysis identified several shoaling areas within the NE Cape Fear River Navigation Project. The shoaling analysis provided a quantitative characterization of each shoaling area. However, as the majority of the surveys do not completely overlap, the shoaling rates calculated in this study should be applied carefully. A decrease in the shoaling area or shoaling rate in a section of river does not necessarily reflect a true decrease in the shoal area. Such results may indicate a short term change or a problem with data coverage. Longer term monitoring may shed a more definitive light on overall trends.

Of the three turning basin options, Option *C3* appears to require less maintenance dredging. The more recent change rates (2003 to 2005) show erosion (negative change rates) dominates the sediment dynamics of the area particularly on the west side of the channel. However, historical change rates (1995 to 2005) in the same area indicate that deposition has been the dominant process in the past 10 years. As

such, the erosional trend may not persist in the long term. Finally, the Option *C3* channel template requires the least amount of sediment removal during construction.

The DMS-Data Manager and DMS-Manual provided insight into historical shoaling trends and possible processes that contribute to those trends. Accordingly the next step in the DMS methodology, the DMS-Analytical Toolbox, recommends circulation modeling to analyze the complex flow patterns in the project area. Since most of the shoaling categories identified by the DMS-Data Manager and DMS-Manual deal with flow associated with tidal circulation, the DMS-Analytical Toolbox recommends the USACE's RMA2 hydrodynamic model applied in unsteady conditions. Chapters 4 through 5 detail the RMA2 application.

4.0 DMS-ANALYTICAL TOOLOBOX: HYDRODYNAMIC MODEL

A comparison of flow regimes for various dredged turning basin alternatives requires knowledge of specific hydraulic parameters. Determining these parameters requires a detailed hydraulic analysis of the study area. The complexity of flow conditions at the NE Cape Fear turning basin alternatives required the two-dimensional modeling approach described below.

The NE Cape Fear River flow complexity results from two major factors. First, tidal propagation into the study area overwhelms typical daily base flows and results in periodic flow reversals. This is primarily due to the proximity of the site to the multiple inlet system of the lower Cape Fear River basin. Second, the study area's physiography and the proximity of multiple connecting creeks with meandering channels through marsh fringes influences the flow characteristics at the project site. These conditions make a single-reach, steady-state hydraulic analysis inappropriate. Rather, these conditions point toward a time-dependent, two-dimensional hydrodynamic modeling approach for the multi-inlet system to predict the complex nature of daily flows. For this study, the RMA2 software package, supported by the U.S. Army Corps of Engineers and a recommended component of the DMS Analytical Toolbox, provides the two-dimensional modeling component. This chapter outlines the hydrodynamic modeling efforts that determine the daily mean-tide conditions.

4.1 Model Setup

RMA2 is a one- and two-dimensional, transient, depth-averaged, finite-element, hydrodynamic model. The governing equations treat conservation of mass, conservation of momentum in the x- and y-direction, and turbulence closure. Model capabilities include wetting and drying, Coriolis acceleration, wind stress, dynamic friction assignment by depth, Peclet number definition of turbulent exchange coefficients, and two choices for boundary conditions (flow or elevation).

The Coastal and Hydraulics Laboratory (CHL) of the U.S. Army Engineer Research and Development Center in Vicksburg, MS developed an RMA2 model mesh for a previous project in the Cape Fear River basin. USACE-Wilmington provided this model to Taylor Engineering to expedite model development for the current project. Taylor Engineering augmented the model domain and increased mesh resolution in the area of interest. The refined model incorporates the lower Cape Fear River basin with high-resolution elements focused on the NE Cape Fear River turning basins. The sections below discuss the mesh generation process, boundary conditions, a limited calibration, and application of the mean tide event

4.1.1 Model Schematization

The model mesh (Figure 4.1) describes the specific bathymetric characteristics of the lower Cape Fear River basin including three rivers (Cape Fear, NE Cape Fear and Black Rivers) and two inlets (Cape Fear River mouth and Carolina Beach Inlet) as well as the interior interconnected waterways. The mesh boundaries extend 40 miles radially (west to east) from the mouth of the Cape Fear River to approximately the -90 ft contour. The mesh further extends north along the shoreline an additional 20 miles from the mouth. Upriver, the mesh extends 50, 47 and 66 miles up the Cape Fear, Black and NE Cape Fear Rivers. The model terminated at USGS gage locations (Kelly, Currie, and Burgaw) for each of the three rivers. Vertically, the mesh extends below the +1-ft-MLW contour. The solution domain contains 6,462 one-dimensional triangular and quadrilateral elements with nodes at the corners and midpoints of the segments (21,312 nodes). In all, the elements cover 3,600 mi^2. The largest element covers 130 mi^2 and the smallest covers 960 ft^2. This translates to a largest area to smallest area ratio of approximately 3,800,000:1.

The model mesh was constructed from multiple sources. CHL referred to scanned NOAA charts (11537, 11539, and 11541) to specify the bathymetry and topography for the original model. The USACE-Wilmington provided recent bathymetry around the turning basins.

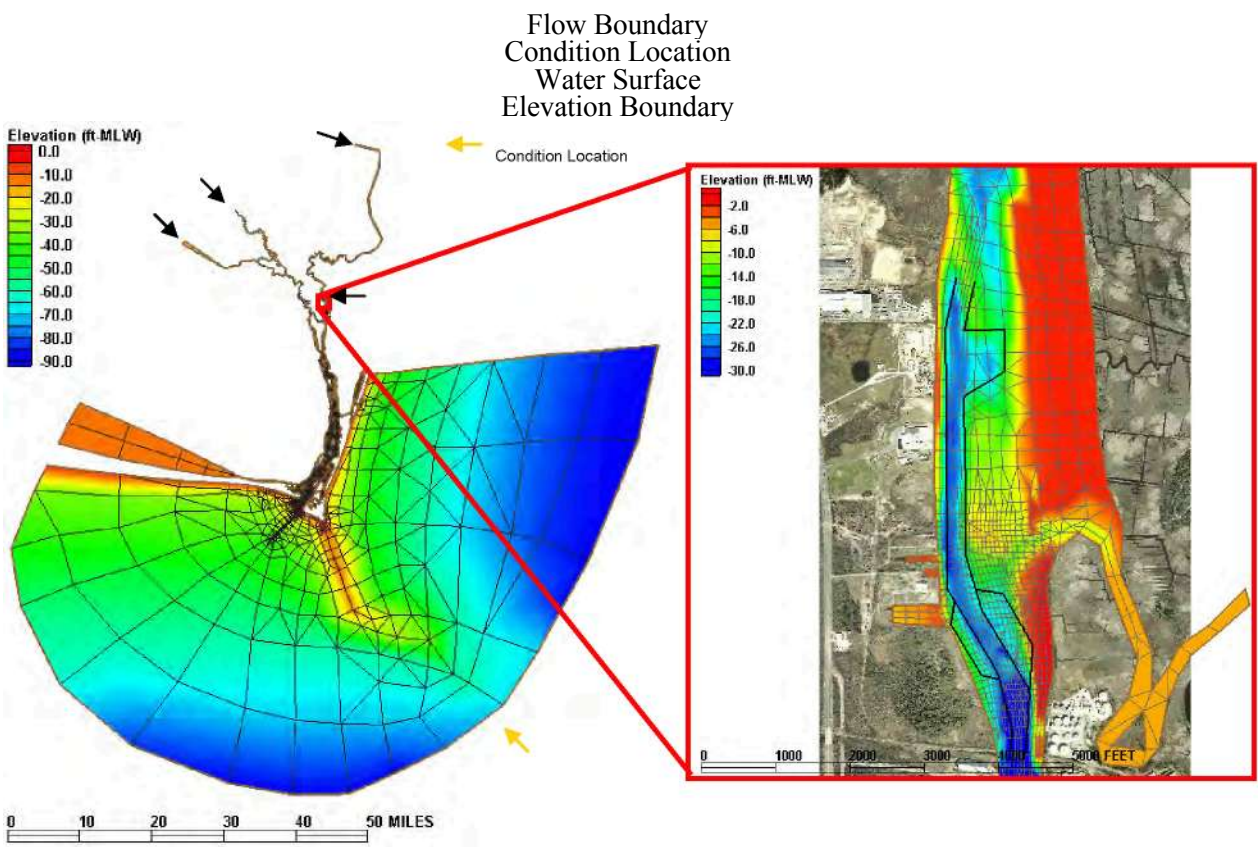


Figure 4.1 Model Mesh

The program SMS (Surface-water Modeling System) served as the user interface for RMA2. SMS provides tools for mesh construction with special emphasis on appropriate resolution in the areas of interest. The program allows the user to input ASCII data files of digitized bathymetry. One of several interpolation schemes — in this case, a linear interpolation method — then interpolates this bathymetry onto the mesh. RMA2 allows the specification of friction and turbulent exchange coefficients on an element-by-element basis. For this model, friction (represented by Manning's *n)* and turbulent exchange coefficient were set according to water depth. Assignment of Manning's *n* ranged from 0.02 for deep areas to 0.1 for the marshy areas of the lower Cape Fear River basin. A Peclet number specified at a value of 20 controls the automatic assignment of the turbulent exchange coefficients throughout the model. Both values fall well within the range the developers recommend for flow in tidal estuaries.

4.1.2 Model Boundary Conditions

The final step required in the model setup involved specification of known or assumed boundary conditions at the external boundaries to the model mesh. For these, RMA2 provides several options. For an unspecified mesh boundary, the program automatically assumes an external barrier with a "slip" boundary condition. In short, the flow at nodes on a slip boundary does not have velocity components perpendicular to the boundary. Specified boundary conditions may include free surface elevations (constant or time-varying) and/or flow (constant or time-varying). The model contains four constant flow boundary conditions and one time-varying elevation boundary condition (Figure 4.1). The flow boundary conditions were set at the base (average) flow rates for the Cape Fear, Black, NE Cape Fear Rivers, and Smith Creek.

In order to determine average flow conditions, Taylor Engineering analyzed U.S. Geological Survey (USGS) gage records on the Cape Fear River, the Black River, and the NE Cape Fear River. The records indicate August 2004 represents flows that best approximate a mean condition. The flow boundary condition for the Cape Fear River at Kelly reflects 35 years of flow record from a USGS gage. The Cape Fear River at Kelly has an average flow of 5,627 ft^3/s. For August 2004, the USGS gage measured an average monthly flowrate of 5,029 ft^3/s. For the Black River, the USGS gage at Currie recorded an average flowrate of 1,456 ft^3/s over seven months from March through September 2004. The Currie gage recorded 1,718 ft^3/s for August 2004. The USGS gage for the NE Cape Fear River at Burgaw recorded an average of 1,331 ft^3/s over an eight month period, with an August 2004 flowrate of 1,941 ft^3/s. The assumption that August 2004 best represents mean conditions is based upon the following observations:

1) The combined August 2004 flowrate approximated the average flowrate (within 3%) for all three river flows combined.

2) The August 2004 flowrate for the longest gage in service (USGS gage at Kelly) approximated (within 11%) the average flowrate over the 35 year record.

With no gage record at Smith Creek, Taylor Engineering calculated the flow boundary condition from an empirical equation developed from a comparison of USGS daily streamflow recordings and drainage areas. Figure 4.2 presents a plot of the drainage area versus average daily flows over the coastal plain in New Hanover County. Taylor Engineering delineated the Smith Creek basin and calculated a drainage basin area of 36.7 mi^2. The corresponding average daily flowrate, calculated from the regression equation shown in Figure 4.2, is 8.7 ft^3/s.

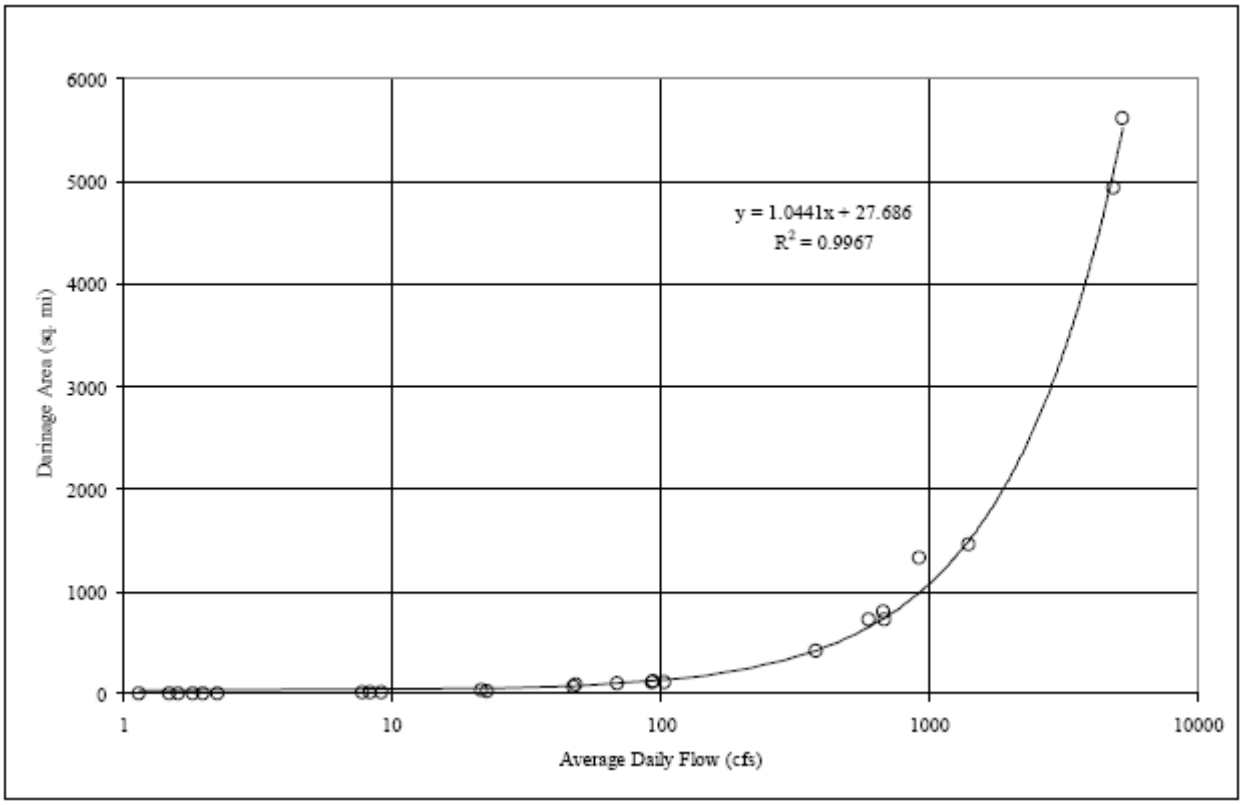


Figure 4.2 Drainage Area vs Average Daily Flows, Coastal Plain, New Hanover County, NC

The offshore boundary condition simulated a time-varying free surface elevation condition. The Tides & Currents software package provided water surface elevations for the most offshore station - Cape Fear (33° 51'N, 77° 58'W). Taylor Engineering adjusted the tide signal's phase and amplitude to convert the near shore tide to the offshore boundary. This method required several iterations of the offshore signal to determine the best fit for tidal signal at the gage location. Thus, the Cape Fear gage station effectively served as an additional calibration gage (Section 4.1.3). Figure 4.3 (Cape Fear station) shows the offshore signal applied to the model.

4.1.3 Limited Model Calibration

Calibration demonstrates a model's ability to reproduce observed hydrodynamic conditions in the study area. Calibration was accomplished through iterative adjustments to model parameters until observed data and model results agreed. For this effort, calibration consisted of a comparison of model results with time series of NOAA tide predictions obtained from the software program Tides & Currents. Because synoptic measurements were only available for the Cape Fear River vicinity, only a limited calibration of the overall model was possible.

As discussed above, the calibration run applied several boundary conditions to the model. Base flow conditions represent the four river and stream boundaries. Calibration was accomplished through iterative adjustments of friction (Manning's *n)* and turbulent exchange coefficients until flow, elevation, and velocity measurements matched measured values. Figure 4.3 shows a comparison of the measured and simulated flow through the lower Cape Fear basin at locations identified in Figure 2.2. Each time series references local mean low water; the dashed lines indicate MHW and MLW levels. Table 4.1 summarizes the calibration results. Model calibration results agreed well with the NOAA-predicted data taken at various locations in the model domain.

Table 4.1 Calibration Summary

Station Name	**Cape Fear**	**Bald Head Island**	**Southport**	**Reaves Point**	**Orton Point**	**Wilmington**
NOAA Station ID	n/a	8658901	8659084	8658622	8658501	8658120
Location	33° 51.0" N 77° 58.0" W	33° 58.2" N 78° 00.1" W	33° 59.4" N 78° 01.1" W	34° 00.2" N 77° 57.3" W	34° 03.4" N 77° 56.4" W	34° 13.6" N 77° 57.2" W
NOAA Predicted Range	n/a	4.49	4.24	4.09	4.17	
Tides & Currents Range around 95 hrs	4.70	4.50	4.40	4.00	4.10	4.20
Simulation Range around 95 hrs	4.79	4.57	4.51	4.44	4.36	4.19
Percent Error	1.9%	1.5%	2.4%	10.9%	6.4%	-0.4%

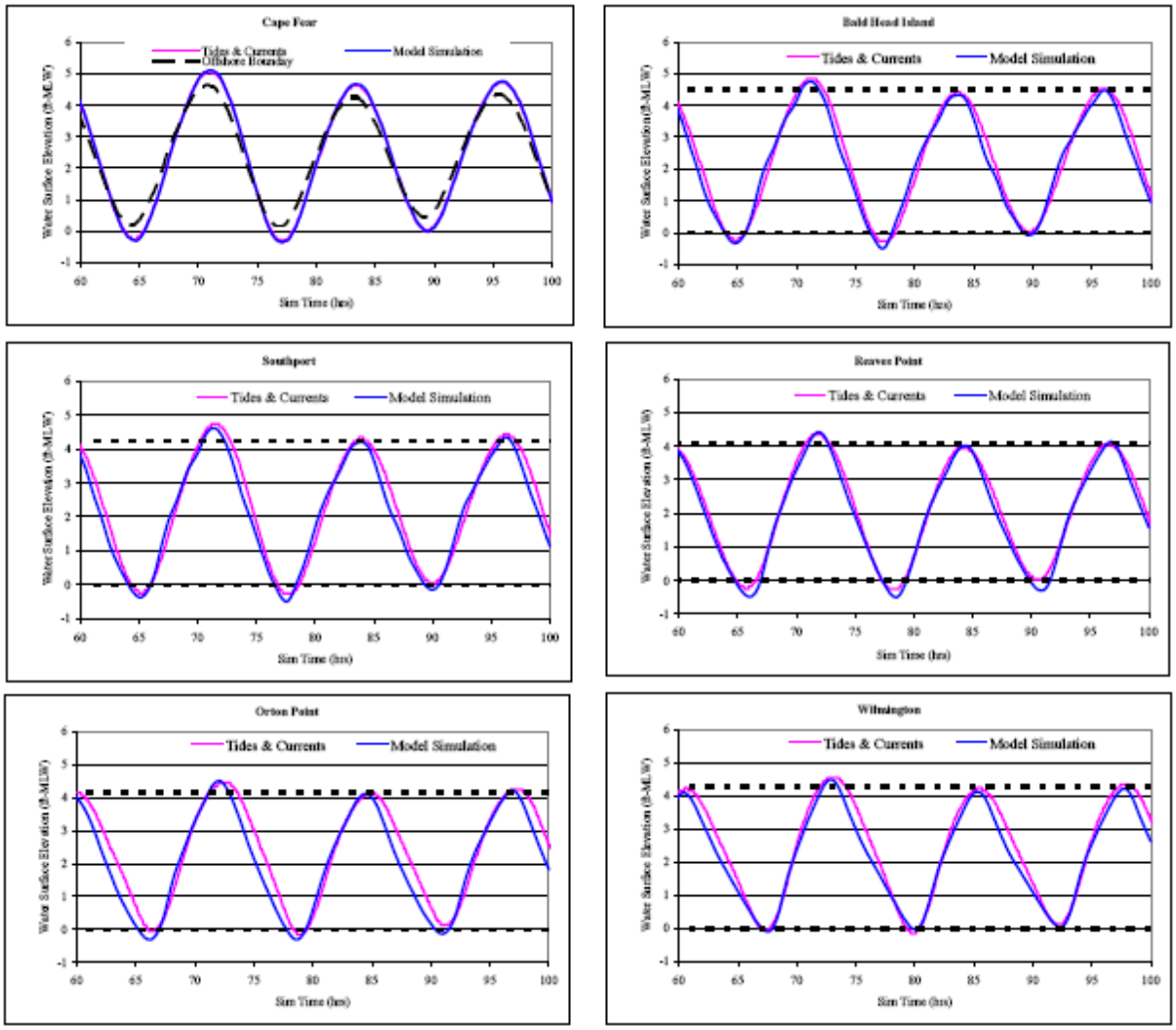


Figure 4.3 Comparison of NOAA-predicted and Simulated Time Series

4.2 Model Simulations

This study seeks to determine the preferred location for the NE Cape Fear River turning basin based on the least shoaling potential. With four alternative locations proposed, Taylor Engineering performed several hydrodynamic simulations. The simulations provide a basis to assess possible causes of shoaling and the effects of depth on flow and sedimentation. The first simulation, an existing conditions run, provided a means of determining calibration parameters for the model (Section 4.1.2). The second simulation, an authorized depth simulation, provided a prediction of the tidal circulation that would exist for the turning basin dredged to the presently authorized depth. This simulation provided a baseline from which to compare simulations containing proposed or planned modifications. Four additional simulations provided the means to examine the effects of the alternative turning basin locations on flows within the

Cape Fear River basin. In total, Taylor Engineering performed six simulations that varied only in bathymetry near the proposed turning basins:

1) *Existing.* The *Existing* simulation helped determine calibration parameters for the model. Taylor Engineering applied the most recent bathymetric data (described in Section 4.1.1) to the model mesh.

2) *Authorized.* The *Authorized* simulation served as a baseline for comparison of proposed turning basin alternatives. For the Authorized simulation, Taylor Engineering set elevations in the lower reach channel at -25 ft-MLLW and in the existing turning basin at -32 ft-MLLW.

3) *B1.* Alternative *B1* applied -39 ft-MLLW elevations to the lower reach and existing turning basin. Additionally, the existing turning basin's area increased as the basin's southern boundary extends approximately 200 ft south of its present location.

4) *C1.* Alternative *C1* includes -39 ft-MLLW elevations in the lower reach and -35 ft-MLLW in the upper reach and existing turning basin. An additional proposed turning basin, located on the east side of the federal channel, extends approximately 3,000 ft south of the existing basin.

5) *C2.* Alternative *C2* includes -39 ft-MLLW elevations in the lower reach and -35 ft-MLLW in the upper reach and existing turning basin. An additional proposed turning basin, located on the east side of the federal channel, extends approximately 3,300 ft south of the existing basin.

6) *C3.* Alternative *C3* includes -39 ft-MLLW elevations in the lower reach and -35 ft-MLLW in the upper reach and existing turning basin. An additional proposed turning basin, located on both the east and west sides of the federal channel, extends approximately 3,800 ft south of the existing basin.

Each of the above model runs simulated the same time frame and boundary conditions as the *Existing* conditions simulation. Given the detailed resolution of the mesh and time constraints, simulations only ran for 100 hours. A period from August 1-5, 2004 encompassed a mean tide condition and provided the tidal boundary conditions for the simulations. As mentioned, the average monthly flows served as interior boundary conditions for river boundaries. The model generated water surface elevation and velocity values at each node of the mesh.

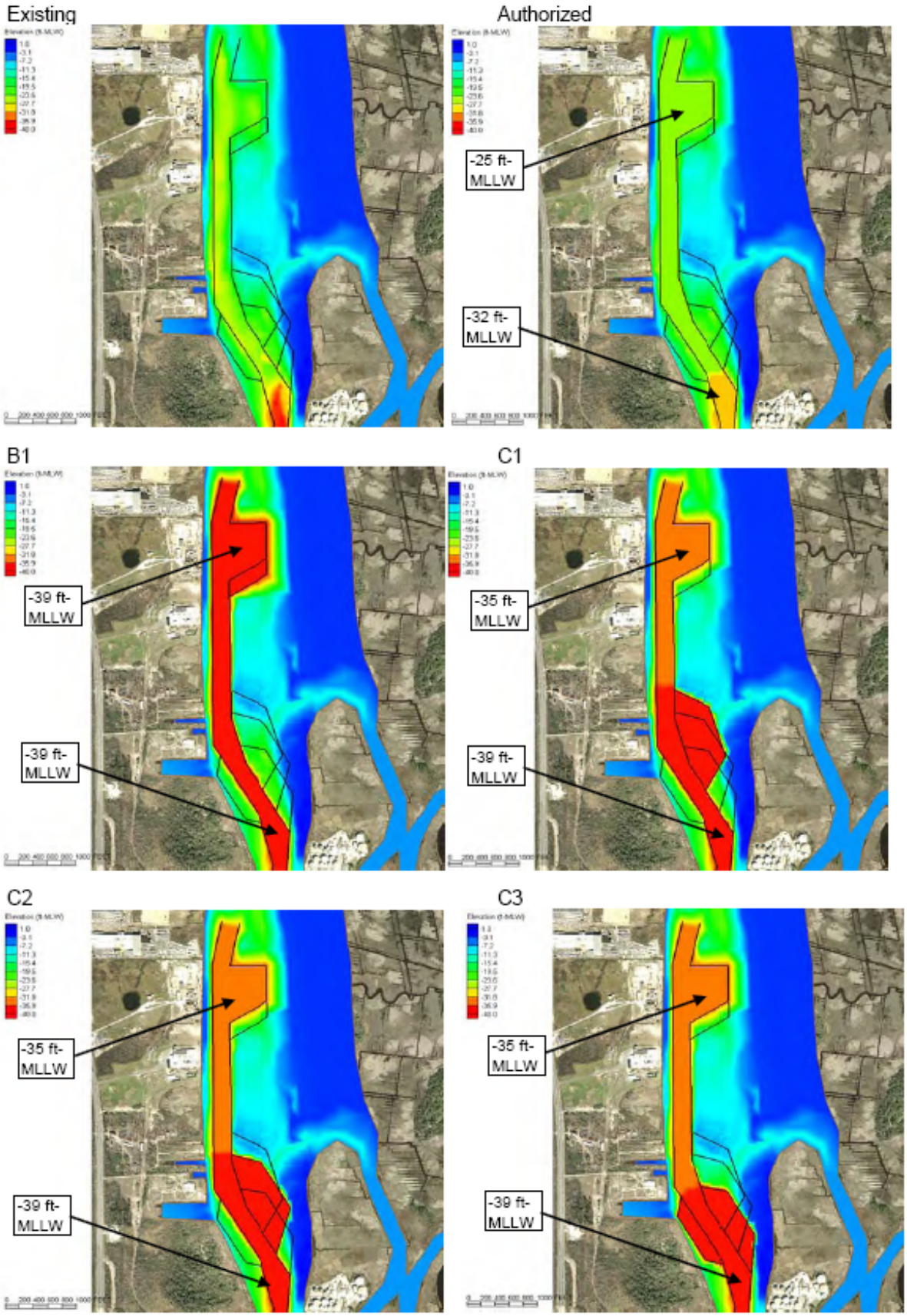


Figure 4.4 Model Simulations

4.3 Sediment Transport Methodology

The following paragraphs introduce the sediment transport methodology that generated the results presented in the next chapter. Chapter 5 looks at sediment transport rates as they pertain to shoaling mechanisms and the effects of the proposed turning basin alternatives on shoaling patterns. Notably, the sediment transport methodology incorporates the results from the tidal circulation modeling discussed in the previous sections.

The sediment transport analysis examined sediment transport at time steps corresponding to maximum flow conditions at the area of interest. The simulation results (dynamic water depth and flow velocity) during each examined time step provided input for a sediment transport equation and yielded an estimate of the sediment transport rate at each node within the domain. Bagnold's energy-based total sediment transport model (Bagnold, 1963 and 1966) adapted by Bailard (1981 and 1982) for time varying sediment transport, provided the sediment transport formulae applied in this study.

Bagnold based his sediment transport model on energy balance concepts. He stated that the available flow power supplies the energy for sediment transport. The model assumes two distinct modes of sediment transport: bedload transport where grain to grain interactions support the sediment and suspended load where the fluid supports the sediment through turbulent diffusion. Bagnold defined the sediment transport efficiency as the rate energy is expended transporting either the bedload or suspended load divided by the total rate of energy production in the fluid due to the flow field.

Bagnold's equation reduces to

$$q_{total} = \frac{\tau_0 V}{\gamma \Delta}\left[\frac{e_b}{\tan\phi - \tan\beta} + \frac{e_s}{\left|w_s/V - \tan\beta\right|}\right] \qquad (1)$$

where

qtotal = volume rate of sediment transport per unit width
yA = immersed weight of the sediment
y = fluid specific gravity ($\gamma = \rho g$)
γ_s = sediment specific gravity ($\gamma_s = \rho_s g$)
A = (γ_s/ρ-1)
$\tau 0$ = bed shear stress
V = mean stream flow velocity
$\tau_0 V$ = stream power (the total rate of energy production in the fluid)
e_b, e_s = bedload and suspended load efficiencies (empirically, 0.13 and 0.01)
$\tan\beta$ = bed slope
$\tan\varphi$ = internal angle of friction of the sediment
w_s = sediment fall velocity.

Classical hydraulics defines the bed shear stress in terms of the mean velocity, the hydraulic radius (R) and the energy grade line (S), equation (2),

$$\tau_0 = \gamma R S. \tag{2}$$

Equating the hydraulic radius to the stream depth (h) in open channel flow gives.

$$\tau_0 = \gamma h S. \tag{3}$$

Chezy's equation relates the energy grade line to the mean stream velocity by

$$V = C\sqrt{hS}, \tag{4}$$

where C = the grain Chezy coefficient. Julien (1998) gives the following equation for the grain Chezy coefficient

$$C = 5.75\sqrt{g}\log\left(\frac{4h}{d_{90}}\right), \tag{5}$$

where d_{90} is the 90% finer grain size.

Combining equations (3) and (4) yields

$$\tau_0 = \gamma\left(\frac{V}{C}\right)^2. \tag{6}$$

Combining equations (1) and (6) gives

$$q_{total} = \frac{V^3}{C^2\Delta}\left[\frac{e_b}{\tan\phi - \tan\beta} + \frac{e_s}{\left|w_s/V - \tan\beta\right|}\right]. \tag{7}$$

Julien (1998) gives a formula for the sediment fall velocity (w_s) in terms of the dimensionless particle size (d*). Equation (8) defines the dimensionless particle size:

$$d_* = d_s\left(\frac{\Delta g}{\nu^2}\right)^{1/3} \tag{8}$$

where d_s = the representative diameter of suspended sediment particles and ν = the kinematic fluid viscosity. Equation (9) gives the sediment fall velocity:

$$\omega = 10\frac{\nu}{d_s}\left(\sqrt{1+0.01d_*^3}-1\right). \tag{9}$$

Most of the values for the variables presented above came from either the numerical simulations (depth, velocity) or from standard or published values (fluid density, sediment density, gravity, etc.). The only additional information required involves a description of the sediment found within the channel. USACE-Wilmington (2005) provided sediment gradation curves at several locations within the existing turning basin. For simplicity, the calculations assume a single distribution applicable over the entire project length. The single distribution reflects an average of 12 surface samples within the turning basin presented in the reference. Table 4.2 presents several of the average sediment properties input into the transport rate equations.

Table 4.2 Average Sediment Properties

Sediment Property	Sediment Size (mm)
D_{90}	1.200
D_{60}	0.279
D_{50}	0.214
D_{30}	0.206
D10	0.084

These equations provided the estimates of sediment transport rate for given flow conditions within the turning basin. Sediment transport rates were calculated for all nodes within the solution mesh at specific time steps from the simulation runs. These rates, presented in the form of contour plots, provide a picture of the theoretical sediment transport at given instances of time. In Chapter 5, these plots present insight into the mechanisms contributing to shoaling and provide a means to evaluate how the channel-deepening project changes the sediment transport.

5.0 DMS-ANALYTICAL TOOLBOX: HYDRODYNAMIC MODEL RESULTS

Chapter 3 of this study presented an examination of shoaling in the NE Cape Fear River turning basin vicinity through an application of DMS concepts. Chapter 3 also assigned categories to each existing shoal based on its location and corresponding characteristics. Toward this end, Chapter 4 detailed the two-dimensional hydrodynamic model setup, calibration, and simulations conducted to develop a better understanding of the shoaling mechanisms at work. With Chapters 3 and 4 as a backdrop, this chapter revisits the shoaling areas to identify, in light of the model results, the mechanisms contributing to shoaling in the existing and proposed turning basins.

The *Existing* simulation provides a means to understand the present NE Cape Fear River shoaling processes in the turning basin's vicinity. The shoaling trends in the *Existing* simulation are qualitatively calibrated by the DMS analysis presented in Chapter 3. The *Authorized* simulation then provides a baseline for comparison of proposed turning basin alternatives. Finally, the proposed alternative locations for the turning basins are presented.

The proposed turning basin alternatives evaluation began with an examination of the results from the tidal circulation simulation. Velocity and sediment transport plots describe the *Existing* and *Authorized* simulations. The evaluation continued with a comparison of the proposed simulations to the *Authorized* simulation. Subtracting the *Authorized* solution from the proposed solution at every node for every time step indicated flow velocity response to deepening. The final step in the evaluation involved examining the sediment transport associated with the channel deepening. In addition to the velocity and sediment transport plots, Taylor Engineering also presented velocity difference plots of velocity for the proposed simulations *(B1, C1, C2,* and *C3).*

5.1 Existing Simulation

As presented in Chapter 3, three distinct shoals form in the existing channel and turning basin vicinity.

1) North Shoal - immediately north of the turning basin in the main channel (Reach 1) and in the existing turning basin,

2) Central East Shoal - on the east side of the main channel (Reach 2)

3) South Shoal - at the southern end of the project area within the main channel where the channel takes a NW-SE alignment (Reach 3).

The following discussion of existing shoal formation processes presents concepts important to understanding the discussion regarding proposed turning basin shoaling trends. While potential shoaling mechanisms proposed by the DMS-Manual are necessary for a general understanding of project conditions, the underlying process by which shoaling occurs, identified in part by hydrodynamic modeling, remains the primary objective of the analysis.

The North Shoal area logically divides into two shoals: 1) within the existing turning basin and 2) north of the existing turning basin. The DMS-Manual category for the existing turning basin shoal corresponds to both *Vertical Channel Expansion - In-Line Channel Flow* and *Horizontal Channel Expansion - In-Line Channel Flow* categories. The tidal circulation modeling results can help confirm this classification. Figures 5.1 and 5.2 display the circulation modeling results. The figures indicate both direction and magnitude of flow at the time of maximum ebb and flood velocity. From Figure 5.1, velocities decrease approximately 50% during ebb flow as the flow propagates over the existing turning basin. This area corresponds exactly to the area of shoaling noted in Chapter 3. The negative velocity gradient reduces the sediment transport potential and thus leads to shoaling. The cross-sectional flow area increases as depths increase from a dredged depth of 15 ft to 22 ft upon entering the turning basin. The river width also increases from approximately 700 ft to 1250 ft. As the cross-sectional area increases, the associated decrease in velocity satisfies the continuity equation. Figure 5.2, the time of peak flood, indicates a decrease in velocity over the area of shoaling and then an increase in velocity as the flow moves to the shallower maintained depths.

Figures 5.3 and 5.4 represent sediment transport rates during peak ebb and flood flow. Importantly, any attempt to interpret these figures in terms of sediment deposition or erosion requires caution. The critical parameters are the flow direction and the magnitude of the sediment transport contours. Increasing sediment transport magnitude along a particular flowpath indicates more sediment leaves than enters a specific area. This sediment imbalance results in bed erosion. Conversely, decreases in sediment transport magnitude along a particular flowpath indicate more sediment enters than leaves a specified area — hence, sediment deposition. Figure 5.3 indicates that relatively high sediment transport occurs north of the turning basin and decreases rapidly in the turning basin as flow exits during ebb. Figure 5.4 shows the same trends on flood. Therefore, the results confirm that the DMS-Manual shoaling classifications are indeed *Vertical Channel Expansion - In-Line Channel Flow* and *Horizontal Channel Expansion - In-Line Channel Flow.*

The DMS-Manual category for the shoal to the northwest of the existing turning basin also corresponds to *Vertical Channel Expansion - In-Line Channel Flow* and *Horizontal Channel Expansion - In-Line Channel Flow* categories. From Figure 5.1, flow across the existing turning basin decreases in

velocity during ebb flow. Again, this area corresponds exactly to the area of shoaling noted in Chapter 3. The negative velocity gradient leads to reduced sediment transport potential and shoaling. The cross-sectional flow area increases with depths (from 15 ft to 22 ft) and river width (700 to 1,200 ft) upon entering the main channel. The increase in cross-sectional area results in an associated decrease in velocity. Figure 5.2, the time of peak flood, however, does not indicate a substantial velocity gradient. This leads to the hypothesis than ebb flows predominately contribute to the shoal development. This hypothesis is verified in the sediment transport figures in Figures 5.3 and 5.4. Figure 5.3 indicates that relatively high sediment transport occurs north of the channel and decreases rapidly in the channel during ebb, whereas Figure 5.4 shows little variation in sediment transport during flood. Nonetheless, the results confirm that the DMS shoaling classification are indeed *Vertical Channel Expansion - In-Line Channel Flow* and *Horizontal Channel Expansion - In-Line Channel Flow.*

The east-central shoal develops along the eastern edge of the main federal channel. As the shoal pattern is both uniform and consistent along the entire Reach, channel bank instability and collapse may be the prime factor in shoal development. The hydrodynamic model simulations cannot address this gravity-based process as it is not directly attributed to tidal flows. Periodic storm runoff from Smith Creek may serve as another possible cause. Smith Creek's deposition fan lies immediately opposite the federal channel. Importantly, the Smith Creek shoal lies approximately 10 ft shallower than the surrounding elevations to the north and south and could serve as a sediment source during periods of high flow from the Creek. As Taylor Engineering assumed mean daily flows for all flow boundary conditions input into the hydrodynamic model, specific determination of the contribution of storm-induced shoaling adjacent to Smith Creek is inappropriate. Finally, another minor contribution to the shoal is the deposition of sediment along the inner bend of the main river. Both ebb and flood sediment transport plots display a negative gradient in sediment transport over this shoal area. This indicates deposition of material in the shoal. From these results, the likely shoaling classifications are *Riverine Sediment Sources* and *Channel Alignment - Inside of Channel Bends.*

The third shoal, located at the southern end of the NW-SE directed portion of the federal channel, is a product of two opposing hydrodynamic interactions. The existing bathymetry in this area deepens from approximately 25 ft to 39 ft from north to south. On ebb, this depth increase would tend to decrease flow velocities. However, over the same distance, the river width decreases from 1,100 ft to 580 ft. On ebb, the narrower river width would tend to increase flow velocities. On flood, the opposite effects are expected - a shallower channel would increase flow velocities whereas a wider river width would decrease flow velocities. The velocity plots in Figure 5.1 and 5.2 indicate that the narrower areas of the river show higher average velocities. Thus the defining controlling mechanism for the flow regime is the channel width.

On ebb, the velocity vectors do not necessarily follow the channel thalweg; instead the main conveyance focuses to the west of the channel centerline. This flow regime - flow focused on the inside of a river bend - is opposite to the typical flow regime for a river bend alignment. Typical flow patterns would show flows concentrated on the outside of a bend. The relatively higher flow on the outside of a river bend would result in erosion on the outside and accretion on the inside of the bend. However, at this location accretion occurs on both the inside and outside of the river bend.

If one follows the velocity vectors along the channel centerline, the velocity magnitude decreases (indicated by a color variation from red to green). Although not as pronounced, Figure 5.2 shows a similar velocity decrease on flood. The sediment transport plots in Figures 5.3 and 5.4 show that sediment transport decreases in the channel on both ebb and flood in the same location as the historical shoal vicinity; this confirms that the model predicts shoaling. Notably, sediment transport does not vary in magnitude to the west of the channel - hence shoaling is not expected. Therefore, the DMS shoaling classification is combined *Horizontal Channel Expansion - In-Line Channel Flow with Channel Alignment - Inside of Channel Bends.*

A comparison between Figures 5.1 and 5.2 show that flood velocities are generally smaller than ebb throughout all reaches. This is due primarily to river flow (directed downstream from North to South) reinforcing the ebb tidal flow. Additionally, water surface elevations at time of peak ebb flow are slightly lower than at peak flood flow. With higher water surface elevations at peak flood, the cross-sectional area of the river increases both vertically and horizontally (as the banks of the channel become inundated). Thus for equal flow rates and no river discharge, river velocities are smaller on flood than ebb. Section 5.7.2 details the maximum flow rates experienced through the turning basins and adjacent channels for ebb and flood tides.

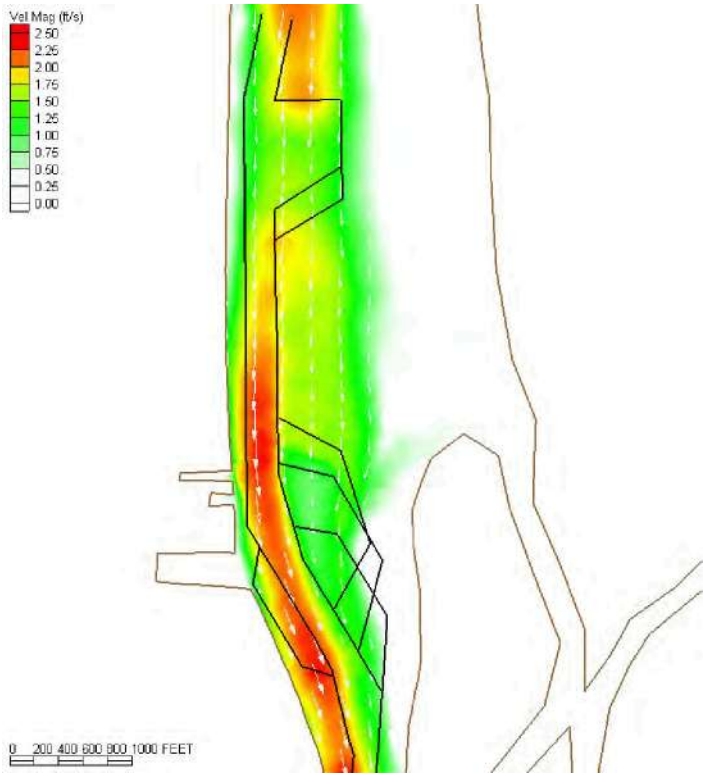


Figure 5.1 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *Existing* Simulation

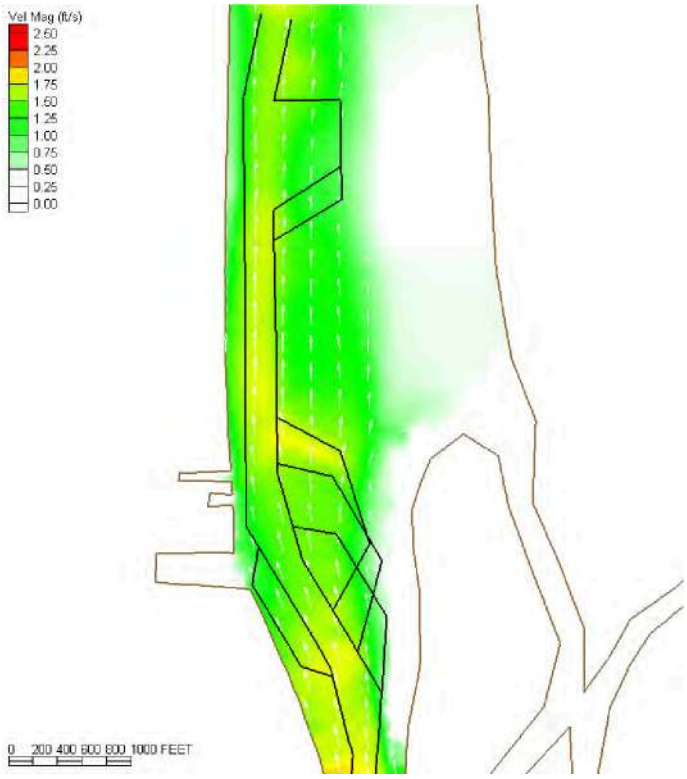


Figure 5.2 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *Existing* Simulation

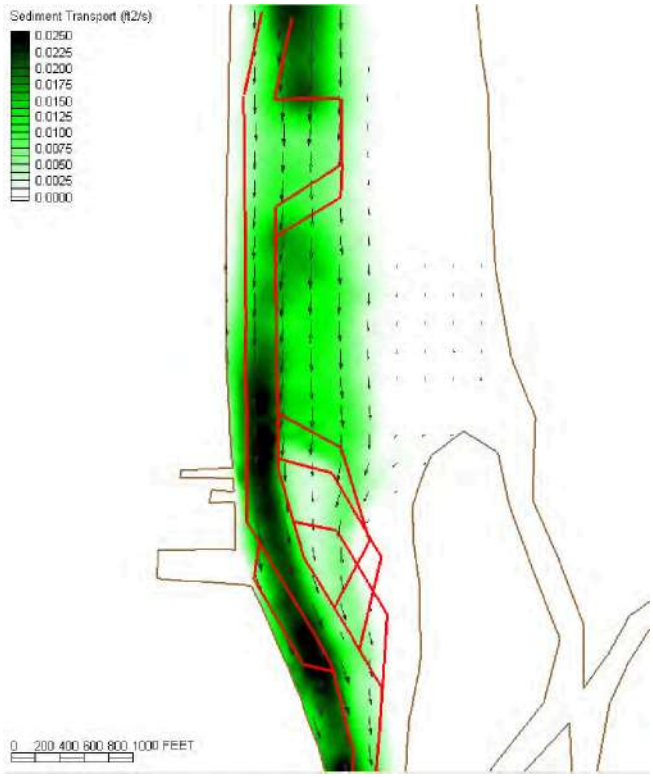


Figure 5.3 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *Existing* Simulation

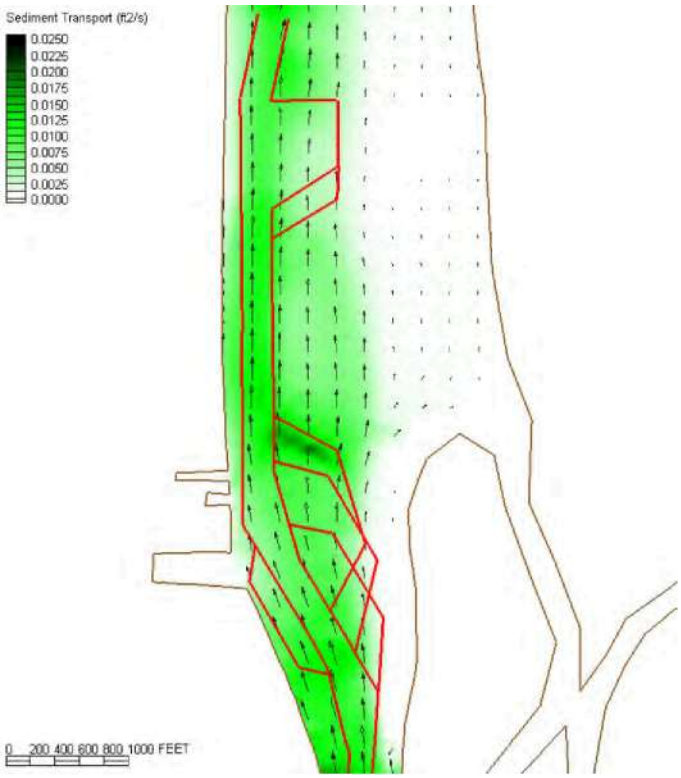


Figure 5.4 Sediment transport contours (ft /s/ft) in the vicinity of the turning basin during peak flood flow for the *Existing* Simulation

5.2 Authorized Simulation

For the *Authorized* simulation, Taylor Engineering applied the authorized dredged depths within the federal channel and turning basin. The *Authorized* basin simulation represents a channel and basin configuration immediately after maintenance dredging. The proposed turning basin alternatives similarly describe bathymetries immediately after dredging. Thus, the *Authorized* simulation served as a baseline for comparison alternatives. For the *Authorized* simulation, Taylor Engineering set elevations in the lower reach channel at -25 ft-MLLW and in the existing turning basin at -32 ft-MLLW (Figure 3.1).

As existing depths in the channel and turning basin approximate the authorized depths, shoaling trends during the *Authorized* simulation do not vary substantially from those seen in the *Existing* simulation. The three shoaling locations described in Section 5.1 apply to the *Authorized* simulation - the north shoal, the central east shoal, and the south shoal.

Figures 5.5 and 5.6 displays the flow direction and magnitude at the time of maximum ebb and flood velocity. Figures 5.7 and 5.8 represent sediment transport rates during peak ebb and flood flow. These plots provide a baseline comparison for the proposed alternative turning basin locations.

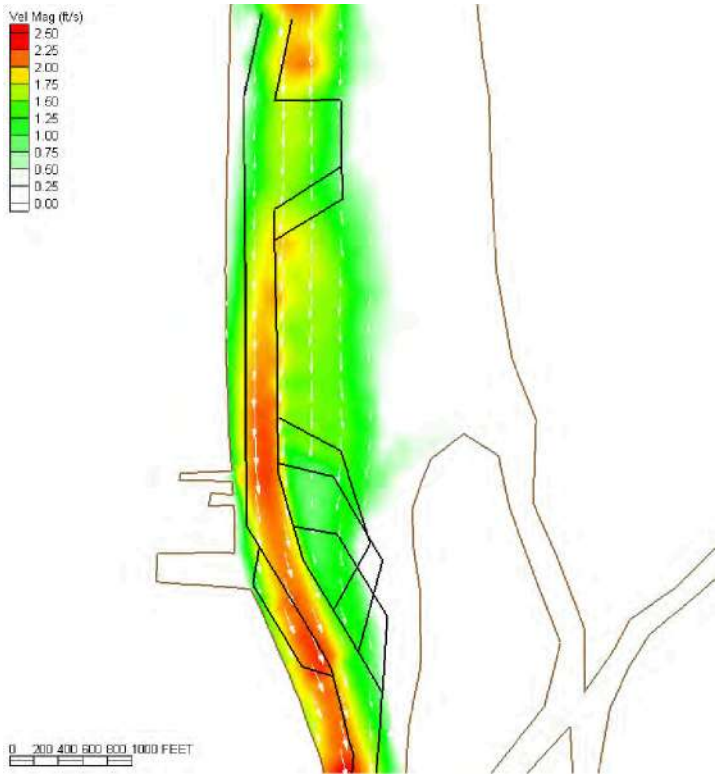


Figure 5.5 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *Authorized* Simulation

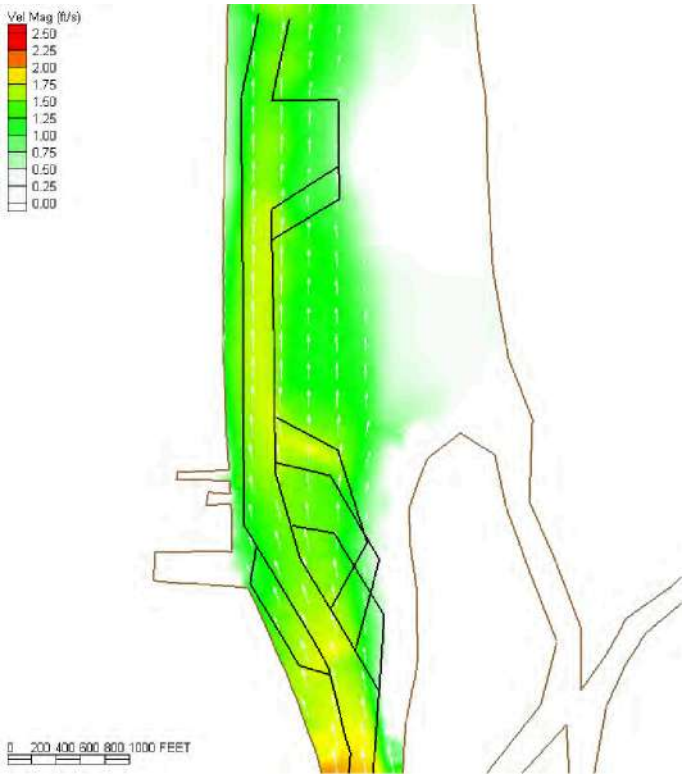


Figure 5.6 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *Authorized* Simulation

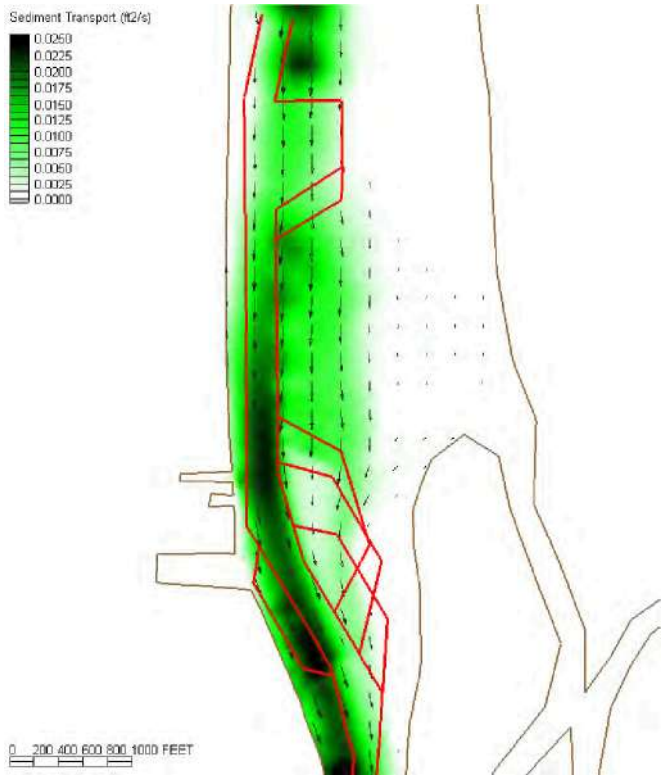


Figure 5.7 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *Authorized* Simulation

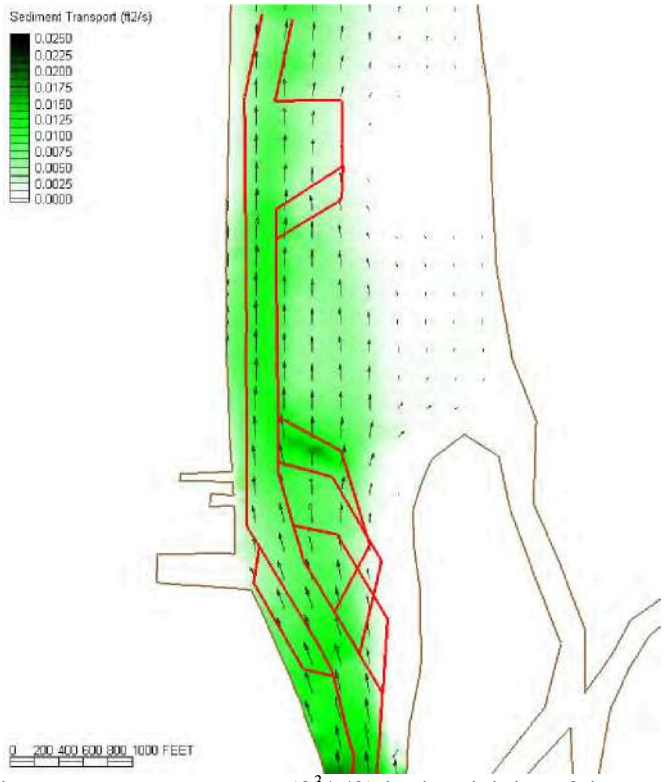


Figure 5.8 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *Authorized* Simulation

5.3 B1 Simulation

The *B1* alternative included -39 ft-MLLW elevations in the lower reach and exiting turning basin. Additionally, the 200 ft southern extension of the existing turning basin's south boundary effectively increased the basin's area.

Figure 5.9 and 5.10 show contoured velocity magnitudes at time of maximum ebb and flood flows. On ebb, velocities generally follow the channel centerline. Flood velocities are smaller than ebb throughout the reaches, with slightly larger velocities near the constricted sections in the southern reach. Figures 5.11 and 5.12 show velocity difference plots between the *B1* and *Authorized* simulations. Ebb flows produce velocity decreases up to 1 ft/s within the proposed turning basin and localized velocity increases north of the basin. Flood flows produce similar velocity decreases and small velocity increases east of the basin.

Figures 5.13 and 5.14 show the sediment transport plots for peak ebb and flood. As stated earlier, sediment transport is a complex function of both local velocity and depth. As such, discerning how the flow field modifications affect sediment transport is not as simple as examining the regions of velocity increase or decrease. However, when examining the results from the sediment transport calculations, the controlling concept remains that shear stress (i.e., sediment transport) is a much stronger function of velocity than depth. Sediment transport rate calculations followed the procedure outlined in Chapter 4. Importantly, discerning shoaling activity from examination of sediment transport rates requires caution. The gradients of sediment transport rather than the rate magnitudes contain the important information. *Increases in sediment transport rate in the direction of flow indicate regions of erosion, while decreases indicate regions of deposition.*

In the vicinity of the proposed turning basin, sediment transport rates decrease substantially from upstream of the basin to inside the basin during ebb (Figure 5.13). This indicates a likelihood of shoaling. On flood, no significant changes are apparent.

Figure 5.9 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *B1* Simulation

Figure 5.10 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *B1* Simulation

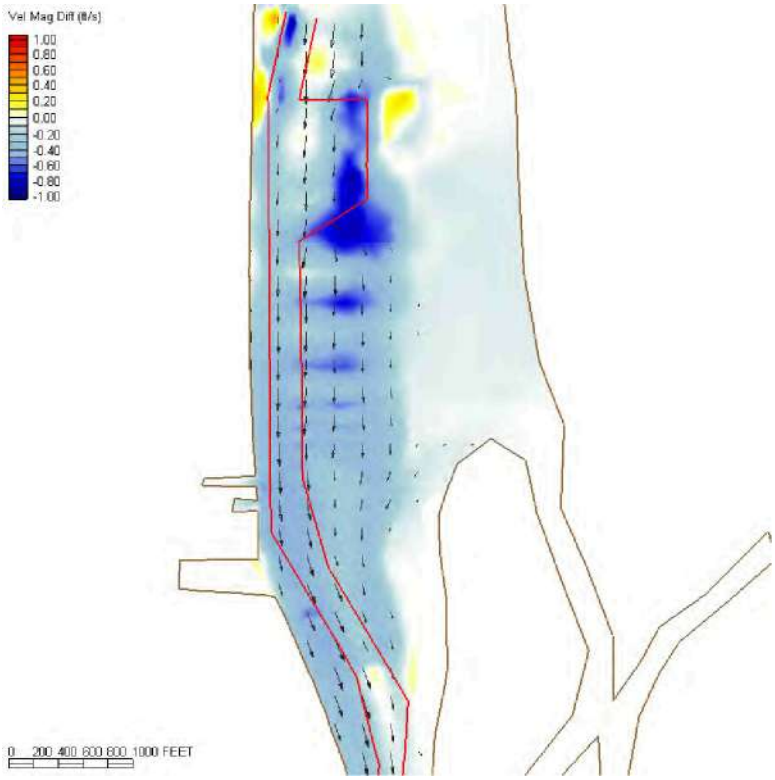


Figure 5.11 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *B1* Simulation

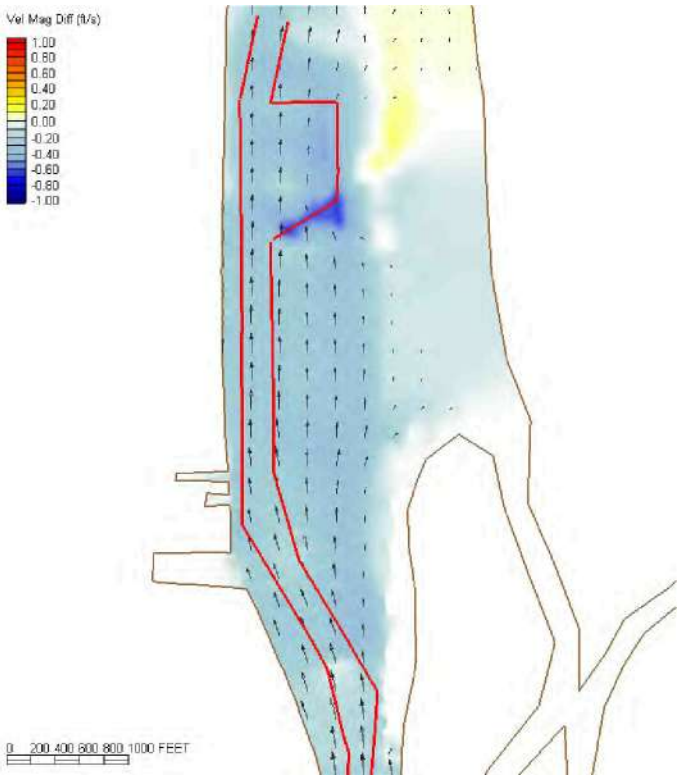


Figure 5.12 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak flood flow for *B1* Simulation

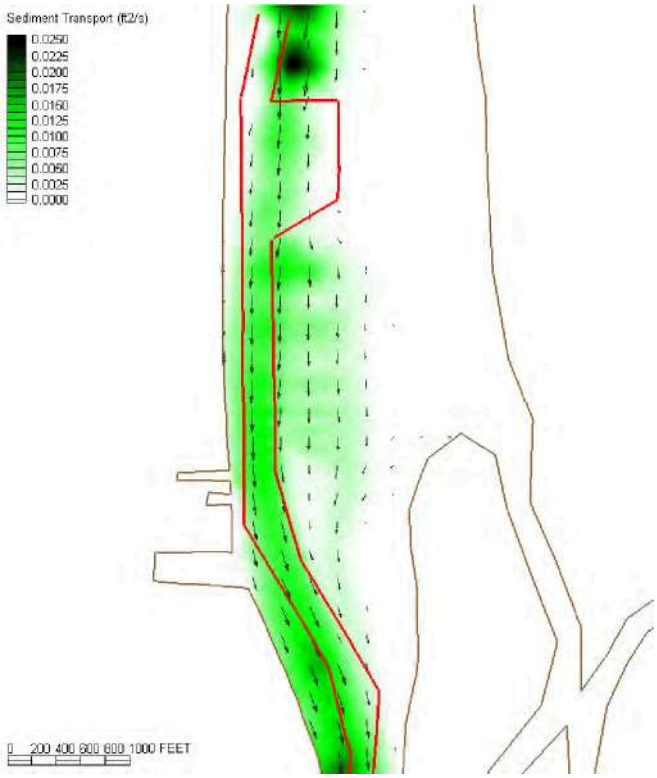


Figure 5.13 Sediment transport contours (ft /s/ft) in the vicinity of the turning basin during peak ebb flow for the *B1* Simulation

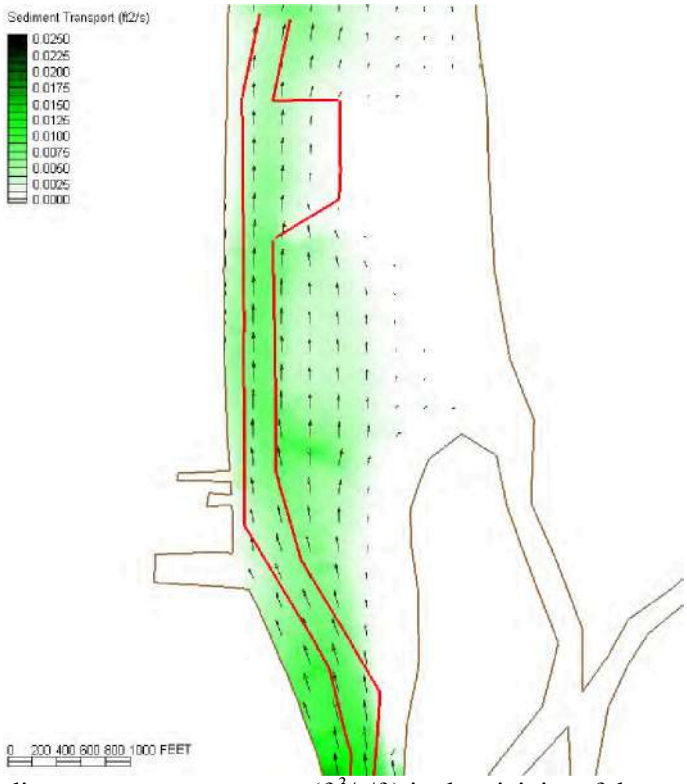


Figure 5.14 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *B1* Simulation

5.4 C1 Simulation

The C1 alternative included construction of a new turning basin with -39 ft-MLLW elevations in the lower reach and -35 ft-MLLW elevations in the exiting turning basin.

Figure 5.15 and 5.16 show contoured velocity magnitudes at time of maximum ebb and flood flows. On ebb, flows north of the existing turning basin experience high velocities greater than 2 ft/s. Velocities then slow considerably upon entering the existing turning basin. South of the existing turning basin, flows remain within the main channel. Flows fall off significantly within the proposed turning basin. South of the proposed turning basin, flows accelerate through the narrow, constricted portion of the river. On flood, flows are largely unremarkable with localized velocity increases north of the proposed turning basin. Figures 5.17 and 5.18 show velocity difference plots between the *C1* and *Authorized* simulations. On ebb, localized velocity increases exist north of the existing turning basin. At the proposed turning basin, velocities decrease dramatically around the outer perimeter of the basin. South of the proposed basin, velocities increase slightly. A similar large decrease in velocity around the outer perimeter occurs on flood tide. Also similar to the ebb difference, velocities increase immediately north of the turning basin. Small increases in flows are apparent east of the main river conveyance.

Figures 5.19 and 5.20 show the sediment transport plots for peak ebb and flood. On ebb, large gradients of sediment transport potential lead into both the existing and proposed turning basins. On flood, the sediment transport rates show a similar gradient when exiting the proposed turning basin. The coincidence of both the ebb and flood gradient location indicates an active sediment transport zone. Alternating accretion and erosion will occur with the changes in tide.

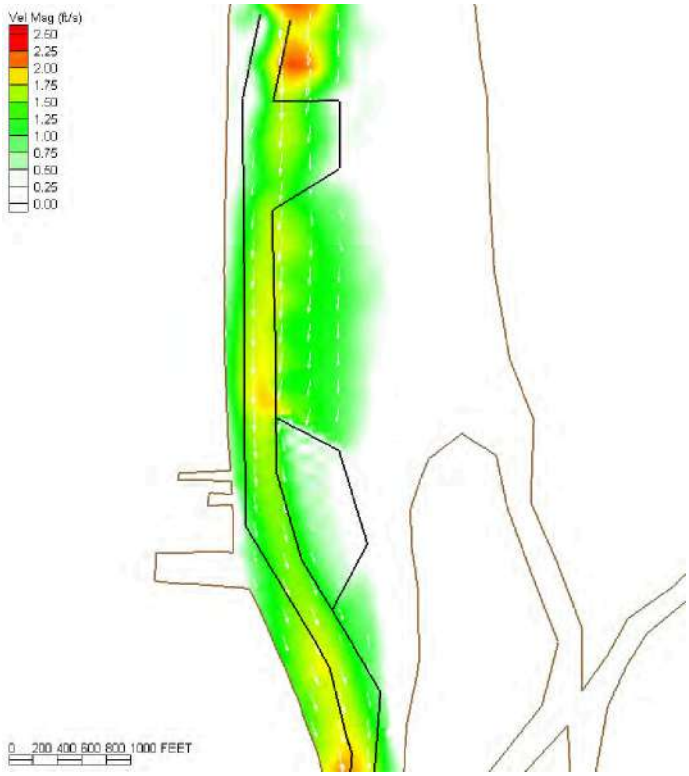


Figure 5.15 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C1* Simulation

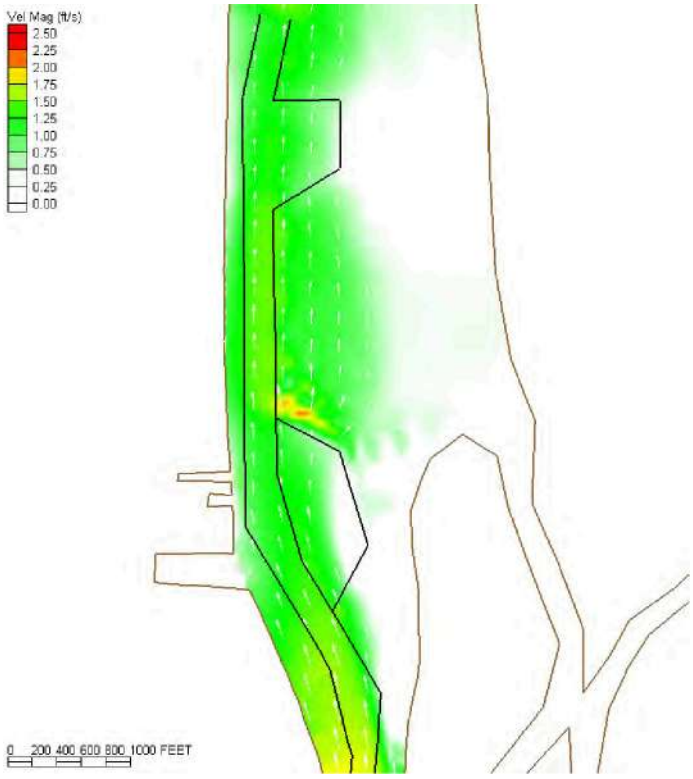


Figure 5.16 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C1* Simulation

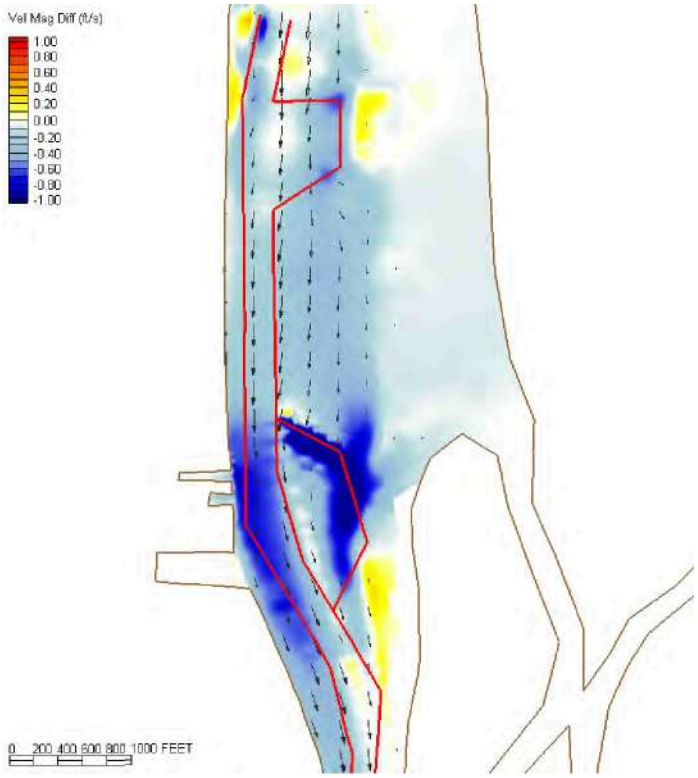


Figure 5.17 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C1* Simulation

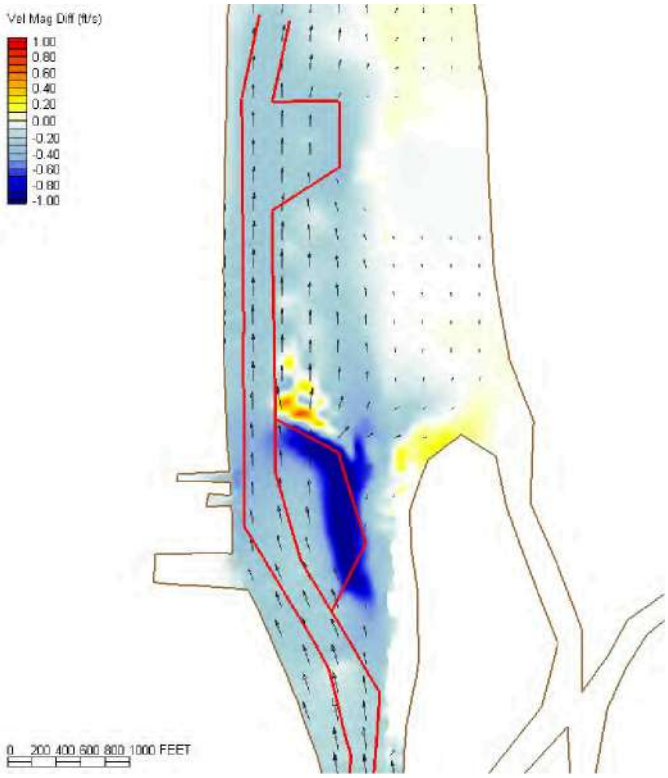


Figure 5.18 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C1* Simulation

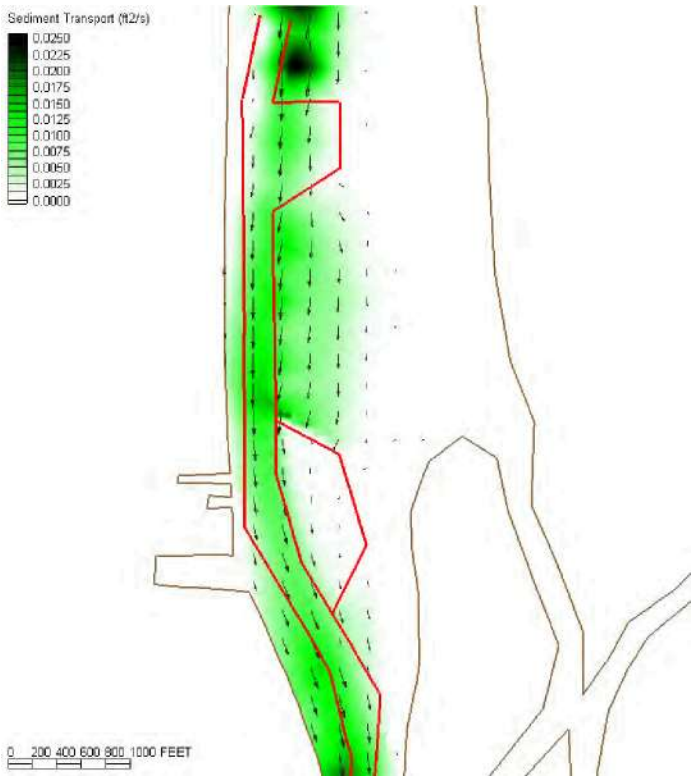


Figure 5.19 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *C1* Simulation

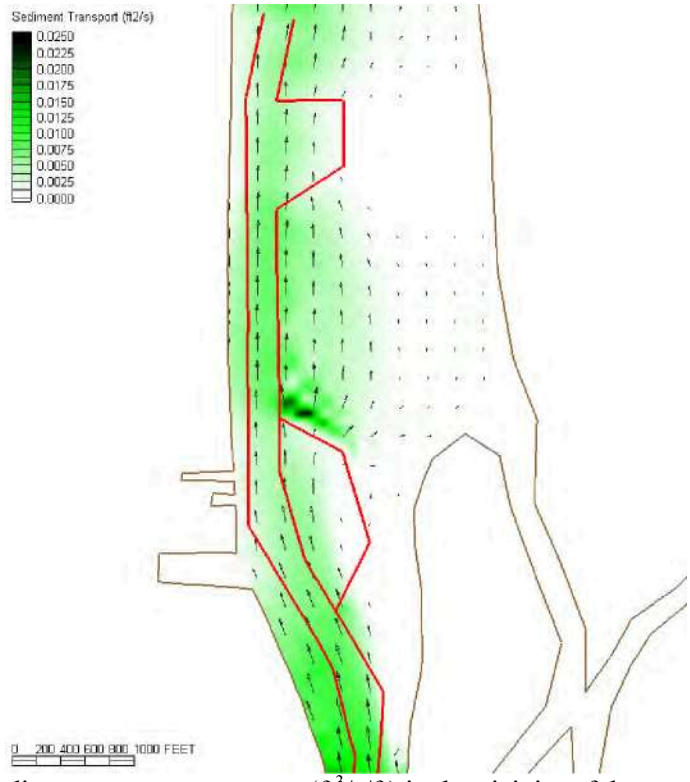


Figure 5.20 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak flood flow for the *C1* Simulation

5.5 C2 Simulation

The *C2* alternative included construction of a new turning basin with -39 ft-MLLW elevations in the lower reach and -35 ft-MLLW elevations in the existing turning basin. The proposed *C2* turning basin is located approximately 300 ft south of the proposed *C1* turning basin.

Figure 5.21 and 5.22 show contoured velocity magnitudes at time of maximum ebb and flood flows. On ebb, trends similar to the C1 simulation occur for the *C2* simulation - velocities slow down into the existing turning basin, flows mainly confined to the channel, flows fall off dramatically within the proposed basin and accelerate through the confined southern reach. Similarly, the trends are duplicated on flood with only a slight increase in velocity north of the proposed basin. Figures 5.23 and 5.24 show velocity difference plots between the *C2* and *Authorized* simulations. Since the *C2* velocity magnitude plots approximate the *C1* results, the velocity magnitude difference plots are also similar. However, although the trends are similar to *C1,* the magnitude differences are not as pronounced. Thus, the flow regime for the proposed *C2* turning basin will have lower impact than the proposed *C1* turning basin.

With less aggressive flow velocities, sediment transport potential should decrease. Figures 5.25 and 5.26 support this hypothesis. On ebb, negligible sediment transport occurs within the proposed turning basin. Lower transport magnitude gradients lead into the basin than observed during the *C1* simulation. The flood transport plot is virtually identical to the *C1* simulation; however, lower sediment transport magnitudes occur within the reach adjacent to the proposed basin.

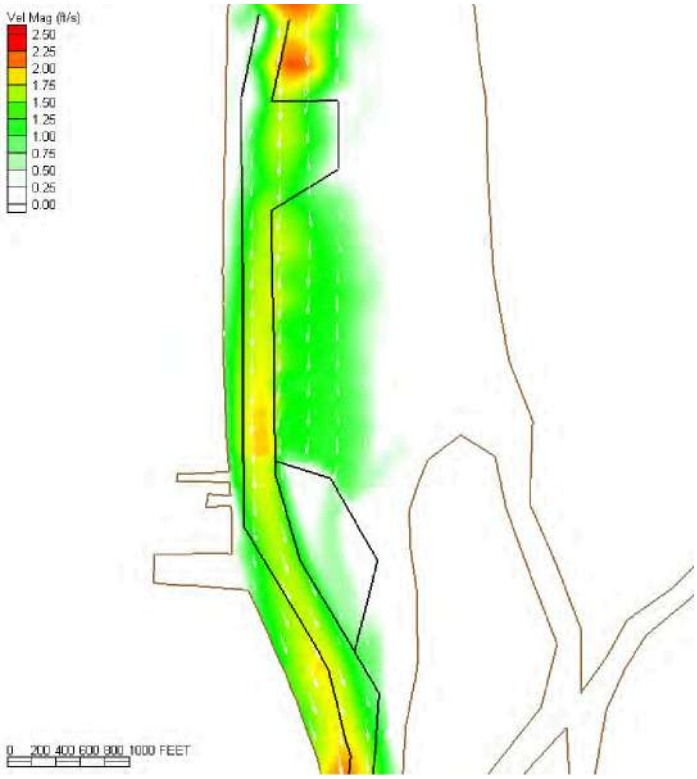


Figure 5.21 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C2* Simulation

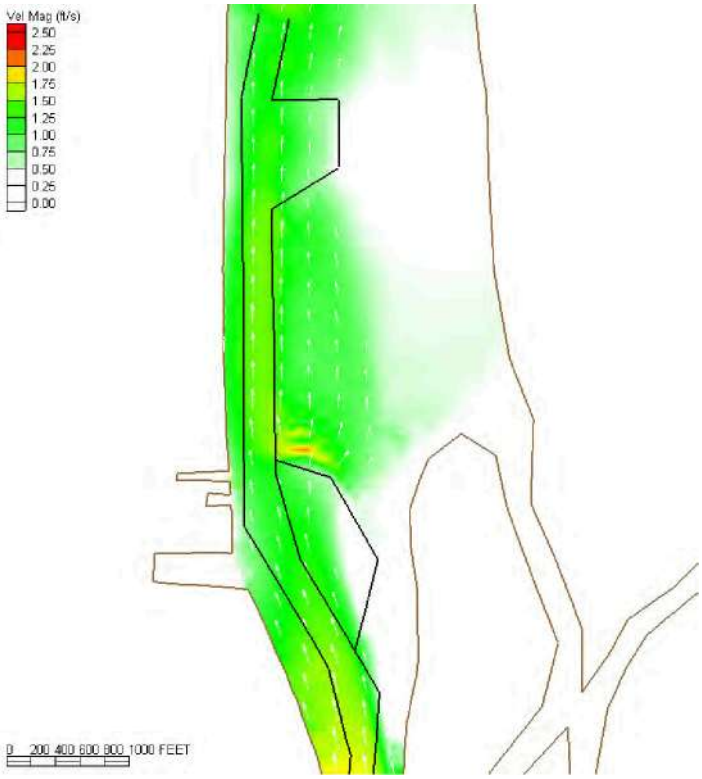


Figure 5.22 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C2* Simulation

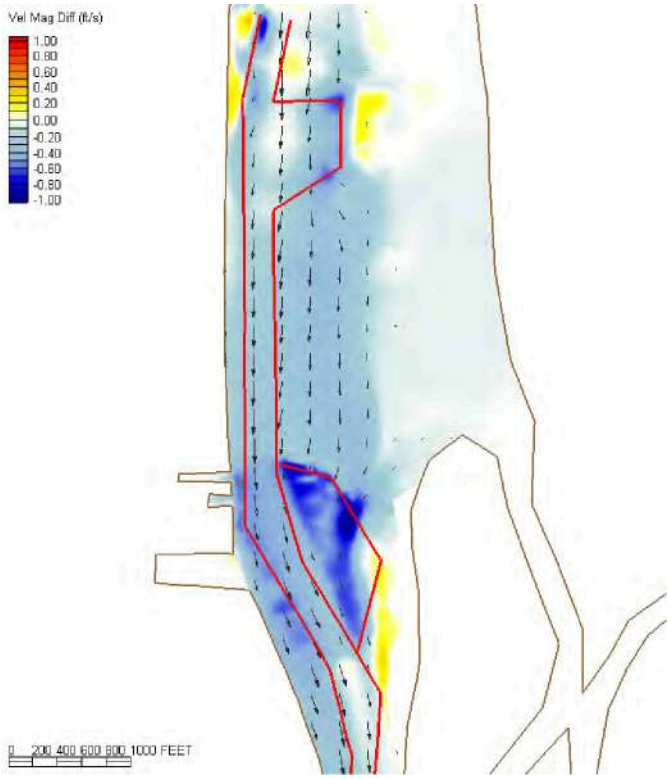


Figure 5.23 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C2* Simulation

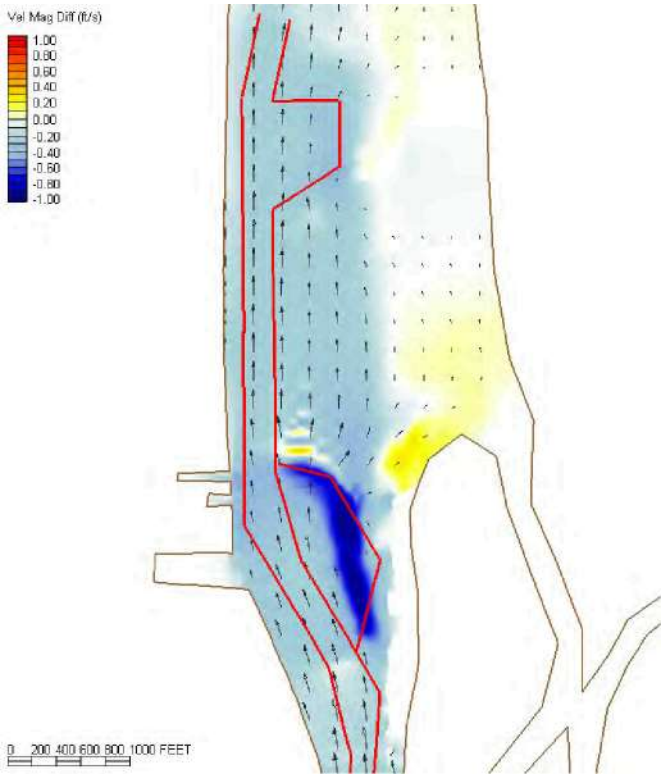


Figure 5.24 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C2* Simulation

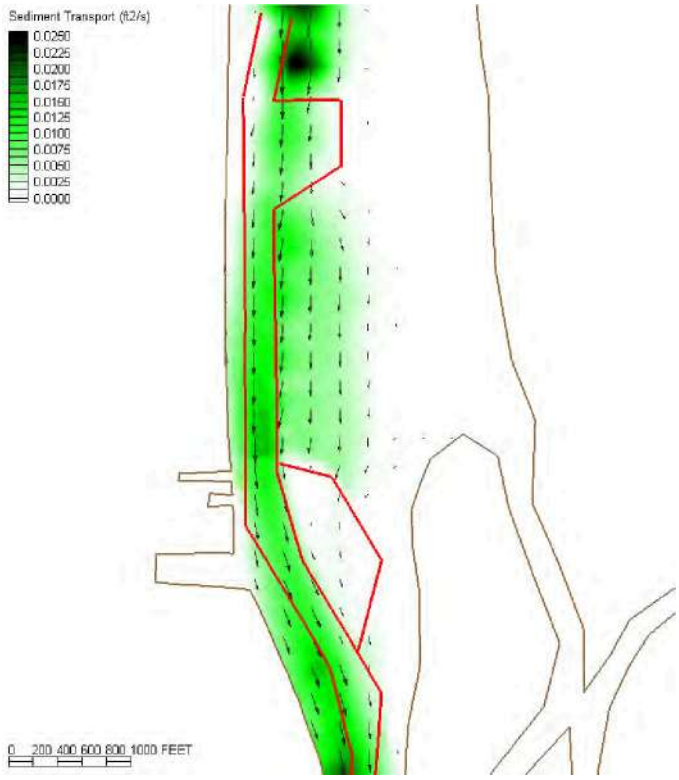


Figure 5.25 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *C2* Simulation

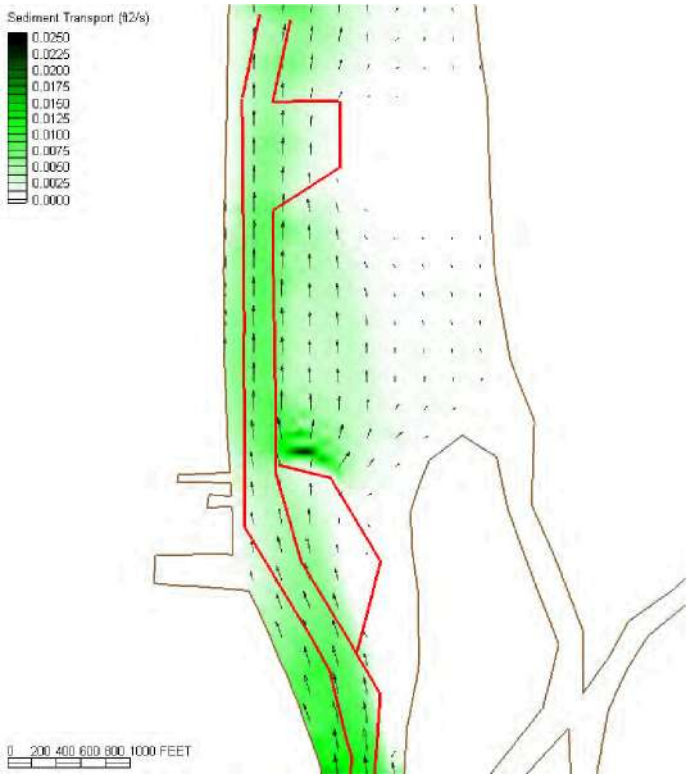


Figure 5.26 Sediment transport contours (ft /s/ft) in the vicinity of the turning basin during peak flood flow for the *C2* Simulation

5.6 C3 Simulation

The *C3* alternative included construction of a new turning basin with -39 ft-MLLW elevations in the lower reach and -35 ft-MLLW elevations in the existing turning basin. The proposed *C3* turning basin lies approximately 800 ft and 500 ft south of the proposed *C1* and *C2* turning basins. Unlike the proposed *C1* and *C2* basins (which extend to the east of the federal channel), the *C3* basin extends both to the west and east of the federal channel.

Figure 5.27 and 5.28 show contoured velocity magnitudes at time of maximum ebb and flood flows. Although the geometry for the *C3* basin differs from the *C1* and *C2* basins, the velocity magnitude trends are similar. During ebb flow velocities slow down upon entering the existing turning basin, flows are mainly confined to the channel, flows drop off dramatically at the eastern side of the proposed basin and accelerate through the confined southern reach. During flood flow, velocities decrease upon entering the western side of the proposed basin. Figures 5.29 and 5.30 show velocity difference plots between the *C3* and *Authorized* simulations. Again, the velocity magnitude difference plots are similar to previous simulations. During ebb, isolated velocity increases occur north of the existing turning basin. In the proposed basin, the velocity decreases up to 1 ft/s throughout the proposed basin. Small velocity increases exist to the east and south of the basin. During flood, the most apparent velocity change occurs on the eastern side of the proposed basin where velocities decrease up to 1 ft/s. Small increases occur south of the western side of the proposed basin.

Figures 5.31 and 5.32 show plots of sediment transport potential. During ebb, negligible sediment transport occurs within the proposed basin. The lower transport magnitude gradients leading into the basin on the eastern side of the channel occur than previous simulations. On the western side, sediment transport increases south of the basin and indicate erosion. During flood, slight decreases in sediment transport occur on both sides of the proposed basin.

Figure 5.27 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C3* Simulation

Figure 5.28 Velocity vectors and velocity magnitude contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C3* Simulation

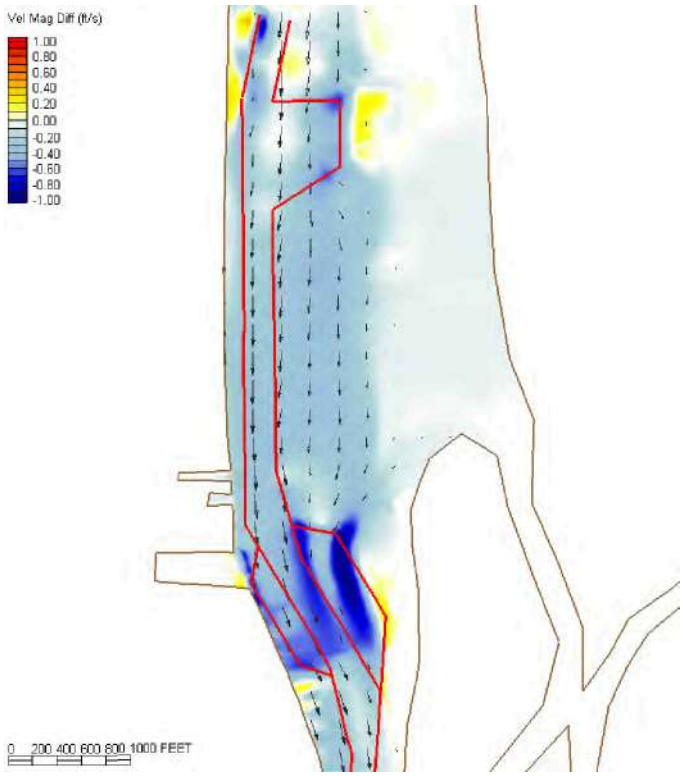


Figure 5.29 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak ebb flow for *C3* Simulation

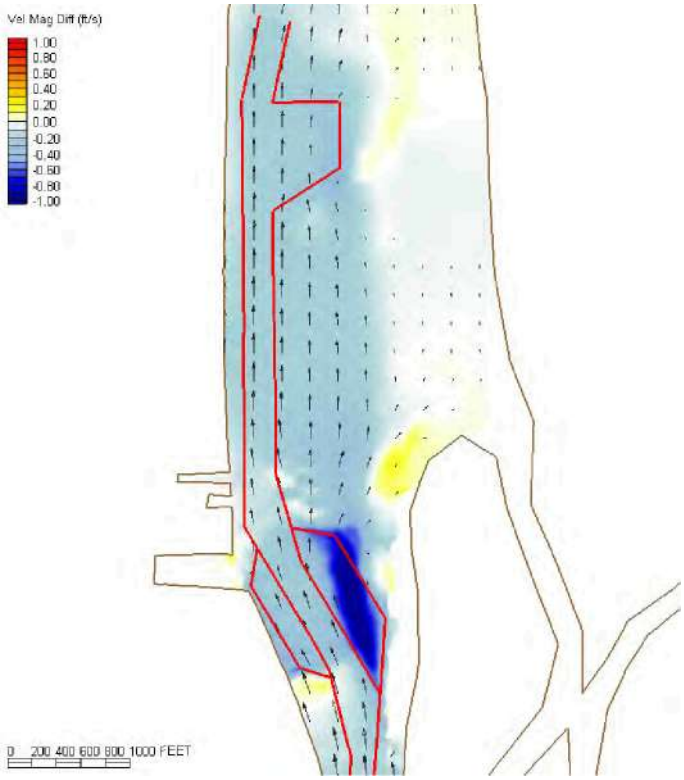


Figure 5.30 Velocity difference contours (ft/s) in the vicinity of the turning basin during peak flood flow for *C3* Simulation

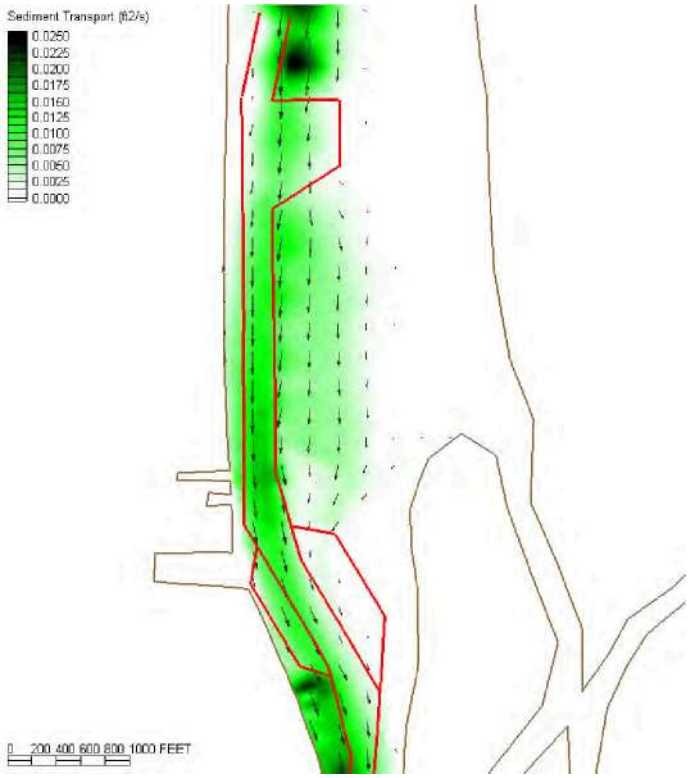


Figure 5.31 Sediment transport contours ($ft^3/s/ft$) in the vicinity of the turning basin during peak ebb flow for the *C3* Simulation

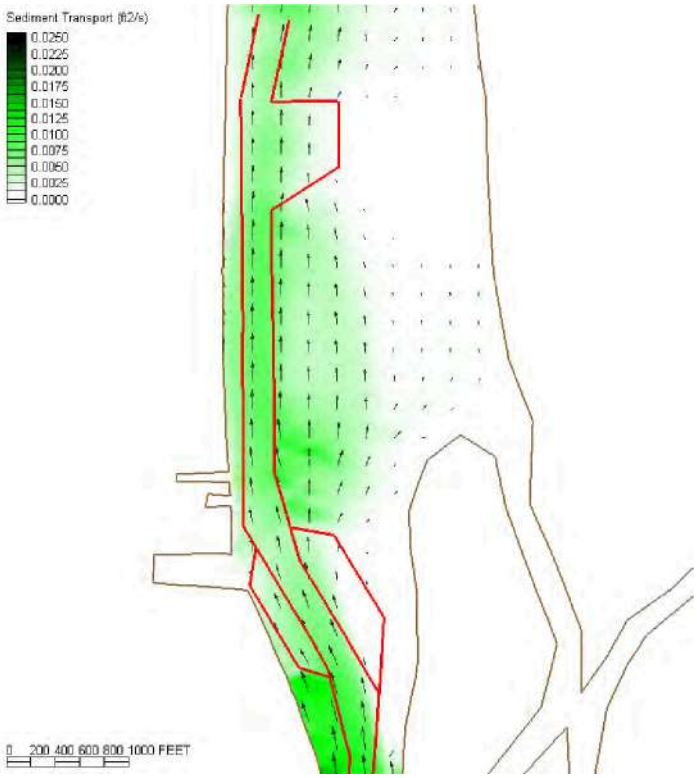


Figure 5.32 Sediment transport contours (ft /s/ft) in the vicinity of the turning basin during peak flood flow for the *C3* Simulation

5.7 Quantitative Analysis

The proposed turning basins shoaling assessment described in the previous sections provide a qualitative assessment of expected shoaling patterns based on the hydrodynamic modeling. This section addresses quantitative impacts that the proposed turning basin alternatives may have on the NE Cape Fear River flow regime.

5.7.1 Tide Range

Within the numerical model, Taylor Engineering positioned simulation gages in the channel adjacent to the existing and proposed turning basins to examine the potential change to tidal range from the proposed turning basin modifications. Table 5.1 presents the predicted tide range for both existing and proposed dredged bathymetric simulations. The table shows minimal (approximately 1%) increase in tide range for the proposed configurations at the existing (or *B1)* turning basin, and negligible variation in tide range at the proposed turning basins *(C1, C2,* or *C3).*

Table 5.1 Tide Range for *Existing* and Proposed Simulations

Gage Location	Model Run	Tide Range	Gage Location	Model Run	Tide Range
Existing Turning Basin	*Existing*	4.20	Proposed Turning Basin	*Existing*	4.24
	Authorized	4.21		*Authorized*	4.23
	B1	4.24		*B1*	4.24
	C1	4.24		*C1*	4.24
	C2	4.23		*C2*	4.24
	C3	4.23		*C3*	4.24

5.7.2 Flow Rates

Taylor Engineering applied nodestrings across the channel and turning basins to extract flow rates and compare the impact of each alternative on flow rates in the NE Cape Fear River. Table 5.2 details the flow rates (applied within the channel and turning basin boundaries only) for the various alternatives. Flows increase for all proposed alternatives over the existing conditions. In general terms, a maintained channel provides a more efficient flow path than a more natural condition channel. Artificially large depths can transmit more flow relative to a natural channel. The *B1* simulation includes the deepest dredging depths and produces the highest channel and turning basin flow rates. Additionally, with a maintained channel, flows concentrate in path of least resistance - i.e. inside as opposed to outside the channel. The *Authorized* simulation produces larger flow rates within the channel than the turning basin.

Alternatives *C1, C2* and *C3* show flows more confined to the dredged channel. In the turning basin, flows in *C1* and *C2* decrease in comparison to the *Authorized* simulation on ebb and remain relatively unchanged on flood. Flow rates in the proposed *C3* turning basin increase on both ebb and flood.

Table 5.2 Flow Rates for *Existing* and Proposed Simulations

Flow Rate Location	Simulation	Turning Basin Flow Rate (cfs)		Adjacent Channel Flow Rate (cfs)	
		Ebb	**Flood**	**Ebb**	**Flood**
Existing Turning Basin	*Existing*	17,108	15,638	10,517	12,618
	Authorized	20,039	17,155	10,643	12,658
	C1	23,754	18,590	13,614	14,025
	C2	23,755	17,884	13,575	14,730
	C3	23,746	18,011	13,564	14,700
Proposed Turning Basin	*B1*	22,954	15,733	15,042	14,497
	C1	14,346	18,641	19,988	17,633
	C2	12,265	17,493	22,393	16,672
	C3	20,829	22,134	23,275	17,731

5.7.3 Average Velocity

Taylor Engineering calculated average velocities through the middle of the turning basin and adjacent channels. This information is intended to verify ships can safely maneuver in the proposed basin. The average velocities do not vary substantially in the existing turning basin for any of the simulations. Of all the simulations, only alternative *C3* shows an increase in average velocity compared to the *Authorized* simulation. Velocities increase in the adjacent channel on ebb from 1.47 to 1.59 ft/s. However, the average velocity in the *C3* turning basin represents an average across both the east and west portions of the turning basin. The velocity magnitude plots show higher velocities within the western portion of the turning basin. Separated, the average velocity west of the channel (1.61 ft/s on ebb and 1.34 ft/s on flood) is significantly higher than east of the channel (0.54 ft/s on ebb and 0.78 ft/s on flood).

Table 5.3 Average Velocities for *Existing* and Proposed Simulations

Velocity Location	Simulation	Turning Basin Average Velocity (ft/s)		Adjacent Channel Average Velocity (ft/s)	
		Ebb	**Flood**	**Ebb**	**Flood**
Existing	*Existing*	1.52	1.39	1.52	1.82
	Authorized	1.57	1.34	1.47	1.75
	C1	1.33	1.04	1.35	1.39
	C2	1.33	1.00	1.34	1.46
	C3	1.33	1.01	1.34	1.45
Proposed	*B1*	1.15	0.17	1.03	0.99
	C1	0.66	0.86	1.37	1.21
	C2	0.54	0.77	1.53	1.14
	C3	0.92	0.98	1.59	1.21

5.7.4 Shoaling Rates

A tangible shoaling rate for all alternatives considered is typically attained through application of a sediment transport model. Sediment transport models apply one or more sediment transport equations (e.g. Bailard/Bagnold method described earlier) to the hydrodynamic simulation results to derive sediment transport through individual elements in the model domain. The models predict erosion or accretion if the transport out is greater or less than the transport into each element. Although the application of a complex sediment transport model is beyond the scope of this analysis, Taylor Engineering applied a simple sediment transport model to better compare the proposed alternatives.

Taylor Engineering divided each turning basin into 6 cells (3 rows and 2 columns). Simulation gages placed at the boundaries of the cells provide components of velocity magnitude and water depth at every time step throughout the simulation. Applying each element's average water depth and corresponding velocity to the Bailard/Bagnold sediment transport equation determines the transport into and out of each element face. Volume change is computed from the finite difference form of the sand conservation equation

$$\frac{\partial q_x}{\partial x} + \frac{\partial q_y}{\partial y} = \frac{\partial h}{\partial t}$$

where q_x and q_y are the sediment transport components in the two horizontal directions x and y; h is the water depth; and t is time. The process was repeated for all cells over all time steps in one complete mean tidal cycle. Taylor Engineering then extended the shoaling rates to estimate cubic yards of material per

year and divided by the basin areas to estimate average feet of accumulation per year. Table 5.4 shows the results of the sediment transport model.

Table 5.4 Shoaling Rates predicted by the Sediment Transport Model

Shoaling Rate Location	Simulation	Turning Basin Shoaling Rate (cy/yr)	Turning Basin Shoaling Rate (ft/yr)	Adjacent Channel Shoaling Rate (cy/yr)	Adjacent Channel Shoaling Rate (ft/yr)
Existing Turning Basin	*Existing*	14,812	4.27	-1,556	-0.56
	Authorized	4,081	1.18	-3,664	-1.33
	C1	4,219	1.19	-2,731	-0.99
	C2	4,277	1.20	-2,859	-1.04
	C3	4,261	1.20	-2,879	-1.05
Proposed Turning Basin	*B1*	2,706	0.60	-2,598	-0.71
	C1	22,204	3.37	1,918	0.44
	C2	18,934	3.35	2,706	0.66
	C3	-7,405	-1.90	955	0.14

Table 5.4 indicates the existing turning basin shoals at approximately 14,812 cy/yr. This prediction in shoaling rates is more then the average rate calculated by the DMS-Data Manager methodology in Chapter 3 for 1994-2003 (8,477 cy/yr). However, according to the DMS-Data Manager calculations, the turning basin shoaling rate varies between -2,400 and 46,000 cy/yr over all complete surveys. Since the turning basin's shoaling rate fluctuates over the available surveys, the rates listed in Table 5.1 determined from the sediment transport model should be considered in terms of relative comparisons only.

Ships maneuvering in the turning basin must access the adjacent channel. As such, Table 5.4 also lists shoaling rates in the channel adjacent to the turning basins. The sediment transport model predicts the channel immediately adjacent to the existing turning basin will remain stable (no deposition) for all alternatives.

In the existing turning basin, shoaling rates would decrease from *Existing* to *Authorized* conditions due to a decrease in sediment transport gradient across the northern boundary of the basin. Higher shoaling rates for the *Authorized* simulation are expected northwest of the existing turning basin. The shoaling rates increase slightly between the *Authorized* simulation and all alternatives *(C1, C2,* and *C3)* in the existing turning basin. For the *B1* simulation, shoaling rates decrease in the *B1* turning basin (in effect an extension of the existing turning basin) largely due to decreased velocities (and sediment transport gradient) in the approach to the basin.

Further south at the proposed turning basins *C1, C2,* and *C3,* no existing turning basin exists. As such, shoaling rate comparisons are made between the proposed basins only. Of the three basins, *C1* should experience the most shoaling (22,204 cy/yr). *C2* should experience slightly less shoaling of 18,934 cy/yr. Basins *C1* and *C2* suffer from large shoaling rates due to flow propagating across the shallow shoal adjacent to Smith Creek and into the dredged basins. The *C3* calculations show net erosion (negative shoaling rate) of 7,405 cy/yr. However, the shoaling rate for the proposed *C3* basin can be further divided into sections west and east of the channel. West of the channel, the sediment transport model predicts erosion of 9,429 cy/yr, whereas east of the channel, the basin shoals at 2,024 cy/yr. Thus, this modeling approach indicates the area east of the channel will likely shoal over time.

The limitations of the above simple methodology are as follows. First, RMA2 assumes two-dimensional depth-averaged flow in its computational scheme. Three-dimensional flow is typical on the near dredged areas where flow stratification and complex vortex formation exist. The complex flow patterns contribute to complex sediment transport rates.

Second, the drawback to post-processing RMA2 results in a simple sediment transport model is that the two models do not communicate with each other. The RMA2 model assumes a fixed bathymetry from which hydrodynamic parameters are calculated over the tidal cycle. Conversely, the sediment transport model assumes the bathymetry dynamically varies due to sediment transport throughout the tidal cycle. Elevation changes due to sediment transport predicted in the sediment transport model are not accounted for in the RMA2 computations.

Finally, due to the multiple variables and complex phenomena involved with sediment transport, equations predicting transport magnitudes contain a large probability of error. For these reasons, the current methodology should not be applied to specifically extrapolate definitive long-term transport magnitudes and shoaling rates. The sediment transport model results should be considered qualitatively to identify the locations of active elevation change rather than form a quantitative estimation of the magnitude of elevation change. Taylor Engineering presents the long term predictions of elevation change for relative comparison between the proposed alternatives only.

6.0 CONCLUSIONS

Based on the DMS and hydrodynamic modeling analysis performed, **option *C3* is the preferred alternative for the NE Cape Fear River turning basin** in terms of least potential of shoaling and maintenance requirements. The sediment transport calculations show option *C3* to have the least potential for shoals to develop of the proposed alternatives. The location of the basin is furthest away from potential storm-induced shoaling from Smith Creek. However, the proposed *C3* basin may provide navigational concerns as velocity magnitudes are slightly higher and are disparately distributed across the turning basin (higher velocities to the west) than the other alternatives. Finally, the elevations to the east of the proposed *C3* basin are extremely shallow - the banks may slough into the dredged channel.

6.1 Additional Considerations

1) The mesh input for each proposed simulation represents the bathymetry that would occur immediately following construction of the deepening project and full maintenance in the remainder of the channel. As such, the model mesh reflects conditions that will actually occur at a point in the future (following completion of the deepening project). Since different sections of the channel shoal at different rates, maintenance of the entire channel at a single instance in time will most likely never occur.

2) Taylor Engineering recommends that the USACE-Wilmington employ a sediment transport model - such as SED2D - to gain a better understanding of the shoaling processes in the project area.

3) Storm events transport large quantities of sediment over comparatively little time. Storm events that superelevate the water surface change the dynamics of the flow regime. The analysis performed in this report does not address storm-induced shoaling. Taylor Engineering recommends storm simulation modeling - both riverine and surge induced - to quantify the contribution of shoaling related to storm events.

7.0 REFERENCES

Bagnold, R.A. (1963) "Mechanics of Marine Sedimentation." The Sea: Ideas and Observations, vol. 3, Interscience, New York.

Bagnold, R.A. (1966) "An Approach to the Sediment Transport Problem from General Physics." U.S. Geological Survey Professional Paper, 422-1.

Bailard, J.A. (1982) "Modeling On-Offshore Sediment Transport in the Surfzone." Proc. Of the 18th Coastal Engineering Conference, ASCE, pp. 1419-1438, Cape Town, South Africa, November 1982.

Bailard, J.A. (1981) "An Energetics Total Load Sediment Transport Model for a Plane Sloping Beach." J. of Geophysical Research, Vol. 86, No. C11, pp. 10938-10954, Nov. 20, 1981.

Giese, G. L., Wilder, H. B., and Parker, Jr, G. G. (1985). "Hydrology of major estuaries and sounds of North Carolina," Paper 2221, U.S. Geological Survey, Alexandria, VA.

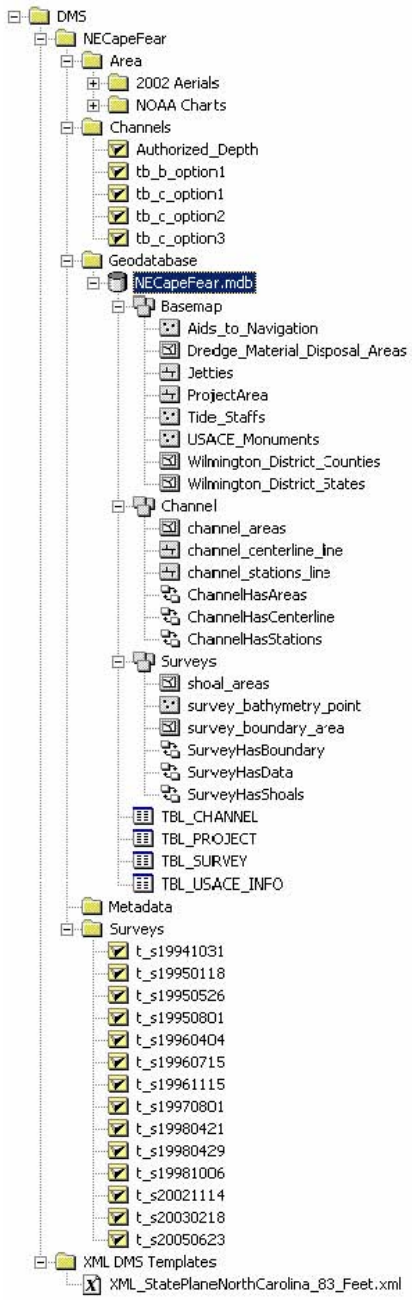
Julien, P.Y. (1998) Erosion and Sedimentation. Cambridge University Press, Cambridge, UK.

MacAdory, R. T. Jr (2000) "Cape Fear-Northeast Cape Fear River, North Carolina: Numerical Model Study," ERDC/CHL TR-00-18, Coastal and Hydraulics Laboratory, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

McCormick, (2005) U.S. Army Corps of Engineers, personal communication with John McCormick, September 2005.

U.S. Army Engineer District, Wilmington. (1960). "Study of shoaling, Cape Fear River and Sunny Point Army Terminal," Wilmington, NC.

APPENDIX A



Northeast Cape Fear River DMS Data Manager Project Folder and Geodatabase

APPENDIX B

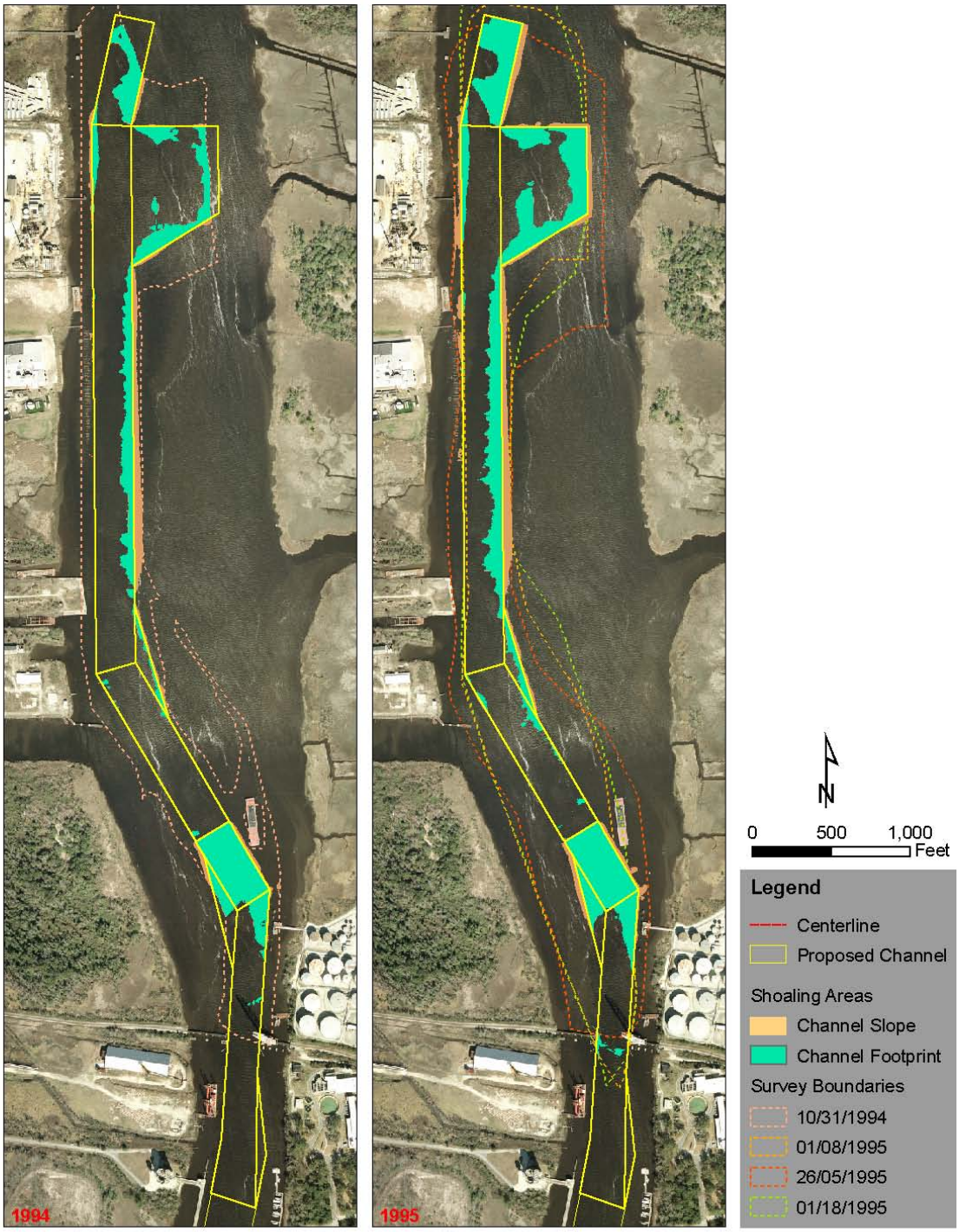


Figure 1 1994 and 1995 Shoaling Area Evolution within the Proposed Channel Footprint (Authorized Channel Depth Template)

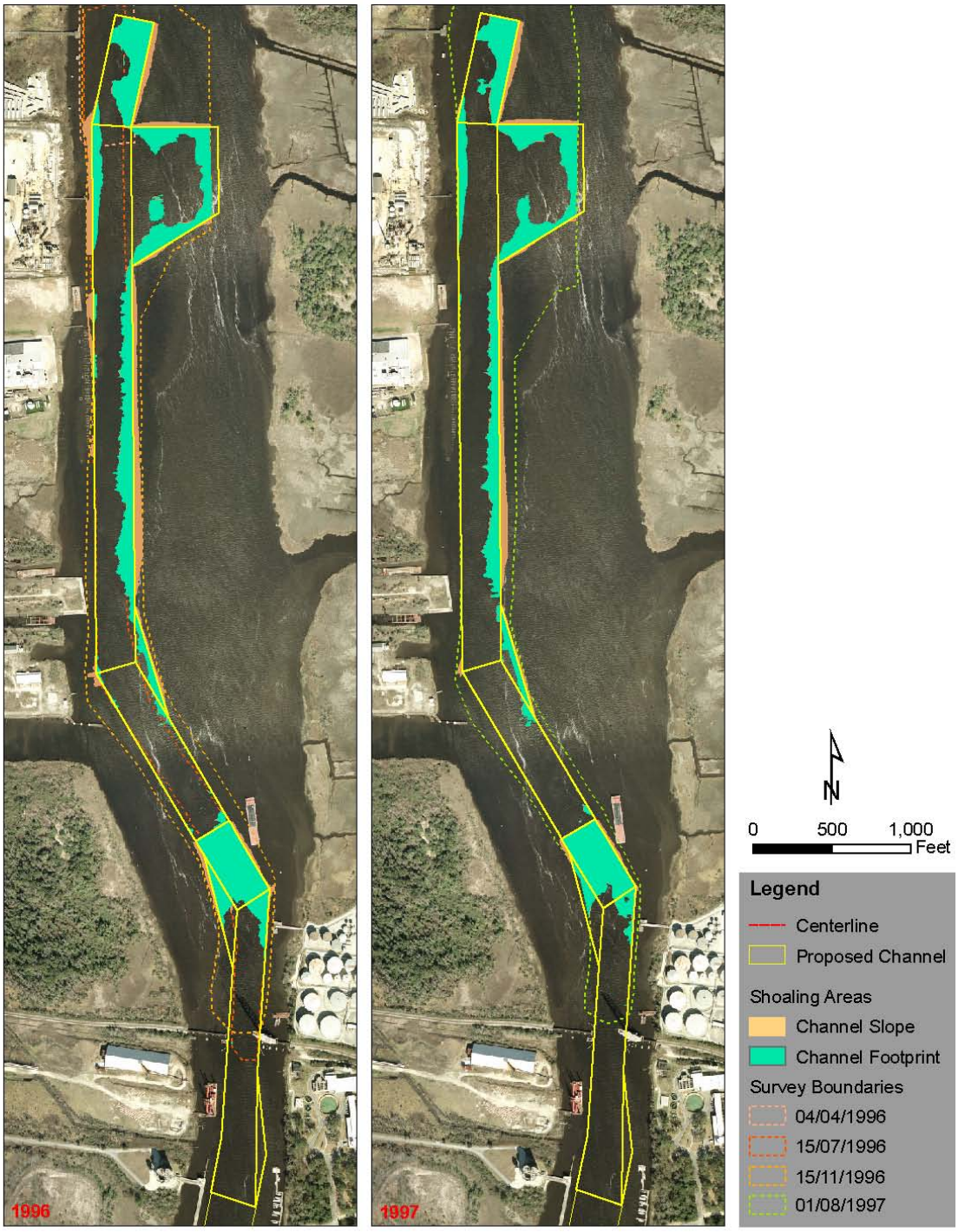


Figure 2. 1996 and 1997 Shoaling Area Evolution within the Proposed Channel Footprint (Authorized Channel Depth Template)

Figure 3. 1998 and 2002 Shoaling Area Evolution within the Proposed Channel Footprint (Authorized Channel Depth Template)

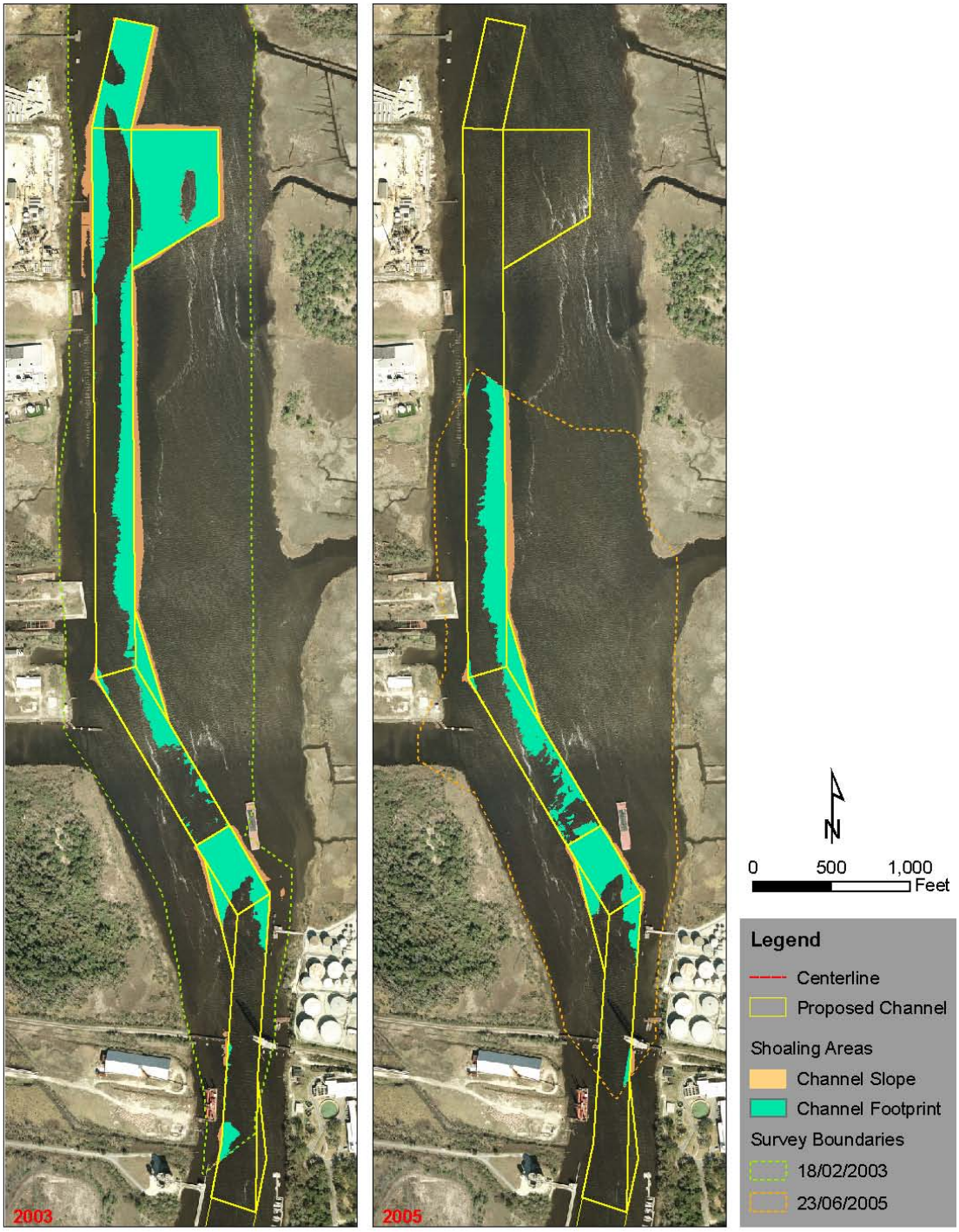


Figure 4. 2003 and 2005 Shoaling Area Evolution within the Proposed Channel Footprint (Authorized Channel Depth Template)

APPENDIX C

Figure 1. Sediment Volume Above the Proposed Turning Basin Option B1 Channel Template in the NE Cape Fear River for 2003 and 2005

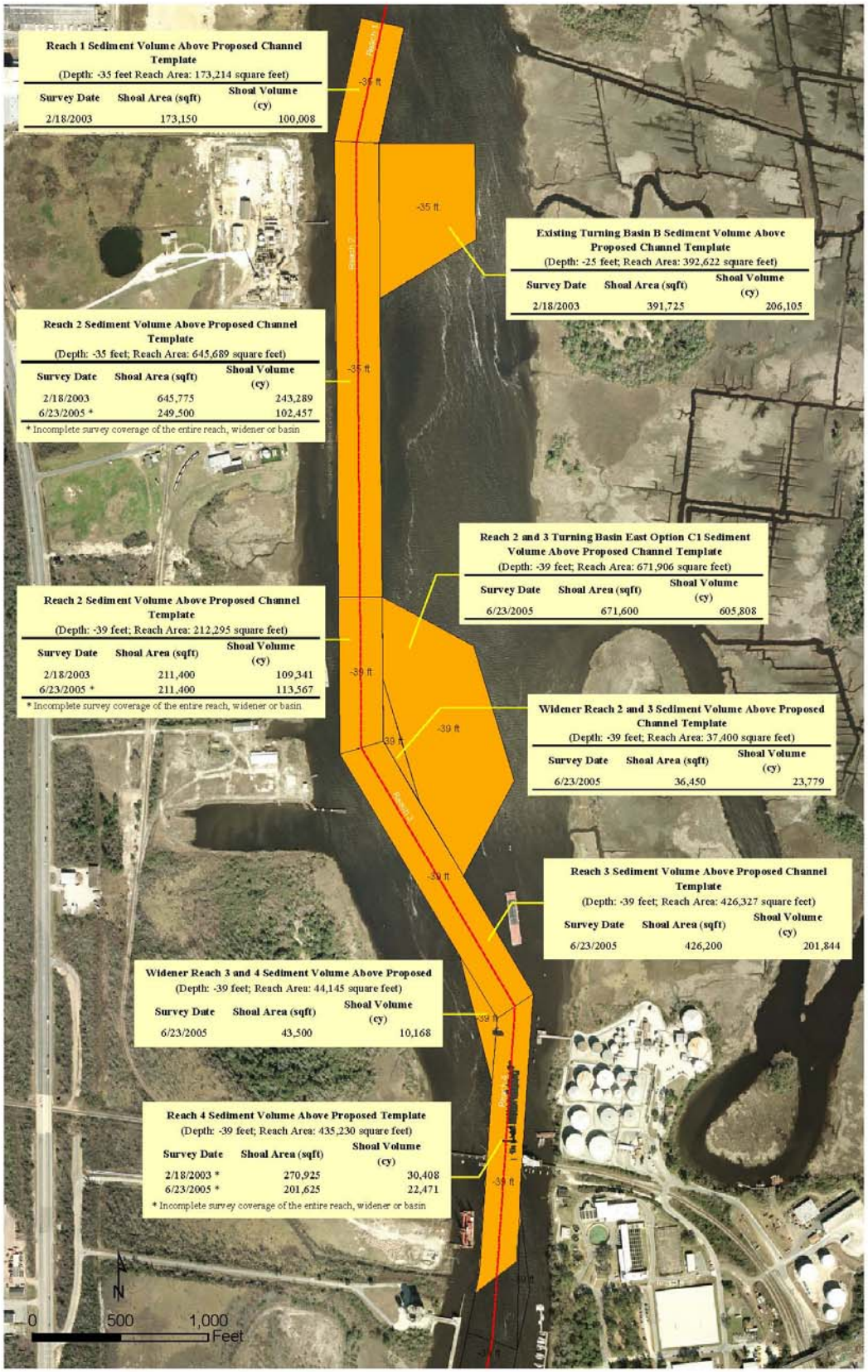


Figure 2. Sediment Volume Above the Proposed Turning Basin Option C1 Channel Template in the NE Cape Fear River for 2003 and 2005

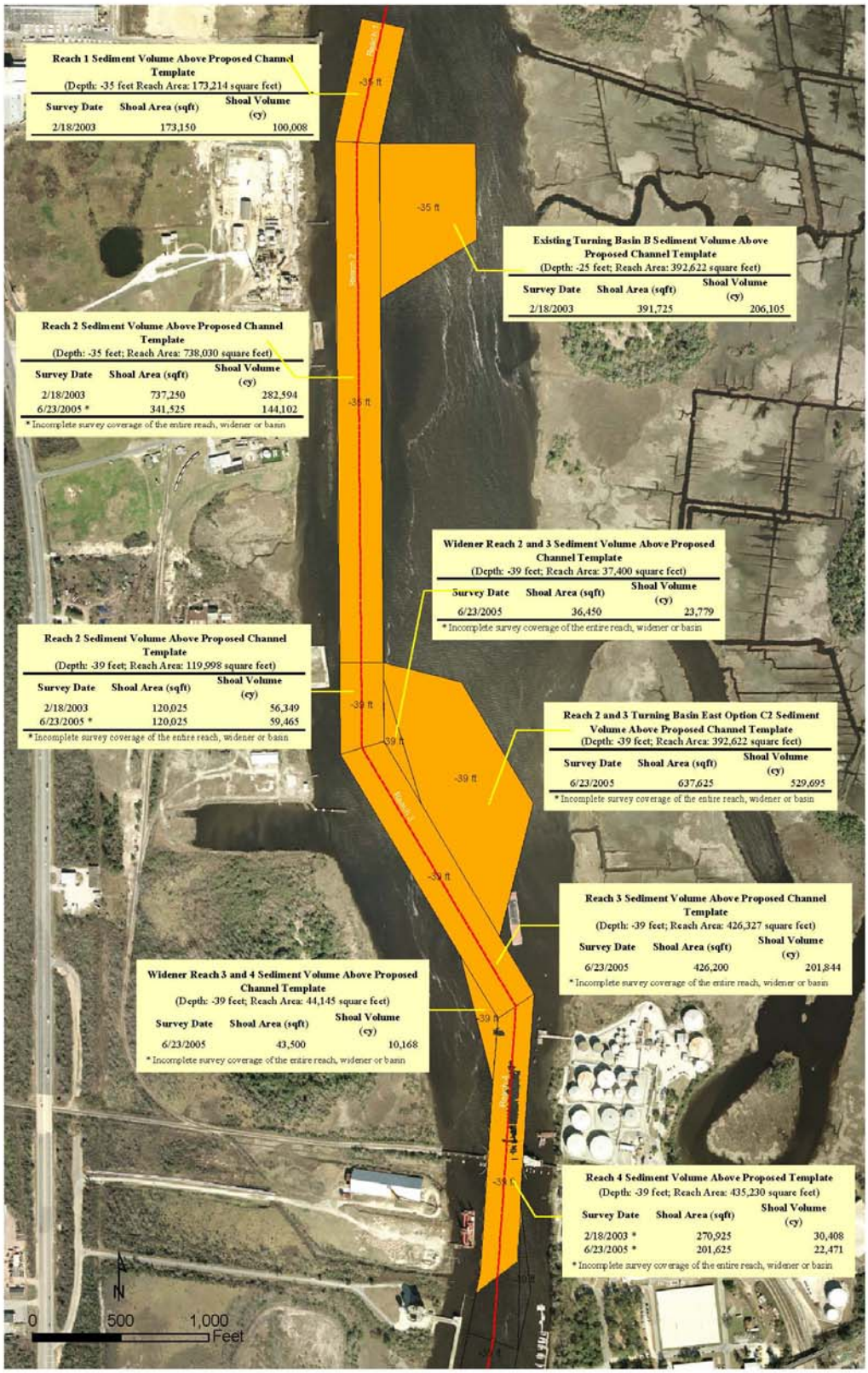


Figure 3. Sediment Volume Above the Proposed Turning Basin Option C2 Channel Template in the NE Cape Fear River for 2003 and 2005

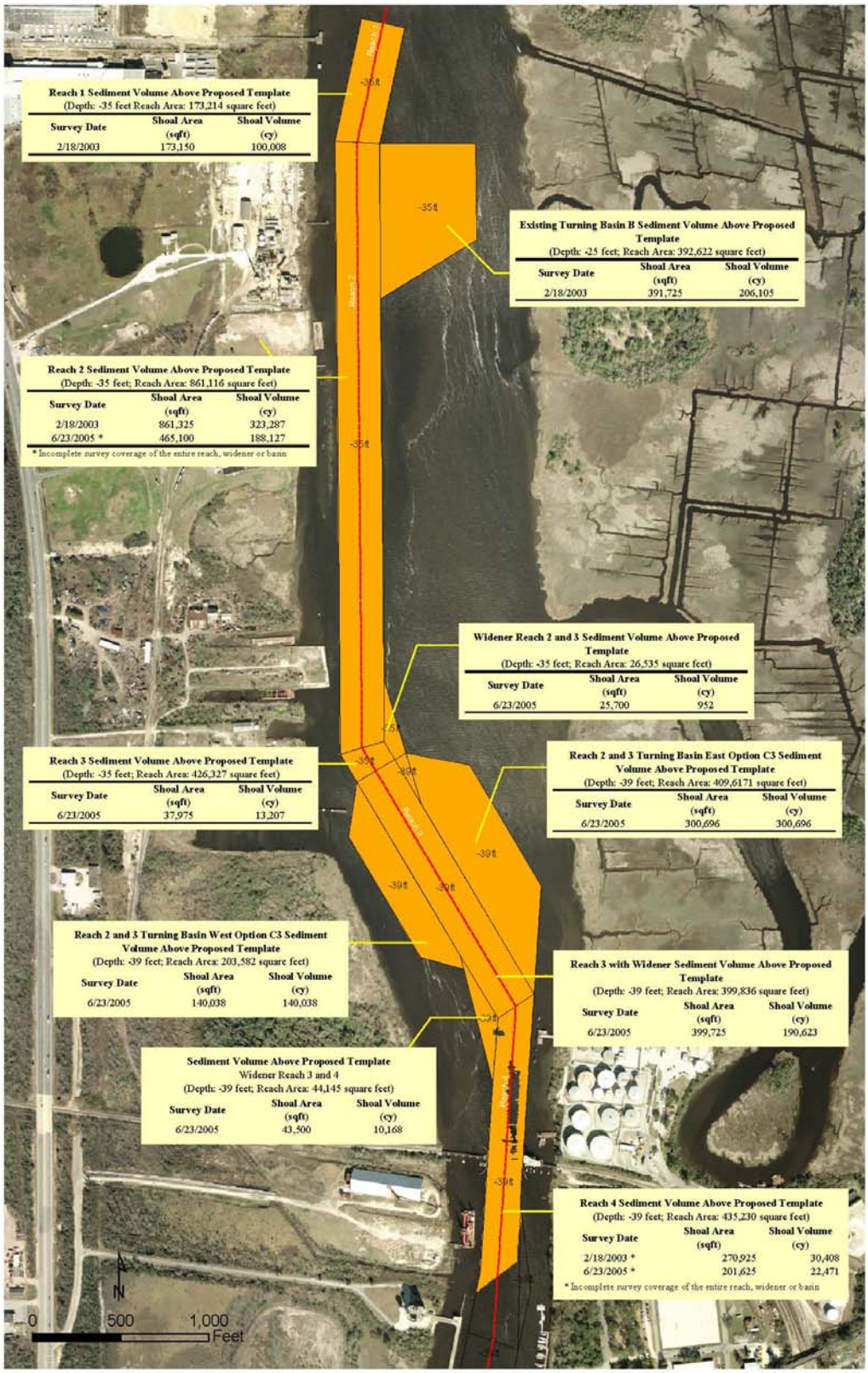


Figure 4. Sediment Volume Above the Proposed Turning Basin Option C3 Channel Template in the NE Cape Fear River for 2003 and 2005

END

Appendix 3.8.1

INTAKE EVALUATION
Mobile District

Wilmington Harbor GRR
Lock and Dam Study
Intake Evaluation

Prepared for:

Army Corps of Engineers
Wilmington District
Wilmington, North Carolina

Prepared by:

Army Corps of Engineers
Mobile District
Mobile, Alabama

MARCH 2007
FINAL

TABLE OF CONTENTS

Narrative

1. General. 1
2. Pump Stations 1
   A. Glenville Pump Station, City of Fayetterville 1
   B. Hoffer Pump Station, City of Fayetteville 2
   C. DuPont Pump Station 3
   D. Lower Cape Fear Water and Sewer Authority Pump Station 3
   E. City of Wilmington Pump Station 5
3. Summary 6
Summary of Elevations on Cape Fear River 8
Water Surface Elevations @ 300 CFS 9

Cost Estimate

Cost Estimate Summary 10
Glenville Intake Cost Estimate 11
Hoffer Intake Cost Estimate 12
DuPont Intake Cost Estimate 13
LCFWASA Intake Cost Estimate 14
City of Wilmington Intake Cost Estimate 15

Drawings

Glenville Pump Station Existing Conditions C-201
Glenville Pump Station Revisions C-202
Hoffer Pump Station Existing Conditions C-203
Hoffer Pump Station Revisions C-204
DuPont Pump Station Existing Conditions C-205
LCFWASA Pump Station Existing Conditions C-206
LCFWASA Pump Station Revisions C-207
City of Wilmington Pump Station Existing Conditions C-208
City of Wilmington Pump Station Revisions C-209

NARRATIVE

Wilmington Harbor GRR
Lock and Dam Study
Intake Evaluation

1. General

This chapter covers the water supply to five existing pump stations on the Cape Fear River from Fayetteville, North Carolina to Wilmington, North Carolina. This study consists of evaluating the impacts to the existing pump stations for three alternatives. Alternative 1 is for the water surface elevation with the locks and dams removed. Alternative 2 is for the water surface elevation with the locks and dams lowered to an elevation of no impact to the intakes. Alternative 3 is for the water surface elevation with the locks and dams lowered so that no rock ramps are required for fish passage. Changes to pump stations and intake structures recommended in this report are offered only if one of the above alternatives is selected through the General Reevaluation Report (GRR) process. A summary of elevations which also contains the maximum daily flow and average daily flow for each of the existing pump stations is on page 8. A spreadsheet provided by the Wilmington District with the existing low water levels and the calculated low water levels for each of the three alternatives for each pump station is on page 9.

2. Pump Stations

A. Glenville Pump Station, City of Fayetteville: The profile of the Glenville Pump Station is shown on Sheet C-201. The Glenville pump station is a square concrete structure. The inside bottom elevation is 15.00 feet. Motors for the vertical turbine pumps and the traveling screen are mounted on the top slab of the pump station and are exposed to the weather. The intake structure is located about 120 feet from the pump station. The intake structure has a screen to stop debris and a sluice gate to control flow of water to the pump station. The carrier pipe from the intake structure to the pump station is a 48" ductile iron pipe inside a 72" casing pipe and is set at a 0% slope with an invert elevation of 22.00. Glenville normally operates a pump station on a lake near Glenville. The Glenville pump station on the Cape Fear River only operates a few months a year based on the water surface elevation at their lake intake. The average daily flow through the Glenville pump station is 3 million gallons per day (mgd). The peak daily flow for the Glenville pump station is 7 mgd.

The minimum water surface elevation for Alternative 2 is 28.0 feet, which will not require any revisions to the intake structure.

The low water levels established for Alternatives 1 and 3 are set at elevation 23.00 and 24.10 respectively. These elevations are well below the top of the intake pipe. Changes to the intake structure will be required for both of these alternatives.

The recommended changes to meet the daily flow requirements for Alternatives 1 and 3 are shown on Sheet C-202 and are as follow:

- Remove the existing intake structure and intake pipe.
- Repair and seal opening into the pump station from the removed pipe.
- Install new 48" intake pipe at invert elevation 18.00.
- Install 2 new 3.5 mgd intake screens with an air cleaning system.

The Glenville pump Station will have to be taken out of operation for incorporation of these modifications.

The repair and seal operation noted above would involve removal of the section of the sump wall from a point some distance above the top of the existing pipe connection to the sump floor, and for a distance either side of the edges of the pipe. A required clearance of 18" around the

top and sides of the pipe connector is recommended. This will result in a section measuring approximately 9′ wide by 12.5 feet high being removed to facilitate removal of the existing pipe and installation of the new 48″ pipe connector. The perimeter of the wall section would be saw cut and the concrete removed taking care to protect and preserve the existing wall reinforcement to provide for splicing with new reinforcement as the concrete wall is renewed. Once the wall section is removed the new wall section would be formed with the new pipe connector supported in place, and the wall reinforcement restored. With these features in place concrete would be placed to restore the section of wall.

The removal and re-placement of the intake pipe will require an extensive amount of excavation. The excavation shall be properly designed in order to protect workmen during placement of the intake pipe. Previous expansions of the intake structures and pump stations in the area sequenced the construction by installing the pump station first followed by intake piping and intake structures. The pipe installation utilized underwater placement methods. After a substantial pipe trench was excavated the trench was allowed to be flooded from the river. This eliminated the need for a cofferdam in the river and a substantial dewatering effort along the pipe trench. Dewatering will still be required for the pump station installation. This construction method is expected to be used for the new pump facilities. Excavated materials shall be stockpiled and re-used, as much as practical, as trench backfill. All scarred areas will be seeded and mulched.

See quantity and cost summary sheets in the cost estimate for listing of items included in the pump station.

Note: There is an existing sediment problem at this site and it will also impact this recommended solution. There is not an adequate depth available for any type of intake structure. Dredging and/or diversion of flow to the right bank needs to be investigated.

B. Hoffer Pump Station, City of Fayetteville: The profile of the Hoffer Pump Station is shown on Sheet C-203. The Hoffer Pump Station is a concrete structure containing a wet well, loading bay, and operating floor. Pumps, motors and controls are located inside the structure. The inside bottom elevation of the wet well is at elevation 15.00 feet. There are two intake structures each consisting of two screens set at a centerline elevation of 25.00 feet. The intake pipes from the screens to the wet well are 60″ in diameter. The centerline of both intake pipes into the wet well are set at elevation 19.00 feet. The average daily flow through the Hoffer Pump Station is 17 million gallons per day (mgd). The peak daily flow for the Hoffer pump stations is 30 mgd.

The minimum water surface elevation for Alternative 2 is 28.0 feet, which will not require any revisions to the intake structure.

The low water levels established for Alternatives 1 and 3 are set at elevation 23.00 and 24.10 respectively. The existing intake screens will not be fully submerged at either of these elevations.

The recommended changes to meet the flow requirements for Alternatives 1 and 3 are shown on Sheet C-204 and are as follow:

- Remove the existing intake pipes to the point of connection with the new intake pipes.
- Replace the existing intake screens with 6 new 5 mgd screens set at a new centerline elevation of 21.00 feet. The existing screens are 36″ in diameter and will not function as designed in 3 to 4 feet of water. The existing air cleaning system will be connected to the new screens.
- Install new 60″ intake pipes from both sets of screens level at a centerline elevation of 21.00 and connect to the existing intake pipes at that same elevation.

The Hoffer Pump station will have to be taken off line for incorporation of these modifications.

The recommended changes to the pump station will require the removal

and replacement of existing slope protection systems. A riprap section will be used for the replacement of the concrete fabriform slope protection. The replacement of the intake pipes will require an extensive excavation. Excavated materials shall be stockpiled and re-used, as much as practical, as trench backfill. The excavation shall be properly designed in order to protect workmen during placement of the intake pipes and screens. The underwater construction method described for the Glenville Pump Station will also be used at this site. All scarred areas will be seeded and mulched. The existing piling used to support the existing intake pipe and screens will be modified to support the new intake pipes and screens.

See quantity and cost summary sheets in the cost estimate for listing of items included in the pumping station.

Note: There is an existing sediment problem at this site and it will also impact this recommended solution. There is not an adequate depth available for the screens. Dredging and /or diversion of flow to the right bank needs to be investigated. If possible, lowering the centerline elevation of the new screens and intake pipe to an elevation of 20.00 feet is preferred.

C. DuPont Pump Station. The profile of the DuPont Pump Station is shown on Sheet C-205. The DuPont pump station is concrete caisson with the top slab set at an elevation of 75.00 feet. The inside bottom elevation is set at an elevation of 3.00 feet. Motors for the vertical turbine pumps and the traveling screen are mounted on the top slab of the pump station and are exposed to the weather. The intake structure is located about 130 feet from the pump station. The intake structure has a screen to stop debris. The intake pipe from the intake structure to the pump station is a 72" pipe set at a 0% slope with an invert elevation of 8.00. The average daily flow through the DuPont Pump Station is 15 mgd. The peak daily flow is 16 mgd. This pump station will remain operational for all alternatives with no changes.

D. Lower Cape Fear Water and Sewer Authority (LCFWASA) Pump Station. The profile of the LCFWASA Station is shown on Sheet C-206. The LCFWASA pump station is a concrete structure containing the wet well and an operations floor. Pumps, motors and controls are located inside the structure. The inside bottom of the wet well is set an elevation of -2.00 feet. The intake structure is a set of intake screens located 1100 feet from the pump station in the river channel. The centerline of the intake pipe entering the wet well is set at an elevation of 3.75 feet. The average daily flow through the LCFWASA Pump Station is 16.8 million gallons per day mgd. The peak daily flow for the LCFWASA pump stations is 45.0 mgd. When the existing pump station is operating at peak capacity there is a head loss of over 4 feet from the river to the wet well. This loss varies dependant on how clean the intake screens are.

The low water levels established for Alternatives 1 and 3 are set at elevation 0.00 and 2.50 respectively. Both of these alternatives will require relocation of the intake screens and a complete replacement of the pump station.

The minimum water surface elevation for Alternative 2 is 10.0 feet, which will not require any revisions to the intake structure or pump station.

The recommended changes to meet the flow requirements for Alternatives 1 and 3 are shown on Sheet C-207 and are as follow:

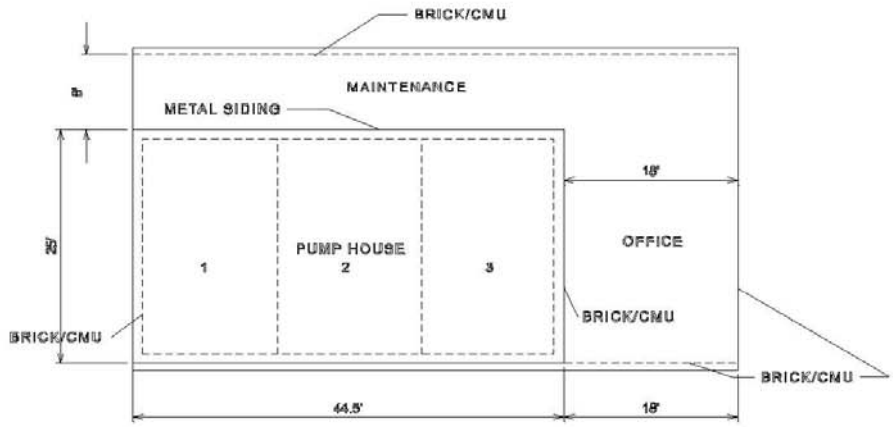
- Install 5 new 9 mgd intake screens with an air cleaning system.
- Construct a new pump station similar to the existing pump station but with the floor of the wet well set at elevation -12.00.
- Install 2 new 1400 HP electric motor driven pumps that can deliver 18,000 gpm at 235 feet TDH at 880 rpm with variable frequency drives. These pumps were selected based on sizes and capacities of the existing

electric pumps.

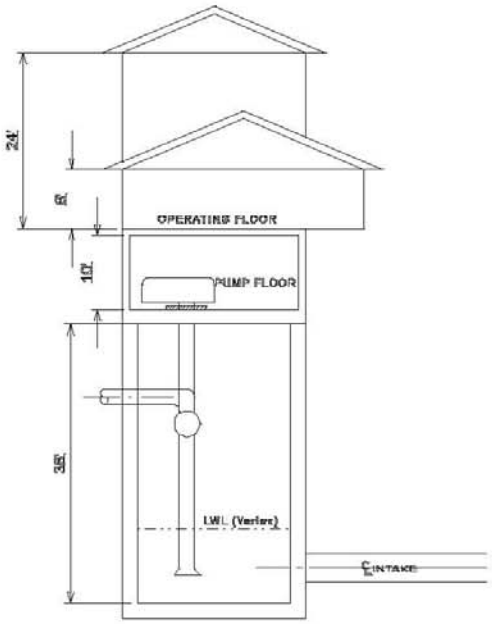
- Install 1 new 1400 HP diesel motor driven pump that can deliver 18,000 gpm at 235 feet TDH at 880 rpm with a right angle drive. This pump was selected based on size and capacity of the existing diesel pump.
- Install a new 60" intake pipe from the relocated screens to the new pump station set at a centerline elevation of -8.00.

The old pump station will remain in operation while the new pump station is being constructed. There may be a minimal downtime required for connections to be made to the existing pressure main from the existing pump station.

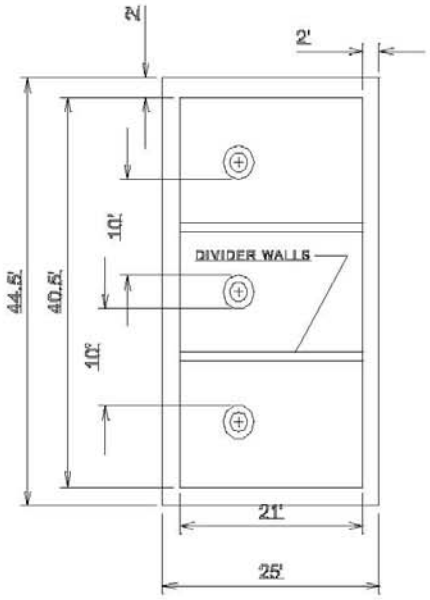
The new pumping facility would have the same operating and administrative floor elevations, and would be configured for an equivalent number of pumps and with the same ancillary facilities as are included in the existing plant. As noted above, all pumps, motors and controls would be located inside the structure. In addition it was assumed that a nominally sized overhead crane would be included in the high bay area to facilitate maintenance of the pumping equipment. For estimating purposes the footprint of the replacement facility was derived using a pump spacing of 10 feet and a wall clearance of 5 feet for all pumps. The operating, administrative, and storage spaces were estimated for square footage and required vertical clearances based on pictures and other data gathered from the existing plant. It was assumed that these areas would be constructed in much the same configuration and with much the same architectural treatment as was used for the existing plant.



FLOOR PLAN



PROFILE VIEW



SUMP FLOOR PLAN

The modifications to this pump station will require significant excavations due to the distance between the pump station and intake structure in the river along with the invert elevation of the intake pipe. The underwater construction method described for the Glenville Pump Station will also be used at this site. Excavated materials shall be stockpiled and re-used, as much as practical, as trench backfill. The excavation shall be properly designed in order to protect workmen during placement of the intake pipe. All scarred areas will be seeded and mulched.

See quantity and cost summary sheets in the cost estimate for listing of items included in the pumping station.

E. City of Wilmington Pump Station. The profile of the City of Wilmington Pump Station is shown on Sheet C-208. The pump station is a concrete and brick structure containing the wet well, pump room, and loading dock. Pumps, motors and controls are located inside the structure. The inside bottom of the wet well is set an elevation of 4.00 feet. The intake structure is a canal from the river to a screened inlet into the wet well. There is a sluice gate on the inlet wall of the wet well to control the flow of water. The average daily flow through the City of Wilmington Pump Station is 9.5 million gallons per day (mgd). The peak daily flow for the pump station is 25.0 mgd.

The low water levels established for Alternatives 1 and 3 are set at elevation 0.00 and 2.50 respectively. No water will enter the wet well at either of these elevations. Alternatives 1 and 3 will require replacement of the existing pump station. New intake screens with an air cleaning system will new intake screens and a complete replacement of the pump station.

The minimum water surface elevation for Alternative 2 is 10.0 feet, which will not require any revisions to the intake structure or pump station.

The recommended changes to meet the flow requirements for alternatives

1 and 3 are shown on Sheet C-209 and are as follow:

- Install 3 new 8.33 mgd intake screens with an air cleaning system.
- Construct a new pump station similar to the LCFWASA pump station but with the floor of the wet well set at elevation -12.00.
- Install a new 48" intake pipe from the relocated screens to the new pump station set at an invert elevation of -10.00.
- Install 2 new 600 HP electric motor driven pumps that can deliver 8400 gpm at 230 feet TDH at 880 rpm with variable frequency drives. These pumps were selected based on sizes and capacities of the existing electric pumps.
- Install 1 new 1200 HP diesel motor driven pump that can deliver 10,500 gpm at 335 feet TDH at 880 rpm with a right angle drive. This pump was selected based on size and capacity of the existing diesel pump.

The old pump station will remain in operation while the new pump station is being constructed. There may be a minimal downtime required for connections to be made to the existing pressure main from the existing pump station.

The new pumping facility would have the same operating and administrative floor elevations, and would be configured for an equivalent number of pumps as are included in the existing plant. As noted above pumps, motors and controls would be located inside the structure. A nominally sized overhead crane would be included in the high bay area to facilitate maintenance of the pumping equipment. For estimating purposes the ancillary facilities for this plant were assumed to be the same as for the LCFWASA Pump Station, noted above. The footprint of the replacement facility was derived using a pump spacing of 10 feet and a wall clearance of 5 feet for all pumps. The operating, administrative, and storage spaces were estimated for square footage and required vertical clearances based on pictures and other data gathered from the existing LCFWASA Station. It was assumed that these areas would be constructed in much the same configuration and with much the same architectural treatment as was used for that plant. See quantity and cost summary sheets for listing of items included in the pump station.

The modifications to this pump station will require significant excavations due to the distance between the pump station and intake structure in the river along with the invert elevation of the intake pipe. Excavated materials shall be stockpiled and re-used, as much as practical, as trench backfill. The underwater construction method described for the Glenville Pump Station will also be used at this site. Excavated materials shall be stockpiled and re-used, as much as practical, as trench backfill. The excavation shall be properly designed in order to protect workmen during placement of the intake pipe. All scarred areas will be seeded and mulched.

See quantity and cost summary sheets in the cost estimate for listing of items included in the pumping station.

3. Summary

This study evaluated the impacts to the existing pump stations on the Cape Fear River for three alternatives. Alternative 1 is for the water surface elevation with the locks and dams removed. Alternative 2 is for the water surface elevation with the locks and dams lowered to an elevation of no impact to the existing intakes. Alternative 3 is for the water surface elevation with the locks and dams lowered so that no rock ramps are required. Results are as follow:

A. Glenville Pump Station: For Alternatives 1 and 3 the existing intake structure will have to be replaced with new screens and intake pipe.

B. Hoffer Pump Station: Existing screens will have to be replaced with smaller diameter screens set at a centerline elevation of 21.00 feet with the intake pipes lowered as indicated on the drawing for both

Alternatives 1 and 3.

C. DuPont Pump Station: This station will remain functional with no changes for alternatives 1 and 3.

D. LCFWASA Pump Station: Alternatives 1 and 3 will require a complete replacement of the pump station, intake pipe, and screens.

E. City of Wilmington Pump Station: Alternatives 1 and 3 will require a complete replacement of the pump station with new intake pipe and new screens.

SUMMARY OF ELEVATIONS ON CAPE FEAR RIVER **7 Dec, 2006**

All elevations are NGVD 1929 feet

Lock and Dam #1
(@ river station 353000)

Dam Crest	11.1

City of Wilmington Intake
(@ river station 353440)

4ft X 4ft Intake Sill	9.14
4ft X 4ft Intake Invert	5.14
Wetwell invert	4.06
Max Capacity	25 MGD
Avg Capacity (1997)	9.47 MGD

Lower Cape Fear Water & Sewer Authority (LCFWASA)
(@ river station 353760)

Centerline of intake screen	4.5
Centerline of 48" intake Pipe in river	-8.0
Wetwell invert	-2.0
Centerline intake pipe @ wetwell	3.75
Max Capacity	45.0 MGD
Avg Capacity (1997)	16.81 MGD

Lock & Dam #2
(@ river station 523670)

Dam Crest	20.1

Lock & Dam #3
(@ river station 645300)

Dam Crest	29.1

DuPont Intake
(@ river station 650700)

Centerline of 61" ID intake pipe	11.0
Finish floor of pump area	75.0
Wet well invert	3.0
Bottom of pump intake	5.5
Max Capacity	16.0 MGD
Avg Capacity (2005)	15.0 MGD

Fayetteville Intakes
(@ river station 764605)

Glenville Pumping Station

Top of Intake Screen	34.5
Invert of intake screen	20.5
Invert of 48" intake pipe	22.00
Invert of intake structure	19.5
Invert of wet well	15.0
Max Capacity	7 MGD
Avg Capacity (1997)	3 MGD

Hoffer Pumping Station

Centerline of 36" intake screen	25.0
Centerline of 60" pipe	19.0
Invert of wet well	15.0
Avg Capacity	17 MGD
Max Capacity	30 MGD
Minimum water level for pumps	21.0
Normal minimum water level For pumps	23.0

WILMINGTON HARBOR GRR - LOCKS AND DAMS STUDY

Water Surface Elevations @ 300 CFS

December 8, 2006

LOCATION DESCRIPTION	River Station Feet from Mouth	Existing conditions FEET NGVD Dam Elevation	Existing conditions FEET NGVD Water Surface Elevation	Alternative 1 Locks and Dams Removed FEET NGVD Dam Elevation	Alternative 1 Locks and Dams Removed FEET NGVD Water Surface Elevation	Alternative 2 L&D's lowered to elevation of minimum impact on intakes FEET NGVD Dam Elevation	Alternative 2 L&D's lowered to elevation of minimum impact on intakes FEET NGVD Water Surface Elevation	Alternative 3 Locks and Dams lowered so that no rock ramps are required FEET NGVD Dam Elevation	Alternative 3 Locks and Dams lowered so that no rock ramps are required FEET NGVD Water Surface Elevation
Lock & Dam 1	353000	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50
City of Wilmington Intake	353440	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50
LCFWASA Intake	353760	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50
Lock & Dam 2	523670	20.10	20.50	2*	3.20	2*	8.60	14.00	14.60
Proposed Smithfield Foods Intake	619290	20.10	20.70	2*	13.20	2*	12.90	14.00	14.90
Lock & Dam 3	645300	29.10	29.40	9.9*	13.30	27.20	27.60	23.00	23.60
Dupont Intake	650700	29.10	29.60	9.9*	14.00	27.20	27.80	23.00	23.60
Fayetteville Glenville Intake	764605	29.10	29.70	9.9*	23.00	27.20	28.00	23.00	24.10
Fayetteville Hoffer Intake	764605	29.10	29.70	9.9*	23.00	27.20	28.00	23.00	24.10

NOTE: * indicates that the dam has been removed and that is natural bottom at the site

COST ESTIMATE

PRELIMINARY PROGRAMMING & PLANNING COST ESTIMATE

PROJECT: Cape Fear River Intake Projects
LOCATION: North Carolina

WORK ITEM: PROJECT SUMMARY

ITEM NO. SUMMARY
SHEET NO. 1 OF 6
PREPARED: Addison DeBoi
CHECKED: George Brown
DATE: 26-Mar-07
BASIS OF ESTIMATE: info furnished per Project Delivery Team
FILE NAME: cfripa

DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT
SUMMARY				
GLENVILLE	1	Job	xxx	500,000
HOFFER	1	Job	xxx	$1,100,000
DUPONT	1	Job	xxx	$0
LCFWASA	1	Job	xxx	8,500,000
CITY OF WILMINGTON	1	Job	xxx	$6,059,900
Total Project Cost Summary (FY-08)				**$16,159,900**

Notes:
Price Level Apr 07
Unit Cost Based on Historical Data, Recent Pricing, & Estimator's Judgment
Estimate Excludes:
- Real Estate
- Environmental
- Recreation
- Mitigation
- Removal or Relocations
- HTRW
- All dredging

PRELIMINARY PROGRAMMING & PLANNING COST ESTIMATE

PROJECT: Cape Fear River Intake Projects
LOCATION: North Carolina
WORK ITEM: Glenville Intake - City of Fayetteville

ITEM NO. 1 DATE: 26-Mar-07
SHEET NO. 2 OF 6
PREPARED: Addison DeBoi CHECKED: George Brown
BASIS OF ESTIMATE: info furnished per Project Delivery Team
FILE NAME: cfripa

DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT
GLENVILLE				
Mobilization, Prepartory Work, Demobilization	1	ls	allow	$15,000
Strip 6 inches off top of area to be excavated	2420	sy	2	4,840
Common Excavation	10880	cy	9	100,096
Dewatering (manned 24hr/day)	1	ls	1000	1,000
Removal of existing 42" inside 72" casing pipe	125	lf	77	9,625
Dispose of pipe debris	210	cy	35	7,350
Saw cut wall around and below existing penetration	39	lf	50	1,950
Break up nand remove concrete inside cut area	189	cf	29	5,481
Remove old pipe thimble	1	ea	400	400
Install new pipe thimble	1	ea	1200	1,200
Install reinforcement	450	lb	3	1,125
Install forms around cut wall area	150	sf	17	2,550
Cast in place structural concrete	4	cy	133	530
Install new ductile iron, class 150 48" pipe	140	lf	244	34,160
Install new pylong/concrete support for pipe/screens	3	ea	7700	23,100
Install new 3.5 mgd intake screens w/existing air burst system	2	ea	7800	15,600
Hand compacted backfill	1000	cy	19	18,840
Equipment compacted backfill	9870	cy	5	48,758
Silt Fenching	1270	lf	3	3,810
Seeding & Mulching	3630	sy	1	3,194
Misc. Site Items & Restoration/silt fencing	1	job	allow	8,000
				306,609
		Contingency	35.0%	107,313
				413,922
		Acct 30 Planning and design	6.0%	24,835
				438,758
		Acct 31 Construction. Management	8.0%	35,101
				473,858
		Escalation, FY-08	4.0%	18,954
				$492,813
Total Project Cost Summary (FY-08)				**$500,000**

PRELIMINARY PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Cape Fear River Intake Projects	ITEM NO. 2		DATE:	26-Mar-07
LOCATION:	North Carolina	SHEET NO.	3	OF	6
		PREPARED:	Addison DeBoi	CHECKED:	George Brown
WORK ITEM:	Hoffer Intake - City of Fayetteville	BASIS OF ESTIMATE:	info furnished per Project Delivery Team		
		FILE NAME:	cfripa		

DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT
HOFFER				
Mobilization, Prepartory Work, Demobilization	1	ls	allow	$15,000
Stripping (6 inch)	1452	sy	2	2,904
Common Excavation	7333	cy	9	67,464
Dewatering (Sump Pumping at Pump Station)	1	ls	1000	1,000
Riprap Removal (for reuse)	500	cy	39	19,250
Removal of existing PCCP 60" pipe	280	lf	69	19,320
Dispose of pipe/concretedebris (bobby, took this qty off your input..seems	35	cy	27	945
Concrete Slope Protection Removal	1620	cy	27	42,930
Riprap (reuse of existing)	500	cy	90	45,000
Riprap (new)	350	cy	120	42,000
Bedding Material	420	sy	90	37,800
Bedding Filter Cloth	1400	sy	2	2,800
Modify Steel Pile Support Structure for pipe and 4 screens	48	ea	3800	182,400
Install 2 new Screen Supports for 2 additonal screens	2	ea	5200	10,400
Install new 5 mgd intake screens w/existing air burst system	6	ea	8000	48,000
Install new 60" ductile iron class c pipe	280	lf	408	114,240
Hand compacted backfill	1150	cy	19	21,666
Equipment compacted backfill	6000	cy	5	29,640
Silt Fenching	1270	lf	3	3,493
Seeding & Mulching	2178	sy	1	1,917
Misc. Site Items & Restoration/silt fencing	1	job	allow	8,000
				716,168
		Contingency	35.0%	250,659
				966,826
		Acct 30 Planning and design	6.0%	58,010
				1,024,836
		Acct 31 Construction. Management	8.0%	81,987
				1,106,823
		Escalation, FY-08	4.0%	38,673
				$1,145,496
Total Project Cost Summary (FY-08)				**$1,100,000**

PRELIMINARY PROGRAMMING & PLANNING COST ESTIMATE

PROJECT:	Cape Fear River Intake Projects	ITEM NO. 3	DATE: 26-Mar-07
LOCATION:	North Carolina	SHEET NO. 4	OF 6
		PREPARED: Addison DeBoi	CHECKED: George Brown
WORK ITEM:	Dupont Intake	BASIS OF ESTIMATE: info furnished per Project Delivery Team	
		FILE NAME: cfripa	

DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT
DUPONT				
NO WORK ON THIS PROJECT				
Total Project Cost Summary (FY-08)				**$0**

PRELIMINARY PROGRAMMING & PLANNING COST ESTIMATE

PROJECT: Cape Fear River Intake Projects
LOCATION: North Carolina

WORK ITEM: LCFWASA Intake

ITEM NO. 4 DATE: 26-Mar-07
SHEET NO. 5 OF 6
PREPARED: Addison DeBoi CHECKED: George Brown
BASIS OF ESTIMATE: info furnished per Project Delivery Team
FILE NAME: cfripa

DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT
LCFWASA				
Mobilization, Prepartory Work, Demobilization	1	ls	allow	$20,000
Strip 6 inches off top of area to be excavated	6776	sy	2	13,552
Common Excavation	32700	cy	9	300,840
Dewatering (Sump Pumping at Pump Station)	1	ls	1000	1,000
5 Ton Brige Crane Installed in Building.	1	ea	54800	54,800
Build New lower pump house (Concrete 1:24 ratio to forms)	618	cy	500	309,000
Build New structure CMU/Brick/sheetmetal	2063	sf	175	361,025
Install new 60" ductile Iron class 150 pipe	1160	lf	408	473,280
Build new pipe/screen supports	6	cy	6000	36,000
Install new 9 mgd intake screens/airburst	5	ea	15000	75,000
Hand compacted backfill	5585	cy	19	106,115
Equipment compacted backfill	26985	cy	5	134,925
New 1400 HP pumps	2	ea	1360000	2,720,000
New 1400 HP Diesel pump	1	ea	580000	580,000
Move Diesel tanks install supports etc.	2	ea	17000	34,000
NEW MCC	2	ea	20000	40,000
Silt Fenching	2500	lf	3	6,875
Seeding & Mulching	10164	sy	1	8,944
Misc. Site Items & Restoration	1	job	allow	20,000
				5,295,356
		Contingency	35.0%	1,853,375
				7,148,731
		Acct 30 Planning and design	6.0%	428,924
				7,577,655
		Acct 31 Construction. Management	8.0%	606,212
				8,183,867
		Escalation, FY-08	4.0%	285,949
				8,469,817
		Total Project Cost Summary (FY-08)		**$ 8,500,000.00**

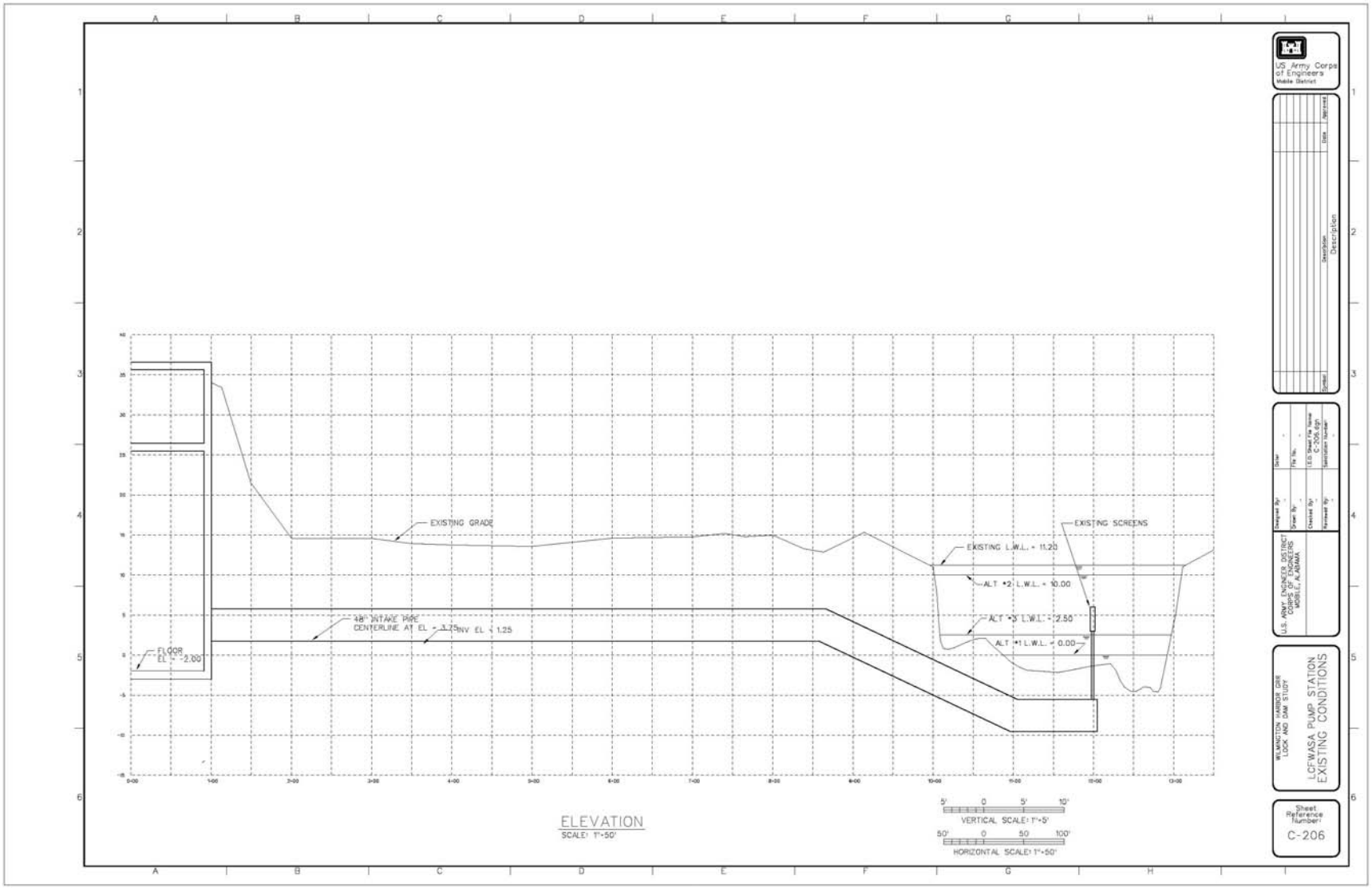
PRELIMINARY PROGRAMMING & PLANNING COST ESTIMATE

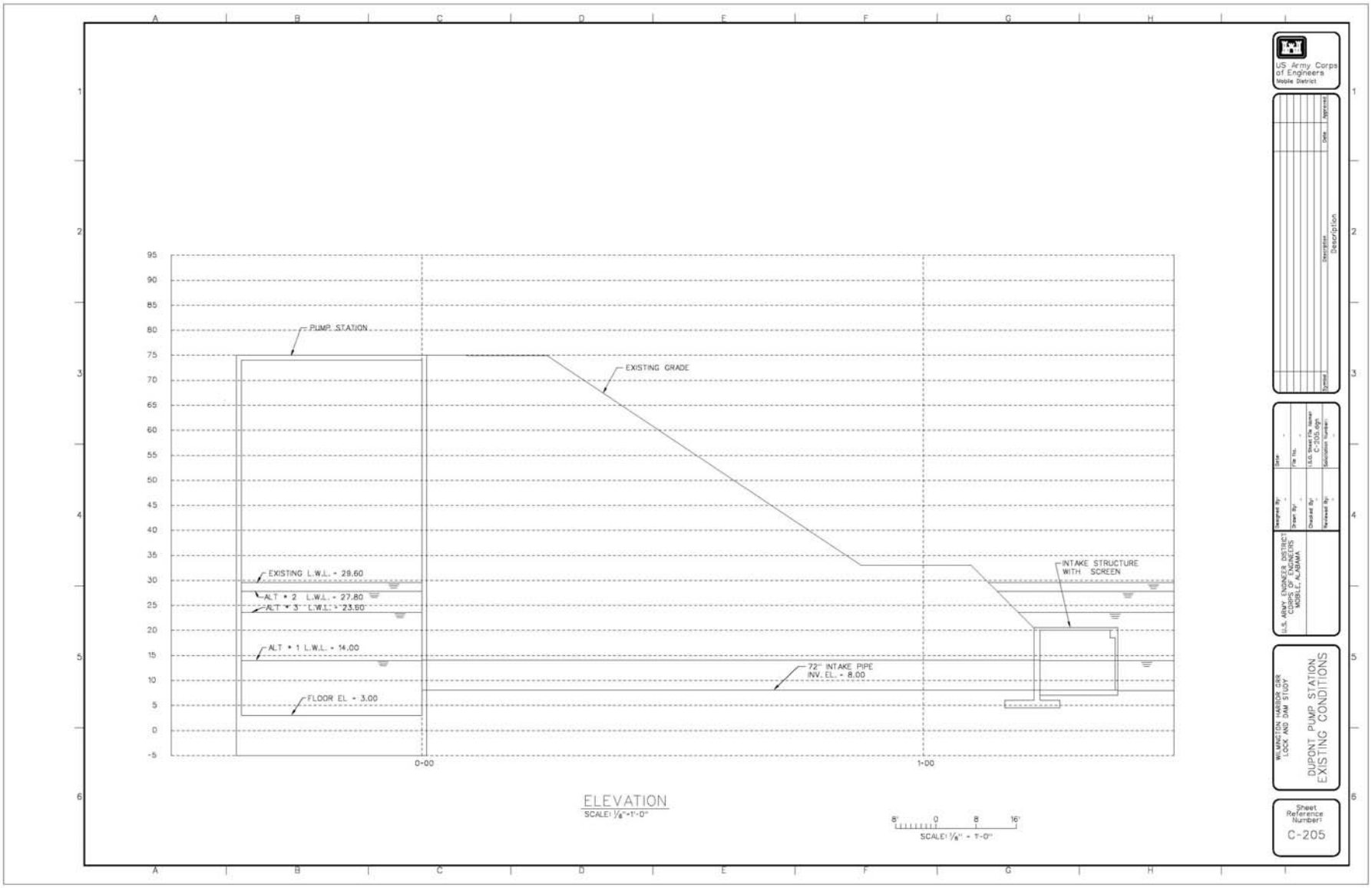
PROJECT: Cape Fear River Intake Projects
LOCATION: North Carolina
WORK ITEM: City of Wilmington Intake

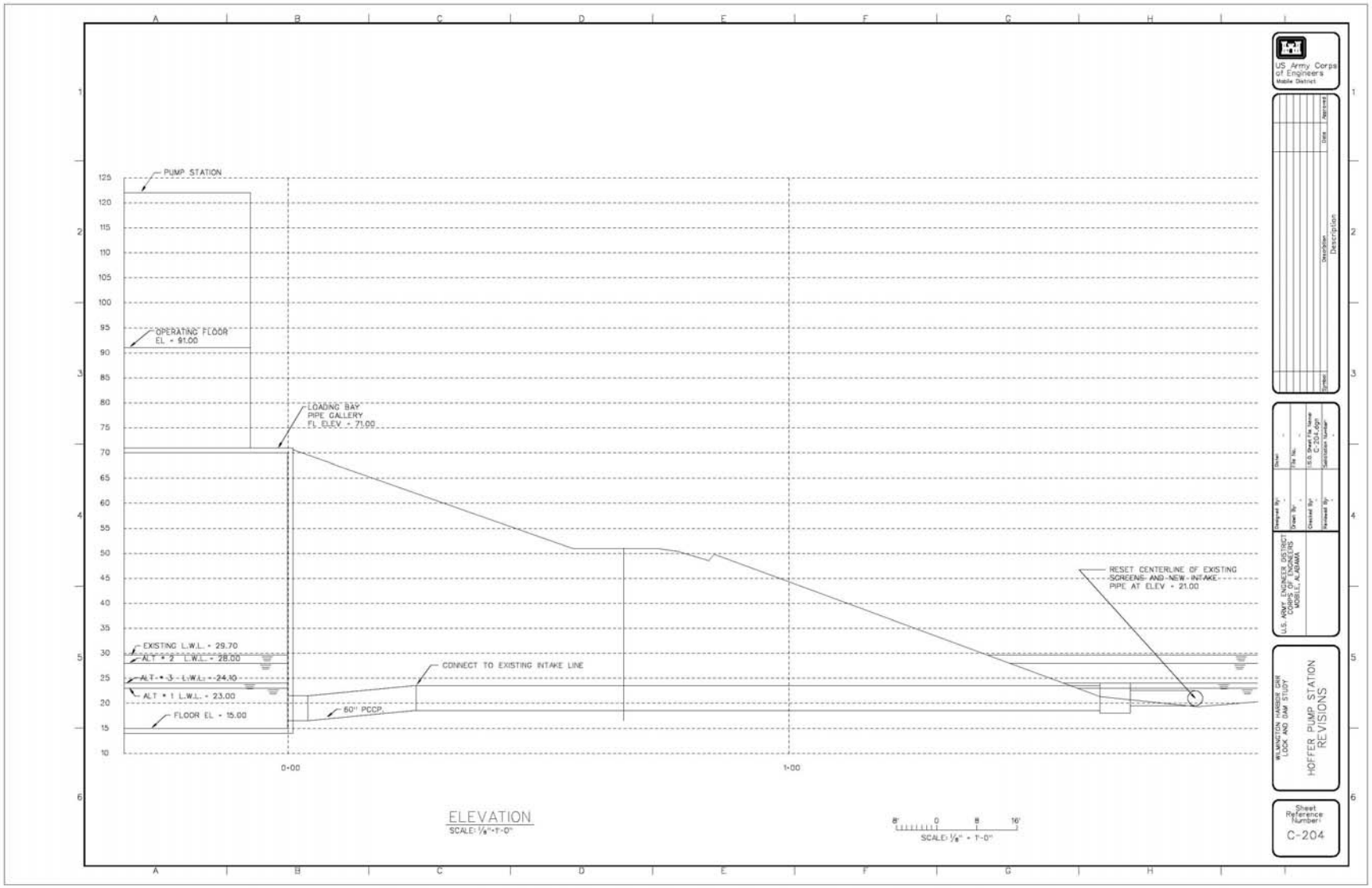
ITEM NO. 5 DATE: 26-Mar-07
SHEET NO. 6 OF 6
PREPARED: Addison DeBoi CHECKED: George Brown
BASIS OF ESTIMATE: info furnished per Project Delivery Team
FILE NAME: cfripa

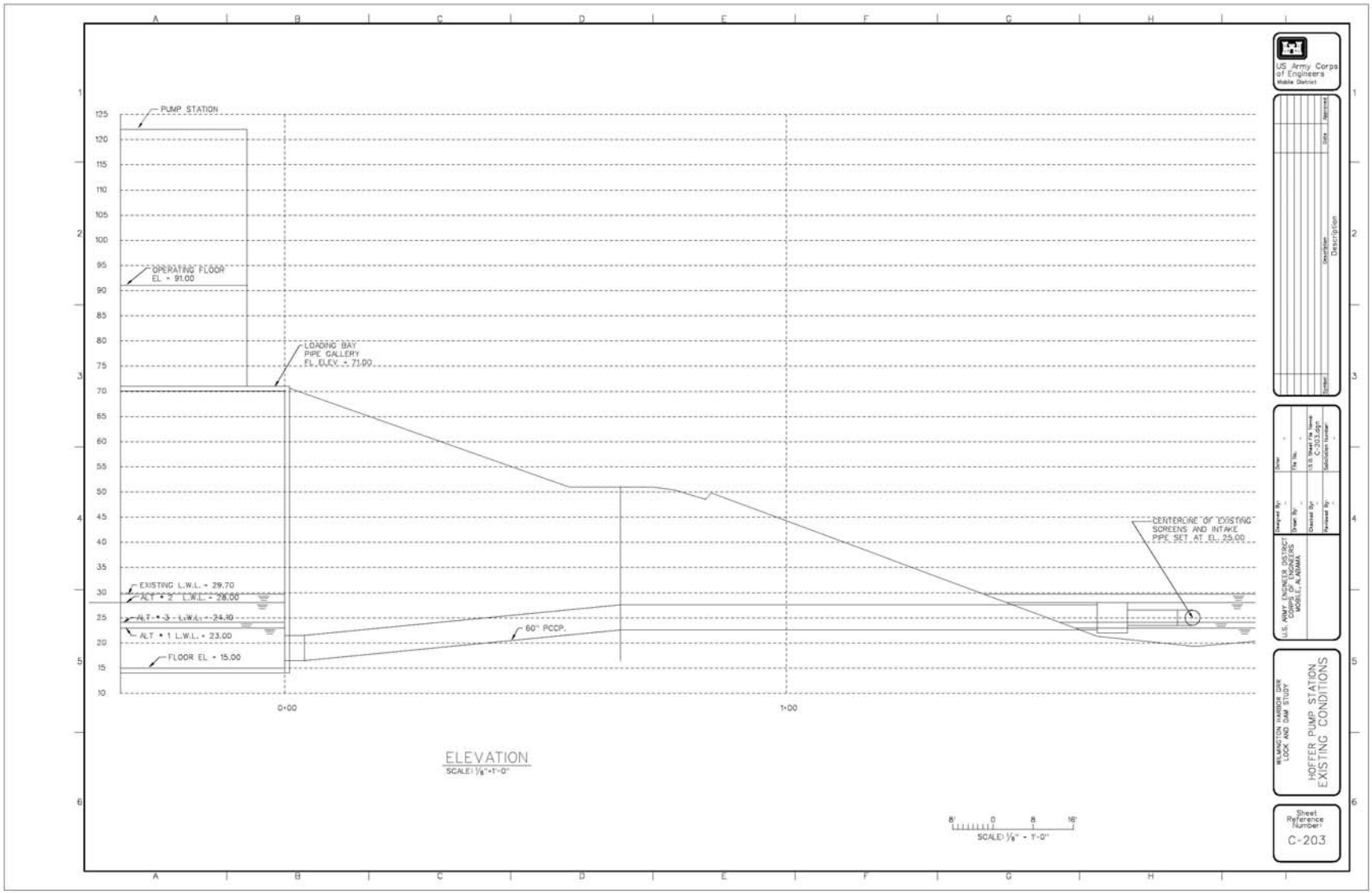
DESCRIPTION	ESTIMATED QUANTITY	UNIT	UNIT PRICE	ESTIMATED AMOUNT
CITY OF WILMINGTON				
Mobilization, Prepartory Work, Demobilization	1	ls	allow	$20,000
Strip 6 inches off top of area to be excavated	6776	sy	2	13,552
Common Excavation	32700	cy	9	300,840
Dewatering (Sump Pumping at Pump Station)	1	ls	1000	1,000
Build New lower pump house (Concrete 1:24 ratio to forms)	618	cy	500	309,000
Build New structure CMU/Brick/sheetmetal	2063	sf	175	361,025
5 Ton Brige Crane Installed in Building.	1	ea	54800	54,800
Install new 48" ductile class 150 pipe	1160	lf	248	287,680
Build Intake screen/pipe support structure	4	ea	6000	24,000
Install new intake screens w/NEW Airburst system	3	ea	15000	45,000
Hand compacted backfill	5585	cy	19	105,221
Equipment compacted backfill	26985	cy	5	133,306
New 600 HP pumps	2	ea	750000	1,500,000
New 1200 HP Diesel Pump	1	ea	505000	505,000
Move Diesel tanks install supports etc.	2	ea	17000	34,000
NEW MCC	2	ea	20000	40,000
Silt Fenching	2500	lf	3	6,875
Seeding & Mulching	10164	sy	1	8,944
Misc. Site Items & Restoration	1	job	allow	20,000
				3,770,244
		Contingency	35.0%	1,319,585
				5,089,829
		Acct 30 Planning and design	6.0%	305,390
				5,395,219
		Acct 31 Construction. Management	8.0%	431,617
				5,826,836
		Escalation, FY-08	4.0%	233,073
				6,059,910
		Total Project Cost Summary (FY-08)		**$6,059,900**

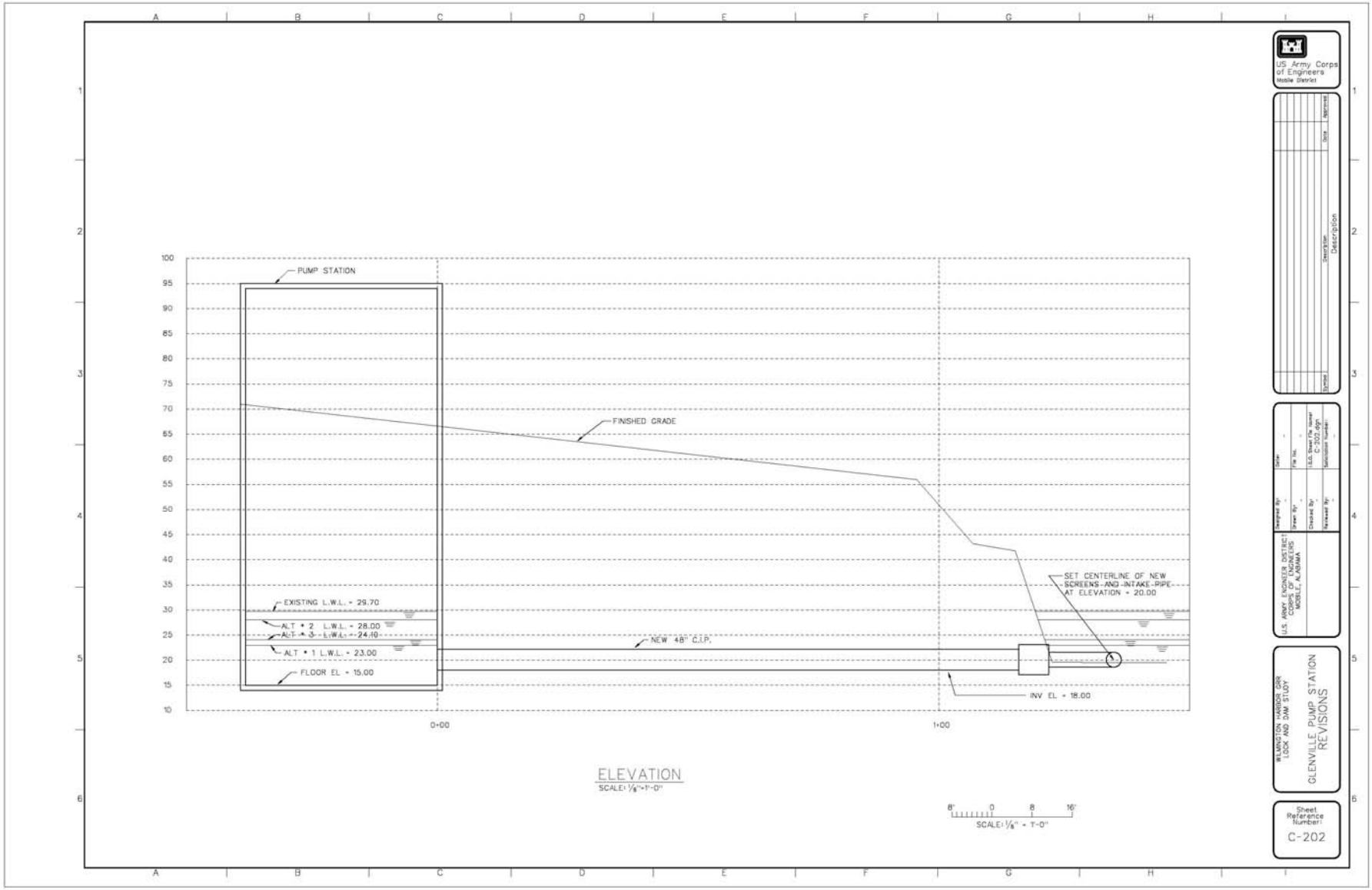
DRAWINGS

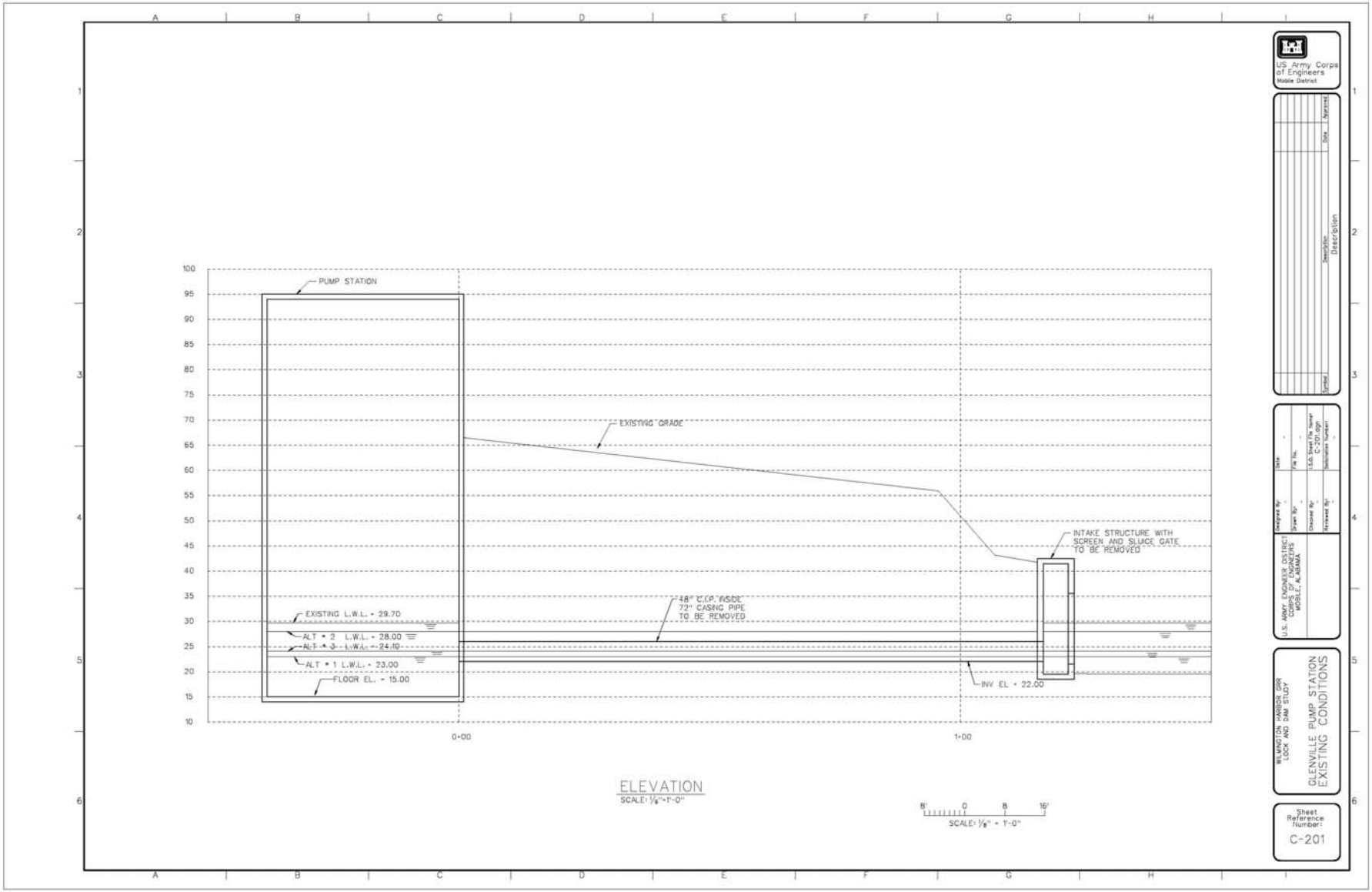

US Army Corps of Engineers
Mobile District
EXISTING GRADE
EXISTING SCREENS
EXISTING L.W.L. = 11.20
ALT #2 L.W.L. = 10.00
ALT #3 L.W.L. = 2.50
ALT #1 L.W.L. = 0.00
48" INTAKE PIPE
CENTERLINE AT EL = 3.25
INV EL = 1.25
FLOOR
EL = -2.00
ELEVATION
SCALE: 1"=50'
VERTICAL SCALE: 1"=5'
HORIZONTAL SCALE: 1"=50'
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MOBILE, ALABAMA
WASHINGTON HARBOR, DPR
LOCK AND DAM STUDY
LCFWASA PUMP STATION
EXISTING CONDITIONS
Sheet Reference Number
C-206

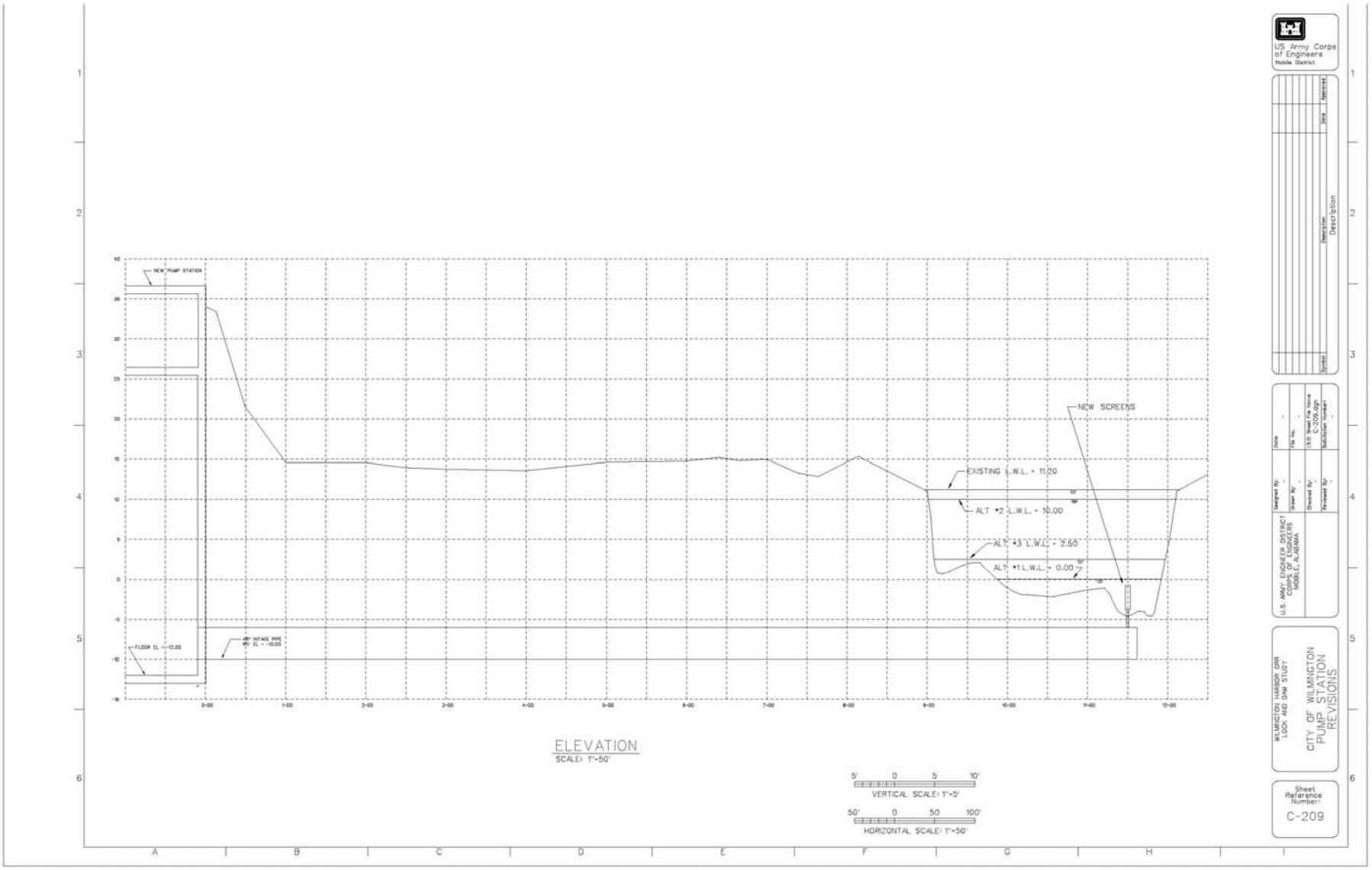

US Army Corps of Engineers
Mobile District
Symbol
Description
Date
Approved
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MOBILE, ALABAMA
Designed By
Drawn By
Checked By
Reviewed By
Date
File No.
U.S.G. Sheet File name
C-205.dgn
Solicitation Number
WILMINGTON HARBOR GRR
LOCK AND DAM STUDY
DUPONT PUMP STATION
EXISTING CONDITIONS
Sheet Reference Number:
C-205
PUMP STATION
EXISTING GRADE
EXISTING L.W.L. = 29.60
ALT # 2 L.W.L. = 27.80
ALT # 3 L.W.L. = 23.80
ALT # 1 L.W.L. = 14.00
FLOOR EL = 3.00
72" INTAKE PIPE
INV. EL. = 8.00
INTAKE STRUCTURE
WITH SCREEN
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0
-5
0+00
1+00
ELEVATION
SCALE: 1/8" = 1'-0"
8'
0
8
16'
SCALE: 1/8" = 1'-0"

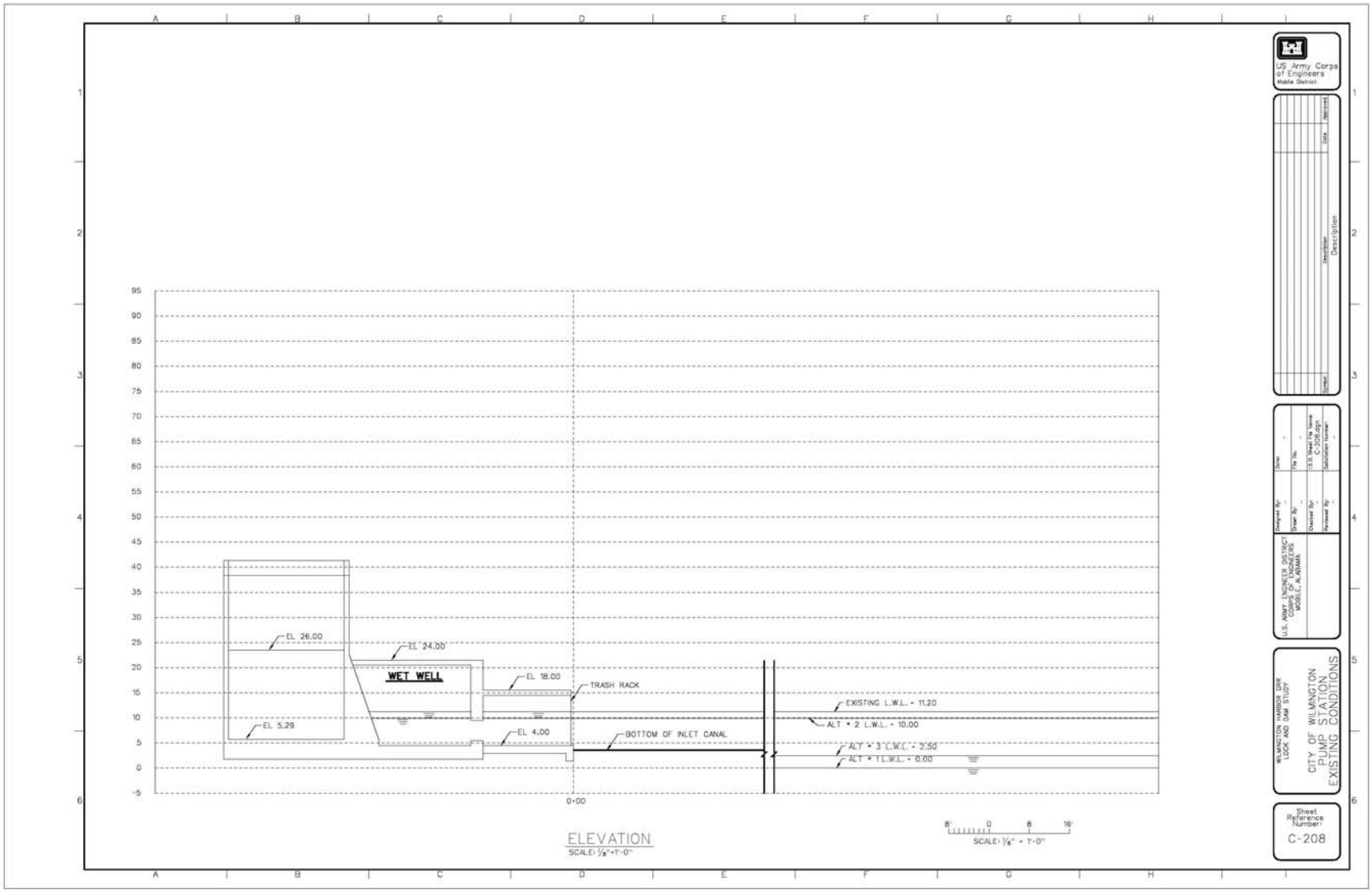

PUMP STATION
OPERATING FLOOR
EL = 91.00
LOADING BAY
PIPE GALLERY
FL ELEV = 71.00
EXISTING L.W.L. = 29.70
ALT = 2 L.W.L. = 28.00
ALT = -3 - L.W.L. = -24.10
ALT = 1 L.W.L. = 23.00
FLOOR EL = 15.00
CONNECT TO EXISTING INTAKE LINE
60" PCCP
RESET CENTERLINE OF EXISTING
SCREENS AND NEW INTAKE
PIPE AT ELEV = 21.00
0+00
1+00
ELEVATION
SCALE: 1/8" = 1'-0"
SCALE: 1/8" = 1'-0"
US Army Corps of Engineers
Mobile District
HOFFER PUMP STATION
REVISIONS
C-204


US Army Corps of Engineers
Mobile District
PUMP STATION
OPERATING FLOOR
EL. = 91.00
LOADING BAY
PIPE GALLERY
EL. ELEV. = 71.00
EXISTING L.W.L. = 29.70
ALT. = 2 L.W.L. = 28.00
ALT. = 3 L.W.L. = 24.10
ALT. = 1 L.W.L. = 23.00
FLOOR EL. = 15.00
60" PCCP.
CENTERLINE OF EXISTING
SCREENS AND INTAKE
PIPE SET AT EL. 25.00
0+00
1+00
ELEVATION
SCALE: 1/8"=1'-0"
SCALE: 1/8" = 1'-0"
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MOBILE, ALABAMA
WILMINGTON HARBOR DPR
LOCK AND DAM STUDY
HOFFER PUMP STATION
EXISTING CONDITIONS
Sheet Reference Number
C-203


US Army Corps of Engineers
Mobile District
PUMP STATION
FINISHED GRADE
EXISTING L.W.L. = 29.70
ALT # 2 L.W.L. = 28.00
ALT # 3 L.W.L. = 24.10
ALT # 1 L.W.L. = 23.00
FLOOR EL = 15.00
NEW 48" C.I.P.
SET CENTERLINE OF NEW SCREENS AND INTAKE PIPE AT ELEVATION = 20.00
INV EL = 18.00
0+00
1+00
ELEVATION
SCALE: 1/8" = 1'-0"
SCALE: 1/8" = 1'-0"
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS MOBILE, ALABAMA
WILMINGTON HARBOR GRR LOCK AND DAM STUDY
GLENVILLE PUMP STATION REVISIONS
Sheet Reference Number:
C-202


PUMP STATION
EXISTING GRADE
INTAKE STRUCTURE WITH SCREEN AND SLUICE GATE TO BE REMOVED
48" C.C.P. INSIDE 72" CASING PIPE TO BE REMOVED
EXISTING L.W.L. = 29.70
ALT = 2 L.W.L. = 28.00
ALT = 3 L.W.L. = 24.10
ALT = 1 L.W.L. = 23.00
FLOOR EL. = 15.00
INV EL = 22.00
100
95
90
85
80
75
70
65
60
55
50
45
40
35
30
25
20
15
10
0+00
1+00
ELEVATION
SCALE: 1/8"=1'-0"
8'
0
8
16'
SCALE: 1/8" = 1'-0"
US Army Corps of Engineers
Mobile District
Description
Date
Approved
Designed By:
Drawn By:
Checked By:
Reviewed By:
Date
File No.
I.S.O. Sheet File Name
C-201.dgn
Sheet Reference Number:
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MOBILE, ALABAMA
WILMINGTON HARBOR DPR
LOCK AND DAM STUDY
GLENVILLE PUMP STATION
EXISTING CONDITIONS
Sheet Reference Number:
C-201


NEW PUMP STATION
NEW SCREENS
EXISTING L.W.L. = 11.20
ALT #2 L.W.L. = 10.00
ALT #3 L.W.L. = 2.50
ALT #1 L.W.L. = 0.00
FLOOR EL. = -12.00
48" INTAKE PIPE INV. EL. = -10.00
ELEVATION
SCALE: 1"=50'
VERTICAL SCALE: 1"=5'
HORIZONTAL SCALE: 1"=50'
US Army Corps of Engineers
Mobile District
Description
Symbol
Date
Approved
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MOBILE, ALABAMA
Designed By:
Drawn By:
Checked By:
Reviewed By:
Date
File No.
Solicitation Number
C-209.dgn
WILMINGTON HARBOR GRR
LOCK AND DAM STUDY
CITY OF WILMINGTON
PUMP STATION
REVISIONS
Sheet Reference Number:
C-209


US Army Corps of Engineers
Mobile District
Date
Description
Designed By:
Drawn By:
Checked By:
Reviewed By:
Solicitation Number:
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
MOBILE, ALABAMA
WILMINGTON HARBOR DPR
LOCK AND DAM STUDY
CITY OF WILMINGTON
PUMP STATION
EXISTING CONDITIONS
Sheet Reference Number:
C-208
EL 26.00
EL 24.00
WET WELL
EL 18.00
TRASH RACK
EL 5.28
EL 4.00
BOTTOM OF INLET CANAL
EXISTING L.W.L. = 11.20
ALT # 2 L.W.L. = 10.00
ALT # 3 L.W.L. = 2.50
ALT # 1 L.W.L. = 0.00
0+00
ELEVATION
SCALE: 1/8"=1'-0"
SCALE: 1/8" = 1'-0"

END

Appendix 3.8.3

HYDRAULICS

WILMINGTON HARBOR GRR
LOCKS AND DAMS
HYDRAULICS APPENDIX 3.8.3
27 July 2007

The backwater model used for this project is a combination of two existing HEC-2 models. One of the models starts at NC 11 just below Lock and Dam #1 and runs all the way to the confluence of the Haw and Deep Rivers, which come together to form the Cape Fear River. This model was developed in 1974 for the B.E. Jordan study. The other model was developed for the 1983 Cumberland County Flood Insurance Study and used part of the Cape Fear River model with additional cross sections added, but had different stations and reach lengths than the Cape Fear River model, as it was based on better mapping than the Cape Fear River model. These two models were merged and duplicate sections removed. Using the current Cumberland County and Bladen County aerial photographs the sections were located on the maps by measuring upstream and downstream from the road and railroad crossings and the Locks and Dams. The station of each section was established by starting the stationing at the mouth of the Cape Fear River and using the U.S. Quadrangle Sheets, Bladen County, and Cumberland County aerial photographs. The stations above Cumberland County were based on the reach lengths in the back water model. Once the new stations and reach lengths were established in the HEC-2 model, it was converted to a HEC-RAS model. The HEC-RAS model was run using a series of discharges and the computed elevations were checked against the elevations at the gages. The channel and overbank "N" values were adjusted to give the best fit at the gages. To establish the starting conditions for the HEC-RAS steady state runs a model was prepared by extending the steady state model downstream approximately 10 miles using topographic information from the U.S. Geological Survey Quadrangle sheets and bottom survey information and terminating the model just below Lock and Dam #2. The unsteady flow option of HEC-RAS was used along with tide information from the **Cape Fear-Northeast Cape Fear River, North Carolina Numerical Model Study** by Robert T. McAdory of the US Army Corps of Engineers, Engineer Research Development Center's Coastal and Hydraulics Laboratory and several discharges to develop a rating curve for the selection of the starting elevations for the steady state model, used to evaluate the various alternatives, see Plate 1.
All of the model elevations are based on the National Geodetic Vertical Datum 1929, (NGVD '29).

There are four alternatives being considered, (1) remove the locks and dams, (2) drop the top of the dams to the lowest elevation with reduced water supply capacity, (3) lower the tops of the dams so that no rock ramps are needed for fish passage, (4) lock and dams 1 & 3 as is and lock and dam 2 removed. Alternatives 1, 2 & 4 were evaluated using the minimum discharge of 300 cfs. Alternative 3 was evaluated using the spawning discharge of 5,000 cfs to establish the maximum elevation of the tops of the dams. The minimum discharge was also used with alternative 3 to evaluate the impact on the water supply facilities. The water surface profiles for the existing conditions and for the alternatives are located at the end of the appendix. A summary of the water surface elevations for the 300 cfs discharge at selected locations for each of the alternatives is summarized in Table 1.

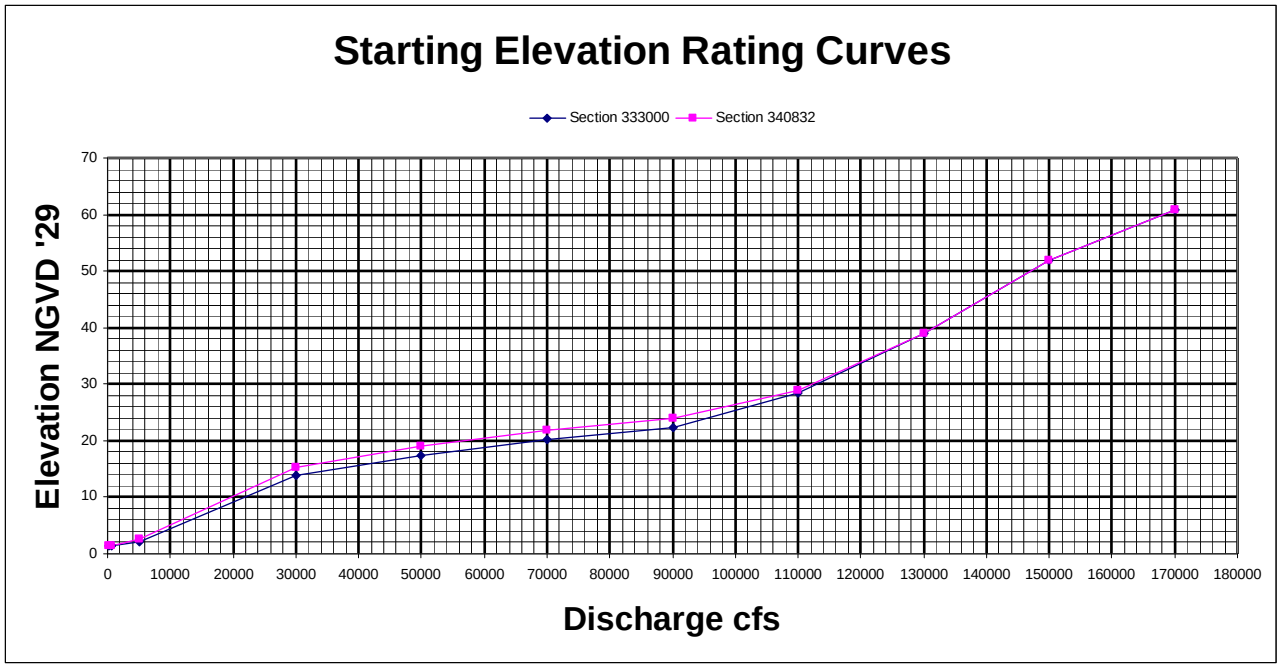


Plate 1
Unsteady Flow Rating Curve

To evaluate the effects of the alternatives on the water supply intakes a discharge that was deemed to be the lowest flow that could be expected to happen was chosen. The historical records for the following gages were examined: Lillington, Fayetteville (discontinued), lock and Dam #3, and Lock and Dam #1. The discharge that was used was the lowest flow that lasted at least three continuous days (300 cfs). Even this flow would provide 195 Mg/d flow, which is greater than the combined requirement of the users between Fayetteville and Lock and Dam #1. The US Army Corps of Engineers Mobile District was contracted to evaluate the effects that each alternative would have on each of the water supply facilities. The final report **Wilmington Harbor GRR Lock and Dam Study Intake Evaluation** prepared by the U.S. Army Corps of Engineers Mobile District, dated March 2007 has the information on each facility, what modifications would be necessary to keep the facility functional for each alternative, and a cost estimate of the modifications for each alternative.

To evaluate the effects that the modifications to the Locks and Dams would have on the fish passage, the historical records of the 4 gages were examined for a representative flow for the fish spawning months, Table 2 shows the spawning discharges that were used, by river station. These discharges were used to determine the highest elevation that the dams could be and still allow fish passage. The elevation of the dams was chosen so that the water surface elevation downstream was at or above the top of the dam and that there was sufficient depth over the weir to keep the velocity below 8 feet per second. The resulting profile is shown in Plates 2/1, 2/2, and 2/3, located at the end of the appendix. The average channel velocity at each section was calculated and plotted against channel station, see Plate 3. Using the average channel velocity at each section travel time from the Buckhorn Dam was calculated in order to determine where the 50 hours required for hatching would be. See Plate 4.

There was concern that the lowering or removal of the dams would have an effect on the flood plains and adjacent wetlands. In consultation with the Wilmington Regulatory office it was decided that if the existing 1-year flood plain would still be flooded by the 2-year or even the 5-year flood after the removal of the locks and dams that the existing hydric soils would not be converted to non-hydric soils. Portions of the adjacent wetlands are being flooded by the flows coming down the slopes toward the river and would continue to be flooded on a frequent enough basis to keep them hydric. Using the 1-year existing conditions flood plain and comparing the 2-year flood plain with the locks and dams removed showed that there was not enough of a change in the flooded area to cause a loss in the wetlands. The removal of the locks and dams will have some impact on the 10-year flood profile, a minor effect on the 50-year flood profile, a negligible impact on the 100-year flood profile and no impact on the 500-year flood profile. The tendency of the changes in the profiles would be for them to drop, which will lessen the flooding of adjacent lands.

WILMINGTON HARBOR GRR - LOCKS AND DAMS STUDY

Water Surface Elevations @ 300 CFS

July 27, 2007

LOCATION DESCRIPTION	River Station Feet from Mouth	**Existing conditions** FEET NGVD Dam Elevation	Water Surface Elevation	**Alternative 1 Locks and Dams Removed** FEET NGVD Dam Elevation	Water Surface Elevation	**Alternative 2 L&D's lowered to elevation of reduced water supply capacity** FEET NGVD Dam Elevation	Water Surface Elevation	**Alternative 3 Locks and Dams lowered so that no rock ramps are required** FEET NGVD Dam Elevation	Water Surface Elevation	**Alternative 4 L&D's 1&3 as is L&D 2 removed** FEET NGVD Dam Elevation	Water Surface Elevation
Lock & Dam 1	353000	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50	10.70	11.20
City of Wilmington Intake	353440	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50	10.70	11.20
LCFWASA Intake	353760	10.70	11.20	-5.5*	0.00	9.50	10.00	2.00	2.50	10.70	11.20
Elwell's Ferry	399290	10.70	11.20	-5.5*	1.60	8.00	8.56	2.00	2.70	10.70	11.20
Lock & Dam 2	523670	20.10	20.50	2*	3.20	2*	8.60	14.00	14.60	2*	11.2
Proposed Smithfield Foods Intake	619290	20.10	20.70	2*	13.20	2*	12.90	14.00	14.90	2*	13.1
Lock & Dam 3	645300	29.10	29.40	9.9*	13.30	27.20	27.60	23.00	23.60	29.10	29.40
Dupont Intake	650700	29.10	29.60	9.9*	14.00	27.20	27.80	23.00	23.60	29.10	29.60
Fayetteville Glenville Intake	764605	29.10	29.70	9.9*	23.00	27.20	28.00	23.00	24.10	29.10	29.70
Fayetteville Hoffer Intake	764605	29.10	29.70	9.9*	23.00	27.20	28.00	23.00	24.10	29.10	29.70

NOTE: * indicates that the dam has been removed and that is natural bottom at the site

TABLE -1
Summary of water surface elevations

CAPE FEAR RIVER SPAWNING DISCHARGES	
RIVER STATION	DISCHARGE cfs
1039922	2000
902102	2200
787430	3500
701640	4100
645000	4800
523671	4900
353013	5000

TABLE -2
Spawning Discharges by river station

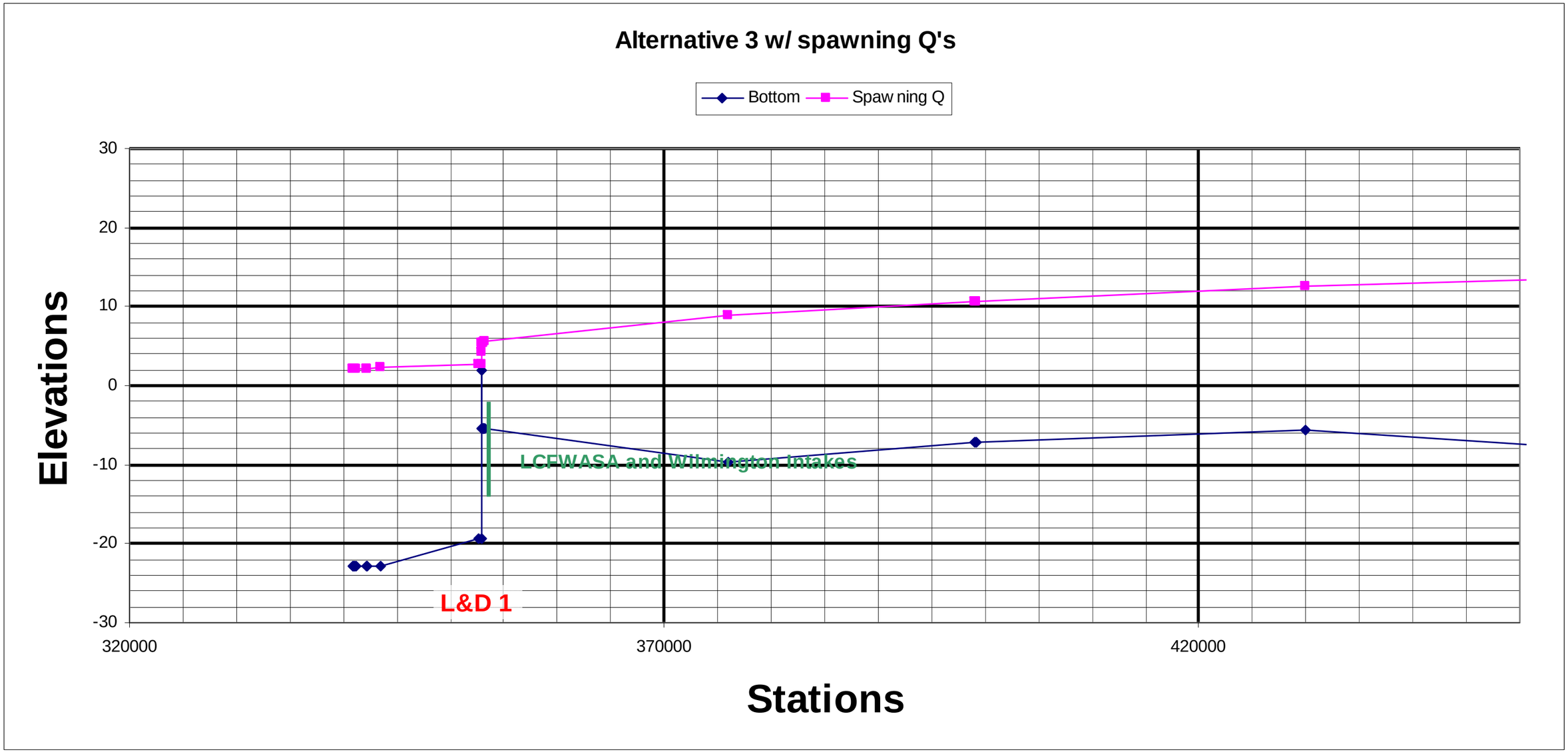


Plate 2/1
Alternative 3 w/ Spawning Q's sheet 1 of 3

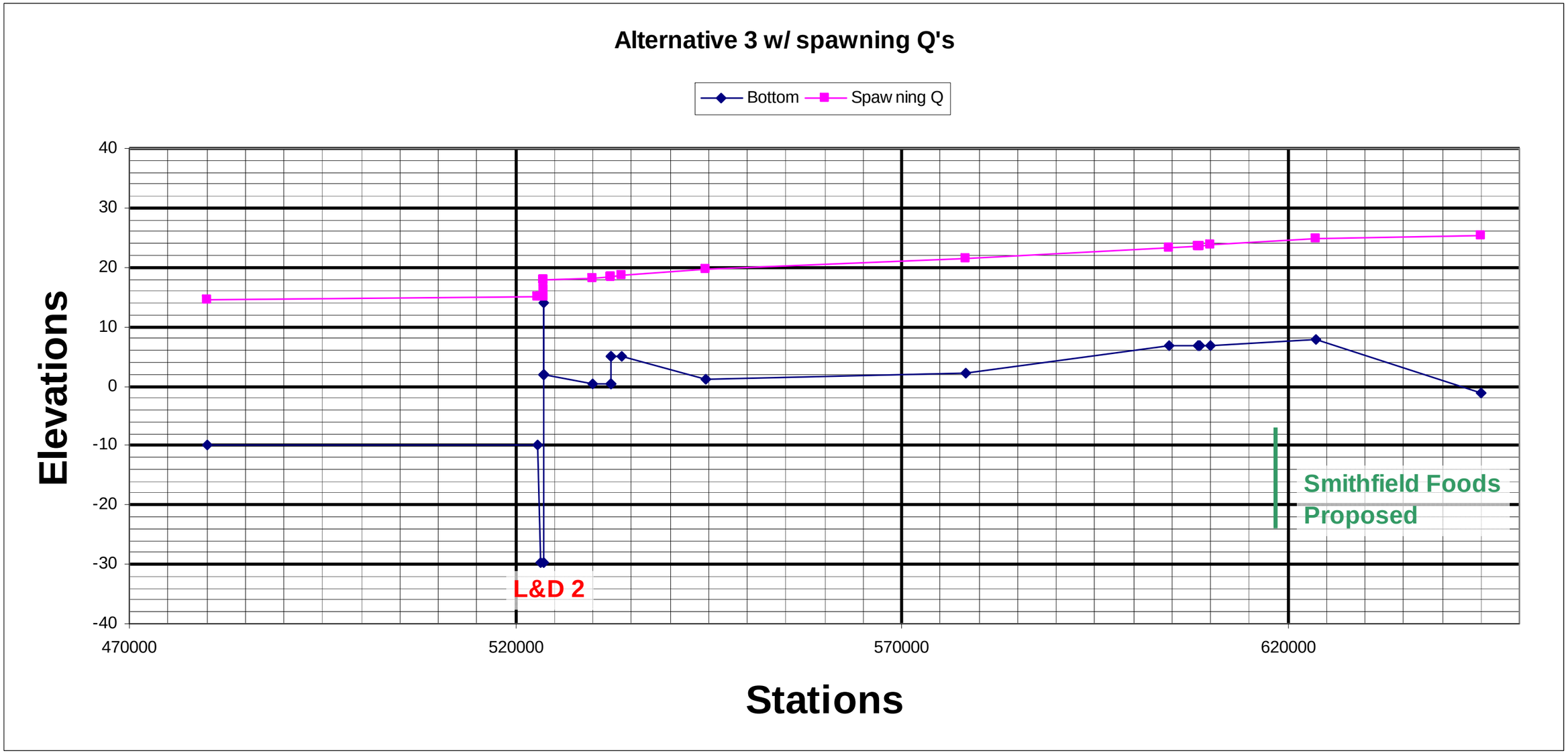


Plate 2/2
Alternative 3 w/ Spawning Q's sheet 2 of 3

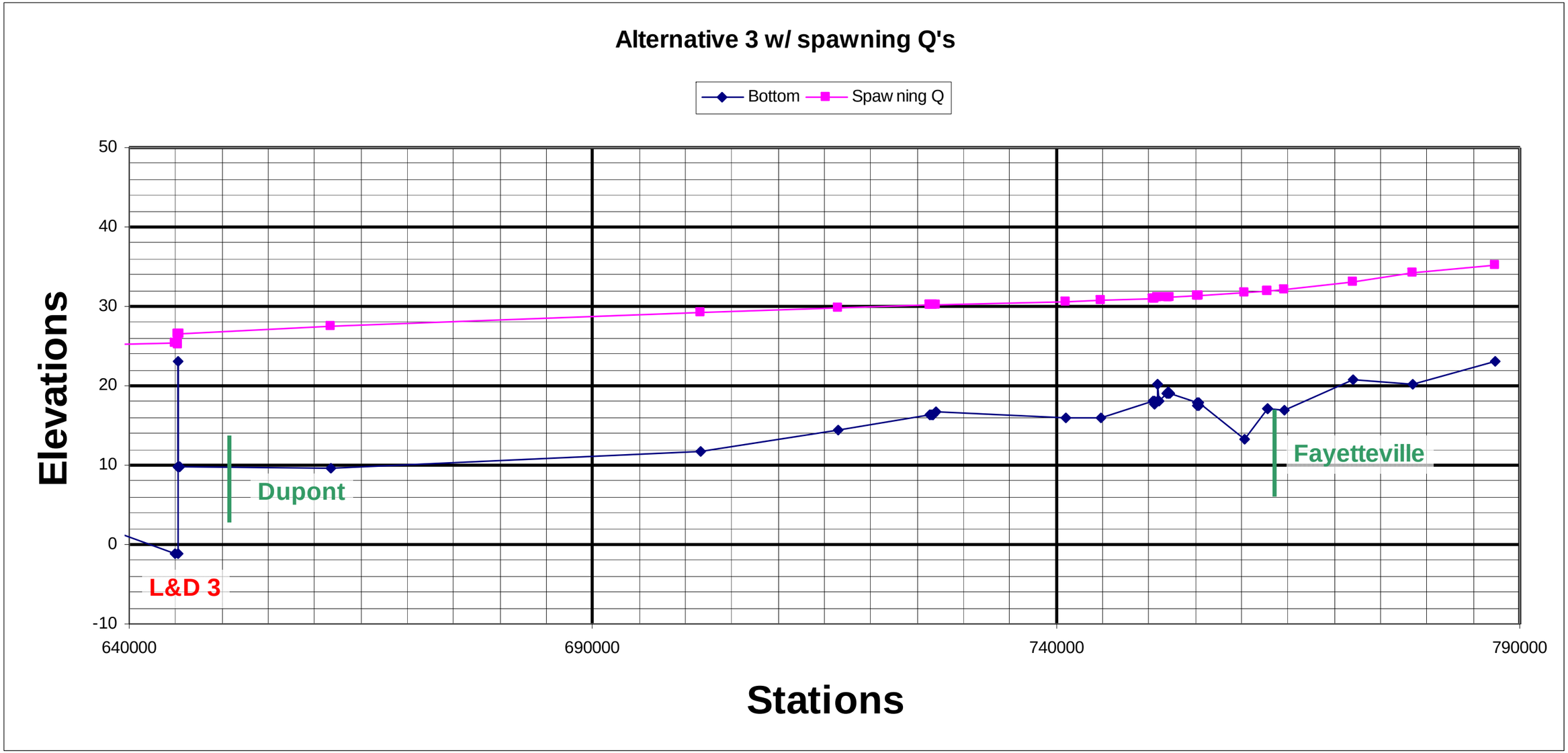


Plate 2/3
Alternative 3 w/ Spawning Q's sheet 3 of 3

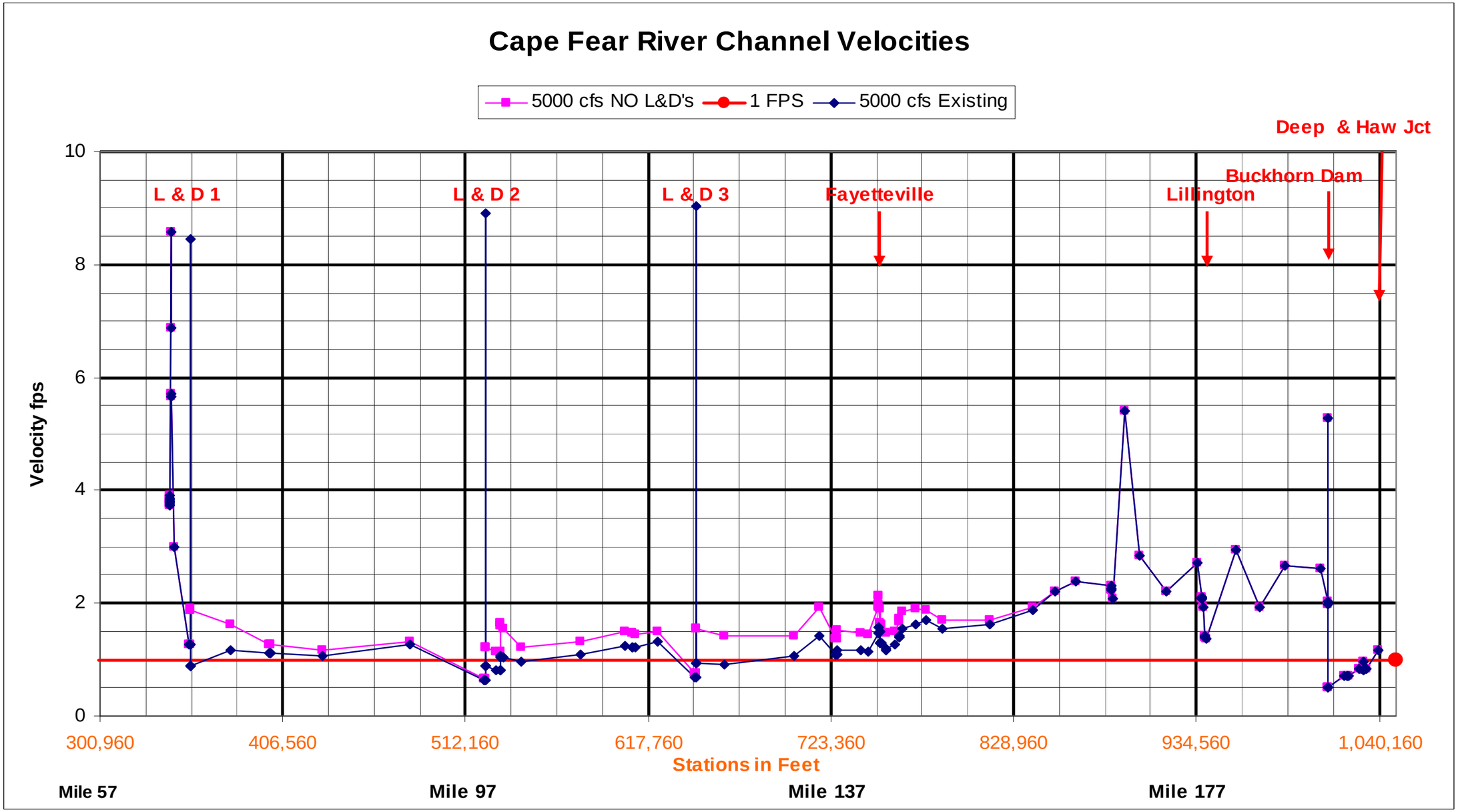


Plate 3
Plot of average channel velocity

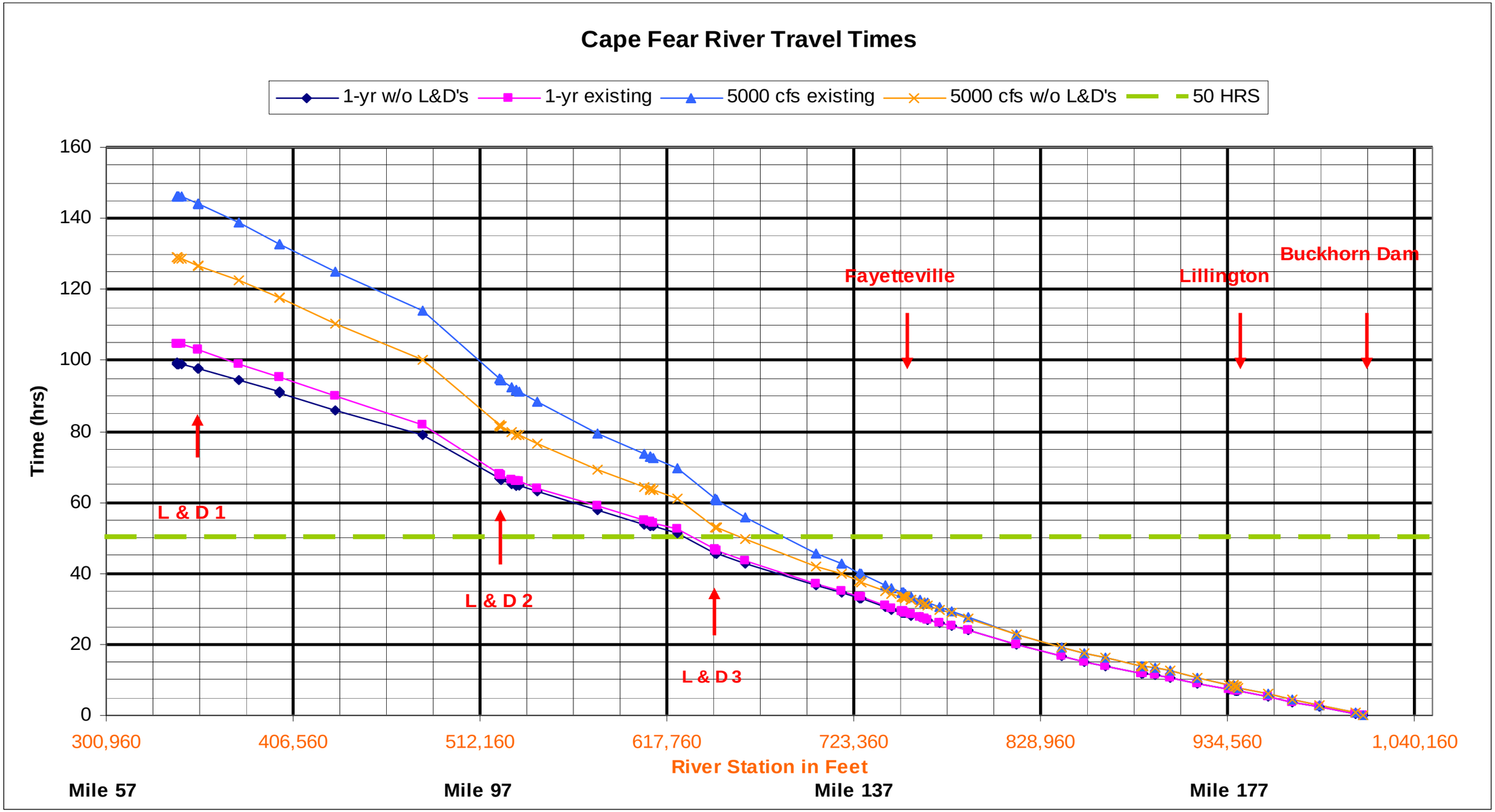


Plate 4
Travel time below Buckhorn Dam

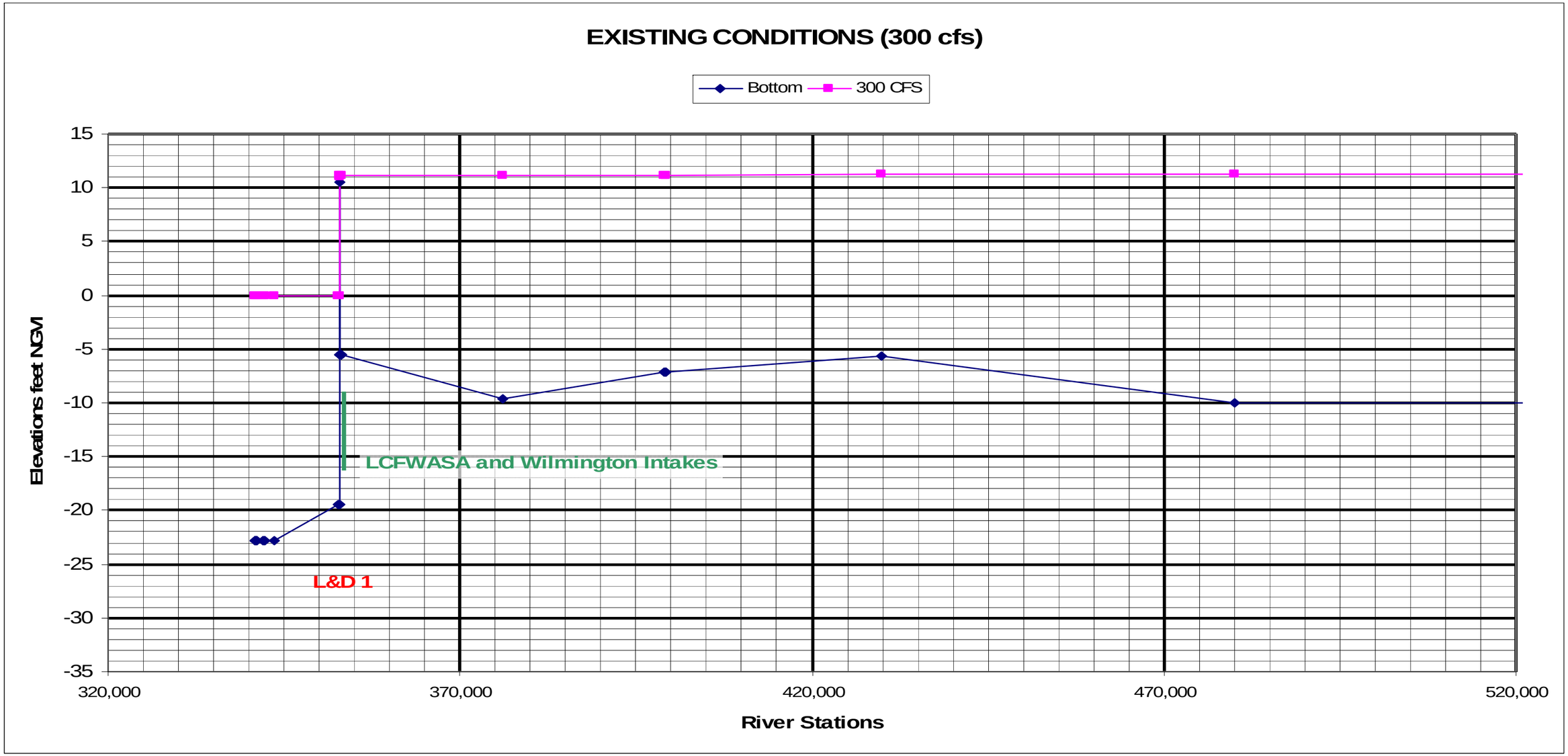


PLATE 5/1
Existing water surface profile sheet 1 of 3

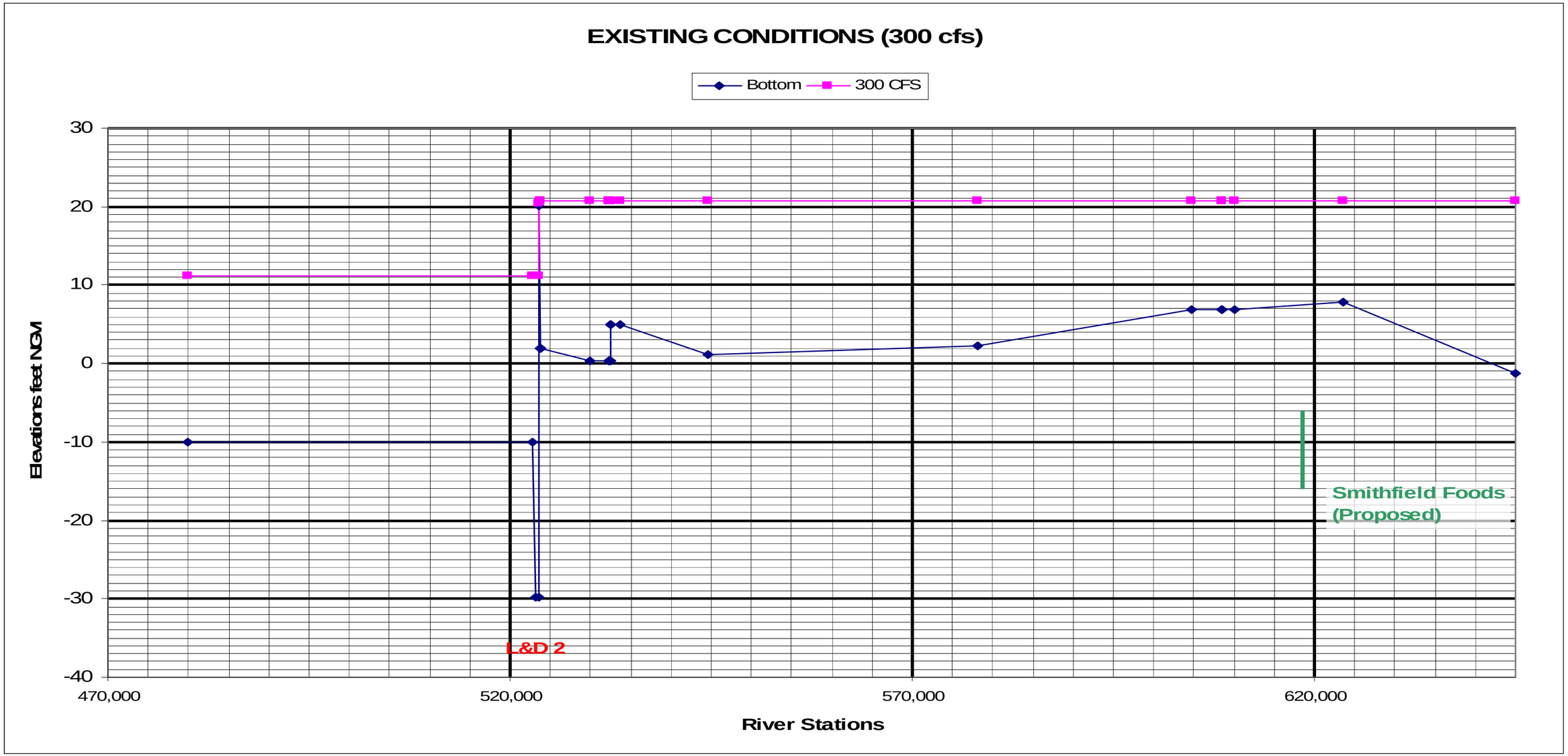


PLATE 5/2
Existing water surface profile sheet 2 of 3

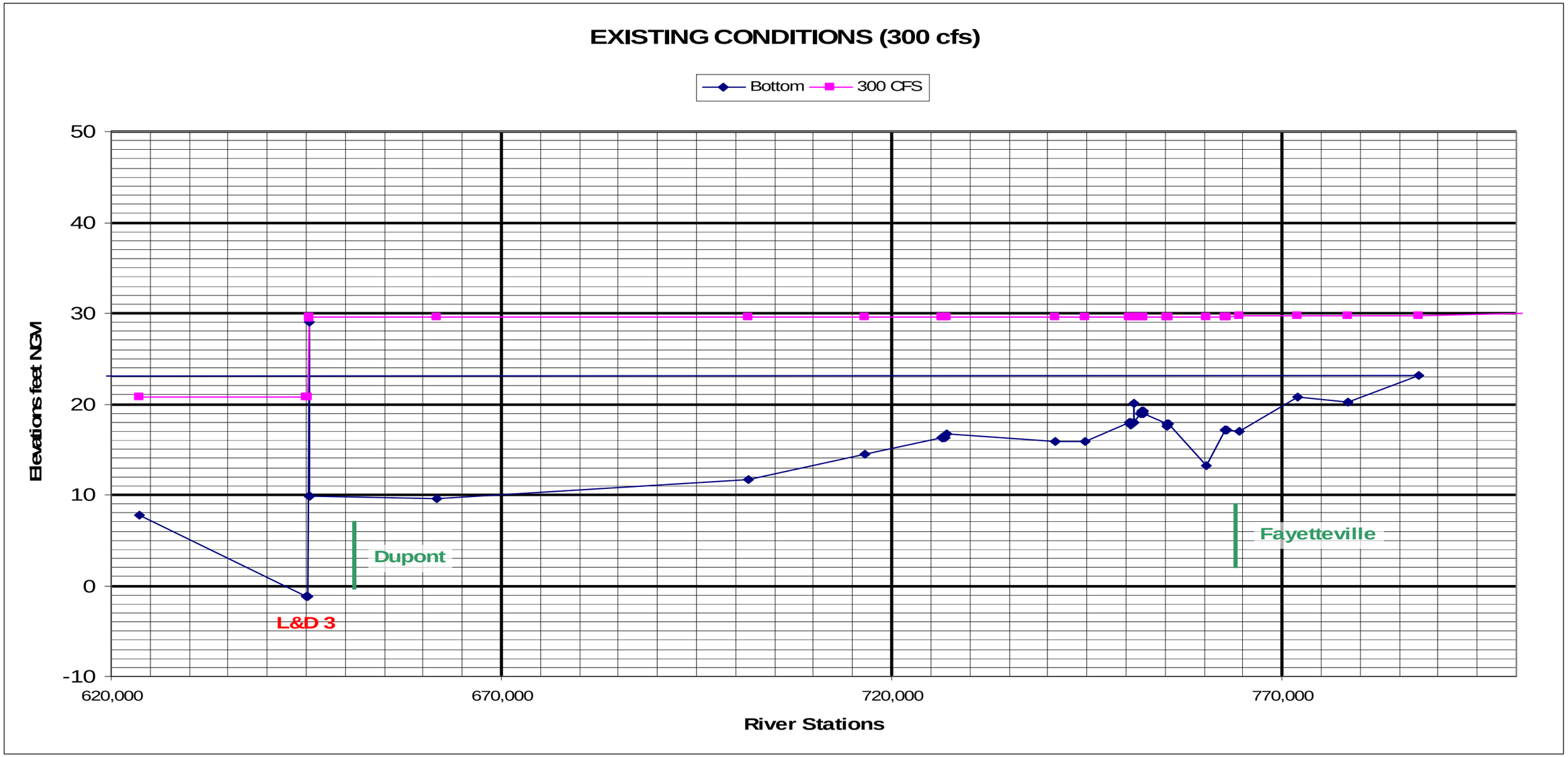


PLATE 5/3
Existing water surface profile sheet 3 of 3

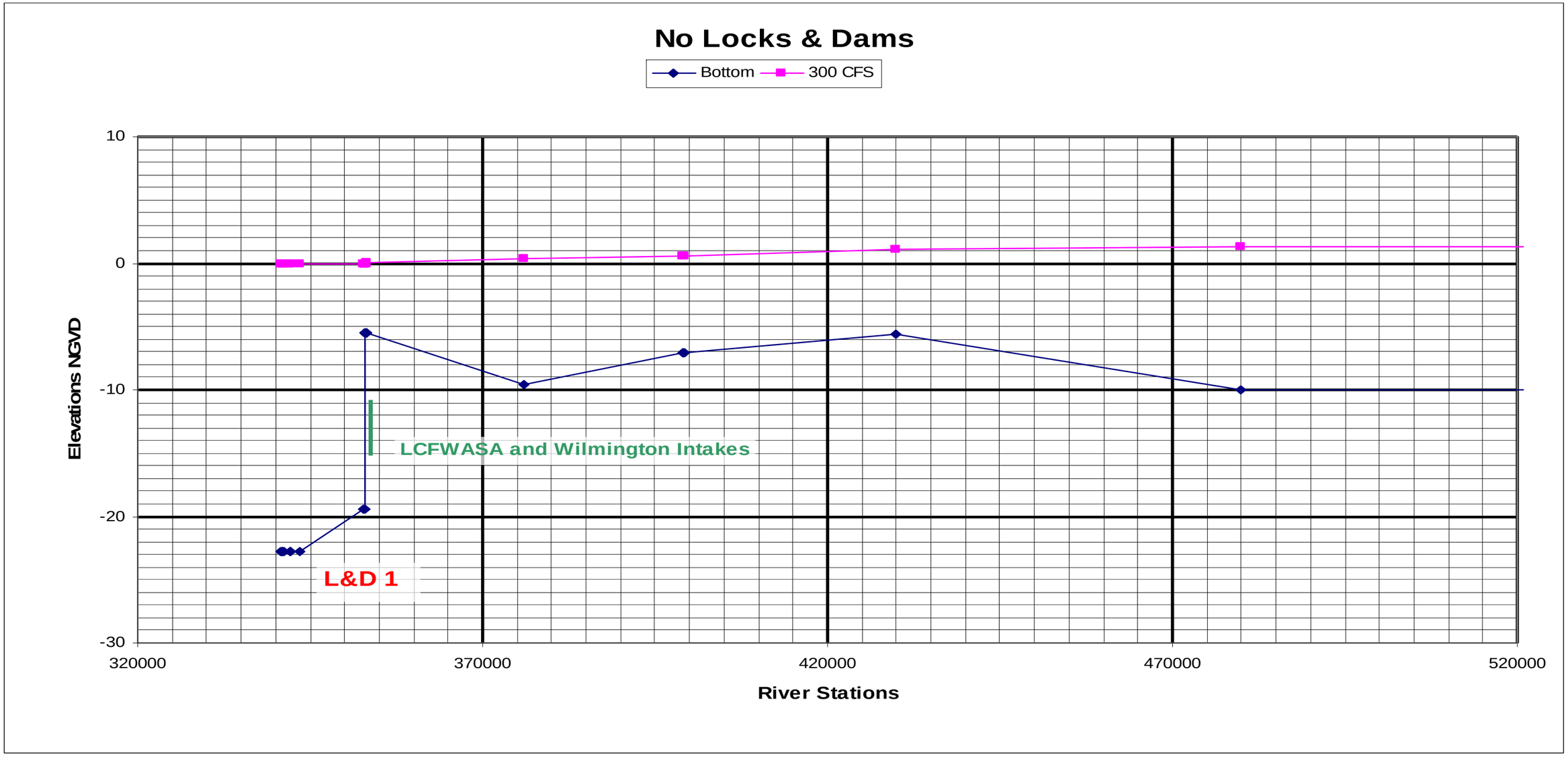


Plate 6/1
Alternative 1 Water surface profile sheet 1 of 3

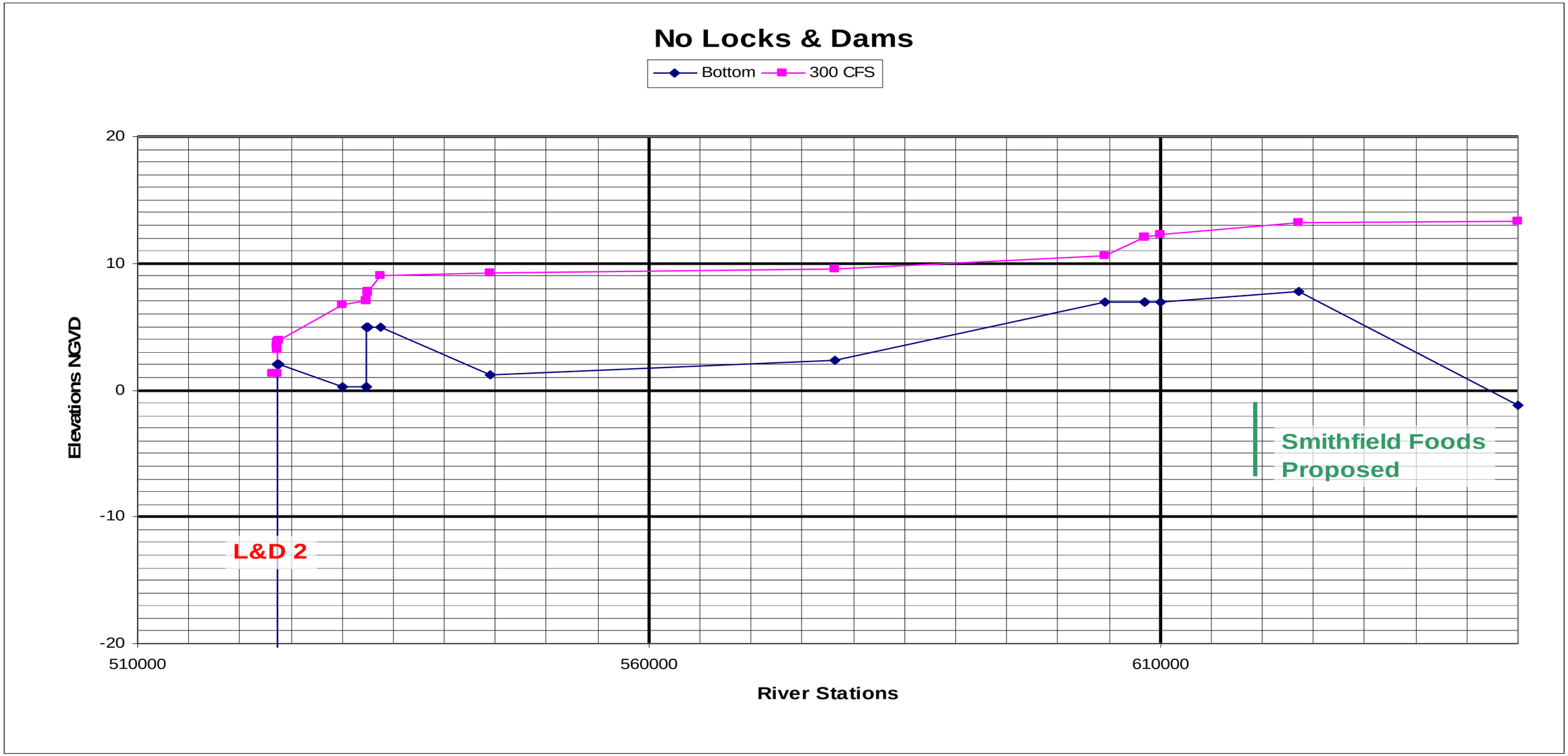


Plate 6/2
Alternative 1 Water surface profile sheet 2 of 3

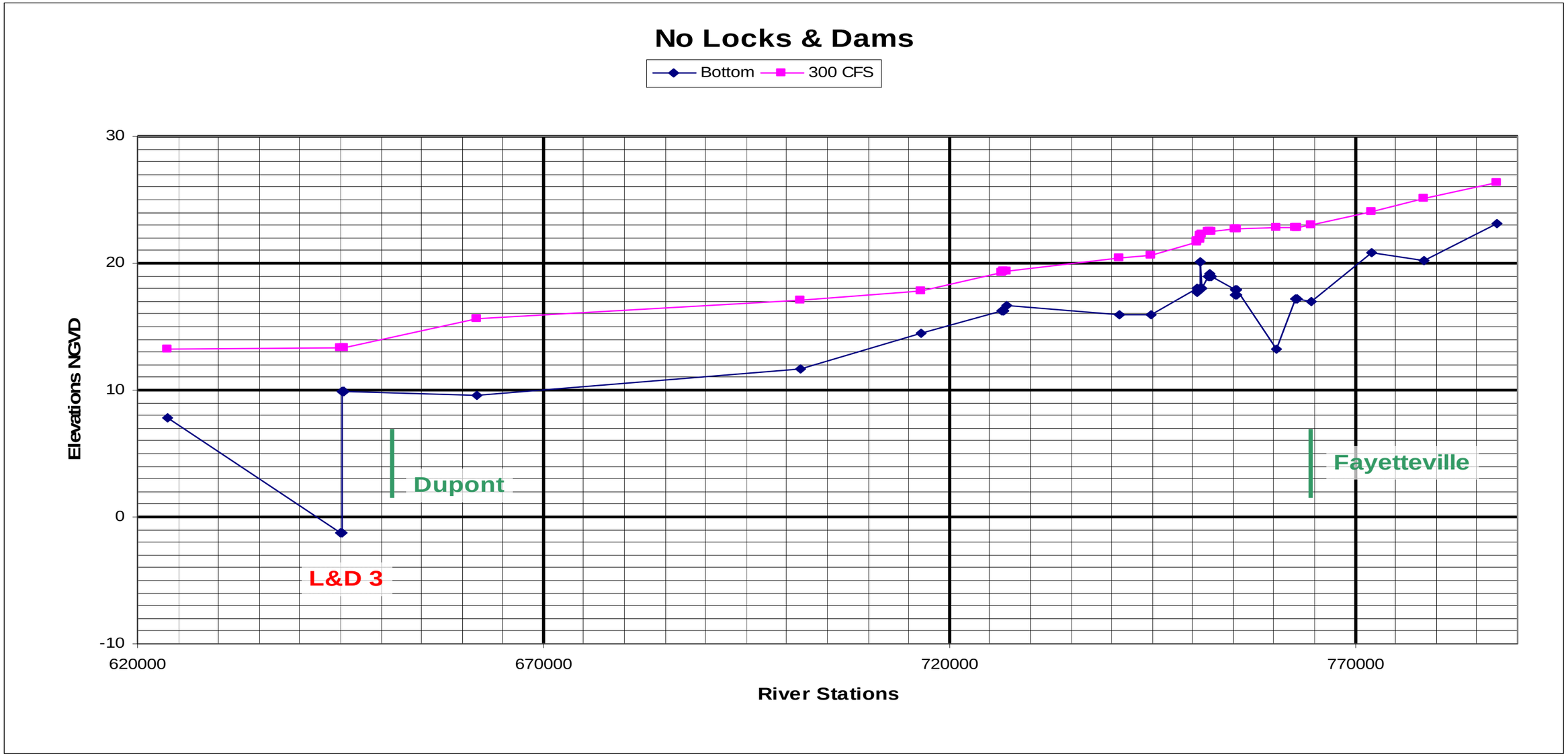


Plate 6/3
Alternative 1 Water surface profile sheet 3 of 3

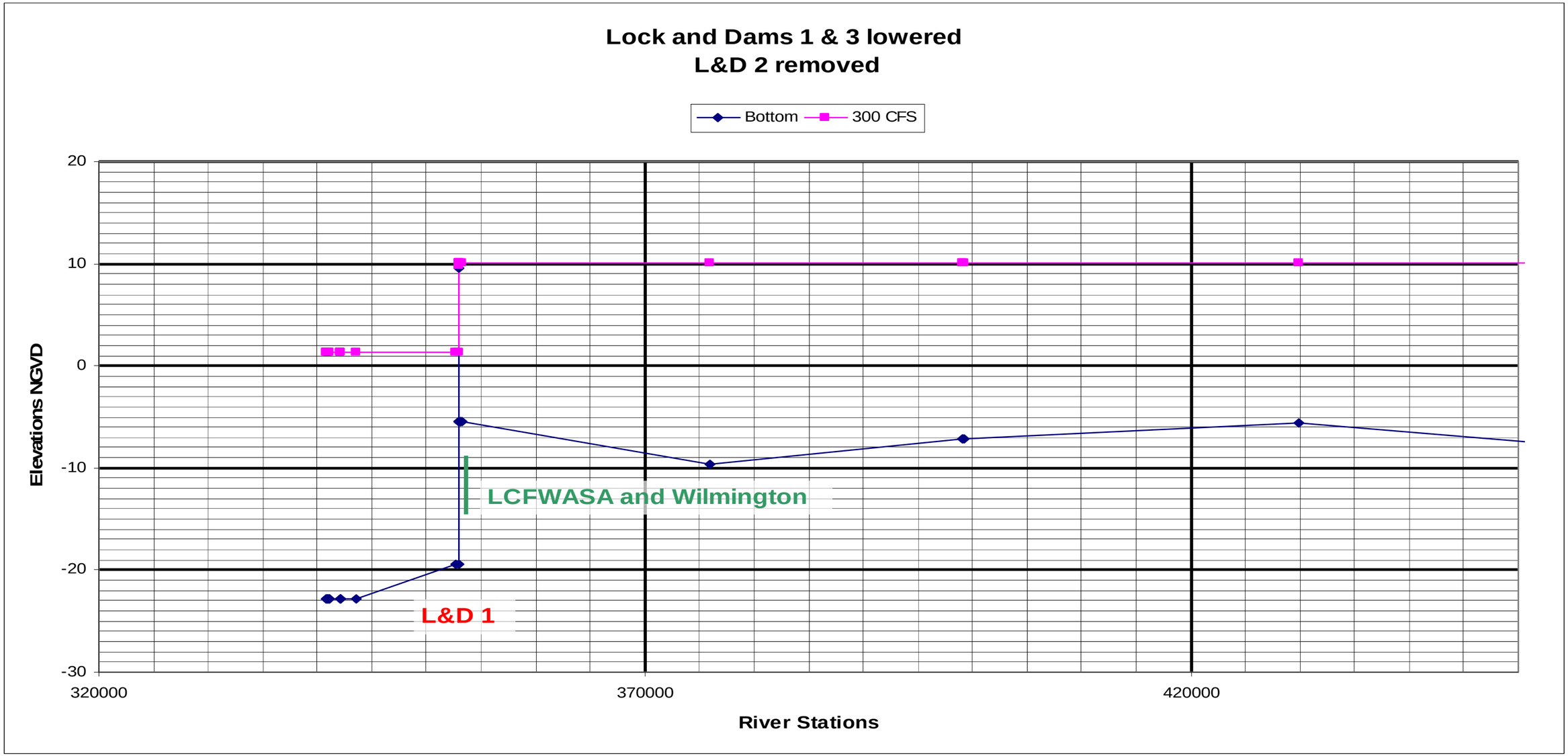


Plate 7/1
Alternative 2 water surface profile sheet 1 of 3

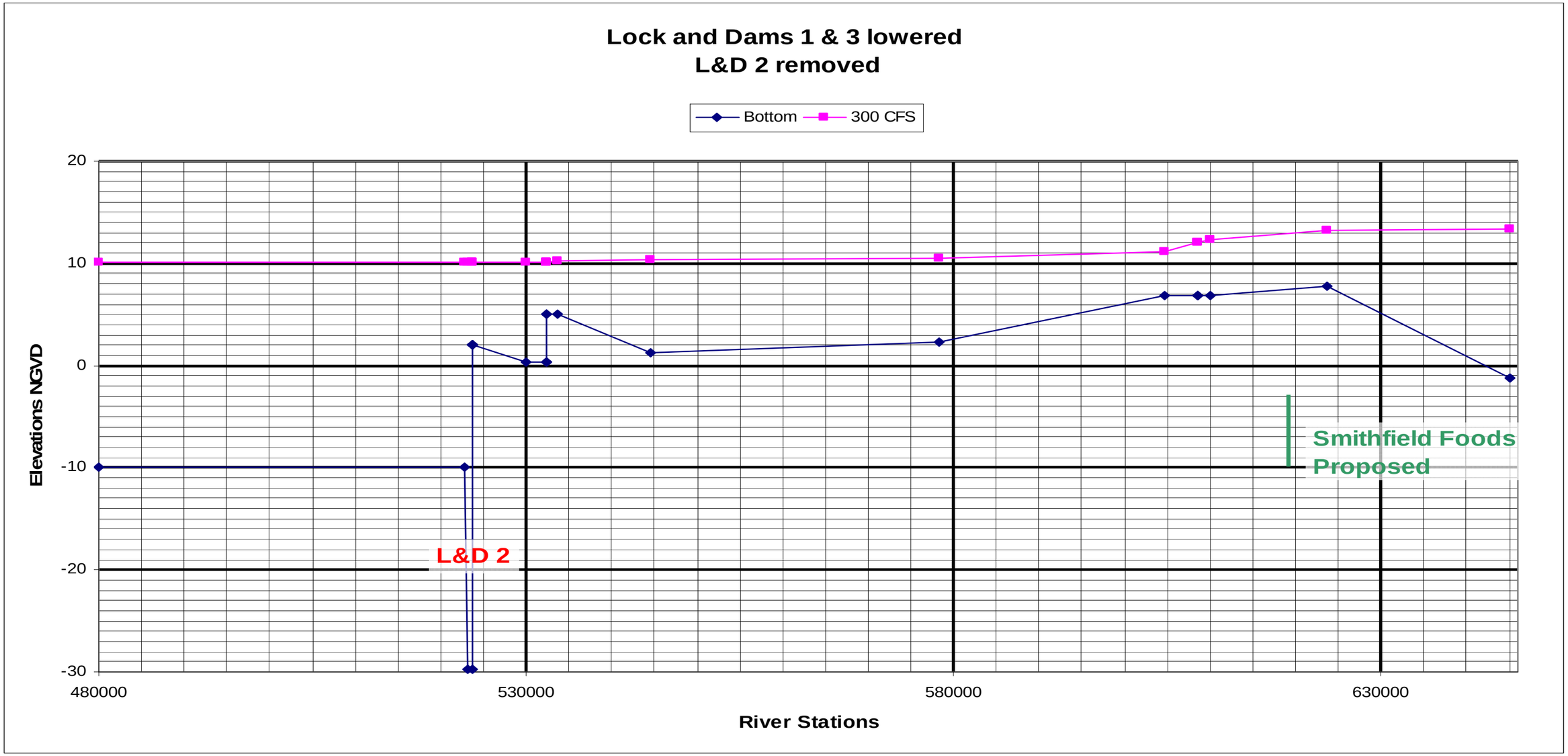


Plate 7/2
Alternative 2 water surface profile sheet 2 of 3

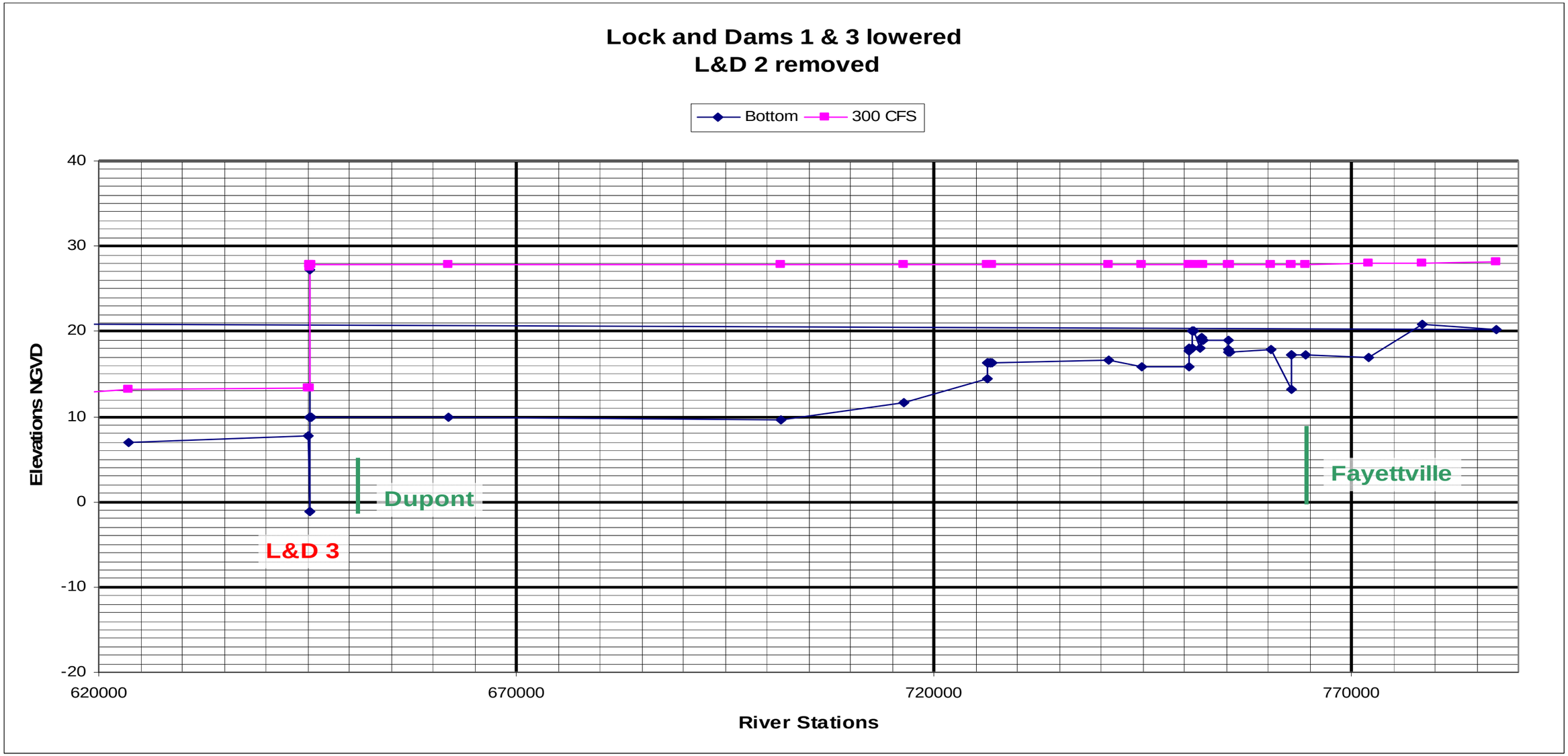


Plate 7/3
Alternative 2 water surface profile sheet 3 of 3

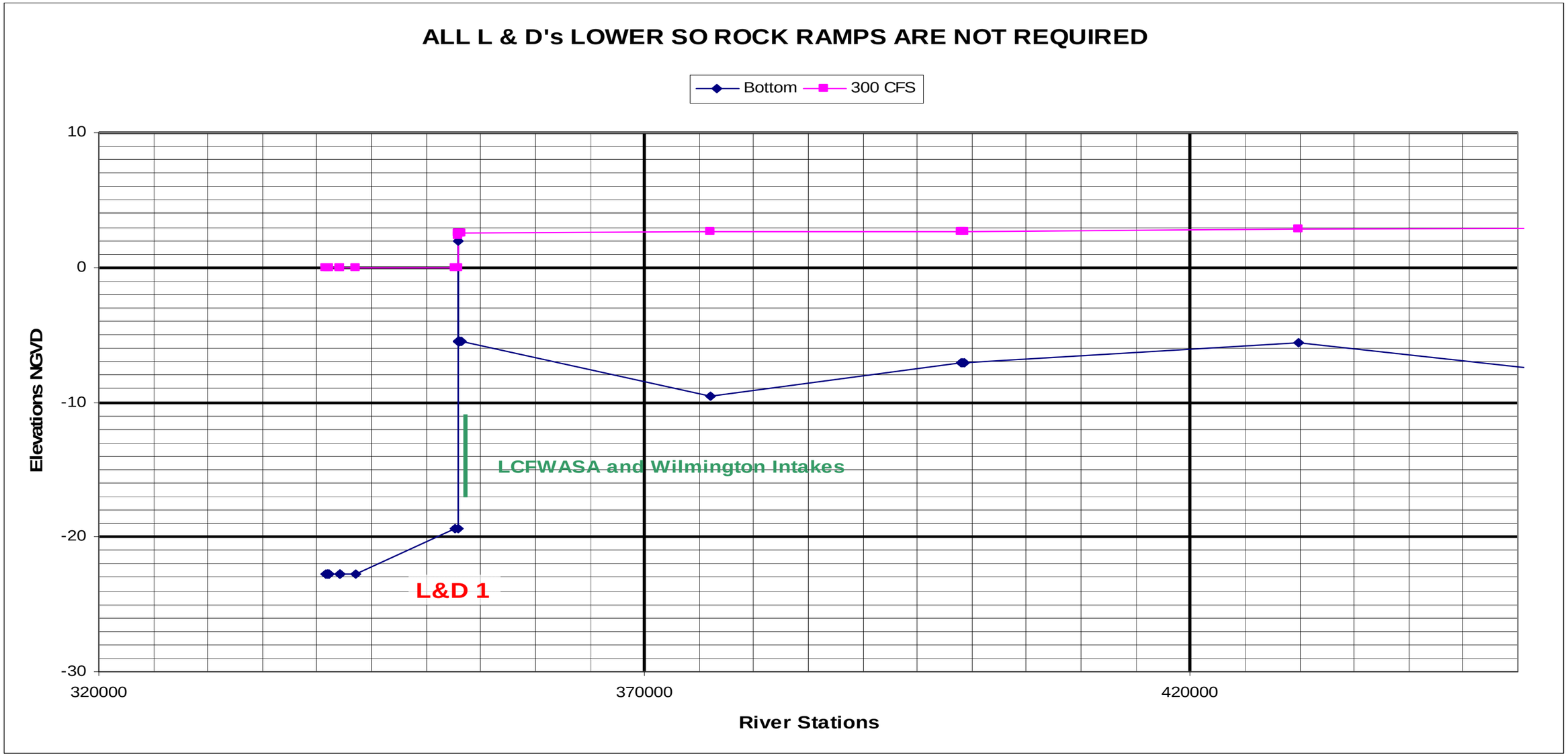


Plate 8/1
Alternative 3 Water surface profile sheet 1 of 3

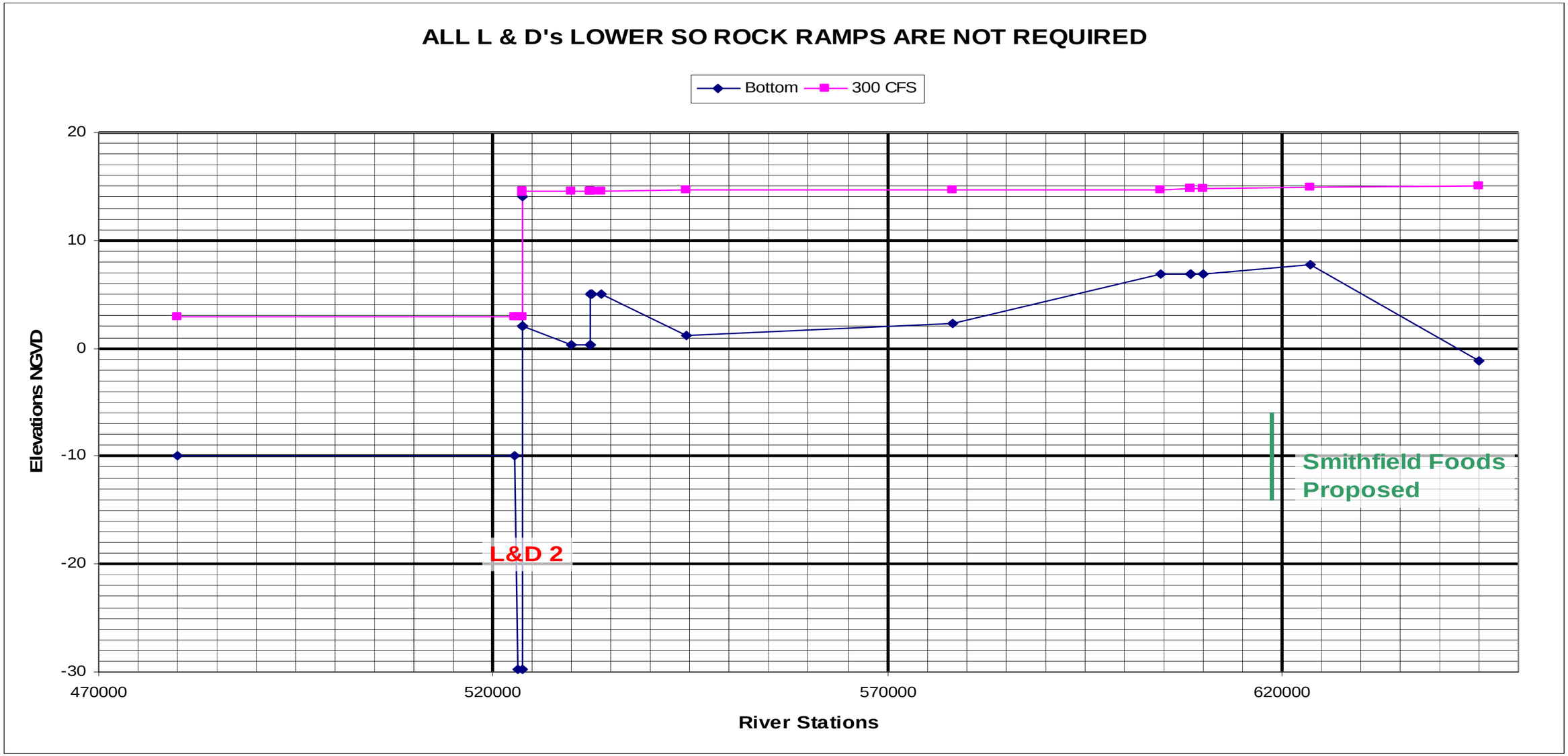


Plate 8/2
Alternative 3 Water surface profile sheet 2 of 3

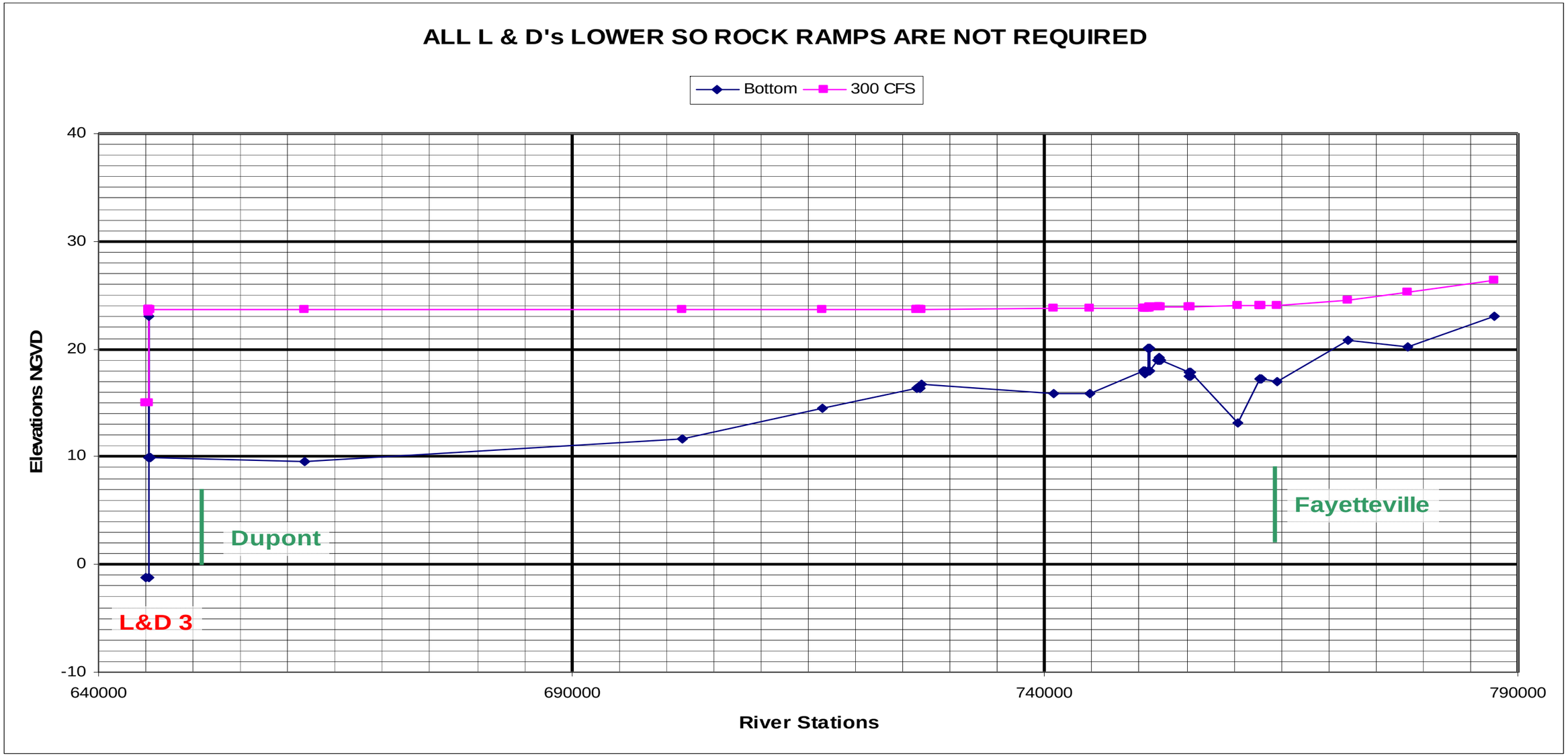


Plate 8/3
Alternative 3 Water surface profile sheet 3 of 3

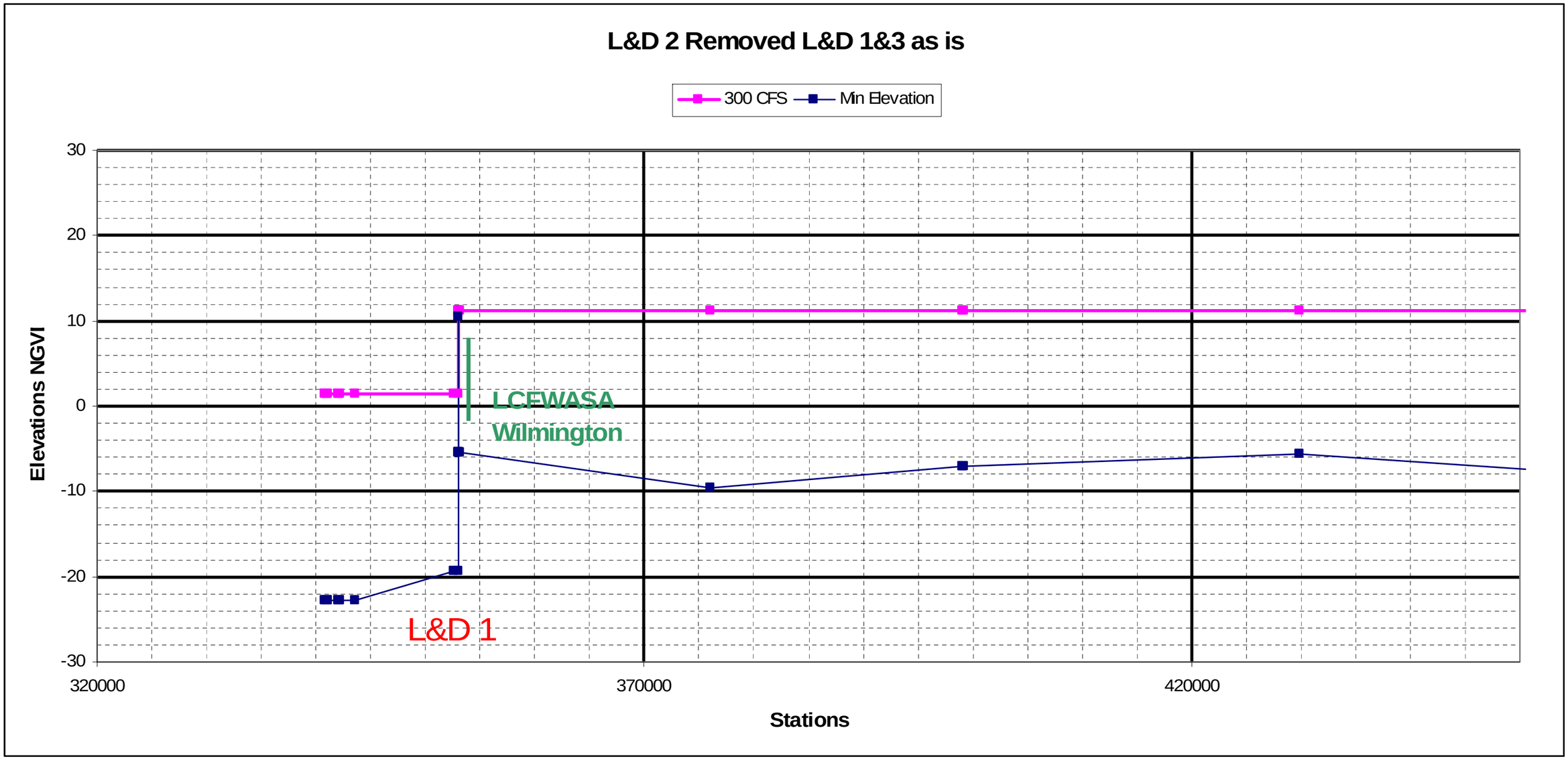


Plate 9/1
Alternative 4 Water surface profile sheet 1 of 3

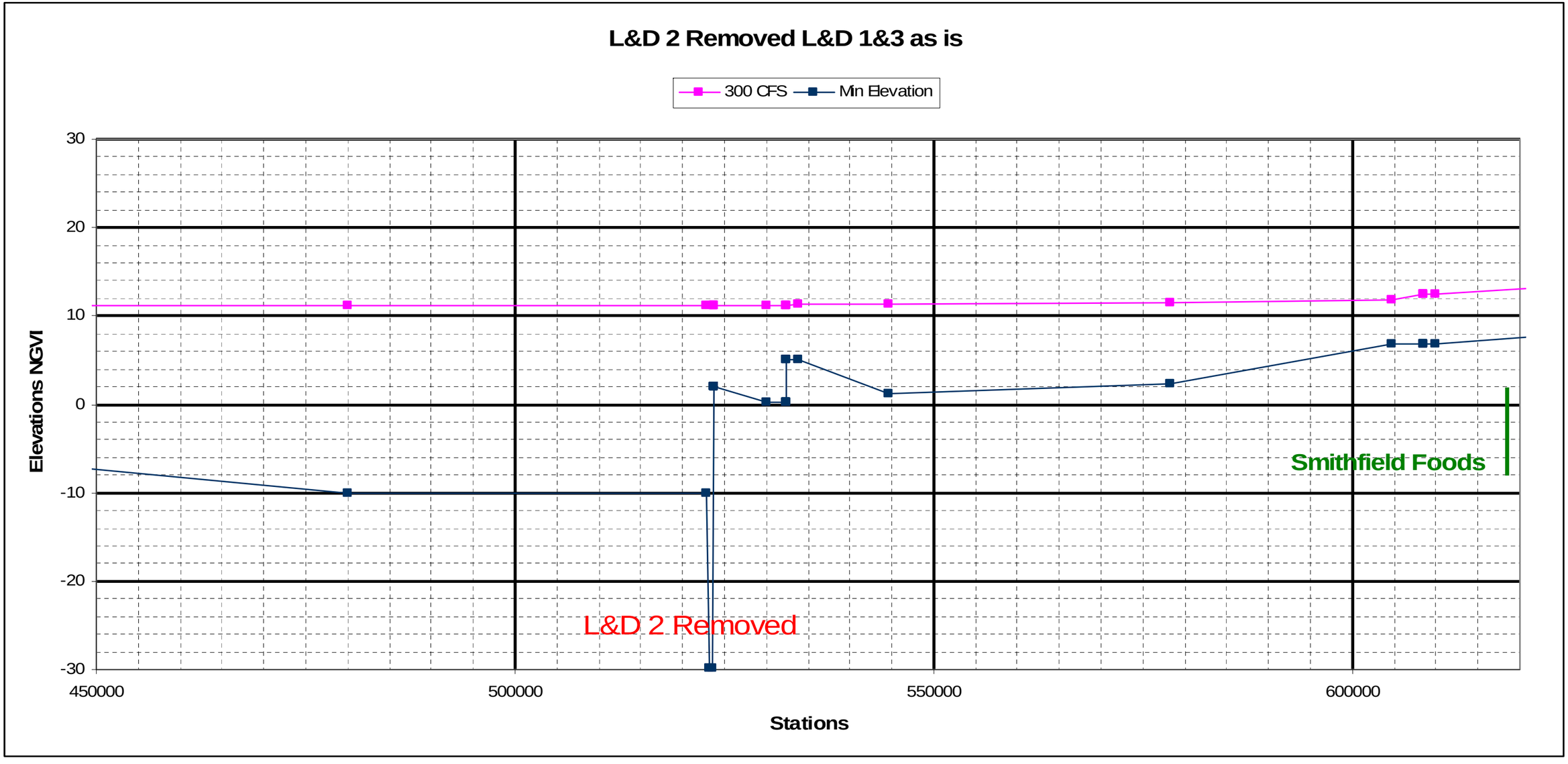


Plate 9/2
Alternative 4 Water surface profile sheet 2 of 3

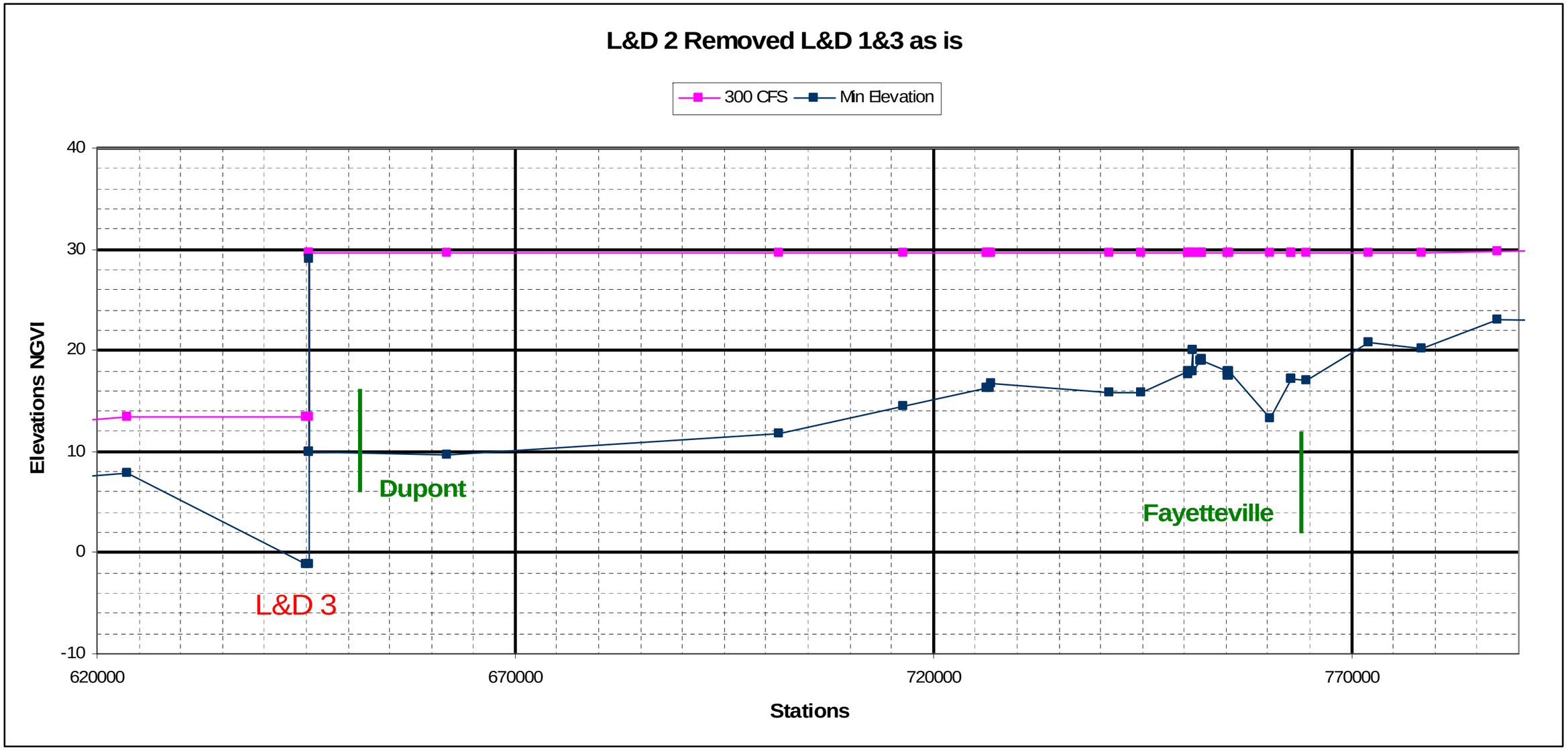


Plate 9/3
Alternative 4 Water surface profile sheet 3 of 3

END

Appendix 6.3

Value Engineering Study Summary Report

US Army Corps
of Engineers

OFFICE OF THE CHIEF OF ENGINEERS
Value Engineering Study Team

VALUE ENGINEERING STUDY SUMMARY REPORT

WILMINGTON HARBOR 96 ACT GENERAL RE-EVALUATION REPORT FOR RELOCATED TURNING BASIN AND REMAINING MITIGATION FEATURES

WILMINGTON, NORTH CAROLINA

Sponsored By: The U.S. Army
Engineering District, Wilmington

July 2007

DOD SERVICE: USACE
VALUE ENGINEERING OFFICER: Don Carmen

Value Engineering Study Report on

WILMINGTON HARBOR 96 ACT GENERAL RE-EVALUATION REPORT FOR RELOCATED TURNING BASIN AND REMAINING MITIGATION FEATURES

WILMINGTON, NORTH CAROLINA

JULY 2007

Sponsor: The U.S. Army Engineering District, Wilmington

VALUE ENGINEERING FIRM NAME: Office of the Chief of Engineers
Value Engineering Study Team (OVEST)

ADDRESS: 69A Hagood Avenue
Charleston, SC
29403-5107
PHONE: (843) 329-8063

VALUE ENGINEERING STUDY TEAM LEADERS: Bill Easley 843-329-8152
Rick Lambert 843-329-8063

VALUE ENGINEERING STUDY TEAM MEMBERS

Aaron Buesing CEMVP-EC-H
John Caldwell CESAW-TS-EE
Frank Yelverton CESAW-TS-PP

VALUE ENGINEERING TEAM STUDY

TABLE OF CONTENTS

Page No.

Cover.....1

Table of Contents 2

Project Description and Background 4

Vicinity and Locality Map 5

Project Plan 6

Executive Summary 7

Summary of Recommendations 9

Proposals:

1 Optimize Use of Local Materials to Fill Scour Holes and Build Rock Ramps 10

2 If Lock & Dam #2 (or any other dam) is removed, do not use coffercells 27

3 Remove L&D 2; lengthen ramp at L&D 3 29

4 Use Rock Dredged from Turning Basin or Authorized NE Cape Fear River Project to Fill Scour Hole and Build Ramp(s) 34

5 Investigate tailwater levels for design of rock ramp fishways 36

6 Use Dock Area Adjacent to Sigma Marine in lieu of New Turning Basin 37

TABLE OF CONTENTS

Page No.

Value Engineering Comments 39

Supporting Documents 43

Appendix A: Contact Directory 44

Appendix B: Issues and Concerns 47

Appendix C: Cost Model 49

Appendix D: Function Analysis System Technique (FAST) Diagram 51

Appendix E: Speculation List 53

VALUE ENGINEERING TEAM STUDY

PROJECT DESCRIPTION AND BACKGROUND

PROJECT TITLE: WILMINGTON HARBOR 96 ACT GENERAL RE-EVALUATION REPORT FOR RELOCATED TURNING BASIN AND REMAINING MITIGATION FEATURES

PROJECT LOCATION: WILMINGTON, NORTH CAROLINA

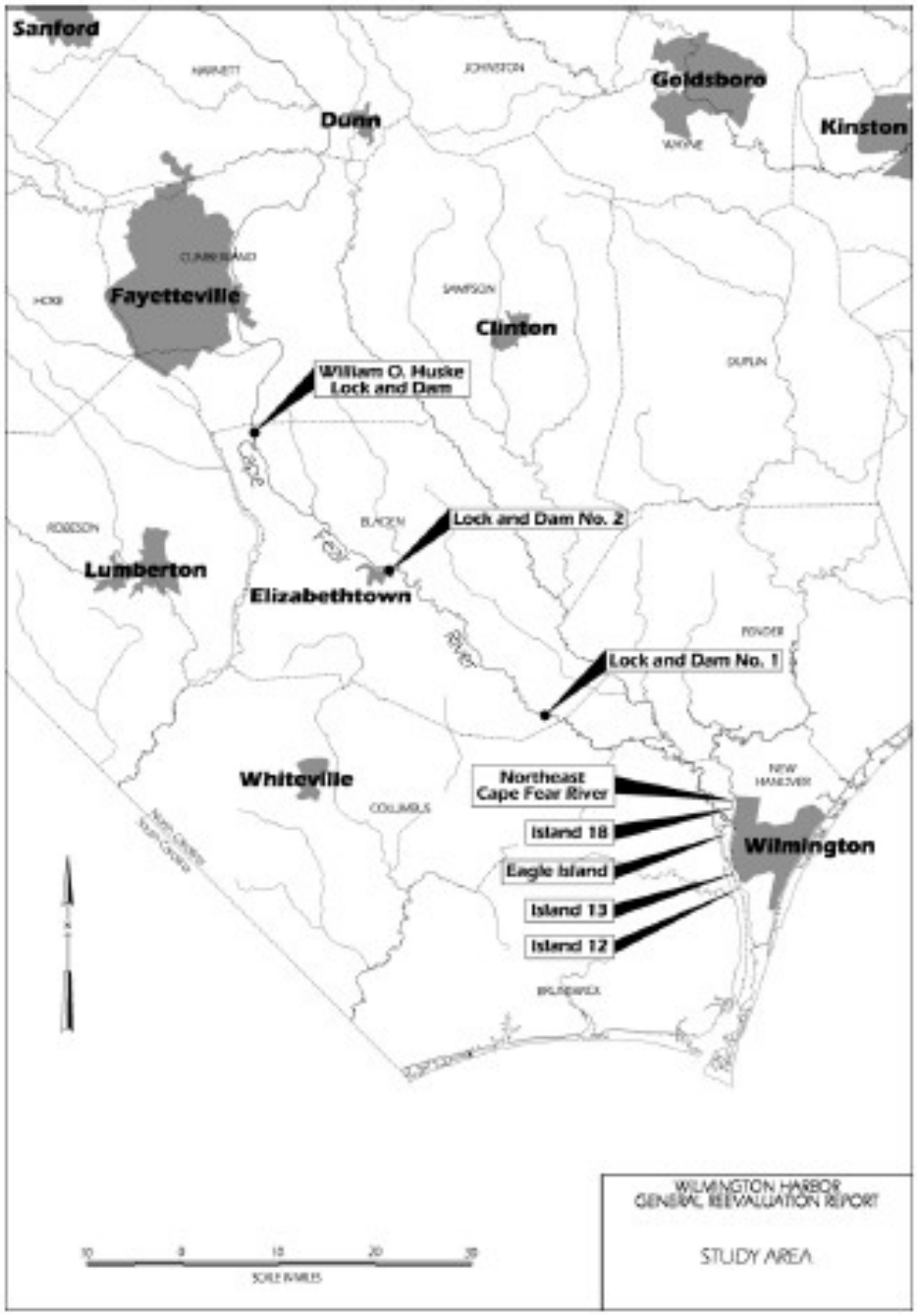
Wilmington Harbor is a Federal navigation project located along the Cape Fear and Northeast Cape Fear Rivers in southeastern North Carolina. The project extends from the Atlantic Ocean to the Port of Wilmington. Wilmington Harbor requires deepening in order to accommodate the vessels that currently call at the port. The turning basin needs to be relocated in order to allow large vessels to turn around to safely exit bow first through several bridges. Environmental mitigation is required in order to protect the endangered shortnose sturgeon from harm if blasting is required to remove rock on the channel bottom. Restoration of primary nursery area for fish and wetlands will be needed.

The work in the proposed alternative includes dredging and possibly blasting to construct a new turning basin and either constructing rock fishway ramps or removing existing Locks & Dam structures upstream in the Cape Fear River.

The total estimated project cost presented in the GRR is approximately $30 Million.

Also see Supporting Documents Appendices for project Cost Model and the Function Analysis System Technique (FAST) diagram, a logic diagram identifying project critical functions.

VICINITY AND LOCALITY MAP



VALUE ENGINEERING TEAM STUDY

PROJECT DESCRIPTION AND BACKGROUND

PROJECT TITLE: WILMINGTON HARBOR 96 ACT GENERAL RE-EVALUATION REPORT FOR RELOCATED TURNING BASIN AND REMAINING MITIGATION FEATURES

PROJECT LOCATION: WILMINGTON, NORTH CAROLINA

PROJECT PLAN

VALUE ENGINEERING TEAM STUDY
EXECUTIVE SUMMARY

The Value Engineering Study was conducted at the Wilmington District office on 16-20 July 2007. The study was based on the General Re-evaluation Report – Alternative Formulation Briefing Preconference Materials dated July 2007. The VE team was comprised of members of Wilmington, Savannah and St Paul Districts, NC Water Resources Commission, NC State Ports Authority, US Fish and Wildlife Service, and OVEST (See Appendix A).

Value Engineering (VE) is a process used to study the functions a project is to achieve. VE takes a critical look at how these functions are proposed to be met and it identifies 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. The project was studied using the Corps of Engineers standard Value Engineering (VE) methodology, consisting of five phases:

Information Phase: The Team studied the General Re-evaluation Report and performed a field trip to visit the project site to fully understand the work to be performed and the functions to be achieved. Project specific issues and concerns were listed (see Appendix B). A Cost Model was prepared in order to determine areas of relative high cost to ensure that the team focused on those parts of the project that offered the most potential for cost savings (See Appendix C). The function analysis system technique was used to identify and organize critical project functions and a FAST diagram was developed (see Appendix D).

Speculation Phase: The Team speculated by conducting a brainstorming session to generate ideas for alternative designs. All team members contributed ideas and critical analysis of the ideas was discouraged (see Appendix E).

Analysis Phase: Evaluation, testing and critical analysis of all ideas generated during speculation was performed to determine potential for savings and possibilities for risk. Ideas were ranked by priority for development. Ideas that did not survive critical analysis were deleted.

Development Phase: The priority ideas were developed into written proposals by VE team members during an intensive technical development session. Proposal descriptions, along with sketches, technical support documentation, and cost estimates were prepared to support implementation of ideas. Additional VE Team Comments were included for items of interest that were not developed as proposals, and these comments follow the study proposals.

Presentation Phase: An informal outbriefing was conducted to review the proposals. This VE Study Report will be distributed to all appropriate project supporters and decision-makers. Final decisions on implementation will be coordinated with the PDE by the Value Engineering Officer.

VALUE ENGINEERING TEAM STUDY

SUMMARY OF RECOMMENDATIONS/ACTION

PROPOSAL NUMBER	DESCRIPTION	POTENTIAL SAVINGS	RECOMMENED ACTION
1	Optimize Use of Local Materials to Fill Scour Holes and Build Rock Ramps	$ 3.47 M max $ 83.7 K min	_____
2	If Lock & Dam #2 (or any other dam) is removed, do not use coffercells	$ 1.5 M	_____
3	Remove L&D 2; lengthen ramp at L&D 3	$ 3.3 M	_____
4	Use Rock Dredged from Turning Basin or Authorized NE Cape Fear River Project to Fill Scour Hole and Build Ramp(s)	$?	_____
5	Investigate tailwater levels for design of rock ramp fishways	$ Zero	_____
6	Use Dock Area Adjacent to Sigma Marine in lieu of New Turning Basin	$?	_____

ORIGINAL DESIGN:

Existing dams have a 1V:5H slope. Current design calls for building up a ramp for fish with a rock face at a 1V:20H slope. Ramps continued at that slope from the crest of the dam to one foot below the no-flow water surface, based solely on the profile backtracking from downstream dams. At that point, the slope changes to 1V:5H to the toe of the ramp. Downstream from Lock & Dams 1 and 2 are large scour holes, apparently resulting from rolling current during high flow events. The scour holes would only be filled to the extent necessary to toe out the new ramps.

The design does not specify the fill material, but includes at least two feet of granite bedding stone, four feet of granite wearing stone, and a final layer of boulders.

PROPOSED DESIGN:

Several alternatives should be considered to fill the scour hole and build up the ramp, since some of the material could be lost during the two weeks expected to do the work.

First alternative is to use hydraulic dredging along the shore from the downstream end of the lock chamber to the downstream boat ramp to fill geobags with sand. Once the stacked bags meet the profile, cover with the bedding stone, wearing stone and boulders.

The second alternative is to dredge directly into the scour hole, sloping up to the dam, and periodically cap it with a thin layer of limestone as an underwater choker course. Once the profile matches design, bring the top layer of limestone or bedding stone up to two feet in depth, and then place the wearing stone and boulder layers.

The third alternative is to use local limestone to provide the bulk of the scour hole fill and core of rock ramp fishway and serve as a bedding layer.

The final alternative is to use biodegradable materials to construct silt curtains to keep dredged material in place while being placed, and then covering with bedding stone, wearing stone and boulders. The curtains would only need to hold up for a few weeks.

ADVANTAGES:

- Hydraulically dredged sand is more economical fill
- Either hydraulically dredged sand or locally quarried limestone will reduce cost compared to granite
- Combination of hydraulically dredged sand and geobags will also dredge cost compared to granite
- Using geobags or choking courses keeps sand in place
- Keeping sand in place prevents need to overbuild
- Keeping sand in place reduces turbidity

DISADVANTAGES:

- Surveys would have to confirm whether there is enough sand in the river to fill the template
- Pumping into geobags would take longer than pumping directly
- Using intermittent choking courses without interrupting dredging would require close attention
- Anchoring silt curtains for fourth alternative would be difficult, even for a short time
- Using sand bag core would settle more than a full rock ramp, which might affect how long it takes to determine if the contractor has achieved the correct final slope
- Placing sand without using geobags or intermittent capping will allow sand to escape and increase turbidity

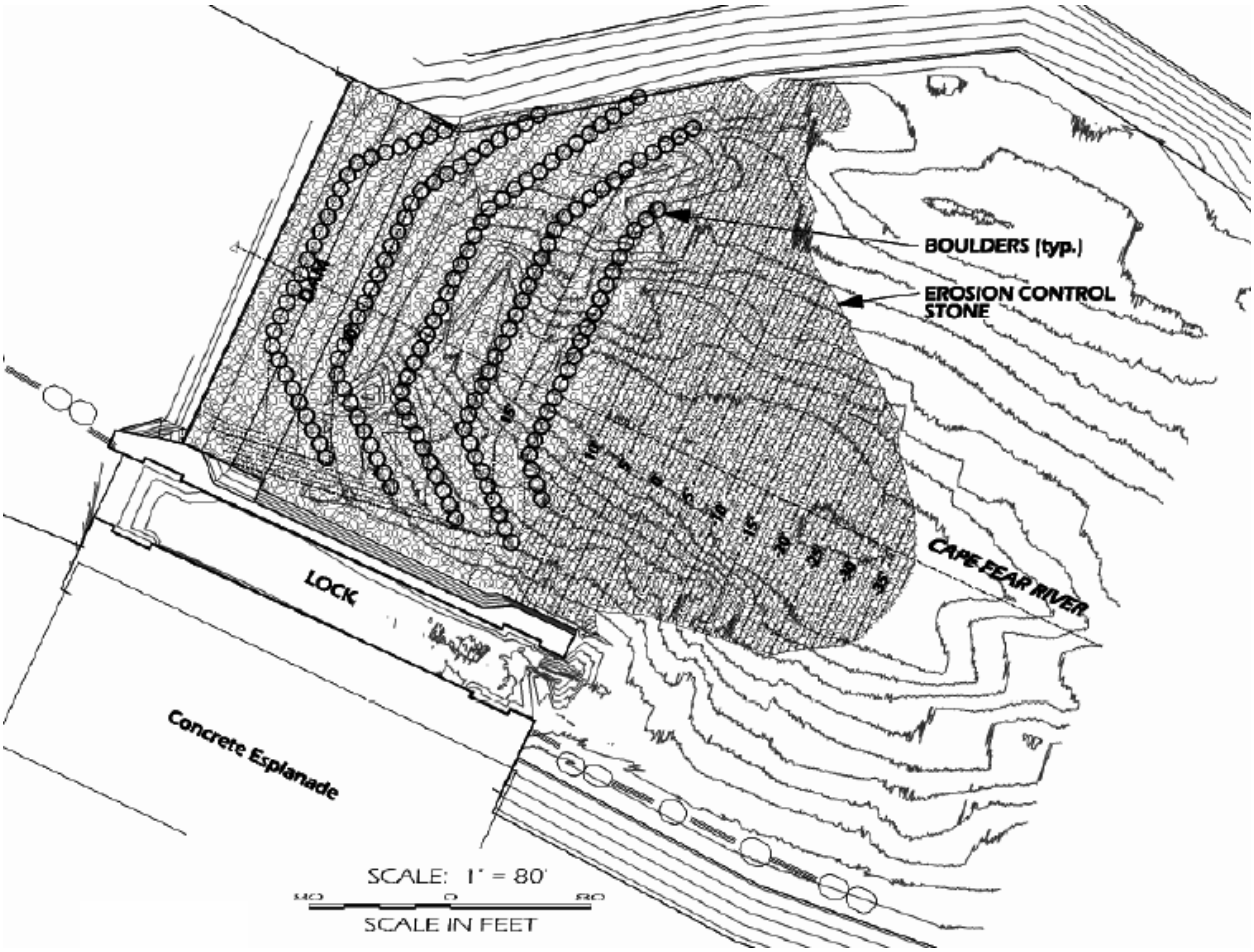
JUSTIFICATION:

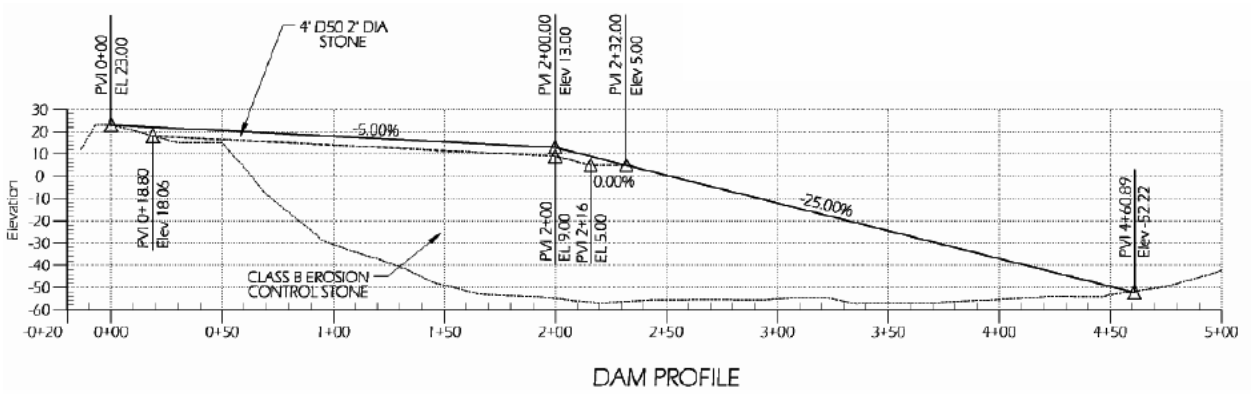
Cost Avoidance for Alternatives:

Description of Alternative	L&D 1	L&D 2	L&D 3
1A: Use Sand Underlayment Placed in GeoBags, Limestone Bedding, and Granite Wearing Course in lieu of All Granite	$2,622,540	$1,506,292	$193,464
1B: Use Limestone Bedding and Granite Wearing in Lieu of All Granite	$302,062	$212,071	$83,712
1C: Use Sand Underlayment, with Intermittent Limestone Choker Courses, Limestone Bedding Course and Granite Wearing Course in lieu of All Granite	$3,472,494	$2,003,125	$896,369
1D: Use Sand Underlayment, with Allowance for Sand Losses, Limestone Bedding Course and Granite Wearing Course in lieu of All Granite	$3,366,250	$1,941,020	$524,001

As shown above, the most cost-effective method would be to use intermittent layers of limestone or gravel to cap the sand and reduce loss during placement. However, even with allowance for 50% sand loss during placement and prior to capping, there is not a significant difference to filling the scour holes and building the core of the fish ramps is hydraulic dredging, and only capping the material with limestone or other local bedding material once the dredging is complete, prior to placing the granite wearing course and boulders. Therefore, if there is enough sand available in the river, it would be most cost-effective to hydraulically dredge the sand and capping it once the template is achieved.

Note that each method is mutually exclusive with the other methods.


BOULDERS (typ.)
EROSION CONTROL STONE
CAPE FEAR RIVER
LOCK
Concrete Esplanade
SCALE: 1" = 80'
SCALE IN FEET


4" D50 2" DIA STONE
PVI 0+00 EL 23.00
PVI 2+00.00 Elev 13.00
PVI 2+32.00 Elev 5.00
-5.00%
0.00%
-25.00%
PVI 0+18.80 Elev 18.06
PVI 2+00 EL 9.00
PVI 2+16 EL 5.00
PVI 4+60.89 Elev -52.22
CLASS B EROSION CONTROL STONE
Elevation
30
20
10
0
-10
-20
-30
-40
-50
-60
-0+20
0+00
0+50
1+00
1+50
2+00
2+50
3+00
3+50
4+00
4+50
5+00
DAM PROFILE

ASSUMPTIONS FOR COST COMPARISON

Mobilization and demobilization cost added for using small truck-carried dredge is $100,000, for equipment and operator. This is assumed to include non-productive time for equipment and operator.

Placing sand by hydraulic dredging, including markups, is $8.00/CY

Cost of stone material per ton delivered to the site, is as follows:

	L&D #1	L&D #2	L&D #3
Limestone for bedding layer material	$ 20	$ 22	$25
Wearing course or granite material	$ 46	$ 48	$?
Riprap or boulders	$ 30	$ 27	$ 25

Note: Add $4 per Ton if gravel is used for bedding layer rather than limestone.

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1A1: Use Sand Underlayment Placed in GeoBags, Limestone Bedding, and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
MATERIAL, L&D 1				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	43300	$80.66	$3,492,578
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	14500	$135.17	$1,959,965
Boulders	T	1315	$271.00	$356,365
				$0
		Total Deletions		$6,796,993
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
MATERIAL, L&D 1				$0
Sand in GeoBags	CY	17743	$40.00	$709,714
Limestone Bedding	T	12250	$76.00	$931,000
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	14500	$135.17	$1,959,965
Boulders	T	1315	$271.00	$356,365
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$5,045,129
		Net Savings		$1,751,864
		Markups	49.70%	$870,676
		Total Savings		$2,622,540
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1A2: Use Sand Underlayment Placed in GeoBags, Limestone Bedding, and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 2				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	30400	$80.66	$2,452,064
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
		Total Deletions		$5,431,939
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 2				$0
Sand in GeoBags	CY	10371	$40.00	$414,857
Limestone Bedding	T	12250	$76.00	$931,000
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$4,425,732
		Net Savings		$1,006,207
		Markups	49.70%	$500,085
		Total Savings		$1,506,292
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1A3: Use Sand Underlayment Placed in GeoBags, Limestone Bedding, and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 3				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	12000	$80.66	$967,920
Granite Filter	T	6300	$80.66	$508,158
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
		Total Deletions		$3,467,868
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 3				$0
Sand in GeoBags	CY	6497	$40.00	$259,886
Limestone Bedding	T	6300	$76.00	$478,800
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	6300	$80.66	$508,158
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$3,338,634
		Net Savings		$129,234
		Markups	49.70%	$64,229
		Total Savings		$193,464
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1B1: Use Limestone Bedding and Granite Wearing in Lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
MATERIAL, L&D 1				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	43300	$80.66	$3,492,578
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	14500	$135.17	$1,959,965
Boulders	T	1315	$271.00	$356,365
				$0
		Total Deletions		$6,796,993
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
MATERIAL, L&D 1				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	43300	$76.00	$3,290,800
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	14500	$135.17	$1,959,965
Boulders	T	1315	$271.00	$356,365
				$0
				$0
		Total Additions		$6,595,215
		Net Savings		$201,778
		Markups	49.70%	$100,284
		Total Savings		$302,062
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1B2: Use Limestone Bedding and Granite Wearing in Lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 2				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	30400	$80.66	$2,452,064
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
		Total Deletions		$5,431,939
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 2				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	30400	$76.00	$2,310,400
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
				$0
				$0
		Total Additions		$5,290,275
		Net Savings		$141,664
		Markups	49.70%	$70,407
		Total Savings		$212,071
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1B3: Use Limestone Bedding and Granite Wearing in Lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 3				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	12000	$80.66	$967,920
Granite Filter	T	6300	$80.66	$508,158
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
		Total Deletions		$3,467,868
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 3				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	12000	$76.00	$912,000
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	6300	$80.66	$508,158
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
				$0
				$0
		Total Additions		$3,411,948
		Net Savings		$55,920
		Markups	49.70%	$27,792
		Total Savings		$83,712
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1C1: Use Sand Underlayment, with Intermittent Limestone Choker				
Courses, Limestone Bedding Course and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
MATERIAL, L&D 1				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	43300	$80.66	$3,492,578
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	14500	$135.17	$1,959,965
Boulders	T	1315	$271.00	$356,365
				$0
		Total Deletions		$6,796,993
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
MATERIAL, L&D 1				$0
Sand	CY	17743	$8.00	$141,943
Limestone Bedding	T	12250	$76.00	$931,000
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	14500	$135.17	$1,959,965
Boulders	T	1315	$271.00	$356,365
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$4,477,358
		Net Savings		$2,319,635
		Markups	49.70%	$1,152,859
		Total Savings		$3,472,494
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1C2: Use Sand Underlayment, with Intermittent Limestone Choker				
Courses, Limestone Bedding Course and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 2				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	30400	$80.66	$2,452,064
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
		Total Deletions		$5,431,939
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 2				$0
Sand	CY	10371	$8.00	$82,971
Limestone Bedding	T	12250	$76.00	$931,000
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$4,093,846
		Net Savings		$1,338,093
		Markups	49.70%	$665,032
		Total Savings		$2,003,125
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1C3: Use Sand Underlayment, with Intermittent Limestone Choker				
Courses, Limestone Bedding Course and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 3				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	12000	$80.66	$967,920
Granite Filter	T	6300	$80.66	$508,158
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
		Total Deletions		$3,467,868
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 3				$0
Sand	CY	5143	$8.00	$41,143
Limestone Bedding	T	3000	$76.00	$228,000
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	6300	$80.66	$508,158
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$2,869,091
		Net Savings		$598,777
		Markups	49.70%	$297,592
		Total Savings		$896,369
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1D1: Use Sand Underlayment, with Allowance for Sand Losses,				
Limestone Bedding Course and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
MATERIAL, L&D 1				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	43300	$80.66	$3,492,578
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	14500	$135.17	$1,959,965
Boulders	T	1315	$271.00	$356,365
				$0
		Total Deletions		$6,796,993
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
MATERIAL, L&D 1				$0
Sand	CY	26614	$8.00	$212,914
Limestone Bedding	T	12250	$76.00	$931,000
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	14500	$135.17	$1,959,965
Boulders	T	1315	$271.00	$356,365
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$4,548,329
		Net Savings		$2,248,664
		Markups	49.70%	$1,117,586
		Total Savings		$3,366,250
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1D2: Use Sand Underlayment, with Allowance for Sand Losses,				
Limestone Bedding Course and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 2				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	30400	$80.66	$2,452,064
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
		Total Deletions		$5,431,939
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 2				$0
Sand	CY	15557	$8.00	$124,457
Limestone Bedding	T	12250	$76.00	$931,000
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	12250	$80.66	$988,085
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$4,135,332
		Net Savings		$1,296,607
		Markups	49.70%	$644,414
		Total Savings		$1,941,020
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

COST ESTIMATE WORKSHEET				
PROPOSAL NO.: 1D3: Use Sand Underlayment, with Allowance for Sand Losses,				
Limestone Bedding Course and Granite Wearing Course in lieu of All Granite				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 3				$0
Sand	CY	0	$8.00	$0
Limestone Bedding	T	0	$76.00	$0
Granite Bedding	T	12000	$80.66	$967,920
Granite Filter	T	6300	$80.66	$508,158
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
		Total Deletions		$3,467,868
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
				$0
MATERIAL, L&D 3				$0
Sand	CY	4886	$8.00	$39,086
Limestone Bedding	T	6300	$76.00	$478,800
Granite Bedding	T	0	$80.66	$0
Granite Filter	T	6300	$80.66	$508,158
Granite Wearing	T	12500	$135.17	$1,689,625
Boulders	T	1115	$271.00	$302,165
				$0
Additional Mob/Demob for Dredging	LS	1	$100,000.00	$100,000
				$0
		Total Additions		$3,117,834
		Net Savings		$350,034
		Markups	49.70%	$173,967
		Total Savings		$524,001
Markups include: Contingency (25%); Engineering/Design (10%); and Construction Mgmt (8%)				

VALUE ENGINEERING PROPOSAL

PROPOSAL NO:	2 (Spec List Item 18)	PAGE NO: 1 OF 2
DESCRIPTION:	If Lock & Dam #2 (or any other dam) is removed, do not use coffercells	

ORIGINAL DESIGN: The cost estimate for the General Re-evaluation Report includes constructing a cellular sheet pile cofferdam around Lock & Dam #2 in order to be able to perform the demolition work in the dry.

PROPOSED DESIGN: The proposed design is to eliminate the cofferdam and perform the demolition work in the wet using a barge mounted crane, wrecking ball, and clamshell.

ADVANTAGES:

1. Safer for personnel, since it eliminates risk of personnel working below water surface behind the cofferdam.
2. Eliminates risk of cofferdam failure during storms or high flow events.
3. Saves $1.5 Million dollars.
4. Saves time since work can begin immediately and doesn't have to wait for the cofferdam to be constructed.

DISADVANTAGES:

1. Necessitates using larger, more costly equipment, since a longer reach is needed.

JUSTIFICATION:

Performing the demolition in the wet would be less costly and less risky than performing the work in the dry behind a cofferdam. Demolition should be done during seasonal low flow. The water surface could be drawn down by opening the lock gates, after which the lock would function as the diversion channel. Dam #1 is a timber crib with concrete cap and could be easily broken up with a wrecking ball and removed with a clamshell. Dam #2 is a sheet pile structure with rock fill and could be easily removed with a clamshell. Dam #3 is a concrete structure which would require blasting to break up, after which the rubble could be removed with a clamshell.

COST ESTIMATE WORKSHEET				
Proposal # 2 - If Lock & Dam #2 (or any other dam) is removed, do not use coffercells				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
Cellular sheet pile cofferdam	LF	960	1,654	$1,587,600
Pumping	HRS	1000	31.16	$31,164
				$0
				$0
				$0
				$0
				$0
				$0
				$0
				$0
				$0
				$0
		Total Deletions		$1,618,764
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
Remove Concrete Cap 20% added cost	CY	560	35	$19,600
Rem Cribbing & Rock 20% added cost	CY	11000	8	$88,000
				$0
				$0
				$0
				$0
				$0
				$0
				$0
				$0
				$0
				$0
		Total Additions		$107,600
		Net Cost Decrease		$1,511,164
		Mark-ups	0.00%	$0
		Total Cost Decrease		$1,511,164
All Mark-ups included in Unit Prices. Contingency or S&I applied.				

VALUE ENGINEERING PROPOSAL

PROPOSAL NO: 3 (Spec List Item 63) PAGE NO: 1 OF 5
DESCRIPTION: Remove L&D 2; lengthen ramp at L&D 3

ORIGINAL DESIGN: Rock ramp fishways at both L&D 2 and L&D 3.

PROPOSED DESIGN: Remove L&D 2 and construct longer rock ramp fishway at L&D 3 due to the lower tailwater elevation due to removal of L&D 2.

ADVANTAGES:

- L&D 2 has a large scour hole close to the dam and L&D 3 doesn't. It's likely cheaper to do the proposed design since filling the scour hole at L&D 2 is a large cost. The scour hole at L&D 3 is farther downstream.
- Complete fish passage is assured with dam removal. Rock ramps pass fish but it's not known if they are as effective as dam removal.
- USFWS expressed possibility that they would assume responsibility of dam removal.
- There are no existing water intakes in Pool 2 and only one proposed water intake.
- While the Navy prefers keeping L&D 2, removal does not preclude Navy training. It would create shoals for training that they currently don't have.
- Removal creates natural riverine habitat.

DISADVANTAGES:

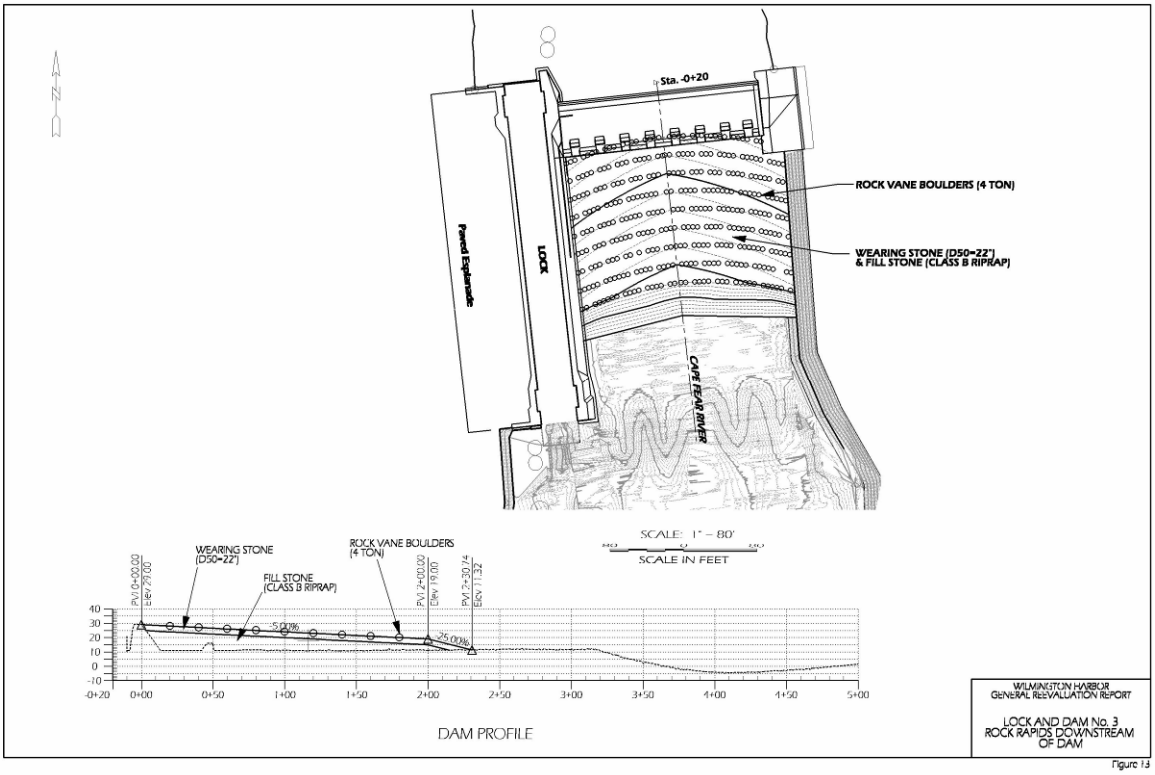
- Minor to moderate bank sloughing is expected upstream. However, there are not a lot of houses near the river between L&D 2 and L&D 3, which minimizes the impact. Slow removal of the dam would likely reduce the amount of sloughing.
- Sediment upstream of dam would be mobilized. However, the amount of sediment impounded by the dam is small and testing has shown that it is clean.
- Removal would limit navigation, but the L&D 2 is not used much (~ 200 recreational lockages per year).
- A longer rock ramp at L&D 3 may be less effective than 2 shorter ramps.
- De-authorization may be difficult and time-consuming. An email from Denise Titus (SAS) to Sharon Haggett (SAW) on February 2, 2006 indicates that the process optimistically typically takes 12 to 24 months.
- Local city and county officials have passed resolutions which states that they would like to keep L&D 2.
- Navy prefers that L&D 2 remains.

JUSTIFICATION:

The assurance of complete fish passage at L&D 2 and the potential cost savings appear to justify investigation of de-authorization. USFWS feels that removal of L&D 2 and one somewhat longer ramp at L&D 3 is better for overall fish passage than having a ramp at L&D 2 and a ramp at L&D 3.

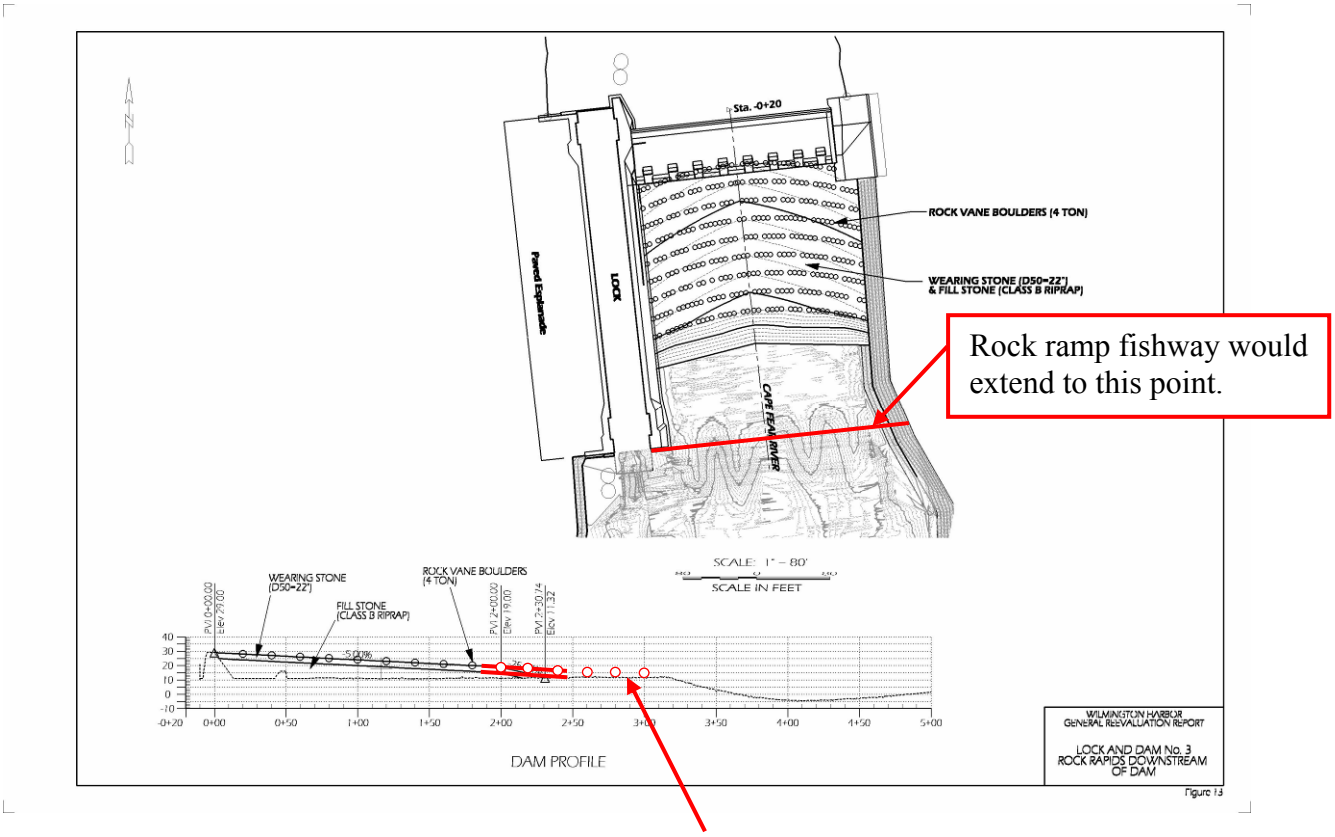
Drawing No. 1

ORIGINAL PLAN



Drawing No. 2

PROPOSED PLAN



It may be possible to place additional boulders on existing erosion protection. Otherwise excavation and placement of granite wearing layer will be needed.

COST ESTIMATE WORKSHEET				
Proposal # 3				
DELETIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
L&D 2 Removal vs. L&D 2 rock ramp				$4,084,003
(cost difference between alternatives)				$0
				$0
				$0
				$0
				$0
				$0
				$0
		Total Deletions		$4,084,003
ADDITIONS				
ITEM	UNITS	QUANTITY	UNIT COST	TOTAL
Granite D50=22" riprap	Tons	3500	$135.17	$473,095
4 ton boulders	Tons	936	$271.00	$253,656
				$0
				$0
				$0
				$0
				$0
				$0
		Total Additions		$726,751
		Net Savings		$3,357,252
		Markups	0.00%	$0
		Total Savings		$3,357,252

PROPOSAL NO:	4 (Spec List Item 31)	PAGE NO: 1 OF 3
DESCRIPTION:	Use Rock Dredged from Turning Basin or Authorized NE Cape Fear River Project to Fill Scour Hole and Build Ramp(s)	

ORIGINAL DESIGN:

Current design calls for the rock ramp fishway to be constructed from granite quarried offsite and trucked to the project site. The only expected effort filling existing scour holes is to the toe of the ramp.

PROPOSED DESIGN:

During the already authorized deepening of the North East Cape Fear River and during excavation of the new or modified turning basin, borings have indicated that rock is expected, as shown on the slide below. It is proposed that the rock be placed on small shallow barges and brought upstream to fill the scour holes at Lock & Dam 1 or Lock & Dam 2. As the holes are filled, any excess rock can be placed on the rock ramp as long as the slope above low flow elevation does not exceed 1V:20H, or along banks adjacent to the site to prevent erosion.

ADVANTAGES:

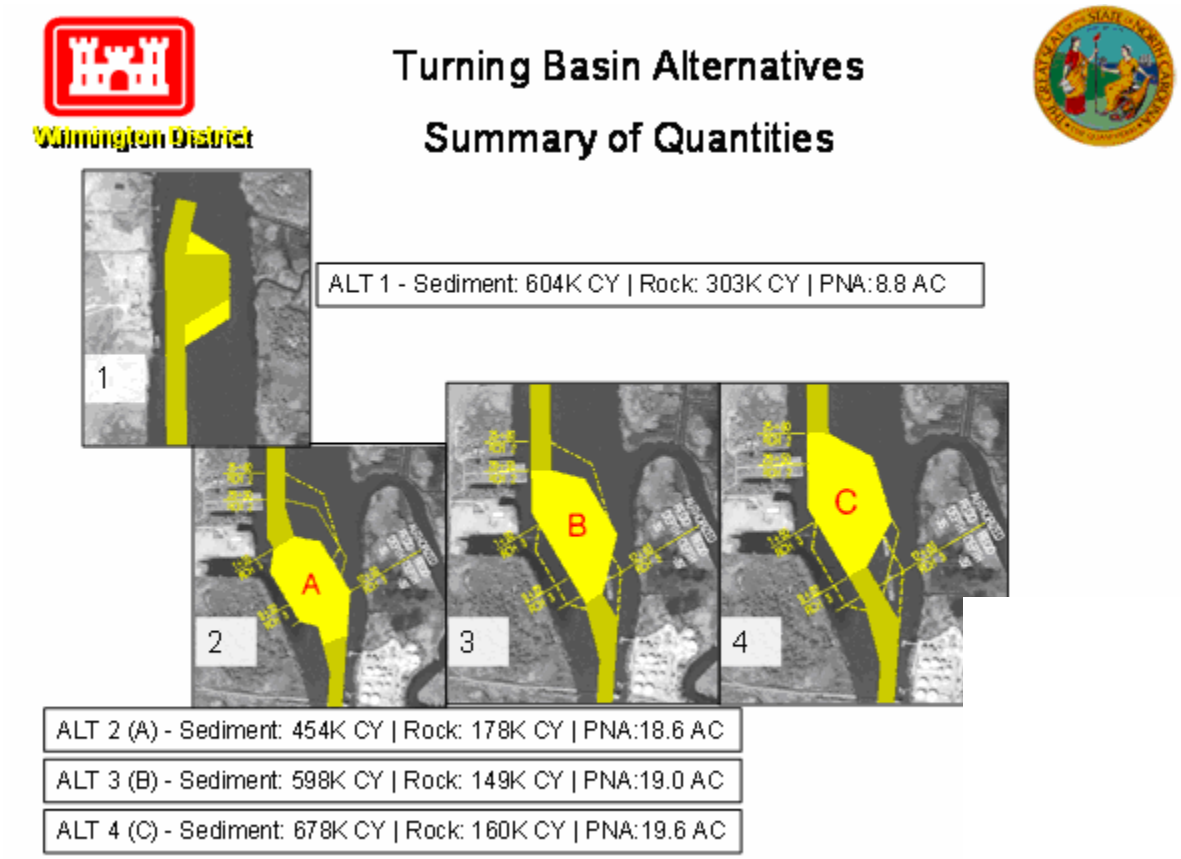
- Saves expense of quarrying
- Provides beneficial use of excavated material

DISADVANTAGES:

- Cost of handling material and transporting it by barge or by truck may be higher than transporting from quarry
- Suitability of rock for wearing surface rather than bedding would need to be determined
- Phasing of excavation may not match when material is needed for fill
- Width, length and draft of water-borne transport through L&D 1 to the L&D 2 scour hole is limited

JUSTIFICATION:

While costs of handling and transporting material the long distance from Wilmington Harbor may render this proposal non-economic, it might be worthwhile to consider stockpiling rock at Eagle Island or Island 12 for future use to be determined. Use of this rock for the fishway rather than requiring the contractor to obtain newly quarried rock might be offered as a contractor option. Insufficient data is readily available on quantities and impact on navigation to perform a meaningful analysis, but this could provide a viable alternative.


Wilmington District
Turning Basin Alternatives
Summary of Quantities
1
ALT 1 - Sediment: 604K CY | Rock: 303K CY | PNA:8.8 AC
A
2
B
3
C
4
ALT 2 (A) - Sediment: 454K CY | Rock: 178K CY | PNA:18.6 AC
ALT 3 (B) - Sediment: 598K CY | Rock: 149K CY | PNA:19.0 AC
ALT 4 (C) - Sediment: 678K CY | Rock: 160K CY | PNA:19.6 AC

VALUE ENGINEERING PROPOSAL

PROPOSAL NO:	5 (Spec List Item 64)	PAGE NO: 1 OF 1
DESCRIPTION:	Investigate tailwater levels for design of rock ramp fishways	

ORIGINAL DESIGN:

Rock ramp fishways were designed assuming 1:20 slope should extend 1 foot below low pool.

PROPOSED DESIGN:

1:20 slope should extend 2 feet below tailwater elevation occurring for minimum flow conditions during fish migration.

ADVANTAGES:

- This may shorten the length of the rock ramp fishway.
- A depth of 2 is less limiting than a depth of 1 foot for fish getting into the fishway.

DISADVANTAGES:

- This change may result in a rock ramp that won't allow fish to pass during very low flows. Maybe this is more important than believed.

JUSTIFICATION:

A review of monthly mean discharges was conducted and discussed with members of the PDT and the USFWS. The minimum discharge during fish migration (March through May) is fairly low (1000 to 1500 cfs). When this proposal was developed, the VE team and PDT members present thought that the minimum discharge during fish migration might be as high as 3000 to 5000 cfs. With it being 1000 cfs to 1500 cfs, the proposal results in the 1:20 slope of the fishways ending at virtually the same elevation as originally proposed. Therefore no change in design is proposed.

Cost impact = $ Zero

VALUE ENGINEERING PROPOSAL

PROPOSAL NO:	6 (Spec List Items 49 and 50)	PAGE NO: 1 OF 2
DESCRIPTION:	Use Dock Area Adjacent to Sigma Marine in lieu of New Turning Basin	

ORIGINAL DESIGN:

Current preferred plan (Alternative A on the aerial photo below) for the turning basin includes excavation on either side of the existing navigation channel, just upstream from the Chemserve terminal and adjacent to the currently unoccupied Sigma Marine facility.

PROPOSED DESIGN:

Use the dock area adjacent to the Sigma Marine facility for turns in lieu of excavating the turning basin in the channel.

ADVANTAGES:

- There are no longer any contaminant issues at the Sigma Marine site
- Using a boat slip for turning would allow other boats to pass
- The boat slip provides emergency harborage
- This may result in less environmental impact than the preferred plan

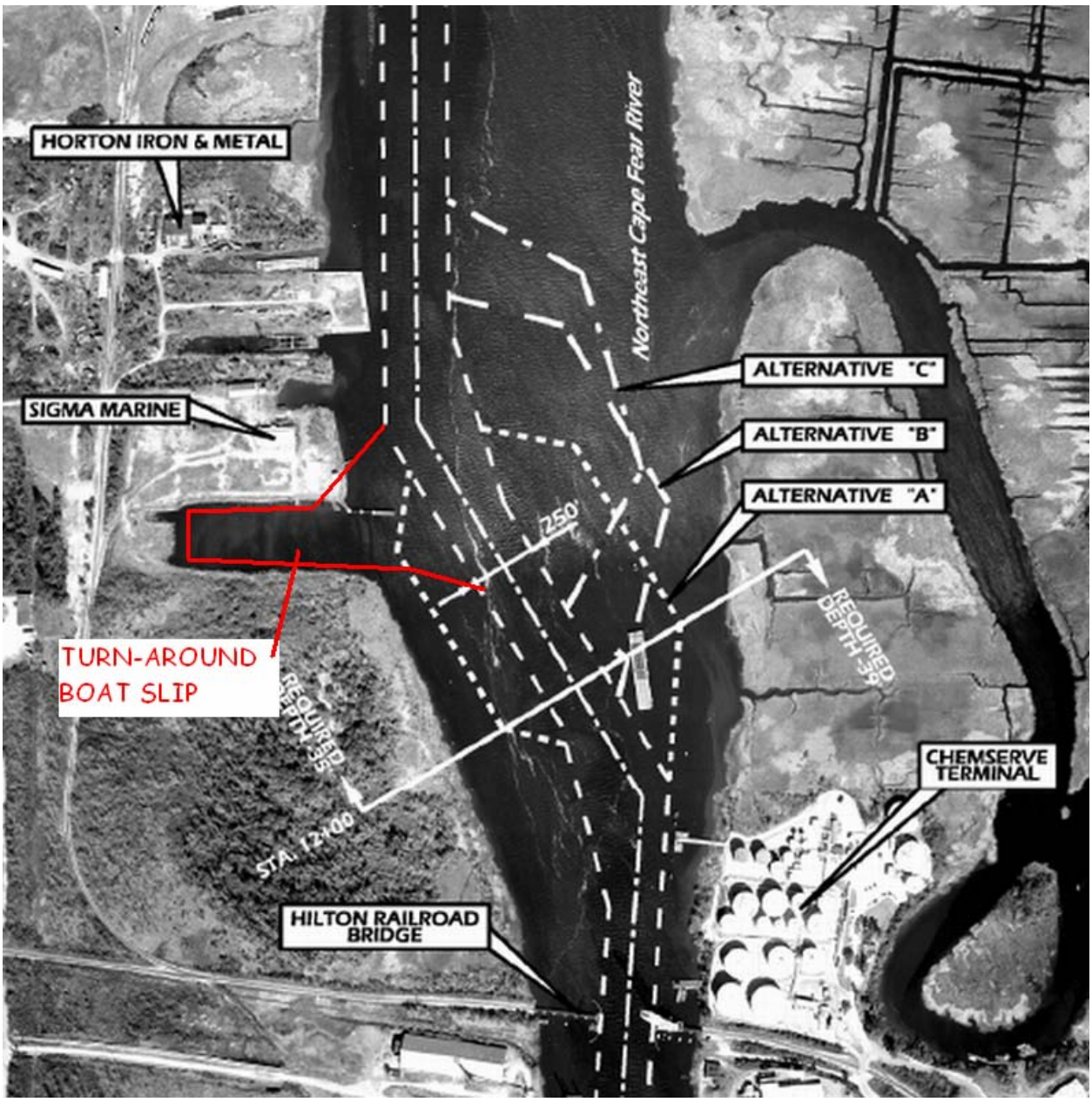
DISADVANTAGES:

- Requires real estate acquisition and upland excavation
- Insufficient data is currently available to determine quantities of rock vs. sediment
- More excavation may be required outside of the vicinity of the channel
- Boat slips tend to silt up, while using a widened channel would be more self-cleaning
- Proposed boat slip turn-around would have to be modeled by ERDC and discussed with the pilots

JUSTIFICATION:

Insufficient data is readily available on quantities and impact on navigation to perform a meaningful analysis, but this could provide a viable alternative.

ALTERNATIVE TURNING BASINS



1. ***Use local limestone or sand fill for rock ramp core and / or scour hole and cap with granite stone. Consider using geobags to contain sand fill (Speculation List Item No. 3:*** Using material other than granite riprap and boulders for the wearing stone and arch vanes is possible, but the following items should be considered in the design:

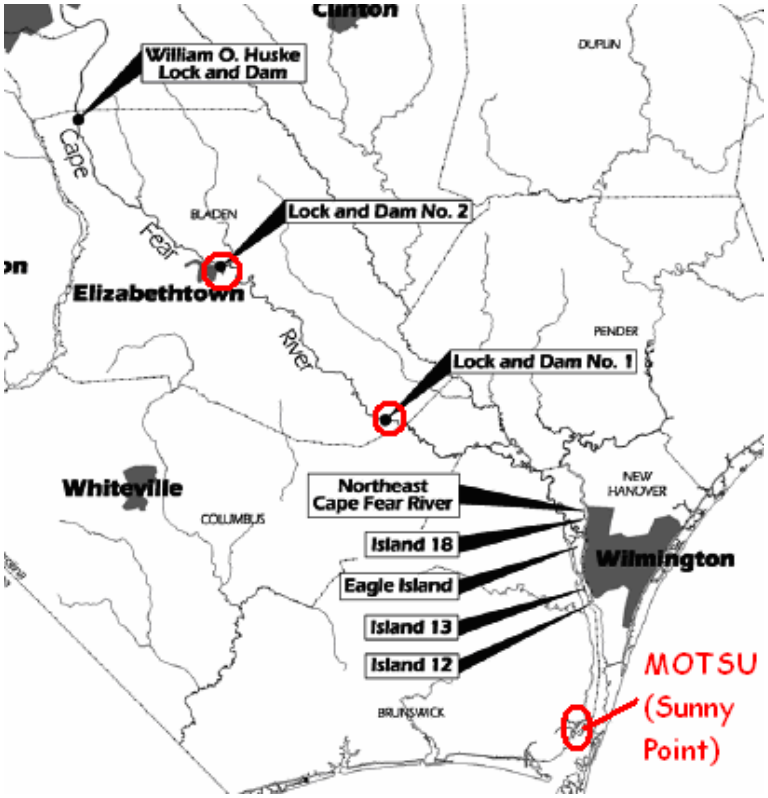
- The top material of the structure must be durable. The existing design calls for granite, which is good.
- With 1 foot of drop over each arch vane, the velocity can be as high as 8 ft/sec if there isn't much roughness through the vanes. Complex shapes are more effective in dissipating energy and providing lower velocity paths through the vanes. This provides more opportunities for fish to traverse the vanes.
- A flexible structure is better able to adjust to settlement without compromising the functionality of the fishway.
- Experience during construction of rock ramp fishways in the Red River of the North basin indicates that the boulder vanes need to be adjusted as they are being built to get the proper head loss across each vane. The design should account for this need to adjust during construction.
- Vane material needs to be large enough to handle forces caused by debris. The existing design calls for 4 ton boulders, which is in line with what's been done on the Red River of the North and has been successful. The large size of material for the vanes should not be compromised.

2. ***Install surface water intake canals and sumps adjacent to Cape Fear River upstream from Lock & Dam #2 in lieu of proposed direct tap at Smithfield (Speculation List Item No 11).*** There is a concern that removing Lock & Dam #2 will lower the water surface in the river and cause problems with a proposed surface water intake for the Smithfield meat packing plant. Consider that the same flow will be available in the river regardless of whether or not the dam is removed. The intake may need to be reconfigured to provide an intake canal or sump in order to assure that the suction end of the pipe is low enough to remain submerged during low water.

3. ***Examine whether there would be contaminant issues at Sigma Marine Site (Speculation List Item No 50).*** According the NC Aquifer Protection Section (Sam Watson), the contamination at the Sigma Marine Site has been cleaned up which has been confirmed by groundwater sampling. The site was closed out in 2000.

4. ***Examine whether placing rock along lock wall associated with rock ramp fishway is a problem (Speculation List Item No 17).*** The lock wall should be checked to see if the lateral force exerted by the rock in the fishway ramp will cause an instability or will overstress the concrete. The unit force from the rock should be no more than that of granular soil, which would increase at a rate of Gamma x H x Ka =.33 x 125 x H = 42 psf. This is less than the pressure from water inside the lock = 62.4 psf, but acts in the opposite direction.

5. ***Relocate rock from sunken barges near Sunny Point to use for rock ramp (Spec Item # 41):*** While costs of handling and transporting material the long distance from Wilmington Harbor may render this idea non-economic, it might be worthwhile to consider stockpiling rock at Eagle Island or Island 12 for future use to be determined. Use of this rock for the fishway rather than requiring the contractor to obtain newly quarried rock might be offered as a contractor option.



6. ***Excess L&D 2 and/or L&D 3 as Corps projects, and transfer to another agency or to the private sector to modify and operate (Spec Item # 44 and 65):*** A couple of speculation ideas were floated to either transfer the property for L&D 2 to Navy as training facility, or to offer L&D 2 and 3 to private sector in exchange for agreement to construct fish passages. To do any of these measures, the project would first have to be deauthorized by Congress. It would then be screened with other DOD agencies within a limited mileage of the property. If there is no interest, a report of excess would be forwarded to the General Services Administration (GSA) for disposal. GSA would screen the property for interest by other Federal agencies and if there were no takers, it would be offered to state and local agencies. If there is still no interest, GSA would sell by public action or sealed bid as a single parcel or divided into marketable portions.

7. ***Allow water supply users to access groundwater during low flow situations when they would have used water from river (Speculation List Item No. 48):*** The proposed water intake between L&D 2 and L&D 3 is for a user that is being directed to move away from using groundwater. Removal of L&D 2 might make it more difficult for that user to get water during periods of drought. To address this concern it may make sense to allow this user to again access groundwater during a drought. It is unknown whether or not the regulatory agency for water usage would allow this type of an exception. Obviously, water usage would have to be investigated in detail if removal of L&D 2 is recommended and it is determined that the lower river level would compromise the proposed water intake.

8. ***Does Military Ocean Terminal Sunny Point (MOTSU) need to earn mitigation credits by building fish passage at L&D 2 and/or 3? (Speculation List Item No 55).*** MOTSU plans to initiate a study to optimize their ship channels and basins and has requested the Wilmington District to provide them an estimate of the District's costs to perform the study. Mitigation will probably be required for some aspects related to the study, but the project is in too early a stage to accurately estimate the need.

9. ***Use standard gradation of limestone (Speculation List Item No. 62:*** Using a readily available gradation of limestone (a gradation commonly produced by a quarry) would likely reduce the cost of getting the limestone. A thicker layer of a readily available gradation may perform satisfactorily and cost less than a thinner layer of a custom gradation. Ultimately an engineer on the PDT will have to come up with the design for any limestone below the granite top layer of the rock ramp fishway.

10. ***Consider allowing the State's Environmental Enhancement Program to construct fish passage at Lock and Dam #2 or #3 using mitigation credits to remove the structures and / or construct rock fishway ramps (Speculation List Item No. P2.*** Consideration should be given to partnering with the State's Environmental Enhancement Program to promote environmental restoration of the Cape Fear River and allow the program to remove the structure at L&D #2 or construct fishway ramps at L&D #2 or #3 using funds obtained for environmental mitigation credit at no cost to the Corps. By reducing the requirement for Federal dollars, this would allow for more environmental restoration work to be accomplished. This would require deauthorization of a Federal Navigation Project which could take a lot of time to accomplish but the benefits would be less Federal expense for mitigation and the end of O&M of the structures. The State and the environment would benefit by allowing greater progress to be made toward improving fish passage up river.

11. ***If Lock and Dam #2 is removed, use the rock and concrete as base material for the rock ramp at Locks and Dams #1 & # 3.*** The project would need to be deauthorized by Congress before Lock and Dam #2 could be removed. Placing a rock ramp at #2 & #3 would not require deauthorization if the lock chambers are not blocked. Therefore fish passage at Locks and Dams #1 & #3 would likely be completed before #2 could be removed. However if the timing is right and the costs of hauling and depositing the material from #2 is cheaper than hauling material from other sources, this alternative should be considered.

SUPPORTING DOCUMENTS

CONTACT DIRECTORY

NAME	ORGANIZATION	TELEPHONE
Bill Easley	OVEST (interdisciplinary / civil engr) william.s.easley@usace.army.mil	843-329-8152
Rick Lambert	OVEST (civil / structural engr) richard.d.lambert@usace.army.mil	843-329-8063
Aaron Buesing	CEMVP-EC-H (hydraulics engr) aaron.w.buesing@usace.army.mil	651-290-5627
Don Carmen	SAW-TS-EE (VE Officer) charles.d.carmen@usace.army.mil	910-251-4656
John Caldwell	SAW-TS-EE (Cost engr) john.c.caldwell@usace.army.mil	910-251-4586
Sharon Haggett	SAW-PM-C (Project Mgr) sharon.f.hagggett@usace.army.mil	910-251-4441
Tom Child	SAW-TS-EG (Civil engr) thomas.child@usace.army.mil	910-251-4708
Jimmy Hargrove	SAW-OPD-N (Civil engr) james.t.hargrove@usace.army.mil	910-251-4879
Frank Yelverton	SAW-TS-PP (Biologist) frank.yelverton@usace.army.mil	910-251-4640
Jim Mullins	SAW-TS-CW (Civil Engr / Cost) james.M.Mullins@usace.army.mil	910-251-4816
Craig Shoe	SAW-OP-LJ (Supv. Park Ranger) craig.shoe@usace.army.mil	919-542-4501 x 23
Phil Edge	SAW-OP-LC (Maint. mechanic) phil.edge@usace.army.mil	910-483-7746

NAME	ORGANIZATION	TELEPHONE
Keith W. Ashley	NC Water Resources Commission ashleykw@intrstau.net	
Steve Everhart	NC Water Resources Commission steve.everhart@ncwildlife.org	970-796-7217
Tom Rachels	NC Water Resources Commission rachelsrt@earthlink.net	
Layton Bedsole	NC State Ports Authority layton_bedsole@ncports.com	
Mike Wicker	US Fish & Wildlife Service mike_wicker@fws.gov	
Elliott Roughen	SAS-EN-ET (Cost engr) elliott.k.roughen@usace.army.mil	912-652-5663
Janelle Mavis	SAW-TS-EE janelle.m.mavis@usace.army.mil	910-251-4916
Frank Reynolds	SAW-TS-PS frank.r.reynolds@usace.army.mil	910-251-4773
Ray Batchelor	SAW-TS-PS (Civil engr) ray.batchelor@usace.army.mil	910-251-4729
Ed Dunlop	SAW-TS-ED (Structural engr) edward.f.dunlop@usace.army.mil	910-251-4992

ISSUES AND CONCERENS

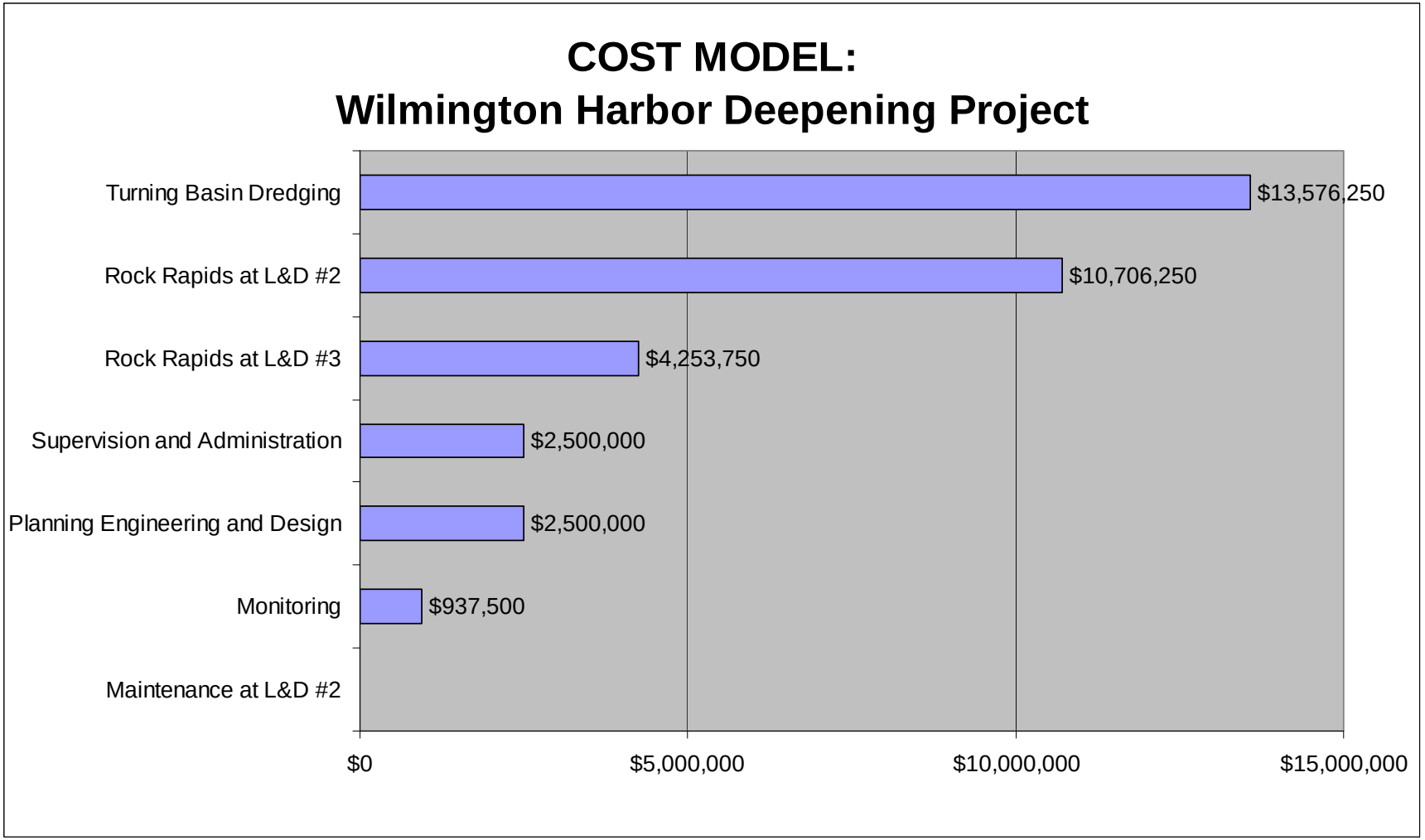
ISSUES AND CONCERNS

Turning Basin and Approach channel
Maneuverability of ships
Safety of ships
Safety of bridges
Distance to turn around
Chemserve
Primary nursery area
Rock excavation
Blasting safety
Shoaling
Turbidity
Cost
Disposal site
Fish mortality
Limits of authorization
Quality of material being placed into former turning basin
Maintaining navigation channel
Utilities

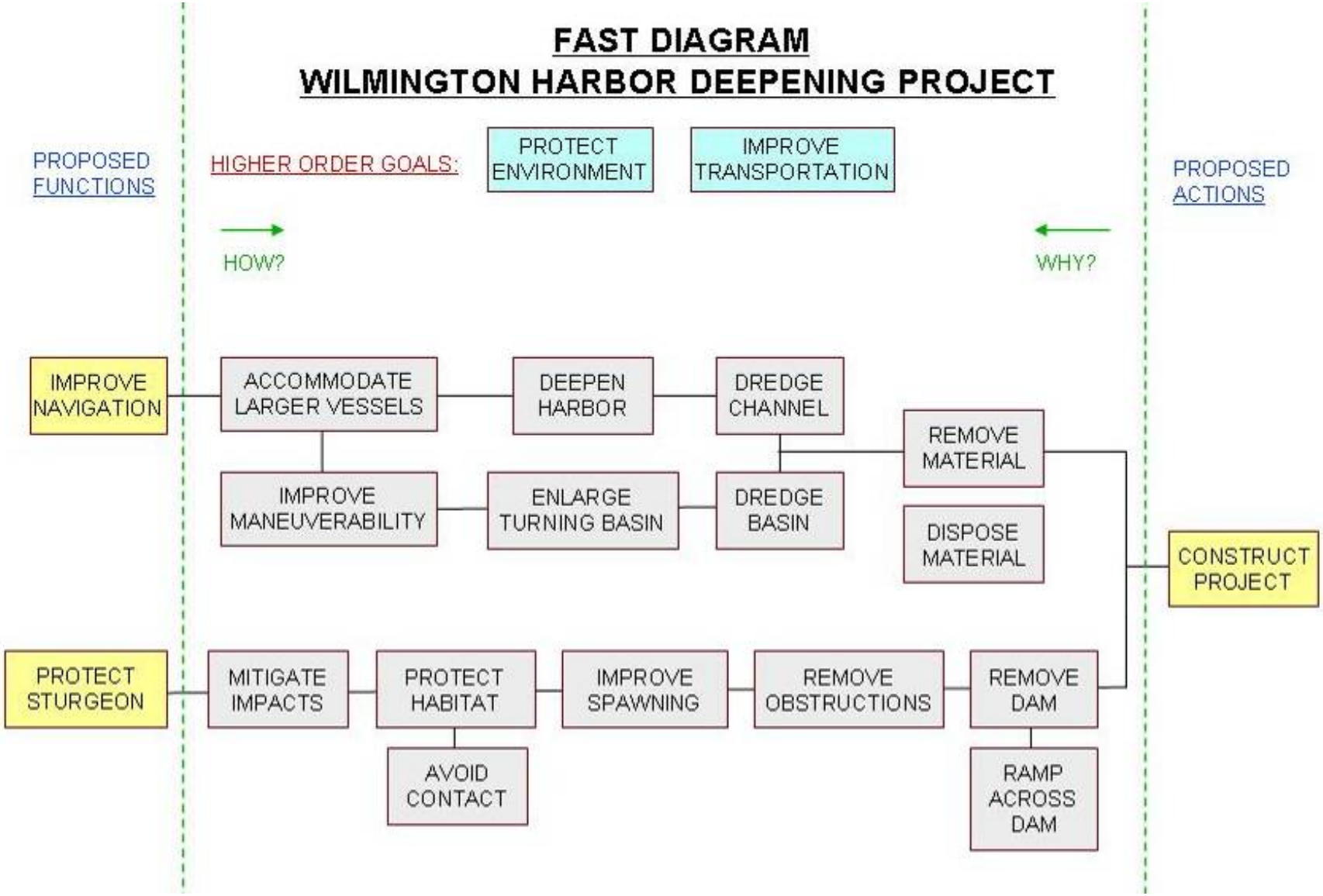
Mitigation Sites (other than Fish Passage)
Cost
Beneficial use of excavated material on channel adjacent to Island 12
Quantity of excavation required vs. habitat units

Fish Passage
Water supply impact
Water level upstream and downstream
Slope
Cost
Rock source
High flow passage
Low flow passage
Proximity to scour holes
Coordination with agencies and users
Real estate
Dam stability
Lock stability
Maintenance
Scouring and deposition

COST MODEL


COST MODEL:
Wilmington Harbor Deepening Project
Turning Basin Dredging
Rock Rapids at L&D #2
Rock Rapids at L&D #3
Supervision and Administration
Planning Engineering and Design
Monitoring
Maintenance at L&D #2
$13,576,250
$10,706,250
$4,253,750
$2,500,000
$2,500,000
$937,500
$0
$5,000,000
$10,000,000
$15,000,000

FUNCTION ANALYSIS SYSTEM TECHNIQUE (FAST) DIAGRAM


FAST DIAGRAM
WILMINGTON HARBOR DEEPENING PROJECT
PROPOSED FUNCTIONS
HIGHER ORDER GOALS:
PROTECT ENVIRONMENT
IMPROVE TRANSPORTATION
PROPOSED ACTIONS
HOW?
WHY?
IMPROVE NAVIGATION
ACCOMMODATE LARGER VESSELS
DEEPEN HARBOR
DREDGE CHANNEL
REMOVE MATERIAL
IMPROVE MANEUVERABILITY
ENLARGE TURNING BASIN
DREDGE BASIN
DISPOSE MATERIAL
CONSTRUCT PROJECT
PROTECT STURGEON
MITIGATE IMPACTS
PROTECT HABITAT
IMPROVE SPAWNING
REMOVE OBSTRUCTIONS
REMOVE DAM
AVOID CONTACT
RAMP ACROSS DAM

SPECULATION LIST

	#	Description
X	1	Use local sand for rock ramp, with geotextile filter, and cap with rock
P	2	Use geocontainers filled with local sand for rock ramp, and cap with rock
C	3	Allow concrete shapes or rubble to cap top of ramp and scour hole in lieu of rock
X	4	Use styrofoam to build up ramp
X	5	Use hollow concrete ramp
X	6	Improve Hilton Railroad Bridge in lieu of turning basin
X	7	Relocate railroad; remove bridge
X	8	Construct loop using Smith Creek
P	9	Offer option to Environmental Enhancement Program to do fish passage at L&D 2 and 3 for mitigation credits
X	10	Remove L&D 2; construct weir to maintain water level for upstream water users
C	11	Install surface water intake canals and sumps adjacent to Cape Fear River upstream from L&D 2 in lieu of proposed direct tap at Smithfield
X	12	Evaluate mitigation benefits for creating aquatic species access to wetland north of US 17
C	13	Compare fish passage for rock ramps at L&D 2 and 3, vs. removing L&D 2 and building much longer ramps at L&D 3 to accommodate lower water level
X	14	Build fish ladder across entire channel in lieu of rock ramps
X	15	Build fish ladder channeling entire flow which currently goes over dam
X	16	Check whether total of mitigation credits for Alligator Creek, Island 12 and area north of US 17 will meet 9.8 units for less than cost of fish ramps
C	17	Examine whether placing rock along lock wall associated with rock ramp is a problem
P	18	If L&D 2 (or any other dams) is removed, do not use coffercells
X	19	Use batching plant onsite to construct concrete shapes in lieu of transporting rock
BD	20	Optimize turning basin dimensions
X	21	Relocate Chemserve to Exxon terminal downstream
X	22	Divert full flow of river through navigational channel during spawning run

X	**23**	**Put grade control structures downstream from L&D 2 to dissipate head differential at L&D 2**
X	**24**	**Use sheet pile weirs 20' apart with 1-foot top elevation difference between them**
X	**25**	**Use steel sheet pile in lieu of concrete sheet pile for weirs**
X	**26**	**Use FRP composite sheet pile for weirs**
X	**27**	**Knock down dam halfway, add elevation control structures between L&D 2 and 3**
BD	**28**	**Request Navy training funds to pay for keeping L&D 2**
BD	**29**	**If L&D 2 is removed, design proposed water intakes to accommodate run of river**
BD	**30**	**Use material from Eagle Island to fill scour holes**
P	**31**	**Use rock dredged from turning basin or authorized NE Cape Fear River project to fill scour hole**
BD	**32**	**Build fish passage before or concurrent with excavation of turning basin**
C	**33**	**Approach Fort Bragg for fill material for scour hole**
X	**34**	**Approach Fort Bragg for hired labor as training exercise**
X	**35**	**Use Chinooks from Fort Bragg to place rock**
C	**36**	**Approach Fort Bragg to remove L&D 2 as training exercise**
X	**37**	**Reconfigure new turning basin (Alt. 3) to smooth out notch on northwest corner**
X	**38**	**Relocate dock for Chemserve to downstream from railroad bridge**
X	**39**	**Relocate manifold from Chemserve to location adjacent to Bennett Brothers Yachts**
X	**40**	**Sink rock barges into scour holes**
C	**41**	**Relocate rock from sunken barges near Sunny Point to use for rock ramp**
X	**42**	**Incorporate whitewater course into rock ramp to stimulate economy**
X	**43**	**Incorporate fishing area into fish passage design**
C	**44**	**Offer L&D 2 and 3 to private sector in exchange for agreement to construct fish passage**
BD	**45**	**Do nothing in turning basin. If pilots can use current configuration, why change it?**

X	**46**	**Use jets to generate eddy to turn ships**
X	**47**	**Build reservoirs for water supply storage, and remove L&D 1 and 3**
C	**48**	**Allow water supply users to access groundwater during low flow situations when they would have used water from river**
X	**49**	**Use Sigma Marine for turns in lieu of turning basin**
X	**50**	**Examine whether there would be contaminant issues at Sigma Marine site**
X	**51**	**Use hydraulic dredging to move material directly from Sigma Marine site to fill existing turning basin**
P/9	**52**	**Add fish passage to L&D 3 for mitigation for turning basin, and offer Environmental Enhancement Program to remove L&D 2 for mitigation credits**
P/9	**53**	**Add fish passage to L&D 3 for mitigation for turning basin, and offer Environmental Enhancement Program to build rock ramp at L&D 2 for mitigation credits**
X	**54**	**Explore whether Navy needs to earn mitigation credits**
C	**55**	**Does MOTSU Ocean Terminal need to earn mitigation credits by building fish passage at L&D 2 and/or 3?**
P	**56**	**While pumping out shoals in front of lock chamber, discharge sand into scour hole in lieu of pumping onto banks**
P/59	**57**	**Use local limestone for rock ramp core capped with granite**
X	**58**	**Use sand for rock ramp core capped with granite, without filter**
P/57	**59**	**Use local limestone to fill scour hole capped with granite**
P	**60**	**Use sand to fill scour hole capped with granite, without filter**
P	**61**	**Use sand to fill scour hole. Cover hole and build ramp with limestone layer, capped with granite**
C	**62**	**Use standard gradation of limestone**
P	**63**	**Remove L&D 2; lengthen ramp at L&D 3**
P	**64**	**Design fish passage rock ramps based on 1,000 - 3,000 cfs (low seasonal average for spawning season (1 Mar - 1 Jun) over ten years) rather than no flow elevations**
C	**65**	**Transfer property for L&D 2 to Navy as training facility**
X	**=**	**Idea not to be developed**
P	**=**	**Proposal**
C	**=**	**Comment**

BD	**=**	**Being Done**

END

Appendix 8

ENGINEERING CONSIDERATIONS

TABLE OF CONTENTS

I. Cape Fear Lock and Dams Description
   A. Lock & Dam No. 1
   B. Lock & Dam No. 2
   C. William O. Huske Lock & Dam

II. Alternatives Considered for Fish Passage
   A. Rock Rapids Downstream of Dams
   B. Rock Rapids Downstream of Dams with Dam Crest Lowered w/ Reduced Water Supply Capacity
   C. Rock Rapids Upstream of Dams
   D. Dams Lowered to Pass Fish w/o Rock Rapids
   E. Short Diversion Channel
   F. Nature Like By-Pass Channel (Lock & Dam No. 1)
   G. Rehab of Lock and Dams
   H. Remove Lock & Dams

FIGURES

Figure 1 - CAPE FEAR RIVER BASIN
Figure 2 - STUDY AREA
Figure 3 – LOCK AND DAM NO. 1
Figure 4 – LOCK AND DAM NO. 2
Figure 5 – LOCK AND DAM NO. 3
Figure 6 – LOCK AND DAM NO. 1 ROCK RAPIDS DOWNSTREAM OF DAM AND INSIDE LOCK
Figure 7 - LOCK AND DAM NO. 1 ROCK RAPIDS DOWNSTREAM OF DAM
Figure 8 - LOCK AND DAM NO. 1 ROCK RAPIDS IN LOCK
Figure 9 - LOCK AND DAM NO. 2 ROCK RAPIDS DOWNSTREAM OF DAM AND INSIDE LOCK
Figure 10 - LOCK AND DAM NO. 2 ROCK RAPIDS DOWNSTREAM OF DAM
Figure 11 - LOCK AND DAM NO. 2 ROCK RAPIDS IN LOCK
Figure 12 - LOCK AND DAM NO. 3 ROCK RAPIDS DOWNSTREAM OF DAM AND INSIDE LOCK
Figure 13 - LOCK AND DAM NO. 3 ROCK RAPIDS DOWNSTREAM OF DAM
Figure 14 - LOCK AND DAM NO. 3 ROCK RAPIDS IN LOCK
Figure 15 – LOCK AND DAM NO. 1 SHORT DIVERSION CHANNEL
Figure 15A NATURE-LIKE BYPASS CHANNEL, CAPE FEAR RIVER, LOCK & DAM NO. 1, SITE PLAN
Figure 16 - LOCK AND DAM NO. 2 SHORT DIVERSION CHANNEL
Figure 17 - LOCK AND DAM NO. 3 SHORT DIVERSION CHANNEL
Figure 18 - LOCK AND DAM NO. 3 SHORT DIVERSION CHANNEL - PROFILE
Figure 19 LOCK & DAM NO. 1 PROFILES
Figure 20 LOCK & DAM NO. 2, PROFILES
Figure 21 LOCK & DAM NO. 3, PROFILES

I. Cape Fear Lock and Dams Description

A. Lock & Dam No. 1

a. Location. Lock and Dam No. 1 is located at Kings Bluff in Bladen County, North Carolina. The project site is approximately 39 river miles from Wilmington, North Carolina and 76 river miles from Fayetteville, North Carolina. It is easily accessible from North Carolina Highway 87, approximately 7 roadway miles northwest of Riegelwood, North Carolina. The National Inventory Number for this dam is NC00182 and the hazard classification is low. See Figure 1 CAPE FEAR RIVER BASIN, and Figure 2 STUDY AREA for location of the Lock and Dam.

b. General. It is the first in a series of three locks and dams on the Cape Fear River above Wilmington, North Carolina and provides an 8-foot navigation channel from Wilmington to Fayetteville, North Carolina. Lock and Dam No. 1 currently has an 11-foot lift with a lock chamber measuring 40 feet by 200 feet. The top of the lock gates and walls are at elevation +21.0 feet, National Geodetic Vertical Datum, 1929 (NGVD 29). The dam is 275 feet long with its crest at elevation +11.0 feet, NGVD 29. A nonfunctioning fish ladder exists on the opposite abutment from the lock. Another fish ladder was installed on top of the dam next to the lock chamber wall, but it is also nonfunctioning. Lockage has been mainly recreation vessels and anadromous fish in the last 5-years with no tugs or barges. Fish lockage is done all year with 3 daily from early March to late May and once a day during the rest of the year. Nine timber dikes upstream of the left abutment (facing downstream) help maintain the pool for navigation purposes and prevent it from escaping around the left abutment during normal flows. See Figure 3 – LOCK AND DAM NO. 1 for a plan view and cross sections of the lock and dam.

c. Construction and Remedial Measures History.

1) The lock and dam was constructed in 1915 with the lock walls made of gravity concrete founded on timber piles and the dam a rock fill timber crib construction. The lock chamber floor was constructed of tremie concrete.
2) In 1935 the lock and dam was raised 3 feet in order to maintain the navigation channel depth between Lock and Dam Nos. 1 and 2 and to reduce the annual dredging requirements. This work included raising the esplanade, lock walls and dam, adding a fish ladder and adding 3 feet to the gates. Raising the dam consisted of constructing a concrete apron that capped the rock filled timber crib dam and constructing a concrete mattress below the dam to protect it from scour and undermining.
3) In 1937 a fender and mooring system was built for the downstream portion of the lock and in 1939 fenders and mooring piles for the upstream portion.
4) In 1940 timber pile dams were constructed upstream of the left abutment in order to help maintain the pool during low flows.
5) In 1953 steel sheet piles were used to build circular cells to act as guide walls for the downstream land wall and upstream river wall. In 1958 a fender and mooring system was built upstream of the lock.
6) In 1961 a major rehabilitation of the dam included replacement of the 12-inch thick concrete spillway apron.

7) In 1980 the wooden sealing system of the miter gates including the sill and miter seals was replaced by a neoprene sealing system and the gates completely refurbished except for the pintle and pintle bushings.
8) In 1986 three new timber dikes were constructed , four existing dikes repaired and deteriorated lock wall concrete replaced.
9) In 1991 three riprap failure areas downstream of the lock were repaired, a cavity under the esplanade filled with gravel and concrete, and holes drilled in the esplanade wall.
10) In 1992 the miter gates were refurbished including replacing existing seals and seal hardware, replacing damaged valves and valve rods, cleaning and painting the gates, and installation of a silt flushing system.
11) In 1999 a Mobile District drill crew investigated settlement of the esplanade at the downstream end by drilling 4 holes through the esplanade.
12) Since the Periodic Inspection (PI) in 2000 the following was accomplished:

    i. in April 2002 Zapata Engineering prepared a Geotechnical Report for a Nature-Like Bypass Channel; in July 2003 Zapata Engineering prepared a supplement to the Geotechnical Report for a Nature-Like Bypass Channel
    ii. in November 2004 a survey was conducted downstream of the lock and dam
    iii. in 2004 a substantial portion of the existing timber quoins at each gate were replaced with nylon quoins Existing quoins at the bottom of each gate (about 3 to 4 LF) were in good condition and not replaced because the bottom bolts were not accessible and could not be removed.

B. Lock & Dam No. 2

a. Location. Lock and Dam No. 2 is located on the Cape Fear River at Browns Landing in Bladen County, North Carolina. The project site is approximately 72 river miles from Wilmington and 43 river miles from Fayetteville. It is easily accessible from North Carolina Highway 87, approximately 2 miles southeast of Elizabethtown, North Carolina. The National Inventory Number for this dam is NC00205 and the hazard classification is low.

b. General. It is the second in a series of three locks and dams on the Cape Fear River above Wilmington, North Carolina and provides an 8-foot navigation channel from Wilmington to Fayetteville, North Carolina. Lock and Dam No. 2 currently has an 8-foot lift with a lock chamber measuring 40 feet by 200 feet. The top of the lock gates and walls is at elevation +34.0 feet, NGVD 29. The dam is 200 feet long with its crest at elevation +20.0 feet, NGVD 29. A nonfunctioning fish ladder exists contiguous to the river wall. Lockage has been mainly recreation vessels and anadromous fish in the last 5-years with no tugs or barges. Fish lockage is done all year with 3 daily from early March to late May and once a day during the rest of the year. See Figure 4 – LOCK AND DAM NO. 2 for a plan view and cross sections of the lock and dam.

c. Construction and Remedial Measures History.

1) Lock and Dam No. 2 was constructed in 1918 with the lock walls made of gravity concrete founded on timber piles and the dam made of rock with an impervious barrier of steel piling and clay fill.
2) In 1934 a major construction effort was made to improve the navigation channel to Fayetteville, North Carolina. It included increasing the project height at Lock and Dam No. 1 which raised the downstream pool of Lock and Dam No. 2 by 3 feet. It also included making modifications to Lock and Dam No. 2 as follows:
    i. a silt barrier upstream of the head gate and downstream of the tailgate to keep sand from building up around the gates
    ii. a 1-foot-thick layer of concrete added to the lock floor
    iii. an extension to the base of the river side lock wall by adding additional concrete
    iv. and construction of a fish ladder.
3) In 1937 a fender and mooring system was built for the downstream portion of the lock and in 1939 fenders and mooring piles for the upstream portion.
4) In 1953 steel sheet piles were used to build circular cells to act as guide walls for the downstream land wall and upstream river wall.
5) In 1957 the concrete esplanade was replaced with new concrete.
6) In 1960 structural and mechanical repairs were made to the lock gates.
7) In 1980 the wooden sealing system of the miter gates including the sill and miter seals was replaced by a neoprene sealing system and the gates completely refurbished except for the pintle and pintle bushings.
8) In 1986 deteriorated lock wall concrete was replaced.
9) In 1991 approximately 578 linear feet of the upstream right bank was replaced with new riprap.
10) In 1993 the miter gates were refurbished including replacing existing seals and seal hardware, replacing damaged valves and valve rods, cleaning and painting the gates, and installation of a silt flushing system.
11) Since the last PI in 2000 the following was accomplished: in 2004 a substantial portion of the existing timber quoins at each gate were replaced with nylon quoins. Existing quoins at the bottom of the downstream gates (about 3 to 4 LF each) were in good condition and not replaced because the bottom bolts were not accessible and could not be removed. Existing quoins at the bottom of the upstream gates (about 9 to 10 LF each) were in good condition and not replaced because they matched the new sections.

C. William O. Huske Lock & Dam (Lock and Dam No. 3)

a. Location. Wm. O. Huske Lock and Dam is located on the Cape Fear River at Tolars Landing in Bladen County, North Carolina. The project site is approximately 95 river miles from Wilmington and 20 river miles from Fayetteville. It is easily accessible from North Carolina Highway 87, approximately 9 roadway miles north of Tar Heel, North Carolina. The National Inventory Number for this dam is NC00206 and the hazard classification is low.

b. General. It is the third in a series of three locks and dams on the Cape Fear River above Wilmington, North Carolina and provides an 8-foot navigation channel from Wilmington to

Fayetteville, North Carolina. The lock at Wm. O. Huske currently has a 9-foot lift with a lock chamber measuring 40 feet by 300 feet. The top of the lock gates and walls is at elevation +39.0 feet, NGVD 29. The dam is 220 feet long with its crest at elevation +29.0 feet, NGVD 29. A nonfunctioning fish ladder exists on the opposite abutment from the lock. Lockage has been mainly recreation vessels and anadromous fish in the last 5-years with no tugs or barges. Fish lockage is done all year with 3 daily from early March to late May and once a day during the rest of the year. See Figure 5 – LOCK AND DAM NO. 3 for a plan view and cross sections.

c. Construction and Remedial Measures History.

1) Wm. O. Huske Lock and Dam was constructed in 1935 with the lock walls and overflow dam made of gravity concrete founded on steel piles and the lock chamber floor constructed of concrete panels and struts. Additional construction work included a fish ladder on the left abutment, concrete energy dissipaters, a riprap mattress below the dam, riprap protection on both abutments and an esplanade.
2) In 1937 a fender and mooring system was built for the downstream portion of the lock and in 1939 fenders and mooring piles for the upstream portion.
3) In 1940 additional riprap protection was placed downstream of the dam in the riverbed.
4) In 1946 a new gutter and drop inlet was installed in the esplanade for drainage.
5) In 1954 steel sheet piles were used to build circular cells to act as guide walls for the downstream land wall and upstream river wall.
6) In 1962 the lock gates were rehabilitated to include structural repairs and repairs to the quoin and miter posts.
7) In 1980 the wooden sealing system of the miter gates including the sill and miter seals was replaced by a neoprene sealing system and the gates completely refurbished except for the pintle and pintle bushings.
8) In 1994 the miter gates were refurbished including replacing existing seals and seal hardware, replacing damaged valves and valve rods, cleaning and painting the gates, and installation of a silt flushing system.
9) Since the PI in 2000 the following was accomplished: in 2004 a substantial portion of the existing timber quoins at each gate were replaced with nylon quoins Existing quoins at the bottom of the downstream gates (about 15 LF each) were in good condition and not replaced. Existing quoins at the bottom of the upstream gates (about 19 to 21 LF each) were in good condition and not replaced.

II. Alternatives Considered for Fish Passage

A. Rock Rapids Downstream side of Dams

The rock rapids ramps will be constructed of bedding stone or suitable fill material, wearing stone and boulder vanes. The bedding stone may consist of aggregate or debris (demolished concrete or masonry, etc.) of adequate size so that it will not be washed downstream by the river current during construction. Wearing stone shall consist of stone aggregate with an average diameter of 22 inches (D_{50} = 22" calculation based on Corps of Engineers Hydraulic Design Criteria, Sheet 712-1). The thickness of the wearing stone shall be 4 feet. Boulders shall be 4 tons in weight and placed in arch vanes. The arch vanes shall be spaced 20 feet horizontally or 1

foot vertically. The slope of the rock rapids shall be 20H:1V from the crest of the dam to 2 feet below the elevation of the downstream pool. The ramp shall continue at a 5H:1V slope to existing river bottom. Plan view and profiles of the rack rapids for the dams and the lock chambers are shown in the following figures:

Figure 6 – LOCK AND DAM NO. 1 ROCK RAPIDS DOWNSTREAM OF DAM AND INSIDE LOCK
Figure 7 - LOCK AND DAM NO. 1 ROCK RAPIDS DOWNSTREAM OF DAM
Figure 8 - LOCK AND DAM NO. 1 ROCK RAPIDS IN LOCK
Figure 9 - LOCK AND DAM NO. 2 ROCK RAPIDS DOWNSTREAM OF DAM AND INSIDE LOCK
Figure 10 - LOCK AND DAM NO. 2 ROCK RAPIDS DOWNSTREAM OF DAM
Figure 11 - LOCK AND DAM NO. 2 ROCK RAPIDS IN LOCK
Figure 12 - LOCK AND DAM NO. 3 ROCK RAPIDS DOWNSTREAM OF DAM AND INSIDE LOCK
Figure 13 - LOCK AND DAM NO. 3 ROCK RAPIDS DOWNSTREAM OF DAM
Figure 14 - LOCK AND DAM NO. 3 ROCK RAPIDS IN LOCK

The river walls of the three Locks will have a stability analysis completed to determine the effects of the weight of the rock from the rock rapids. The lock wall stability analysis will be conducted in accordance with the requirements of EM 1110-2-2602 "Planning and Design of Navigation Lock Walls and Appurtenances", EM 1110-2-2502 "Retaining Wall", and EM 1110-2-2406 "Design of Pile Structures and Foundations".

The three dams: Dam No. 1, Dam No. 2, and the William O. Huske Dam will have a stability analysis completed to determine if they are stable in their present condition. The three dams will also have stability analysis completed, which will include conditions with rock rapids. The dam stability analysis will be conducted in accordance with the requirements of EM 110-2-1902 "Slope Stability" and will utilize the U-Texas 4 Stability Analysis Program.

B. Rock Rapids Downstream of Dams with Dam Crest Lowered w/ Reduced Water Supply Capacity

This alternative evaluated quantities and costs to construct rock rapids with the dam lowered to an elevation that would have reduced water supply capacity on the municipal water intakes that are upstream of Lock & Dam No. 1 and the William O. Huske Lock & Dam. Rock rapids shall be constructed as described above. The elevation that the dams could be lowered to with minimal impact to the municipal water supplies was determined in a study conducted by the US Army Corps of Engineers, Mobile District. The study, "Wilmington Harbor GRR, Lock and Dam Study, Intake Evaluation", is included in Appendix 3.8.1. The elevation that the dams could be lowered as determined by the study stated above, are summarized as follow:

Dam No. 1 lowered from +10.7' NGVD 29 to +9.5' NGVD 29
Dam No. 2 lowered from +20.1' NGVD 29 to +2.0' NGVD 29 (no dam)
Dam No. 3 lowered from +29.1' NGVD 29 to +27.2' NGVD 29

The study evaluated five existing municipal water supply intake structures and pump stations.

The City of Wilmington and the City of Fayetteville have stated that they have concerns in regards to the Corps' study. These concerns will be addressed prior to the completion of the Feasibility Report.

C. Rock Rapids Upstream of Dams

This alternative evaluates quantities and costs to construct rock rapids upstream of the existing dam structures. See section 3.5.3.5 of the main text. The toe or downstream side of the new rock rapids would tie into the remains of the existing dams. The lock walls and fendering system would be removed. The bank behind the river wall shall be sloped to a 3H:1V and protected with riprap. The left abutment shall be right in place to provide stability to the new rock rapids structure. The rock rapids bedding/fill material, wearing stone and boulder vanes would be the same as decribed above. The new rock rapids structures shall have a one-foot thick concrete sheet pile wall driven from bank to bank to stabilize the crest of the new structure. The new structure at the Lock & Dam No. 1 location shall have abutments on each bank to protect the ends of the structure from erosion. The new structure at the Lock & Dam No. 2 location shall have the concrete sheetpile extend to the top of each bank to protect the ends of the structure from erosion.

D. Dams Lowered to Pass Fish w/o Rock Rapids

This alternative evaluated quantities and costs to construct rock rapids with the dam lowered to an elevation that would allow the fish to swim over the dams with no rock rapids. The elevation that the dams would have to be lowered to was determined using a HEC-RAS model of the Cape Fear River for a spring time flow of 5000 CFS The elevation that the dams could be lowered to and allow fish to swim over the dam structure are summarized as follow:

Dam No. 1 lowered from +10.7' NGVD 29 to +3.0' NGVD 29
Dam No. 2 lowered from +20.1' NGVD 29 to +12.0' NGVD 29
Dam No. 3 lowered from +29.1' NGVD 29 to +23.0' NGVD 29

E. Short Diversion Channel

Short diversion channels for the three lock and dams consists of excavating a channel to the east of Lock & Dam No. 1 and Lock & Dam No. 2, and to the west (lock side) of Lock & Dam no. 3. The short diversion channels will require unclassified excavation and construction of one inlet structure upstream of the dam and one outfall downstream of the dam. The inlet and outfall structures will be constructed with a sheetpile wall to protect the channel from erosion. The excavated channel will be protected from erosion with wearing stone and will include boulder vanes to create rapids. Plan view and profiles of the short diversion channels are shown in the following figures:

Figure 15 – LOCK AND DAM NO. 1 SHORT DIVERSION CHANNEL
Figure 16 - LOCK AND DAM NO. 2 SHORT DIVERSION CHANNEL

Figure 17 - LOCK AND DAM NO. 3 SHORT DIVERSION CHANNEL
Figure 18 - LOCK AND DAM NO. 3 SHORT DIVERSION CHANNEL - PROFILE

F. Nature Like By-Pass Channel (Lock & Dam No. 1)

The Nature Like Bypass consists of excavating a channel 3,750' in length to the east of Lock & Dam No. 1 and improving approximately 2 miles of an existing unpaved access road. The bypass channel will require approximately 90,900 CY of unclassified excavation and construction of one inlet structure upstream of the dam and one outfall downstream of the dam. Improvement of the access road will require placing gravel, and installation and removal of four temporary bridges across four streams. The inlet and outfall structures will be constructed of a sheetpile wall. The excavated channel will be protected from erosion with riprap. Plan view of the nature like bypass channel is shown in the Figure 15A NATURE-LIKE BYPASS CHANNEL, CAPE FEAR RIVER, LOCK & DAM NO. 1, SITE PLAN.

G. Rehab of Lock and Dams

1) Lock & Dam No. 1

Rehab of the Lock & Dam No. 1 is derived from the Recommendations of PI No. 6, which include:

a. Repair the concrete cap of the dam at two locations, replace missing sheet pile downstream of dam and provide riprap and armor stone as needed in space between the dam and sheet pile. Replace riprap next to upstream sheet pile as needed.

b. Perform a major rehabilitation of the gates to include replacement of damaged members, seals, pintels, cleaning and painting, and general inspection in the dry of the lock walls and gates.

c. Replace the damaged and settling downstream concrete esplanade sections, fill the void under it with rock and prevent the removal of material through joints and cracks. Inspect the wing wall anchor rods while exposed and make necessary repairs to damaged rods.

d. Repair the lock walls by epoxy injection or replace the upper lift. Repair by injection cracks below this lift in the area of the gates.

e. Fill voids under slab on left abutment and seal holes in slab.

f. Repair timber dikes as needed to maintain pool. Perform annual inspections until dikes are repaired.

g. Add slope protection to the scour hole on the downstream side of the dam. The slope protection shall be constructed of bedding stone or suitable fill material and wearing stone. The bedding stone may consist of aggregate or debris (demolished concrete or masonry, etc.) of adequate size so that it will not be washed downstream by the river current during construction. Wearing stone shall consist of stone aggregate with an average diameter of 22 inches (D_{50} = 22"

calculation based on Hydraulic Design Criteria, Sheet 712-1). The thickness of the wearing stone shall be 4 feet. Current and historical conditions of the scour hole are illustrated in Figure 19 LOCK & DAM NO. 1 PROFILES.

2) Lock & Dam No. 2

Rehab of the Lock & Dam No. 2 is derived from the Recommendations of PI No. 6, which include:

a. Perform a major rehabilitation of the gates to include replacement of damaged members, seals, pintels, cleaning and painting, and general inspection in the dry of the lock walls and gates.

b. Replace the damaged and settling downstream concrete esplanade sections, fill the void under it with rock and prevent the removal of material through joints and cracks.

c. Repair the lock walls by epoxy injection or replace the upper lift. Repair by injection cracks below this lift in the area of the gates.

d. Repair damaged wing wall on the north side of the esplanade.

e. Add slope protection to the scour hole on the downstream side of the dam. The slope protection shall be constructed of bedding stone or suitable fill material and wearing stone. The bedding stone may consist of aggregate or debris (demolished concrete or masonry, etc.) of adequate size so that it will not be washed downstream by the river current during construction. Wearing stone shall consist of stone aggregate with an average diameter of 22 inches (D_{50} = 22" calculation based on Hydraulic Design Criteria, Sheet 712-1). The thickness of the wearing stone shall be 4 feet. Current and historical conditions of the scour hole are illustrated in Figure 20 LOCK & DAM NO. 2, PROFILES.

3) William O. Huske Lock & Dam

Rehab of the William O. Huske Lock & Dam is derived from the Recommendations of PI No. 6, which include:

a. Perform a major rehabilitation of the gates to include replacement of damaged members, seals, pintels, cleaning and painting, and general inspection in the dry of the lock walls and gates.

b. Replace the esplanade drop inlets and drain pipe.

c. Replace the damaged and settling sections of the concrete esplanade sections, fill the void under it with rock and prevent the removal of material through joints and cracks.

d. Place large riprap downstream of the left abutment sheet piling to provide stability and protect exposed areas upstream of the sheet piling with riprap or a ground cover.

e. Repair the lock walls by epoxy injection or replace the upper lift. Repair by injection cracks below this lift in the area of the gates.

f. Current and historical conditions of the dam are illustrated in Figure 21 LOCK & DAM NO. 3, PROFILES.

H. Remove Lock & Dams

1) Lock & Dam No. 1

Removal of Lock & Dam No. 1 shall include: demolition of the timber crib rock filled dam to elevation -10.0' NGVD 29; demolition of the mass concrete lock walls to elevation -10.0' NGVD 29, demolition of the right abutment, grading the bank behind the land wall and the abutment to a 2H:1V slope and protecting with riprap. Elevation -10.0' NGVD 29 is the elevation of the floor of the lock chamber.

2) Lock & Dam No. 2

Removal of Lock & Dam No. 2 shall include: demolition of the rock filled dam to elevation -1.0' NGVD 29; demolition of the mass concrete lock walls to elevation -1.0' NGVD 29, demolition of the right abutment, grading the bank behind the land wall and the abutment to a 2H:1V slope and protecting with riprap. Elevation -1.0' NGVD 29 is the elevation of the floor of the lock chamber.

3) William O. Huske Lock & Dam

Removal of Lock & Dam No. 3 shall include: demolition of the concrete dam to elevation +11.0' NGVD 29; demolition of the mass concrete lock walls to elevation +10.0' NGVD 29, demolition of the right abutment, grading the bank behind the land wall and the abutment to a 2H:1V slope and protecting with riprap. Elevation +10.0' NGVD 29 is the elevation of the floor of the lock chamber.

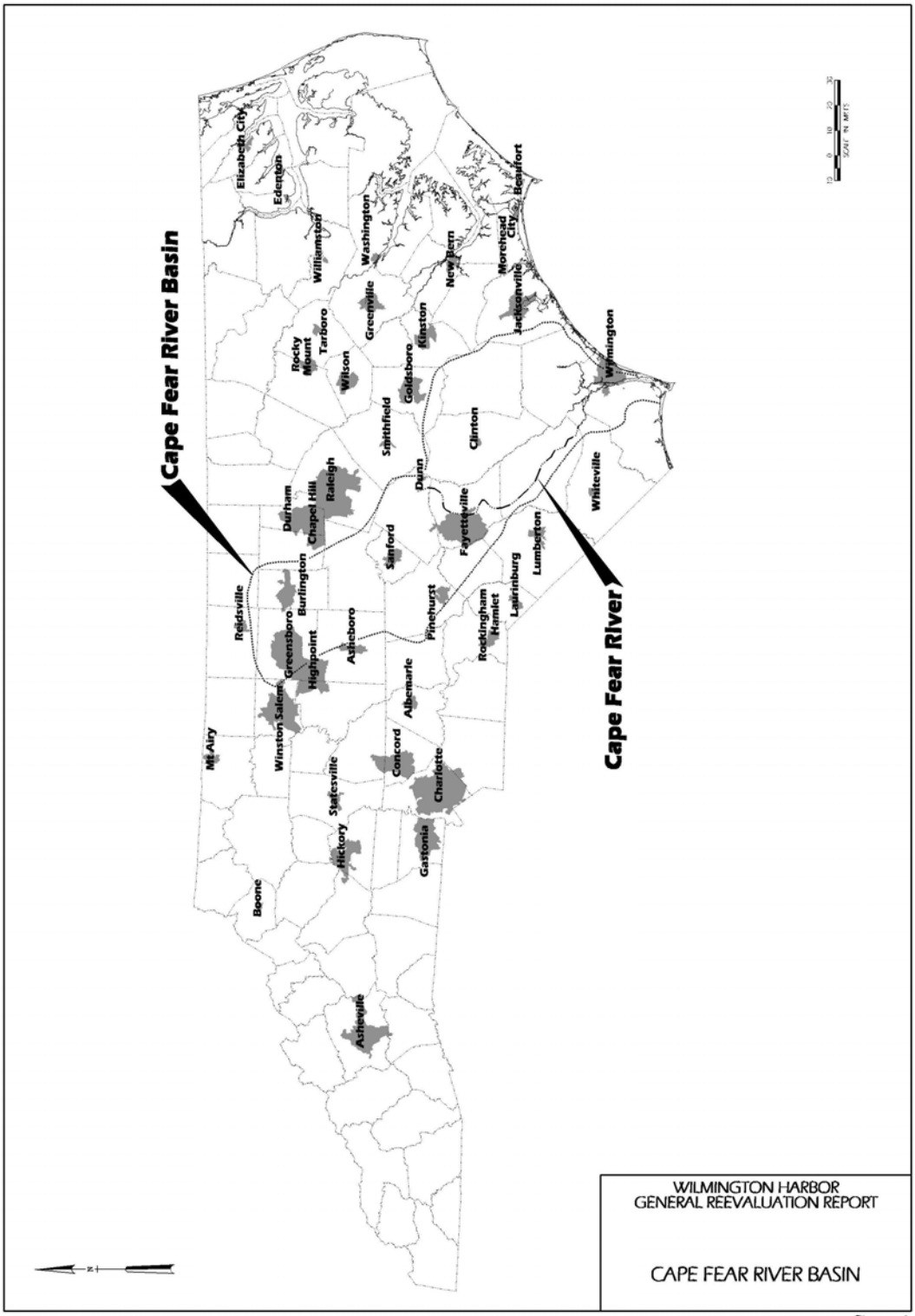

Cape Fear River Basin
Cape Fear River
Elizabeth City
Edenton
Williamston
Washington
Beaufort
Morehead City
New Bern
Jacksonville
Greenville
Kinston
Tarboro
Rocky Mount
Wilson
Goldsboro
Wilmington
Smithfield
Clinton
Dunn
Raleigh
Durham
Chapel Hill
Whiteville
Fayetteville
Sanford
Lumberton
Burlington
Reidsville
Laurinburg
Pinehurst
Asheboro
Greensboro
Highpoint
Rockingham
Hamlet
Albemarle
Winston Salem
Mt Airy
Concord
Statesville
Charlotte
Hickory
Gastonia
Boone
Asheville
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
CAPE FEAR RIVER BASIN

Figure 1

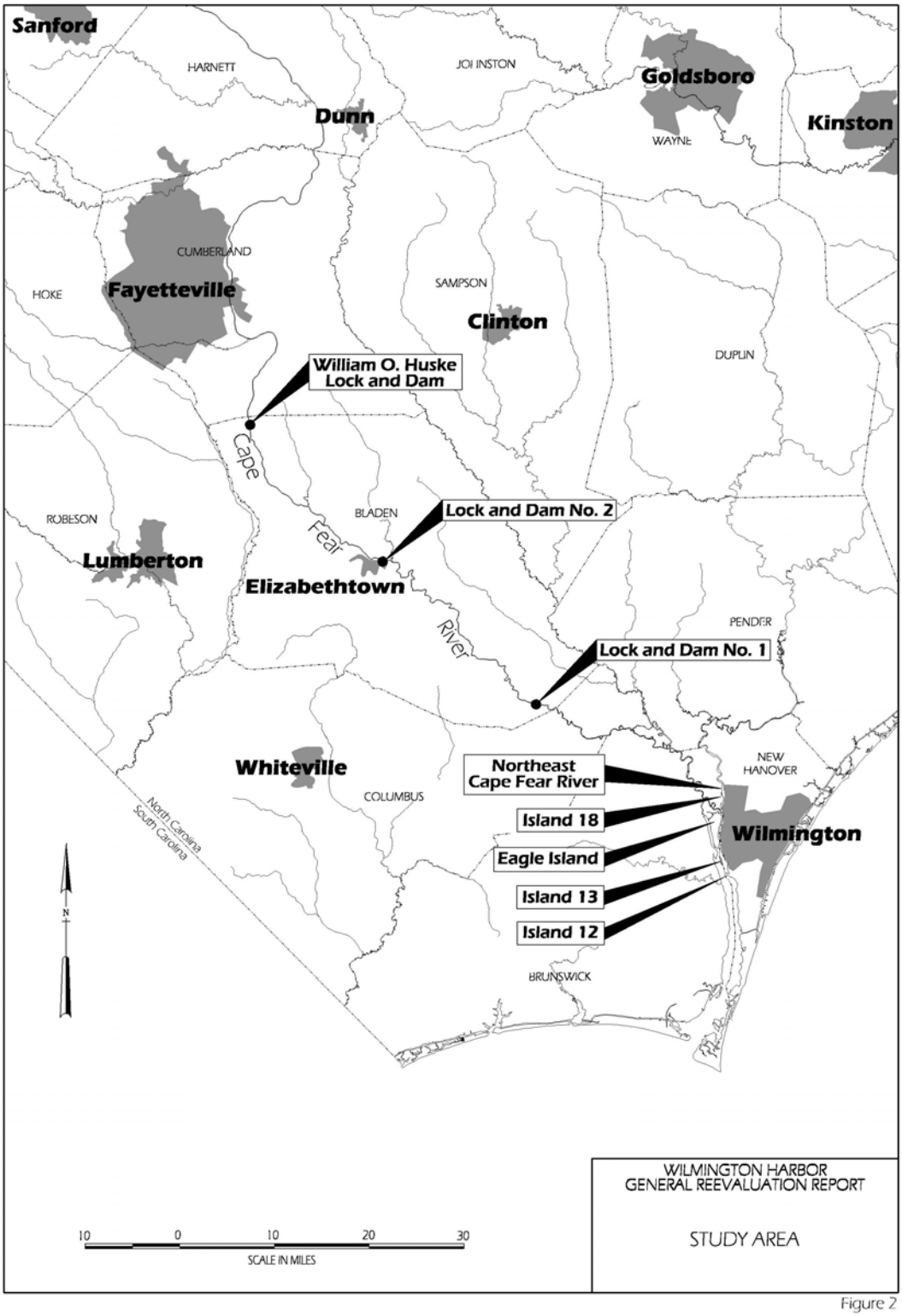

Sanford
HARNETT
JOHNSTON
Goldsboro
Kinston
Dunn
WAYNE
CUMBERLAND
HOKE
Fayetteville
SAMPSON
Clinton
DUPLIN
William O. Huske Lock and Dam
Cape
Fear
River
BLADEN
Lock and Dam No. 2
ROBESON
Lumberton
Elizabethtown
PENDER
Lock and Dam No. 1
Whiteville
COLUMBUS
Northeast Cape Fear River
NEW HANOVER
Island 18
Wilmington
Eagle Island
Island 13
Island 12
BRUNSWICK
North Carolina
South Carolina
N
10 0 10 20 30
SCALE IN MILES
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
STUDY AREA

Figure 2

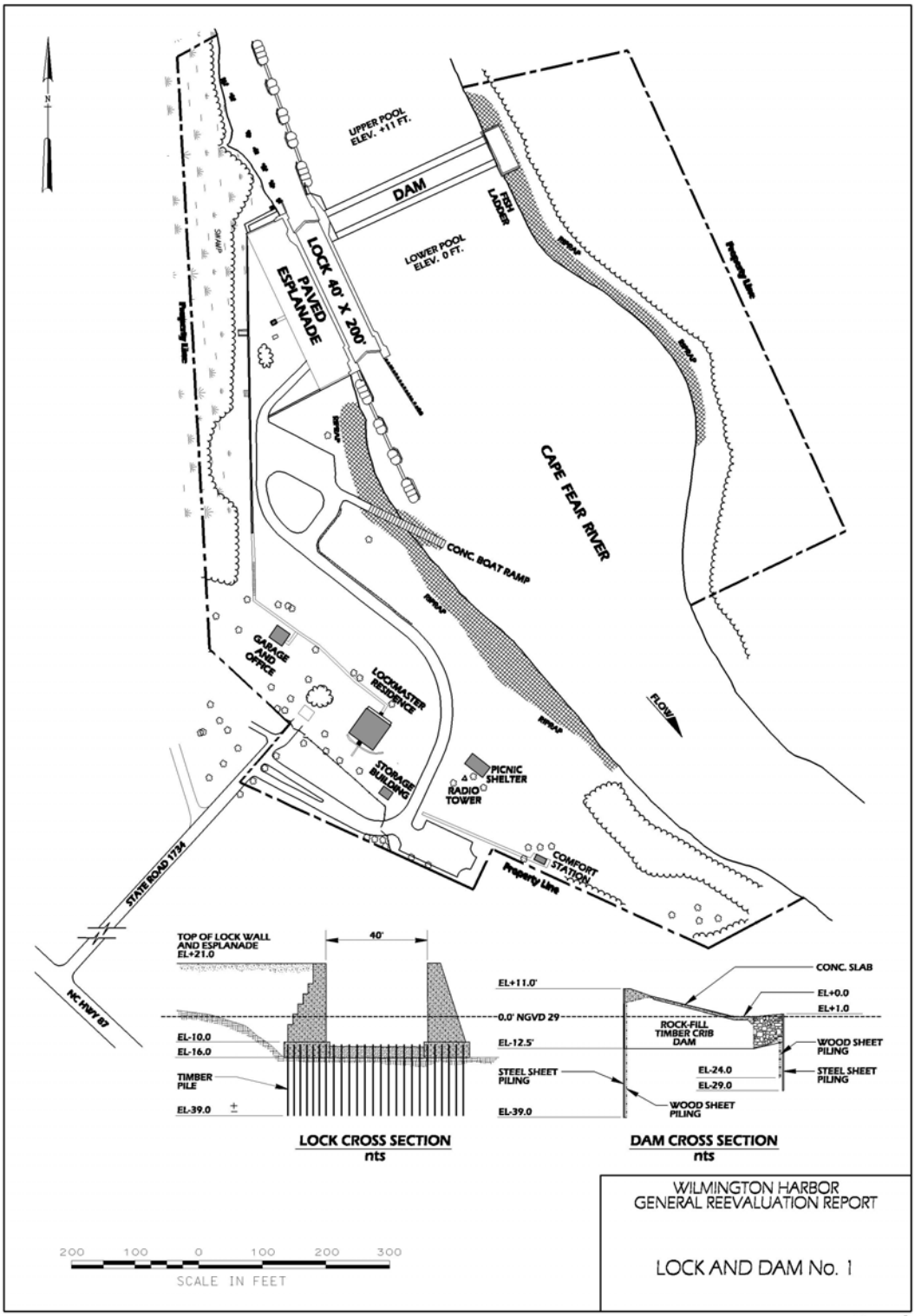

N
UPPER POOL
ELEV. +11 FT.
DAM
FISH LADDER
LOWER POOL
ELEV. 0 FT.
LOCK 40' X 200'
PAVED ESPLANADE
Property Line
RIPRAP
CAPE FEAR RIVER
CONC. BOAT RAMP
GARAGE AND OFFICE
LOCKMASTER RESIDENCE
STORAGE BUILDING
PICNIC SHELTER
RADIO TOWER
FLOW
COMFORT STATION
STATE ROAD 1734
NC HWY 87
TOP OF LOCK WALL
AND ESPLANADE
EL+21.0
40'
EL-10.0
EL-16.0
TIMBER PILE
EL-39.0
LOCK CROSS SECTION
nts
EL+11.0'
0.0' NGVD 29
EL-12.5'
STEEL SHEET PILING
EL-39.0
CONC. SLAB
EL+0.0
EL+1.0
ROCK-FILL TIMBER CRIB DAM
WOOD SHEET PILING
EL-24.0
EL-29.0
STEEL SHEET PILING
WOOD SHEET PILING
DAM CROSS SECTION
nts
200 100 0 100 200 300
SCALE IN FEET
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 1

Figure 3

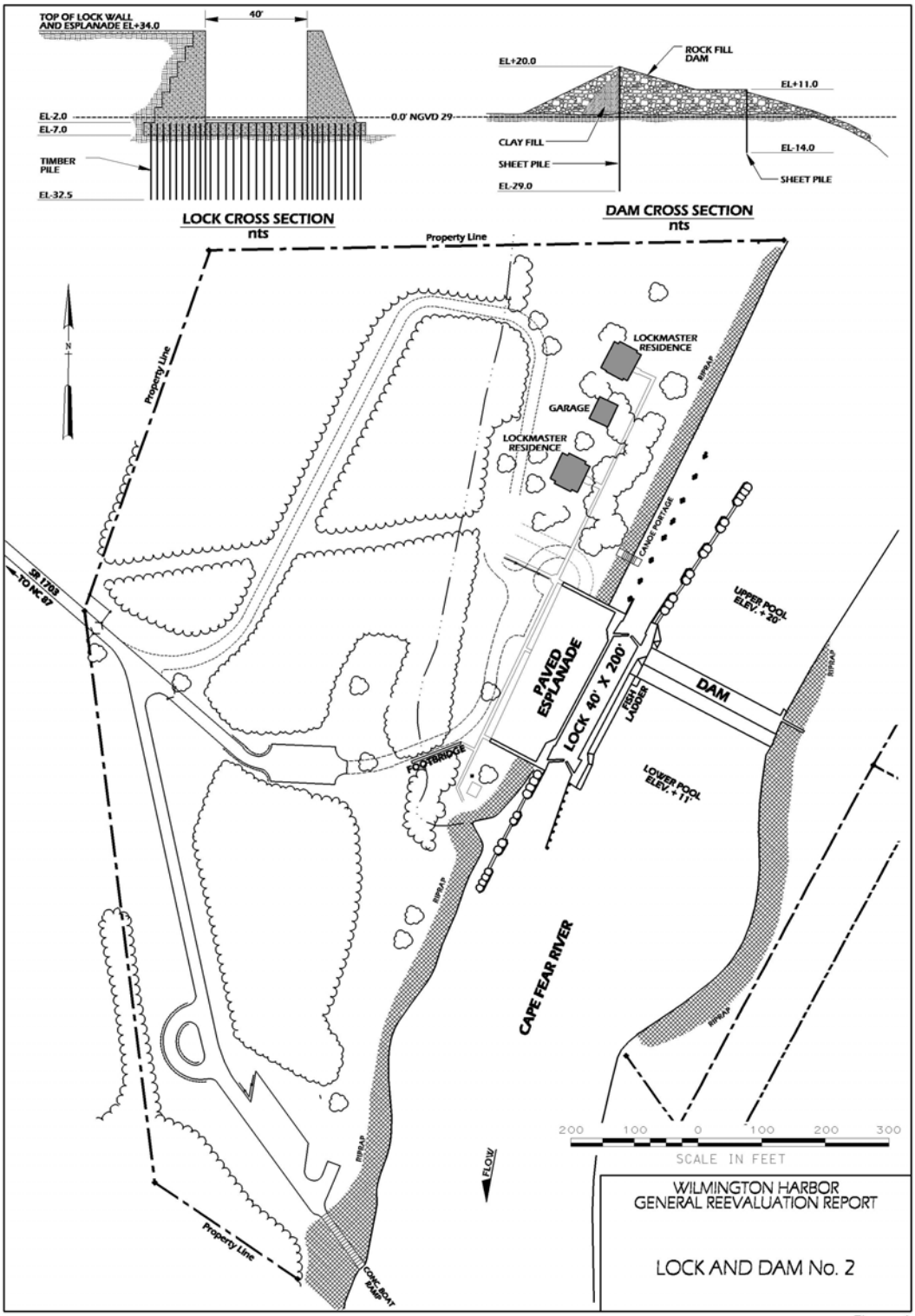

TOP OF LOCK WALL AND ESPLANADE EL+34.0
40'
EL-2.0
EL-7.0
0.0' NGVD 29
TIMBER PILE
EL-32.5
LOCK CROSS SECTION
nts
EL+20.0
ROCK FILL DAM
EL+11.0
CLAY FILL
SHEET PILE
EL-29.0
EL-14.0
SHEET PILE
DAM CROSS SECTION
nts
Property Line
N
LOCKMASTER RESIDENCE
GARAGE
LOCKMASTER RESIDENCE
RIPRAP
CANOE PORTAGE
SR 1703
TO NC 87
UPPER POOL ELEV. +20'
PAVED ESPLANADE
LOCK 40' X 200'
FISH LADDER
DAM
FOOTBRIDGE
LOWER POOL ELEV. +11'
CAPE FEAR RIVER
FLOW
CANOE BOAT RAMP
200 100 0 100 200 300
SCALE IN FEET
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 2

Figure 4

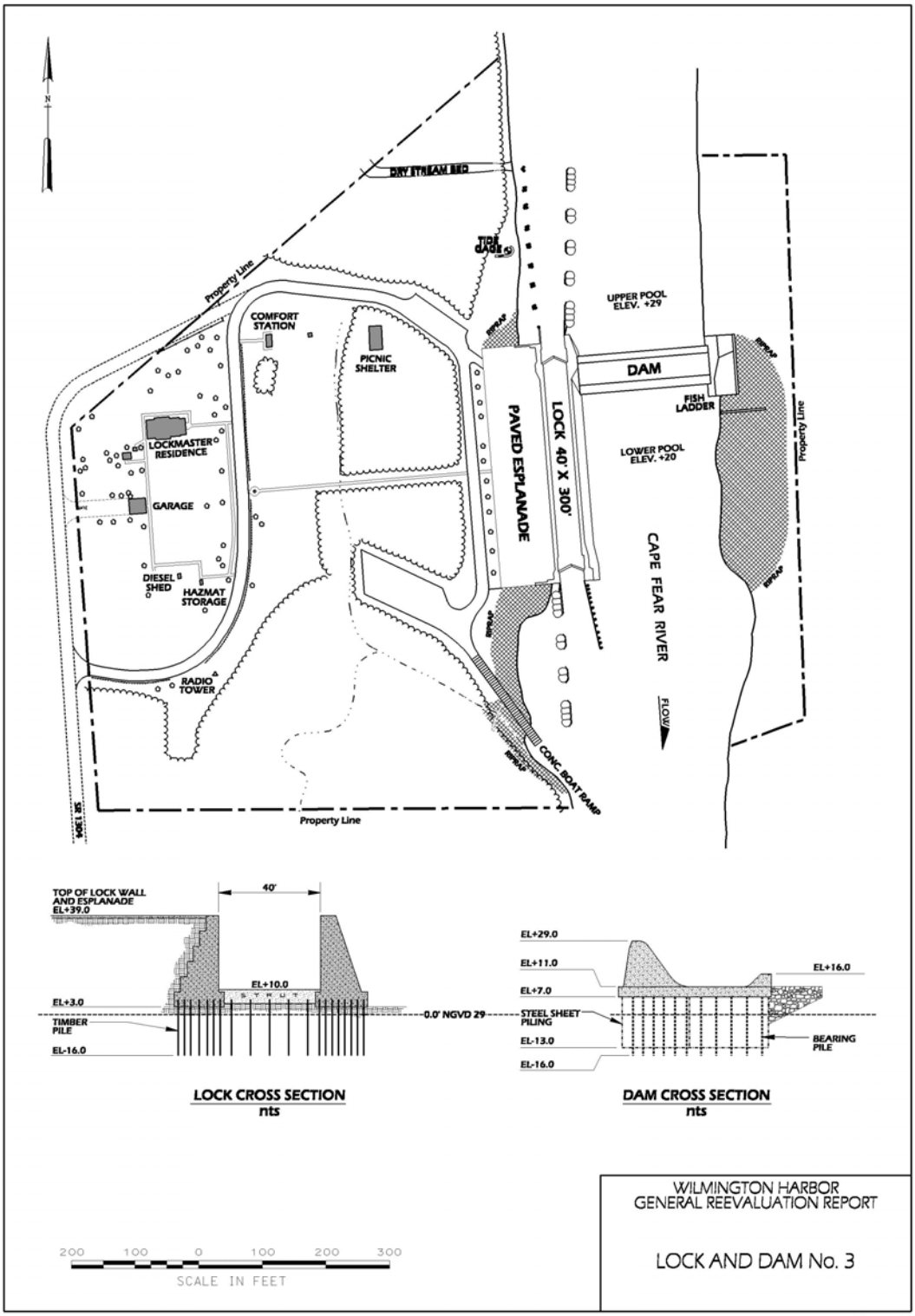
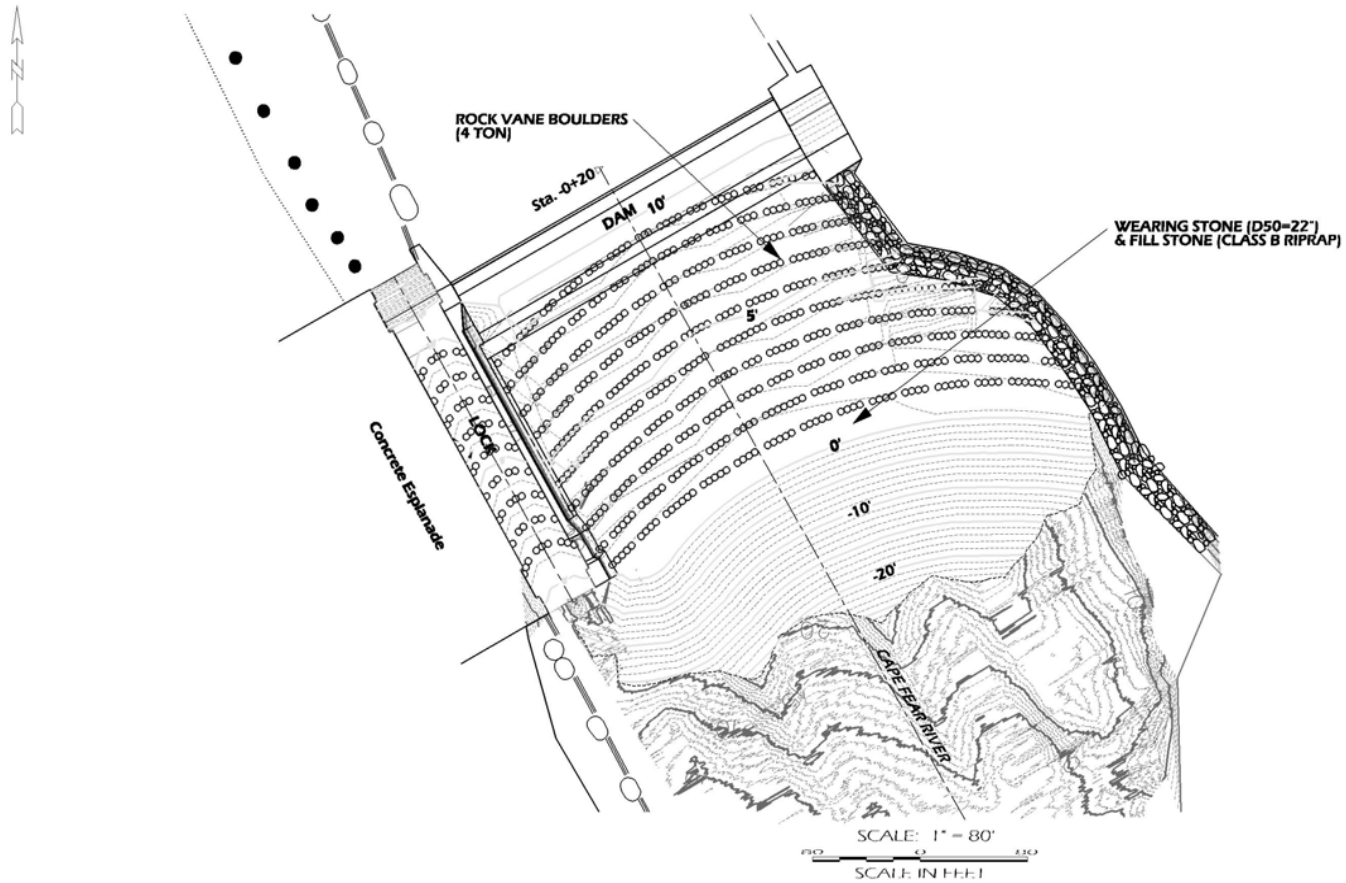

N
DRY STREAM BED
TIDE GAGE
Property Line
COMFORT STATION
PICNIC SHELTER
UPPER POOL ELEV. +29
RIPRAP
DAM
FISH LADDER
LOCKMASTER RESIDENCE
GARAGE
PAVED ESPLANADE
LOCK 40' X 300'
LOWER POOL ELEV. +20
Property Line
CAPE FEAR RIVER
FLOW
DIESEL SHED
HAZMAT STORAGE
RADIO TOWER
CONC. BOAT RAMP
Property Line
TOP OF LOCK WALL AND ESPLANADE EL+39.0
40'
EL+10.0
STRUT
EL+3.0
TIMBER PILE
EL-16.0
0.0' NGVD 29
LOCK CROSS SECTION
nts
EL+29.0
EL+11.0
EL+7.0
STEEL SHEET PILING
EL-13.0
EL-16.0
EL+16.0
BEARING PILE
DAM CROSS SECTION
nts
200 100 0 100 200 300
SCALE IN FEET
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 3

Figure 5



VERTICAL DATUM: NGVD '29

WILMINGTON HARBOR
GENERAL REEVALUATION REPORT

LOCK AND DAM No. 1
ROCK RAPIDS DOWNSTREAM
OF DAM AND INSIDE LOCK

Figure 6

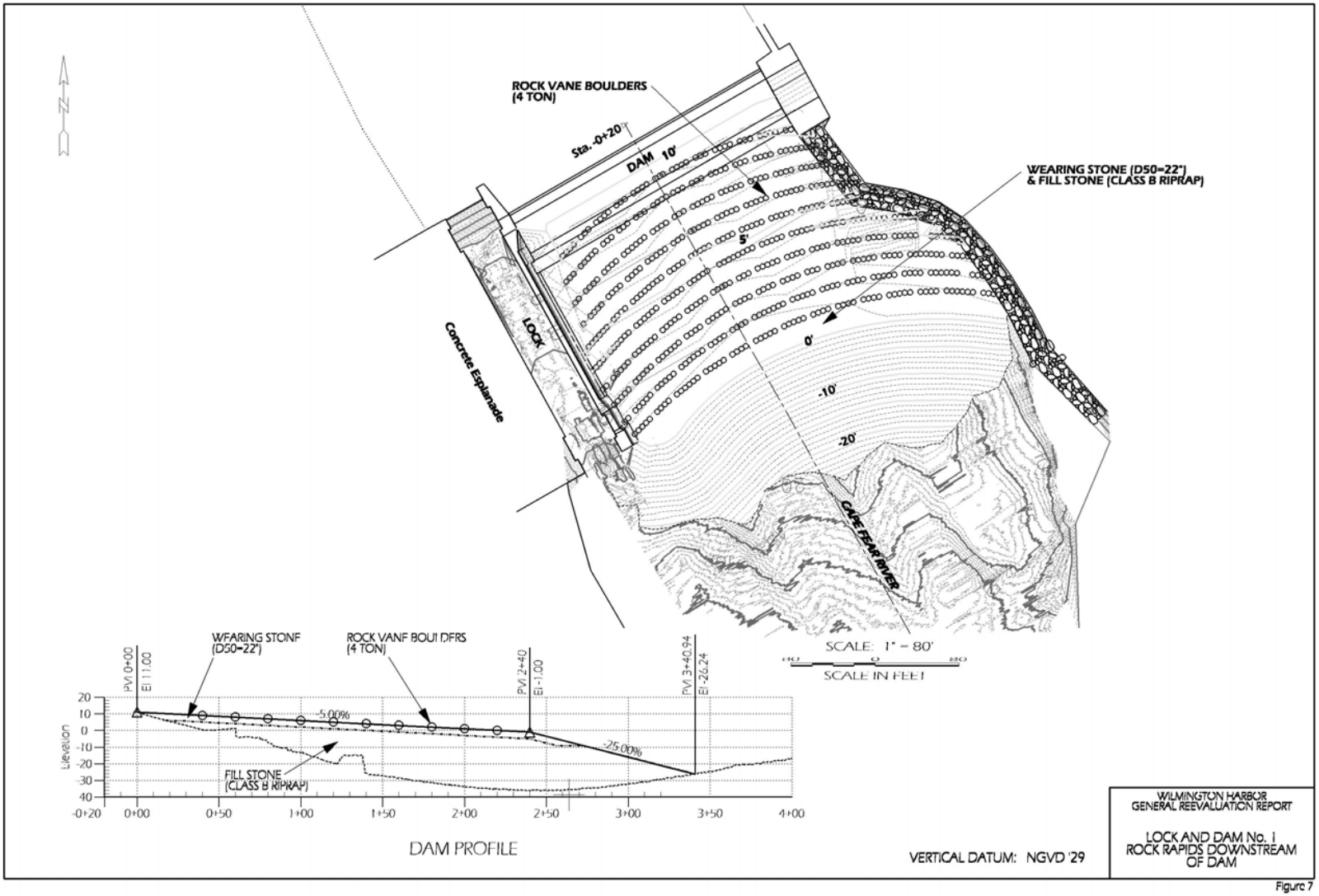

ROCK VANE BOULDERS
(4 TON)
Sta. -0+20
DAM 10'
5'
0'
-10'
-20'
WEARING STONE (D50=22")
& FILL STONE (CLASS B RIPRAP)
Concrete Esplanade
LOCK
CAPE FEAR RIVER
SCALE: 1" = 80'
SCALE IN FEET
WEARING STONE
(D50=22")
ROCK VANE BOULDERS
(4 TON)
PVI 0+00
El 11.00
PVI 2+40
El -1.00
PVI 3+40.94
El -26.24
-5.00%
-25.00%
FILL STONE
(CLASS B RIPRAP)
Elevation
20
10
0
-10
-20
-30
40
-0+20
0+00
0+50
1+00
1+50
2+00
2+50
3+00
3+50
4+00
DAM PROFILE
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 1
ROCK RAPIDS DOWNSTREAM
OF DAM

Figure 7

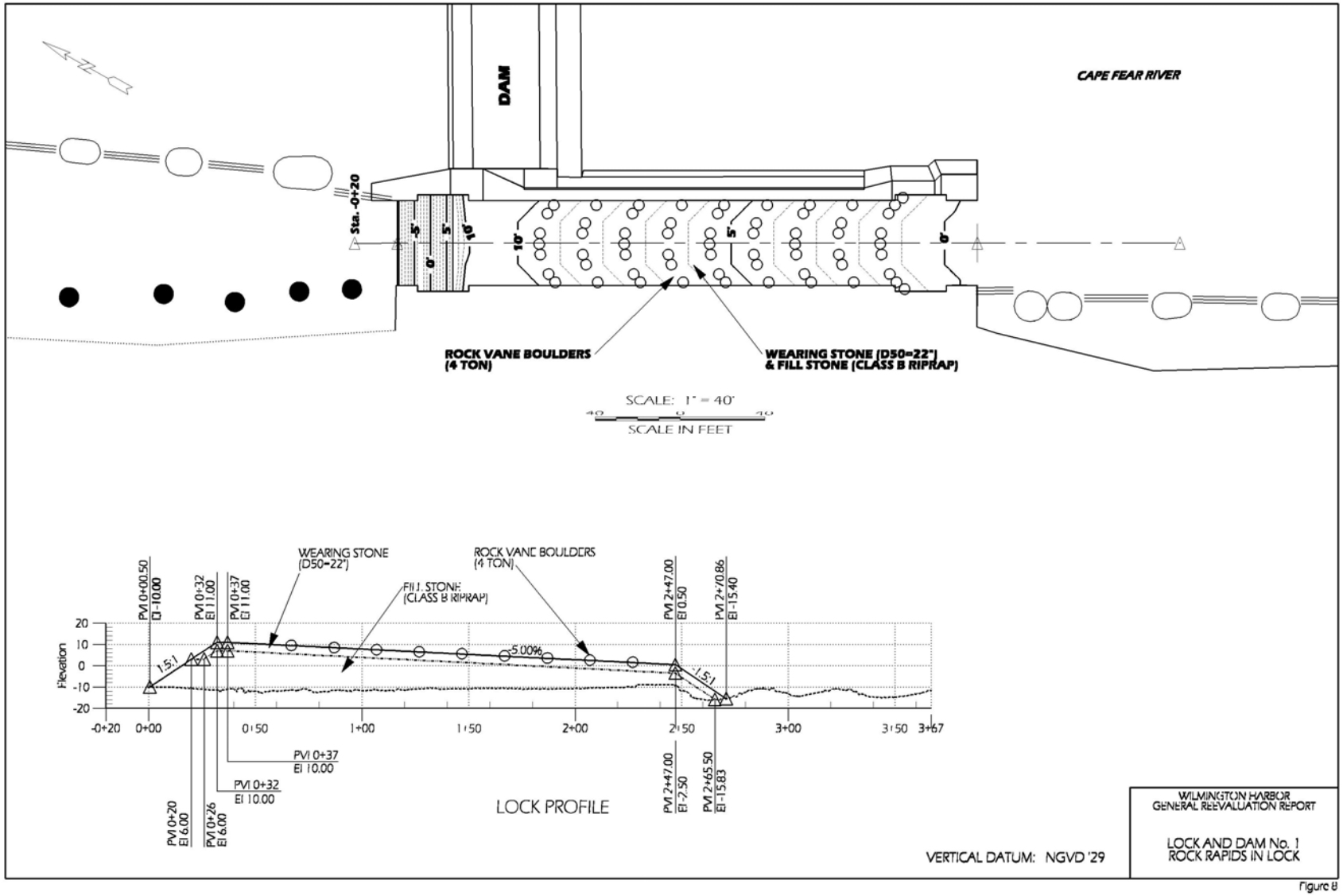

DAM
CAPE FEAR RIVER
Sta. -0+20
-5'
0'
5'
10'
10'
5'
0'
ROCK VANE BOULDERS
(4 TON)
WEARING STONE (D50=22")
& FILL STONE (CLASS B RIPRAP)
SCALE: 1" = 40'
40 0 40
SCALE IN FEET
WEARING STONE
(D50=22")
ROCK VANE BOULDERS
(4 TON)
FILL STONE
(CLASS B RIPRAP)
PVI 0+00.50
El -10.00
PVI 0+32
El 11.00
PVI 0+37
El 11.00
PVI 2+47.00
El 0.50
PVI 2+70.86
El -15.40
Elevation
20
10
0
-10
-20
1.5:1
-5.00%
-1.5:1
-0+20
0+00
0+50
1+00
1+50
2+00
2+50
3+00
3+50
3+67
PVI 0+37
El 10.00
PVI 0+32
El 10.00
PVI 0+20
El 6.00
PVI 0+26
El 6.00
PVI 2+47.00
El -7.50
PVI 2+65.50
El -15.83
LOCK PROFILE
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 1
ROCK RAPIDS IN LOCK

Figure 8

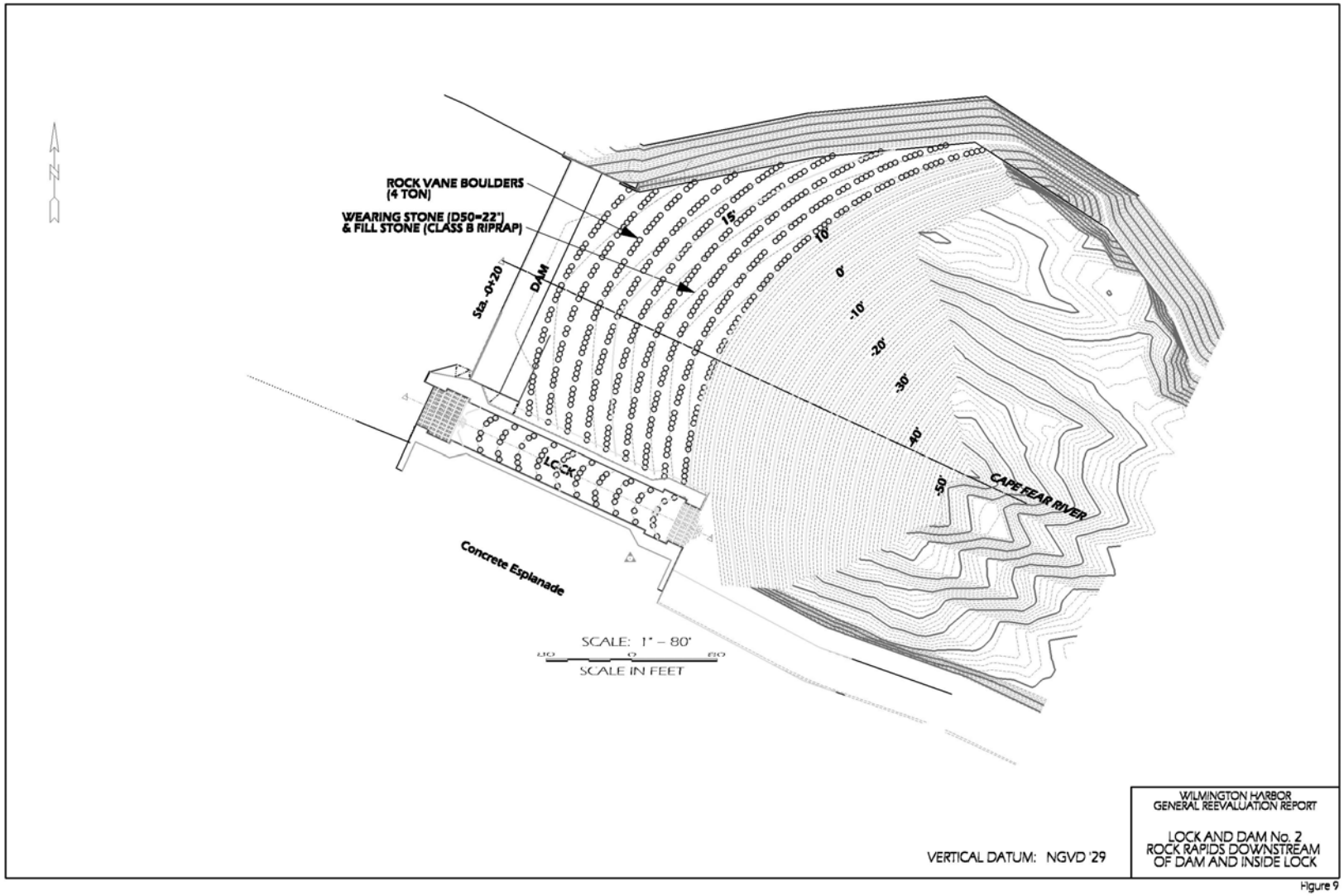

ROCK VANE BOULDERS
(4 TON)
WEARING STONE (D50=22")
& FILL STONE (CLASS B RIPRAP)
Sta. 0+20
DAM
LOCK
Concrete Esplanade
15'
10'
0'
-10'
-20'
-30'
-40'
-50'
CAPE FEAR RIVER
SCALE: 1" = 80'
SCALE IN FEET
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 2
ROCK RAPIDS DOWNSTREAM
OF DAM AND INSIDE LOCK

Figure 9

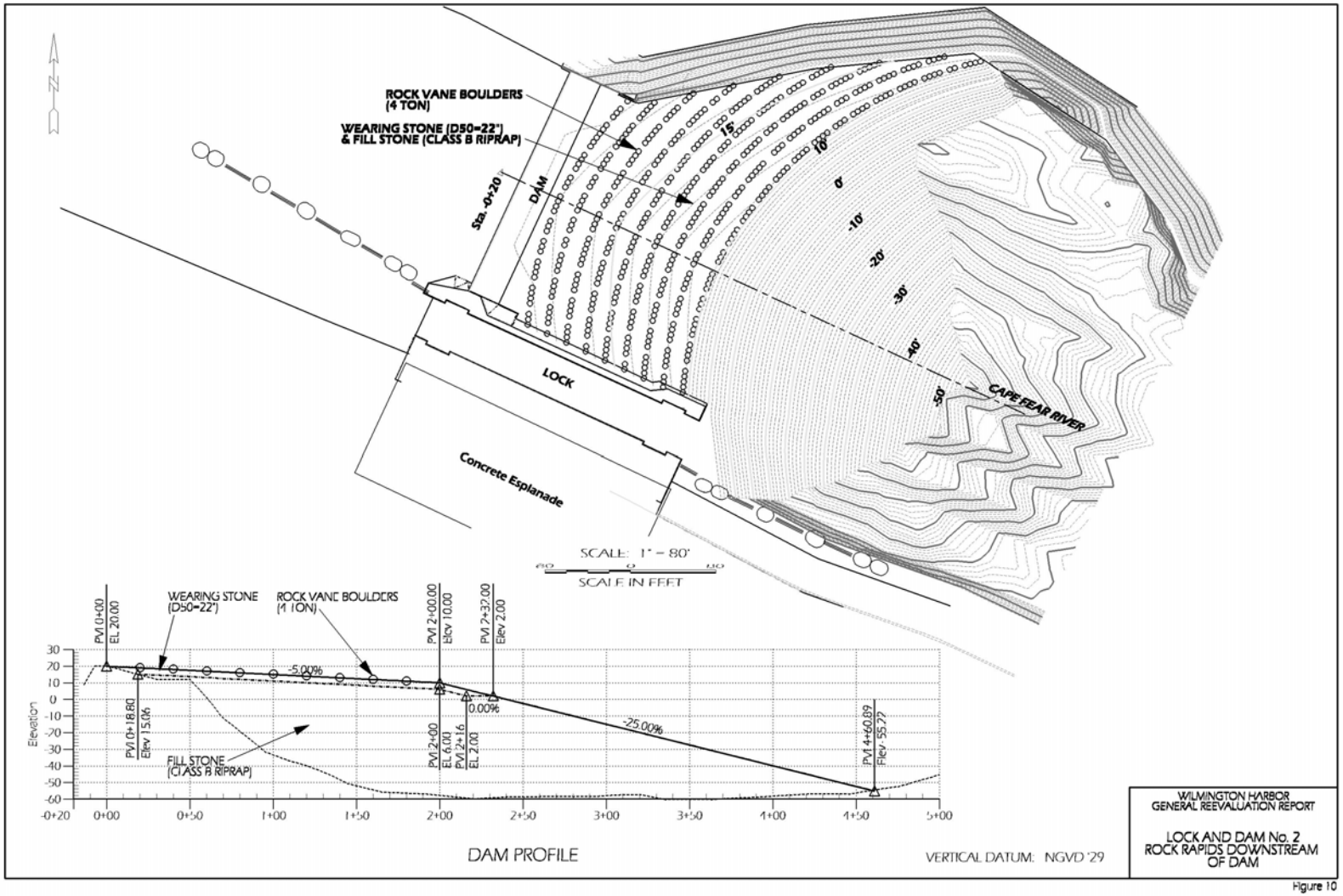

ROCK VANE BOULDERS
(4 TON)
WEARING STONE (D50=22")
& FILL STONE (CLASS B RIPRAP)
Sta. -0+20
DAM
LOCK
Concrete Esplanade
CAPE FEAR RIVER
15'
10'
0'
-10'
-20'
-30'
-40'
-50'
SCALE: 1" = 80'
SCALE IN FEET
WEARING STONE
(D50=22")
ROCK VANE BOULDERS
(4 TON)
FILL STONE
(CLASS B RIPRAP)
PVI 0+00
EL 20.00
PVI 0+18.80
Elev 15.06
PVI 2+00.00
Elev 10.00
PVI 2+37.00
Elev 2.00
PVI 2+00
EL 6.00
PVI 2+16
EL 2.00
PVI 4+60.89
Elev -55.77
-5.00%
0.00%
-25.00%
Elevation
DAM PROFILE
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 2
ROCK RAPIDS DOWNSTREAM
OF DAM

Figure 10

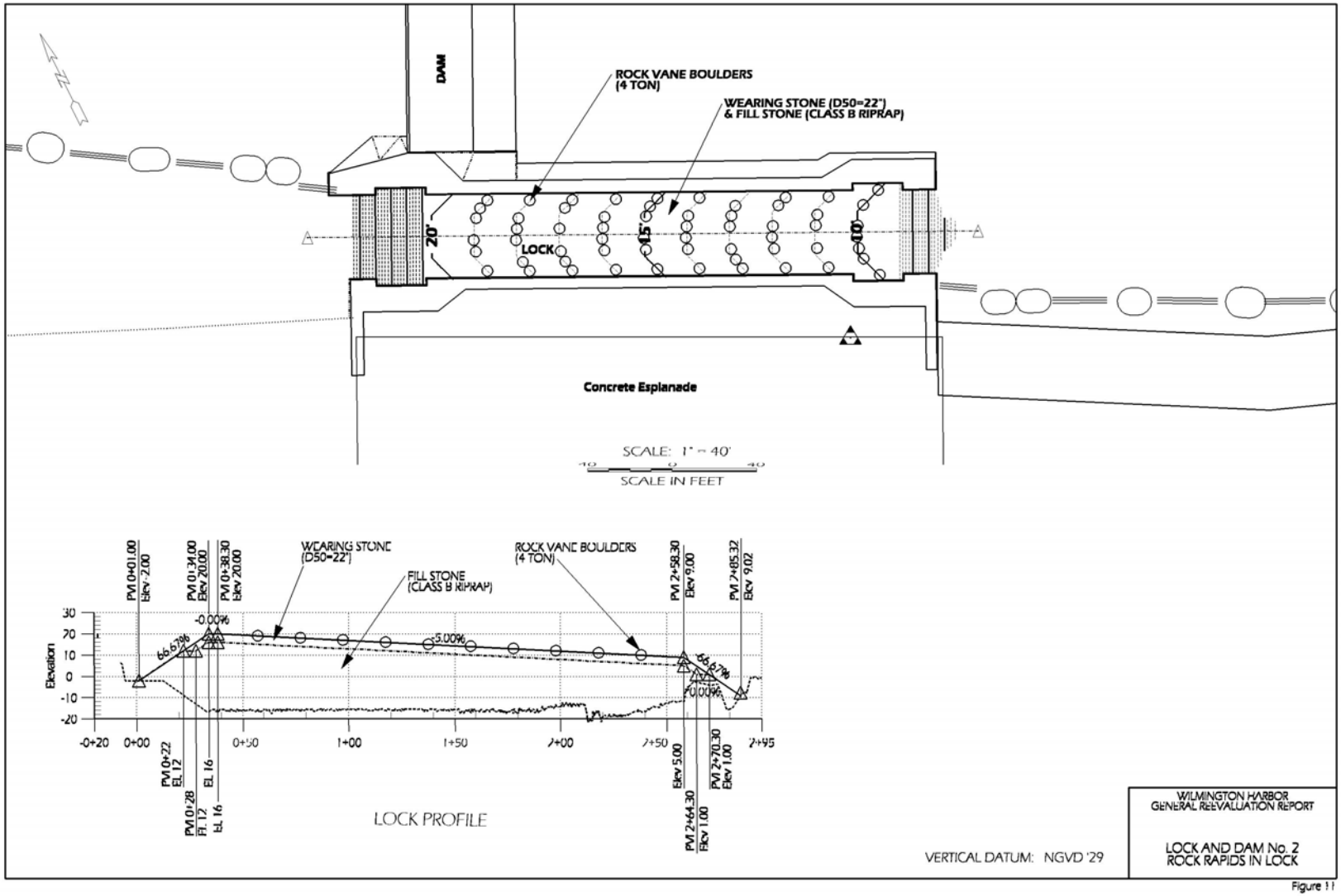
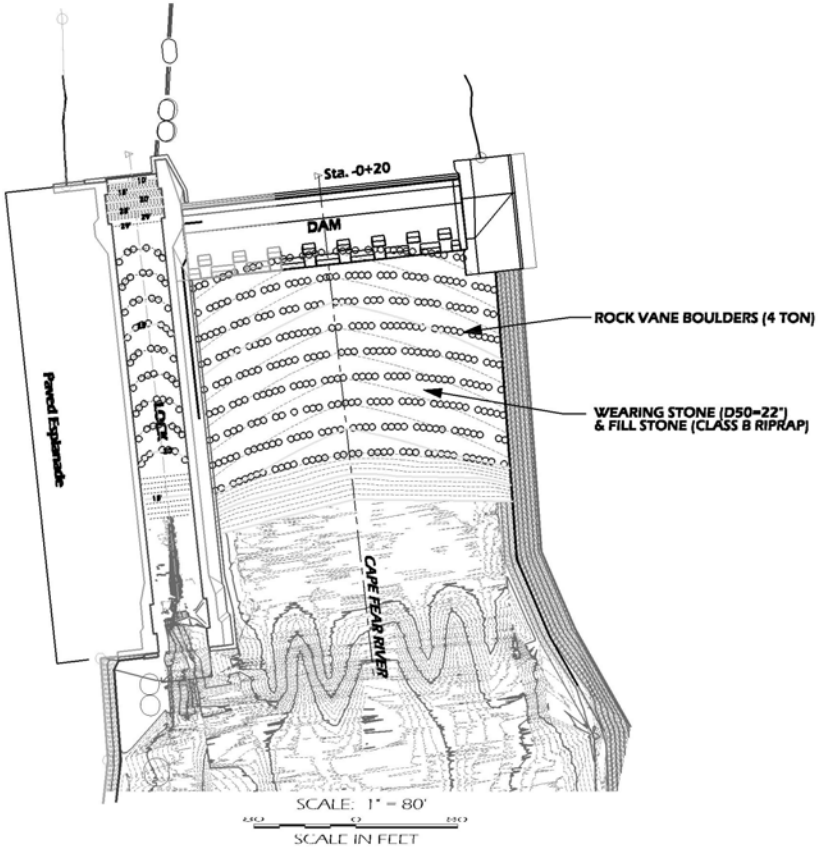

DAM
ROCK VANE BOULDERS
(4 TON)
WEARING STONE (D50=22")
& FILL STONE (CLASS B RIPRAP)
20'
LOCK
15'
10'
Concrete Esplanade
SCALE: 1" = 40'
SCALE IN FEET
WEARING STONE
(D50=22")
FILL STONE
(CLASS B RIPRAP)
ROCK VANE BOULDERS
(4 TON)
PM 0+01.00
Elev -2.00
PM 0+34.00
Elev 20.00
PM 0+38.30
Elev 20.00
PM 2+58.30
Elev 9.00
PM 2+85.32
Elev 9.02
-0.00%
66.67%
-5.00%
66.67%
-0.00%
Elevation
30
20
10
0
-10
-20
-0+20
0+00
0+50
1+00
1+50
2+00
2+50
2+95
PM 0+22
EL 12
PM 0+28
EL 12
EL 16
EL 16
Elev 5.00
PM 2+64.30
Elev 1.00
PM 2+70.30
Elev 1.00
LOCK PROFILE
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 2
ROCK RAPIDS IN LOCK

Figure 11



VERTICAL DATUM: NGVD '29

WILMINGTON HARBOR
GENERAL REEVALUATION REPORT

LOCK AND DAM No. 3
ROCK RAPIDS DOWNSTREAM
OF DAM AND INSIDE LOCK

Figure 12

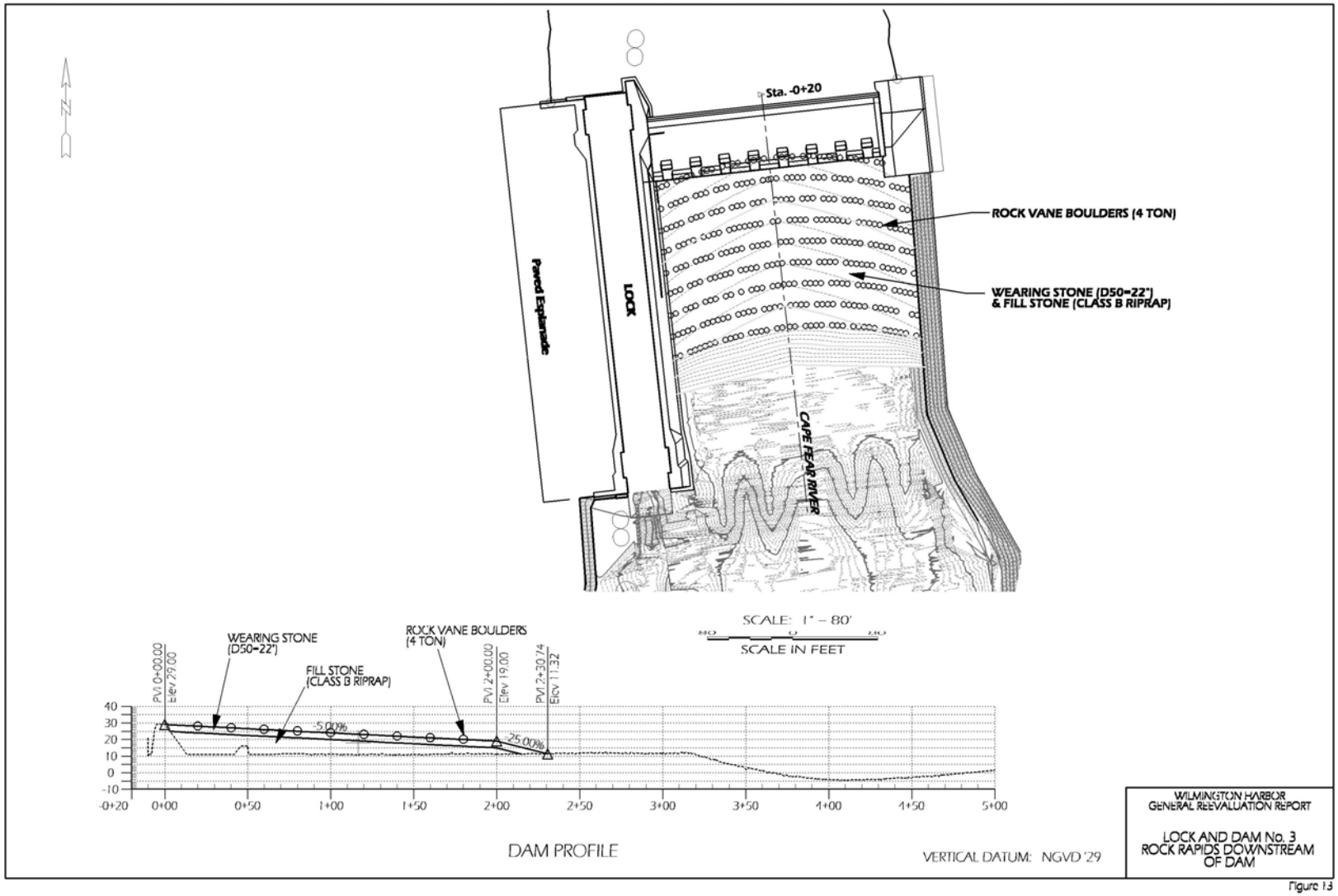

N
Sta. -0+20
ROCK VANE BOULDERS (4 TON)
WEARING STONE (D50=22")
& FILL STONE (CLASS B RIPRAP)
Paved Esplanade
LOCK
CAPE FEAR RIVER
SCALE: 1" = 80'
SCALE IN FEET
WEARING STONE
(D50=22")
FILL STONE
(CLASS B RIPRAP)
ROCK VANE BOULDERS
(4 TON)
PVI 0+00.00
Elev 29.00
PVI 2+00.00
Elev 19.00
PVI 2+30.74
Elev 11.32
5.00%
-25.00%
40
30
20
10
0
-10
-0+20
0+00
0+50
1+00
1+50
2+00
2+50
3+00
3+50
4+00
4+50
5+00
DAM PROFILE
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 3
ROCK RAPIDS DOWNSTREAM
OF DAM

Figure 13

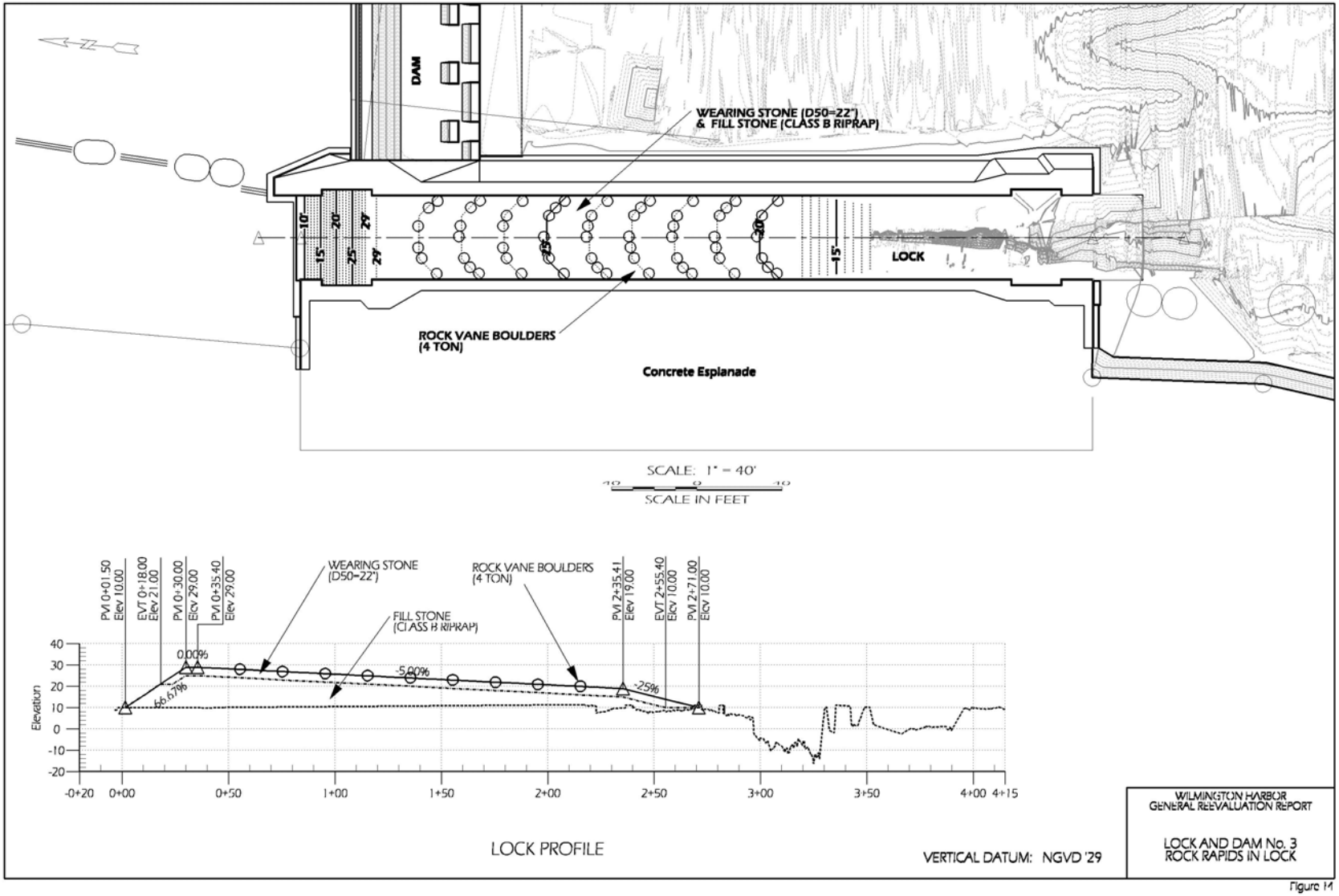

DAM
WEARING STONE (D50=22")
& FILL STONE (CLASS B RIPRAP)
LOCK
ROCK VANE BOULDERS
(4 TON)
Concrete Esplanade
SCALE: 1" = 40'
SCALE IN FEET
PVI 0+01.50
Elev 10.00
EVT 0+18.00
Elev 21.00
PVI 0+30.00
Elev 29.00
PVI 0+35.40
Elev 29.00
WEARING STONE
(D50=22")
FILL STONE
(CLASS B RIPRAP)
ROCK VANE BOULDERS
(4 TON)
PVI 2+35.41
Elev 19.00
EVT 2+55.40
Elev 10.00
PVI 2+71.00
Elev 10.00
0.00%
-5.00%
-25%
66.67%
Elevation
40
30
20
10
0
-10
-20
-0+20
0+00
0+50
1+00
1+50
2+00
2+50
3+00
3+50
4+00
4+15
LOCK PROFILE
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 3
ROCK RAPIDS IN LOCK

Figure 14

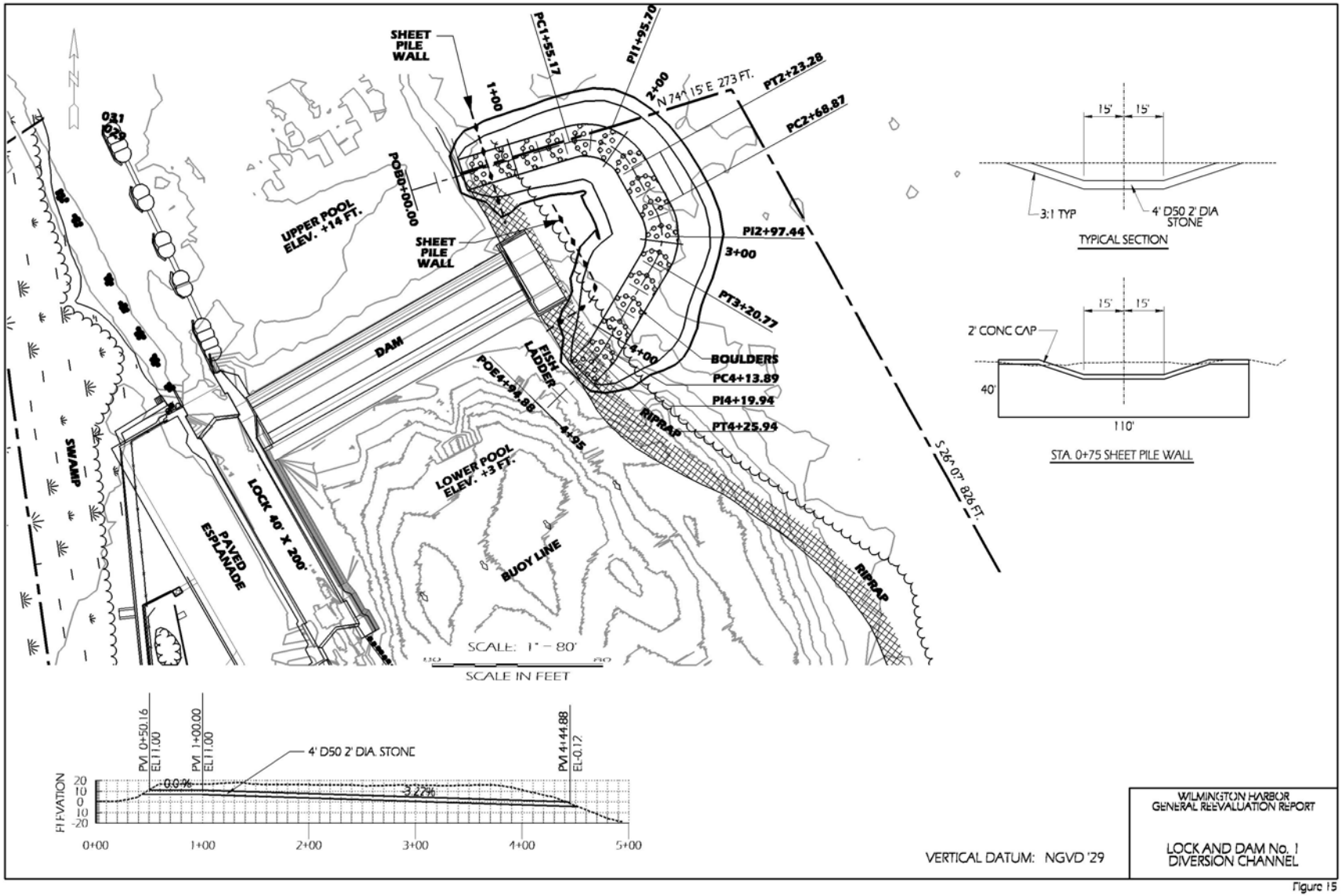
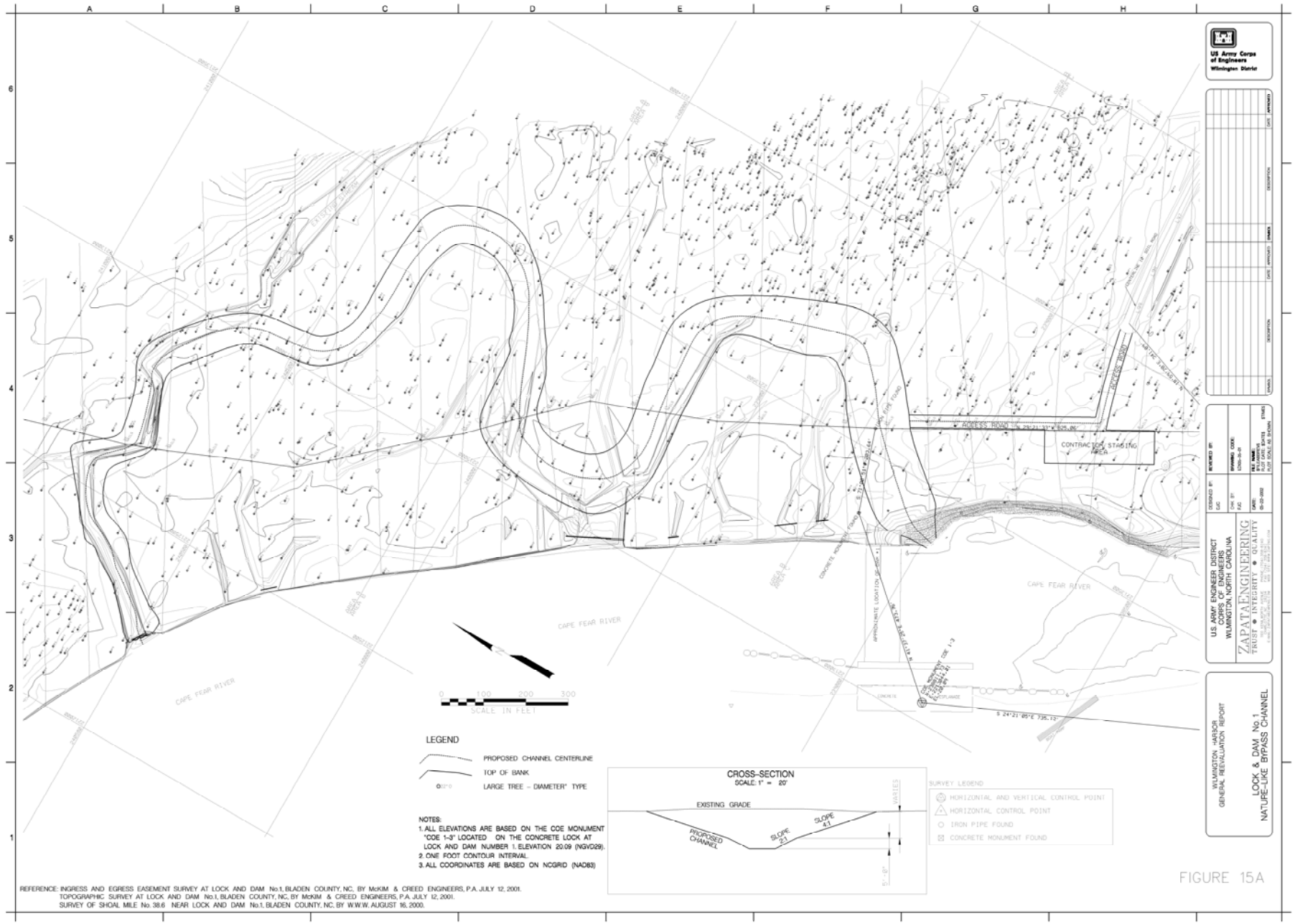

SHEET PILE WALL
PC1+55.17
PI1+95.70
1+00
2+00
N 74° 15' E 273 FT.
PT2+23.28
PC2+68.87
POB0+00.00
UPPER POOL ELEV. +14 FT.
SHEET PILE WALL
PI2+97.44
3+00
PT3+20.77
DAM
FISH LADDER
4+00
BOULDERS
PC4+13.89
POE4+94.98
PI4+19.94
PT4+25.94
4+95
RIPRAP
LOWER POOL ELEV. +3 FT.
S 26° 07' 826 FT.
SWAMP
LOCK 40' X 200'
PAVED ESPLANADE
BUOY LINE
RIPRAP
SCALE: 1" = 80'
SCALE IN FEET
15'
15'
3:1 TYP
4' D50 2' DIA STONE
TYPICAL SECTION
2' CONC CAP
40'
110'
STA. 0+75 SHEET PILE WALL
PVI 0+50.16
EL 11.00
PVI 1+00.00
EL 11.00
4' D50 2' DIA. STONE
PVI 4+44.88
EL-0.17
0.0%
-3.22%
ELEVATION
20
10
0
-10
-20
0+00
1+00
2+00
3+00
4+00
5+00
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 1
DIVERSION CHANNEL

Figure 15


US Army Corps of Engineers
Wilmington District
CAPE FEAR RIVER
CONTRACTOR STAGING AREA
ACCESS ROAD
SCALE IN FEET
LEGEND
PROPOSED CHANNEL CENTERLINE
TOP OF BANK
LARGE TREE – DIAMETER" TYPE
CROSS-SECTION
SCALE: 1" = 20'
EXISTING GRADE
PROPOSED CHANNEL
SLOPE 2:1
SLOPE 4:1
SURVEY LEGEND
HORIZONTAL AND VERTICAL CONTROL POINT
HORIZONTAL CONTROL POINT
IRON PIPE FOUND
CONCRETE MONUMENT FOUND
NOTES:
1. ALL ELEVATIONS ARE BASED ON THE COE MONUMENT "COE 1-3" LOCATED ON THE CONCRETE LOCK AT LOCK AND DAM NUMBER 1. ELEVATION 20.09 (NGVD29).
2. ONE FOOT CONTOUR INTERVAL.
3. ALL COORDINATES ARE BASED ON NCGRID (NAD83)
REFERENCE: INGRESS AND EGRESS EASEMENT SURVEY AT LOCK AND DAM No.1, BLADEN COUNTY, NC, BY McKIM & CREED ENGINEERS, P.A. JULY 12, 2001.
TOPOGRAPHIC SURVEY AT LOCK AND DAM No.1, BLADEN COUNTY, NC, BY McKIM & CREED ENGINEERS, P.A. JULY 12, 2001.
SURVEY OF SHOAL MILE No. 38.6 NEAR LOCK AND DAM No.1, BLADEN COUNTY, NC, BY W.W.W. AUGUST 16, 2000.
U.S. ARMY ENGINEER DISTRICT
CORPS OF ENGINEERS
WILMINGTON, NORTH CAROLINA
ZAPATA ENGINEERING
TRUST • INTEGRITY • QUALITY
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK & DAM No.1
NATURE-LIKE BYPASS CHANNEL
FIGURE 15A

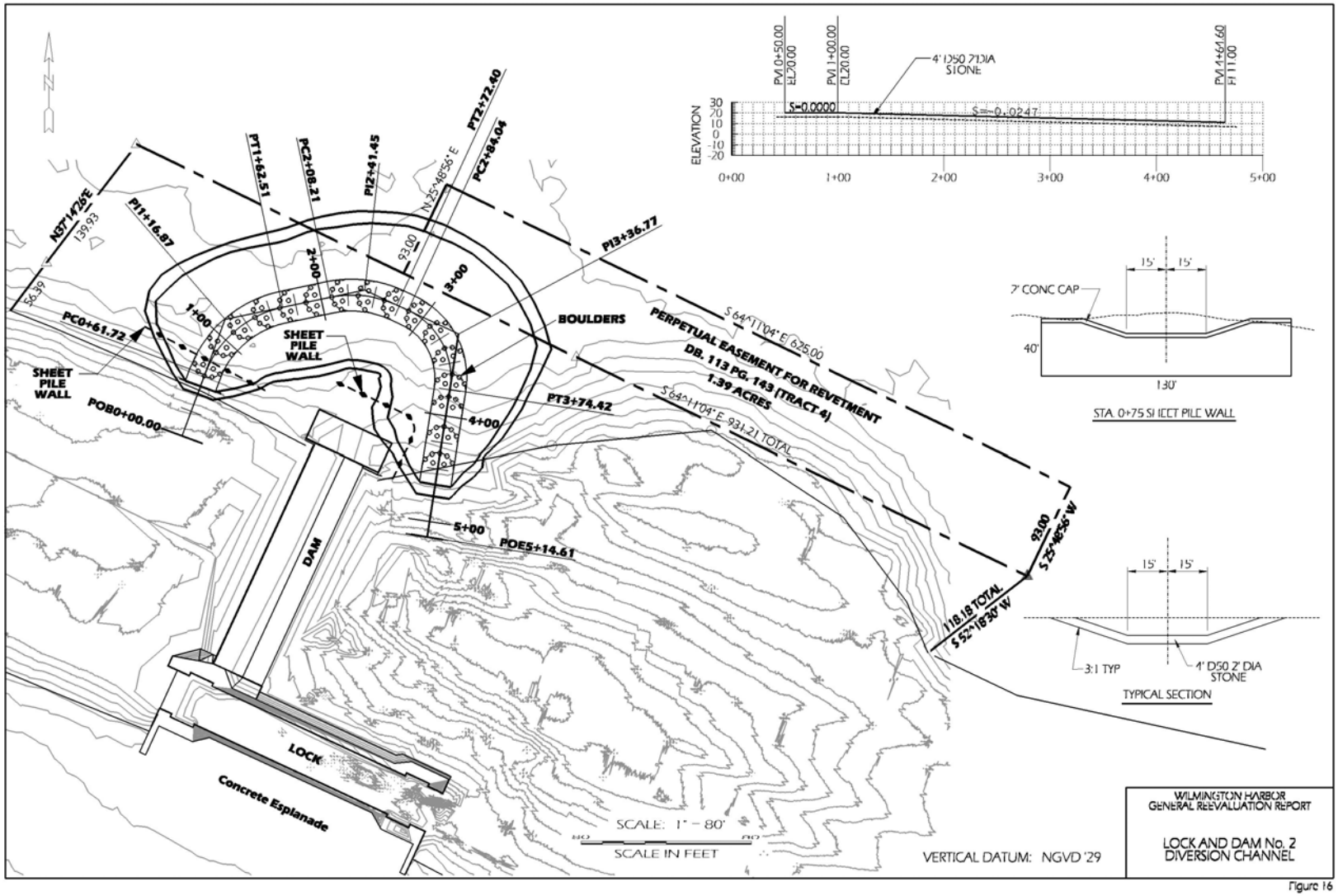

N
PM 0+50.00
EL 7.00
PM 1+00.00
EL 20.00
4' D50 7' DIA STONE
PM 4+61.60
EL 11.00
ELEVATION
S=0.0000
S=-0.0247
0+00
1+00
2+00
3+00
4+00
5+00
PT2+72.40
PC2+84.04
PT1+62.51
PC2+08.21
PI2+41.45
N 25°48'56" E
N37°47'25" E
139.93
56.39
PI1+16.87
PI3+36.77
93.00
PC0+61.72
SHEET PILE WALL
POB0+00.00
BOULDERS
PERPETUAL EASEMENT FOR REVETMENT
DB. 113 PG. 143 (TRACT 4)
1.39 ACRES
S 64°11'04" E 625.00
PT3+74.42
S 64°11'04" E 931.21 TOTAL
DAM
POE5+14.61
118.18 TOTAL
S 52°16'30" W
S 25°48'56" W
LOCK
Concrete Esplanade
SCALE: 1" = 80'
SCALE IN FEET
VERTICAL DATUM: NGVD '29
15'
15'
2' CONC CAP
40'
130'
STA. 0+75 SHEET PILE WALL
3:1 TYP
4' D50 2' DIA STONE
TYPICAL SECTION
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 2
DIVERSION CHANNEL

Figure 16

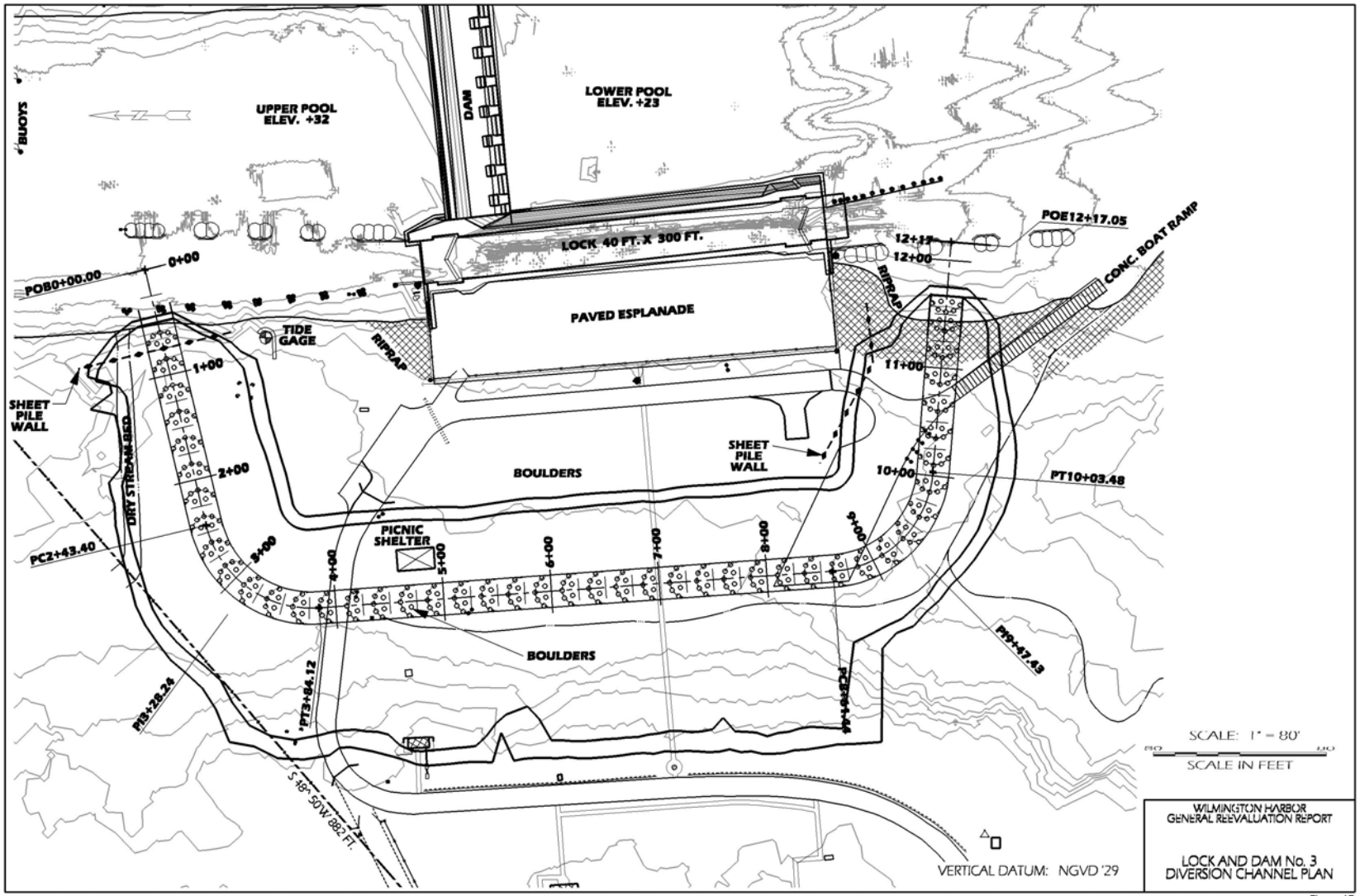
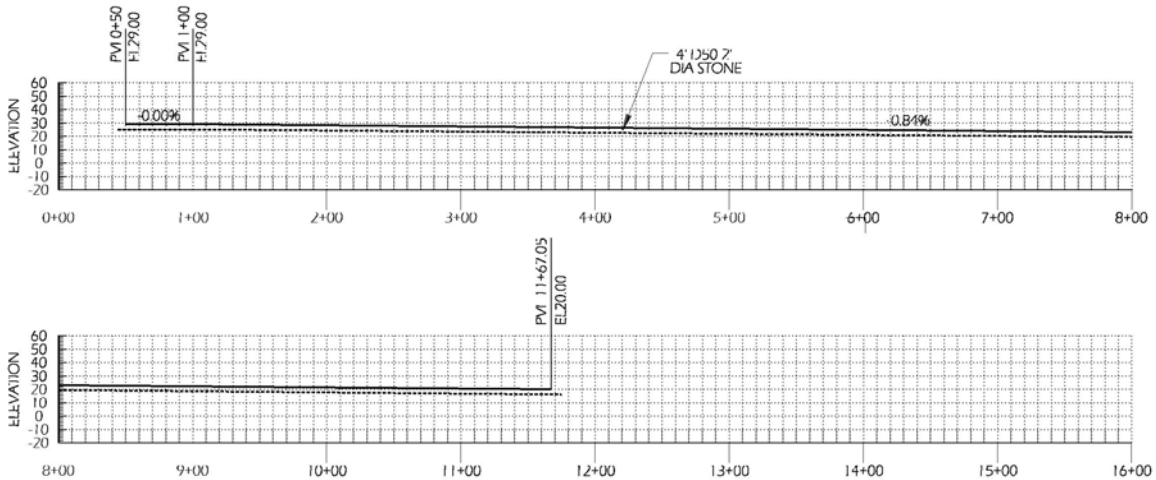
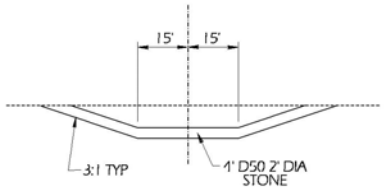
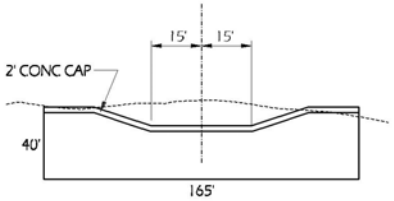

UPPER POOL
ELEV. +32
LOWER POOL
ELEV. +23
DAM
BUOYS
LOCK 40 FT. X 300 FT.
PAVED ESPLANADE
POB0+00.00
0+00
TIDE GAGE
RIPRAP
1+00
2+00
3+00
4+00
5+00
6+00
7+00
8+00
9+00
10+00
11+00
12+00
POE12+17.05
CONC. BOAT RAMP
SHEET PILE WALL
DRY STREAM BED
PC2+43.40
PT10+03.48
PICNIC SHELTER
BOULDERS
PT3+84.12
PT5+28.24
PT9+47.49
S 48° 50' W 882 FT.
SCALE: 1" = 80'
SCALE IN FEET
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 3
DIVERSION CHANNEL PLAN

Figure 17





TYPICAL SECTION



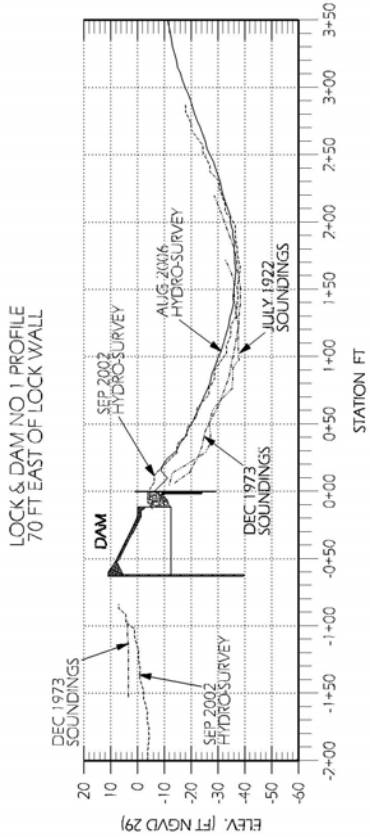
STA. 0+75 SHEET PILE WALL

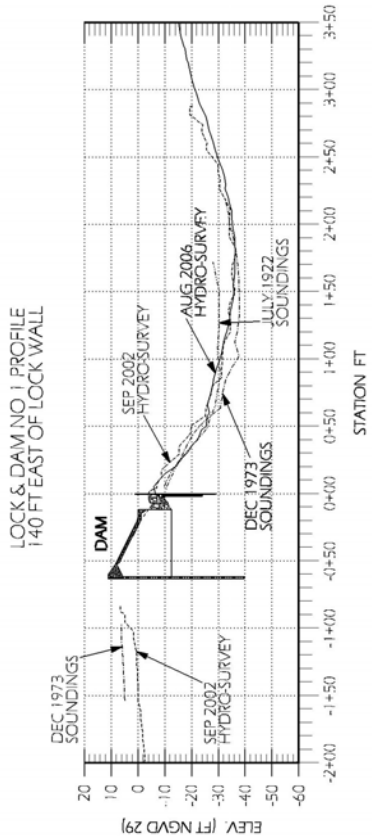
VERTICAL DATUM: NGVD '29

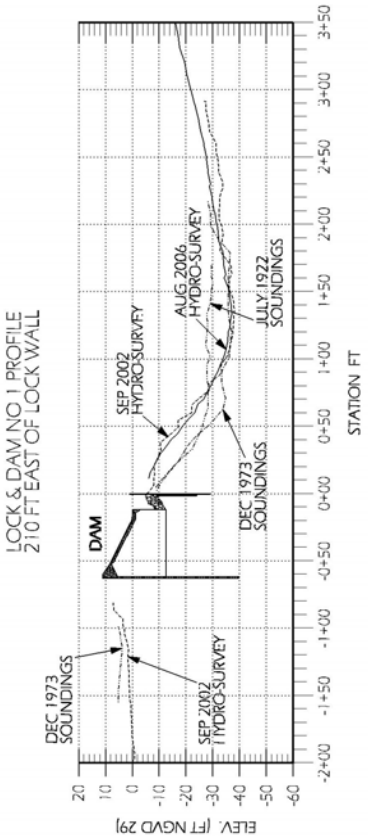
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT

LOCK AND DAM No. 3
DIVERSION CHANNEL PROFILE

Figure 18







SCALE: HORIZ.: 1"=80'
VERT.: 1"= 40'

VERTICAL DATUM: NGVD '29

WILMINGTON HARBOR
GENERAL REEVALUATION REPORT

LOCK AND DAM No. 1
PROFILES

Figure 19

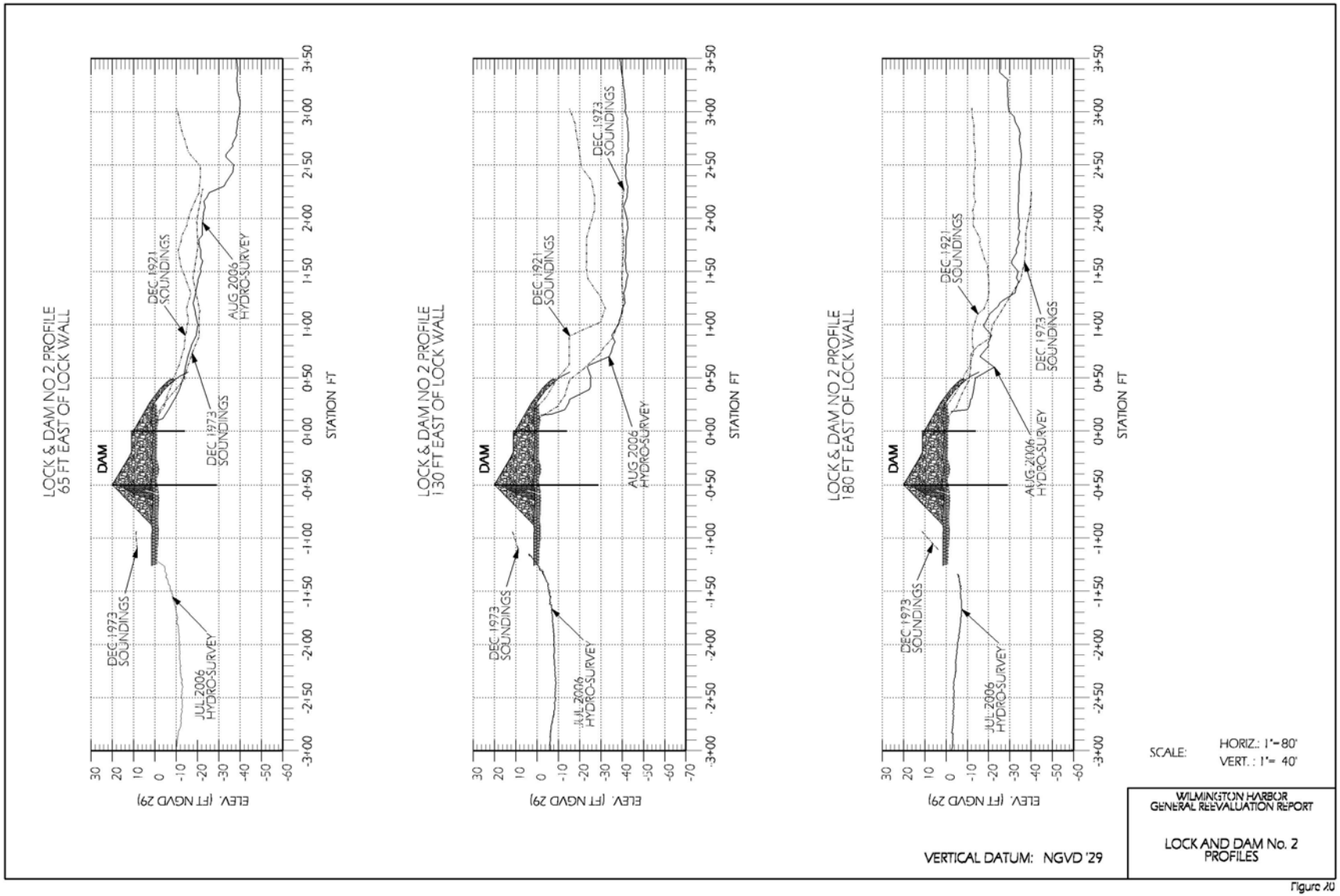
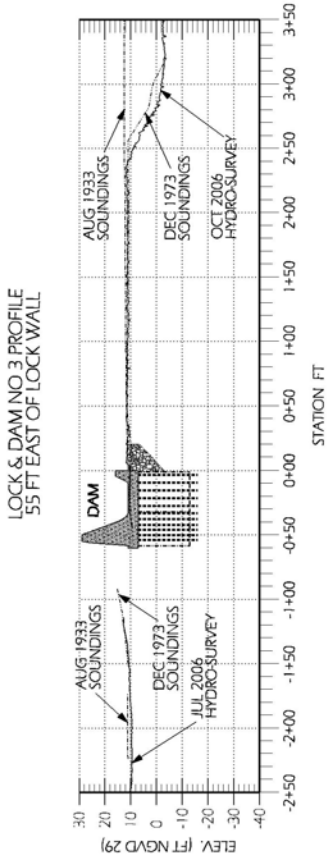
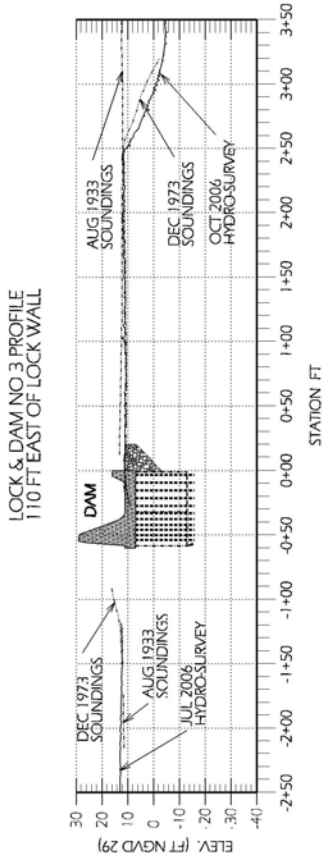
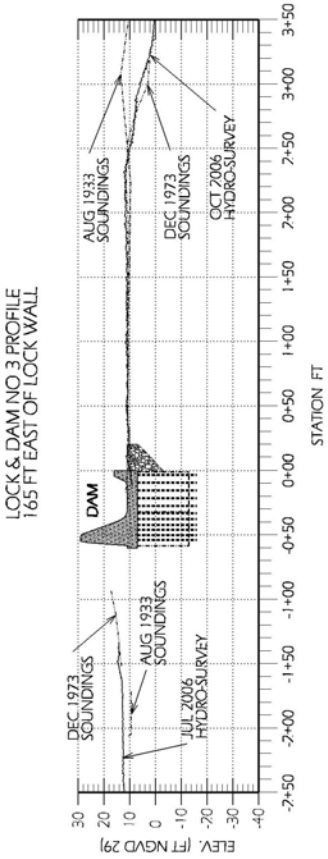

LOCK & DAM NO 2 PROFILE
65 FT EAST OF LOCK WALL
DAM
DEC 1973 SOUNDINGS
JUL 2006 HYDRO-SURVEY
DEC 1921 SOUNDINGS
DEC 1973 SOUNDINGS
AUG 2006 HYDRO-SURVEY
ELEV. (FT NGVD 29)
STATION FT
LOCK & DAM NO 2 PROFILE
130 FT EAST OF LOCK WALL
DAM
DEC 1973 SOUNDINGS
JUL 2006 HYDRO-SURVEY
DEC 1921 SOUNDINGS
DEC 1973 SOUNDINGS
AUG 2006 HYDRO-SURVEY
ELEV. (FT NGVD 29)
STATION FT
LOCK & DAM NO 2 PROFILE
180 FT EAST OF LOCK WALL
DAM
DEC 1973 SOUNDINGS
JUL 2006 HYDRO-SURVEY
DEC 1921 SOUNDINGS
DEC 1973 SOUNDINGS
AUG 2006 HYDRO-SURVEY
ELEV. (FT NGVD 29)
STATION FT
SCALE:
HORIZ.: 1"=80'
VERT.: 1"= 40'
VERTICAL DATUM: NGVD '29
WILMINGTON HARBOR
GENERAL REEVALUATION REPORT
LOCK AND DAM No. 2
PROFILES

Figure A0







SCALE: HORIZ.: 1"= 80'
VERT.: 1"= 10'

VERTICAL DATUM: NGVD '29

WILMINGTON HARBOR
GENERAL REEVALUATION REPORT

LOCK AND DAM No. 3
PROFILES

Figure 21

END

APPENDIX 10

RESOLUTIONS

Resolution In Support Of Construction of a Rock Ramp Cascade On Downstream Side of Lock & Dam #1 by the US Army Corps of Engineers As An Alternative for Removal of Lock & Dam #1 in Bladen County On the Cape Fear River as a Mitigation Measure for the General Re-evaluation Report and Environmental Impact Statement for the Wilmington Harbor Deep Draft Navigation Project

WHEREAS, the US Army Corps of Engineers, (the "Corps") has undertaken a General Re-evaluation Report to include an Environmental Impact Statement for the extension of the Wilmington Harbor Deep Draft Navigation Project which provides for Mitigation Measures to improve the passage of anadromous fish past Lock & Dam #1; and

WHEREAS, the Lock & Dam #1 is located is just downstream of the water supply intakes for the City of Wilmington (the "City") and the Lower Cape Fear Water and Sewer Authority (the "Authority") and provides for an adequate pool of saltwater free water for a population of approximately 250,000 citizens; and

WHEREAS, this water supply pool elevation behind the Lock & Dam is necessary to protect the existing and future water supply intakes for the City and the Authority and performs a vital mission in insuring the quality and quantity of the surface water supply intake requirements as well as an alternative to groundwater sources in the region; and

WHEREAS, the City and the Authority contacted for Professional Technical services to evaluate the various alternative modifications proposed by the Corps.

THEREFORE, BE IT RESOLVED; that the Board of Directors of the Authority support the alternative modification to construct a rock ramp cascade on the downstream side of Lock & Dam #1 on the Cape Fear River by the Corps that maintains the current height and elevation of the structure as the only alternative modification that would not impact the reliability of the water supply for the City's and Authority's customers.

Adopted this 12th day of February 2007

Larry Smith, Authority Chairman

Betty Watkins
Finance Officer/Executive Assistant

#23

Resolution

City Council
City of Wilmington
North Carolina

Introduced By: Sterling B. Cheatham, City Manager **Date:** 02/06/2007

Resolution Supporting the Construction of a Rock Ramp Cascade on the Downstream Side of Lock & Dam #1 by the US Army Corps of Engineers as an Alternative to the Removal of Lock & Dam #1 in Bladen County on the Cape Fear River as a Mitigation Measure for the General Re-evaluation Report and Environmental Impact Statement for the Wilmington Harbor Deep Draft Navigation Project

LEGISLATIVE INTENT/PURPOSE:

WHEREAS, the US Army Corps of Engineers, (the "Corps") has undertaken a General Re-evaluation Report to include an Environmental Impact Statement for the extension of the Wilmington Harbor Deep Draft Navigation Project which provides for Mitigation Measures to improve the passage of anadromous fish passage past Lock & Dam #1; and

WHEREAS, the Lock & Dam #1 is located just downstream of the water supply intakes for the City of Wilmington (the "City") and the Lower Cape Fear Water and Sewer Authority (the "Authority") and provides for an adequate pool of saltwater free water for a population of approximately 250,000 citizens; and

WHEREAS, this water supply pool elevation behind the Lock & Dam is necessary to protect the existing and future water supply intakes for the City and the Authority and performs a vital mission in insuring the quality and quantity of the surface water supply intake requirements as well as an alternative to groundwater sources in the region; and

WHEREAS, the City and the Authority contacted for Professional Technical services to evaluate the various alternative modifications proposed by the Corps.

THEREFORE, BE IT RESOLVED:

THAT, the Wilmington City Council supports the alternative modification to construct a rock ramp cascade on the downstream side of Lock & Dam #1 on the Cape Fear River by the Corps which would maintain the current height and elevation of the structure as the only alternative modification that would not impact the reliability of the water supply for the City's and Authority's customers.

CERTIFIED TO BE A TRUE COPY
Penelope Spicer Sidbury
CITY CLERK

Bill Saffo
Bill Saffo, Mayor

Adopted at a ~~regular~~ meeting
on February 6, 2007

ATTEST:
Penelope Spicer Sidbury
City Clerk

23-3

RESOLUTION
CAPE FEAR RIVER ASSEMBLY
ENCOURAGEMENT OF THE US CORPS OF ENGINEERS
TO CONSIDER ALTERNATIVES TO REMOVAL OF ALL LOCKS AND DAMS
IN BLADEN COUNTY ON THE CAPE FEAR RIVER
AS A MITIGATION MEASURE IN THE GENERAL RE-EVALUATION REPORT
AND ENVIRONMENTAL IMPACT STATEMENT FOR
THE WILMINGTON HARBOR DEEP DRAFT NAVIGATION PROJECT

Whereas, the US Army Corps of Engineers ("the Corps") has undertaken a General Re-evaluation Report to include an Environmental Impact Statement for extension of the Wilmington Harbor Deep Draft Navigation Project which provides for mitigation measures that include an alternative that recommends removal of all Locks and Dams on the Cape Fear Rives in Bladen County as referenced in a report dated January 31, 2006; and,

Whereas, the Corps is seeking to eliminate the annual cost for the operation and maintenance expense of the Locks and Dams, as these structures no longer perform the US Congressional authorization of providing navigation for commercial traffic, but are operated for the annual passage of anadromous fish; and,

Whereas, there are viable options that support successful annual anadromous fish migration up and down the Cape Fear River that do not involve removal of the Locks and Dams; and,

Whereas, the Locks and Dams on the Cape Fear River represent potential dynamic economic value to the struggling and developing local economies along the Cape Fear River including Cumberland, Bladen and New Hanover Counties; and,

Whereas, the presence of the Locks and Dams support a rising vision among local representatives of greater recreational use of the Cape Fear River; and,

Whereas, the raw water intakes behind the Locks and Dams on the Cape Fear River provide a saltwater free pool of water for a significant population in Southeastern North Carolina and,

Whereas, the Locks and Dams on the Cape Fear River are vital for insuring a quantity and quality of raw water supply as an alternative to weakening groundwater sources; and,

Whereas, Lock and Dam No.1 is located immediately downstream of the raw water supply intakes for the Lower Cape Fear Water and Sewer Authority ("the Authority") and the City of Wilmington; and,

Whereas, Locks and Dams Nos. 2 and 3 are located downstream of the raw water supply intakes of the City of Fayetteville and the Dupont Companies Plant location in Bladen County; and,

Whereas, per Lower Cape Fear River Water and Sewer Authority, there are imminent raw water supply intake plans between Lock and Dam No.2 and Lock and Dam No. 3; and,

Whereas, there may be unwanted and not completely understood riparian wetland and sediment issues for the Cape Fear River Basin associated with the removal of Lock and Dams Nos. 1, 2, and 3; and,

Whereas, alternatives are under discussion to keep the Locks and Dams in functional condition that support that would relieve the Corps of ownership and operational responsibilities;

Now Be It Therefore Resolved, that the President and Board of Directors of the Cape Fear River Assembly, by virtue of the adoption of this resolution, encourage the US Corps of Engineers to strongly consider alternatives to the removal of the Locks and Dams on the Cape Fear River in Bladen County; and,

Further, Be It Therefore Resolved, that the President and Board of Directors of the Cape Fear River Assembly, by virtue of the adoption of this resolution, ask the US Corps of Engineers to acknowledge a significant local concern in Southeastern North Carolina over the prospect of removal of the Locks and Dams and that Corps dialogue with State and local government and citizens dealing with the sustained future of the Locks and Dams continue.

Adopted this 17th day of October, 2006.

_____

Roger E. Sheats, Jr.
President

805 West Broad Street
Post Office Box 716
Elizabethtown, N.C. 28337

TOWN OF
ELIZABETHTOWN
TOWN MANAGER'S OFFICE

(910) 862-2066
Fax (910) 862-7117
www.elizabethtownnc.org
Email manager@elizabethtownnc.org

October 4, 2006

Miss Sharon F. Harggett
Project Manager
U.S. Army Corps of Engineers
Wilmington District
P.O. Box 1890
Wilmington, NC 28402-1890

RE: A Resolution for the Removal of All Locks and Dams in Bladen County on the Cape Fear River

Dear Miss Harggett:

Transmitted herewith is an original Resolution as above-referenced and approved by Town Council on October 2, 2006. As is noted in the resolution, of utmost concern is the importance of the lock and dam system to the current and future water supply for much of Southeastern North Carolina.

Currently, the raw water intakes behind Lock & Dam No. 1 provide a saltwater free pool of water for a significant population in Southeast North Carolina. This water supply pool elevation behind the Dam is necessary to protect the existing and future supply intakes for the Authority and the City of Wilmington. Lock and Dam No. 1 performs a vital mission in insuring the quality of the raw water requirements, as an alternative to groundwater sources in the region.

Additionally, the Lower Cape Fear Water and Sewer Authority (LCFWSA) has a proposed additional raw water intake supply project in a planning stage behind Lock & Dam No. 2 that could service Elizabethtown, Bladen County and, potentially, parts of Sampson, Cumberland, and Columbus Counties. The lock and dam system is crucial to this planned facility.

Furthermore, both recreational boaters and fishermen have enjoyed the use of the U. S. Army Corps of Engineers locks and dams on the Cape Fear for many years, and there is a great concern about the recreation impact if the lock and dams were eliminated.

I hope the U. S. Army Corps of Engineers gives the aforementioned items the utmost consideration during its evaluation of lock and dam system on the Cape Fear River. Please let me know if you have any questions about the attached resolution.

Sincerely,

David B. Bone
Town Manager

Attach.
Ccs: Mayor Kenneth R. Kornegay
Congressman Mike McIntyre

TOWN OF ELIZABETHTOWN

RESOLUTION # R-06-017

A RESOLUTION OPPOSING THE US ARMY CORPS OF ENGINEERS PROPOSED ALTERNATIVE FOR REMOVAL OF ALL LOCKS AND DAMS IN BLADEN COUNTY ON THE CAPE FEAR RIVER AS A MITIGATION MEASURE IN THE GENERAL RE-EVALUATION REPORT AND THE ENVIRONMENTAL IMPACT STATEMENT FOR THE WILMINGTON HARBOR DEEP DRAFT NAVIGATION PROJECT

Town Council member Rufus Lloyd, seconded by Town Council member Herman Lewis, moved that the following be adopted:

WHEREAS, the US Army Corps of Engineers (the "Corps") has undertaken a General Reevaluation Report to include an Environmental Impact Statement for extension of the Wilmington Harbor Deep Draft Navigation Project which provides for Mitigation Measures that include an alternative that recommends removal of all Locks and Dams on the Cape Fear River in Bladen County as reference in a report dated January 31, 2006; and

WHEREAS, a primary focus of the Corps project team ahs been the proposed removal of Lock and Dam No. 1, which is located immediately downstream of the raw water supply intakes for the Lower Cape Fear Water & Sewer Authority (the "Authority") and the City of Wilmington. As this would enhance the environmental mitigation measure required for the Wilmington Harbor Deep Draft Navigation Project to improve the passage of anadromous fish past the dam; and

WHEREAS, the Board finds that the preservation of regional groundwater resources in Elizabethtown and surrounding areas necessitates the evaluation of alternative water sources, including surface water; and

WHEREAS, the Corps is seeking to eliminate the annual cost for the operation and maintenance expense of the locks and dams as these structures no longer perform the Congressional authorization of providing navigation for commercial traffic, but are operated for the annual passage of anadromous fish; and

WHEREAS, the raw water intakes behind Lock & Dam No. 1 provide a saltwater free pool of water for a significant population in Southeast North Carolina. This water supply pool elevation behind the Dam is necessary to protect the existing and future supply intakes for the Authority and the City of Wilmington. Lock and Dam No. 1 performs a vital mission in insuring the quality of the raw water requirements, as an alternative to groundwater sources in the region; and

FURTHERMORE, the Authority has a proposed additional raw water intake supply project in a planning stage behind Lock & Dam No. 2 that could service Elizabethtown, Bladen County and, potentially, parts of Sampson, Cumberland, and Columbus Counties;

NOW THEREFORE BE IT RESOLVED, that the Mayor and Town Council of Elizabethtown, by virtual adoption of this Resolution, opposes the removal of all Locks and Dams in Bladen County on the Cape Fear River by the US Army Corps of Engineers.

Adopted this the 2nd day of October, 2006.

Kenneth R. Kornegay
Mayor

ATTEST:

Juanita Hester
Town Clerk

Resolution Opposing the US Army Corps of Engineers Proposed Alternative for Removal of all Locks and Dams in Bladen County on the Cape Fear River as a Mitigation Measure in the General Re-evaluation Report and the Environmental Impact Statement for the Wilmington Harbor Deep Draft Navigation Project

Whereas, the US Army Corps of Engineers (the "Corps") has undertaken a General Reevaluation Report to include an Environmental Impact Statement for extension of the Wilmington Harbor Deep Draft Navigation Project which provides for Mitigation Measures that include an alternative that recommends removal of all Lock and Dams on the Cape Fear River in Bladen County as referenced in a report dated January 31, 2006; and

Whereas, a primary focus of the Corps project team has been the proposed removal of Lock and Dam No. 1, which is located immediately downstream of the raw water supply intakes for the Lower Cape Fear Water & Sewer Authority (the "Authority") and the City of Wilmington, ("City"). As this would enhance the environmental mitigation measure required for the Wilmington Harbor Deep Draft Navigation Project to improve the passage of anadromous fish past the dam; and

Whereas, the Corps is seeking to eliminate the annual cost for the operation and maintenance expense of the locks and dams as these structures no longer perform the Congressional authorization of providing navigation for commercial traffic, but are operated for the annual passage of anadromous fish; and

Whereas, the raw water intakes behind Lock & Dam No. 1 provide a saltwater free pool of water for a significant population in Southeastern North Carolina. This water supply pool elevation behind the Dam is necessary to protect the existing and future supply intakes for the Authority and the City. Lock and Dam No. 1 performs a vital mission in insuring the quality and quantity of the raw water supply requirements, as an alternative to groundwater sources in the region; and

Furthermore, the Authority has a proposed additional raw water intake supply project in a planning stage behind Lock and Dam No. 2.

Now Therefore Be It Resolved, that the Chairman and Board of Directors for the Authority by virtual of adoption of this Resolution opposes the removal of all Lock and Dams in Bladen County on the Cape Fear River by the US Army Corps of Engineers.

Adopted this 8th day of May 2006

William A Carter

William Caster, Chairman

Betty Watkins

Betty Watkins
Finance Officer/Executive Assistant

CITY of WILMINGTON
North Carolina

P.O. BOX 1810
28402

OFFICE OF THE CITY MANAGER
(910) 341-7810
FAX (910) 341-5839
TDD (910) 341-7873

04/04/2006

City Council
City Hall
Wilmington, North Carolina 28401

Dear Mayor and Councilmembers:

The US Army Corps of Engineers (the "Corps") is currently studying the modification and/or removal of Lock and Dam No. 1 to improve the passage of anadromous fish past the dam as an environmental mitigation measure related to the Wilmington Harbor dredging project. The Corps would also like to eliminate the operation and maintenance expense of the lock and dam because they no longer perform the function of passing commercial traffic.

Lock and Dam No. 1 is located in Bladen County on the Cape Fear River. The dam also happens to be located just downstream of the water supply intakes for the City of Wilmington and the Lower Cape Fear Water and Sewer Authority, providing the water supply pool for approximately 250,000 citizens. Lock and Dam No. 1 also provides a physical barrier to salt water contamination of the water supply. Groundwater sources are already stressed and do not present a viable alternative water supply, so the water supply pool elevation is necessary to protect existing water supply intakes. Lock and Dam No. 1 performs a vital role in meeting the water supply needs of Southeastern North Carolina.

The Resolution attached for your consideration asks that the Corps include considerations related to the water supply, economic, and financial impacts to the citizens of the City of Wilmington and the residents of Southeastern North Carolina when taking any and all actions related to the modification and/or removal of Lock and Dam No. 1.

Passage of the attached Resolution is recommended.

Respectfully submitted,

Sterling B. Cheatham
City Manager

Resolution

City Council
City of Wilmington
North Carolina

Introduced By: Sterling B. Cheatham, City Manager **Date:** 04/04/2006

Resolution Requesting that the US Army Corps of Engineers Consider Potential Impacts to the Citizens of Wilmington and Southeastern North Carolina when Studying Alternatives for the Modification and/or Removal of Lock and Dam No. 1

LEGISLATIVE INTENT/PURPOSE:

WHEREAS, the US Army Corps of Engineers (the "Corps") is conducting a study of alternatives for the modification and/or removal of Lock and Dam No. 1 on the Cape Fear River in Bladen County; and

WHEREAS, the dam is located just downstream of the water supply intakes for the City of Wilmington and the Lower Cape Fear Water and Sewer Authority; and

WHEREAS, the Corps is required to improve passage of anadromous fish past the dam as an environmental mitigation measure related to the Wilmington Harbor dredging project; and

WHEREAS, the Corps would like to eliminate the operation and maintenance expense of the lock and dam because they no longer perform the function of passing commercial traffic as authorized by Congress; and

WHEREAS, Lock and Dam No. 1 provides the water supply pool for a population of approximately 250,000 citizens; and

WHEREAS, the water supply pool elevation is necessary to protect existing water supply intakes; and

WHEREAS, Lock and Dam No. 1 provides a physical barrier to salt water contamination of the water supply; and

WHEREAS, groundwater sources are stressed; and

WHEREAS, Lock and Dam No. 1 performs a vital role in meeting the water supply needs of Southeastern North Carolina.

THEREFORE, BE IT RESOLVED:

THAT, the City of Wilmington City Council does hereby request that any and all actions related to the modification and/or removal of Lock and Dam No. 1 include considerations related to the water supply, economic, and financial impacts to the citizens of the City of Wilmington and Southeastern North Carolina.

_____
Spence H. Broadhurst, Mayor

Adopted at a _____ meeting
on _____, 2006.

ATTEST:

_____
City Clerk

Appendix 12

MCACES ESTIMATE

WILMINGTON HARBOR - GRR
ROCK RAPIDS & NEW TURNING BASIN
SUMMARY OF COSTS
CURRENT WORKING ESTIMATE (CWE)
CODE OF ACCOUNTS

Designed By: USACE - WILMINGTON DISTRICT
Estimated By: CESAW-TS-EE

Prepared By: John C. Caldwell
CESAW-TS-EE

Preparation Date: 07/06/07
Effective Date of Pricing: 07/06/07

Sales Tax: 0.00%

This report is not copyrighted, but the information
contained herein is For Official Use Only.

M C A C E S G O L D E D I T I O N
Composer GOLD Software Copyright (c) 1985-2000
by Building Systems Design, Inc.
Release 5.31

LABOR ID: JC2006 EQUIP ID: JC2006 Currency in DOLLARS CREW ID: JC2006 UPB ID: JC2006

The Northeast Cape Fear River deepening was evaluated to determine the best location for turning ships. The current authorized project has two(2) turning basins at Reach 2 near Southern States and Reach 7 near old Almont (now PPD). The authorized turning basins would have to be deepened and widened. As an alternative to these 2 basins, a single turning basin near Reach 3 was considered to replace these 2 basins.

In evaluating the turning basin alternatives, there would be environmental mitigation for disturbance of Primary Nursury Areas constructing any of the alternatives. Mitigation Islands, rock rapids for fish passage at the 3 Locks & Dams, a nature like bypass around the dams, short diversion channels, etc. were evaluated to determine the most economical and effective means of mitigating for the turning basin options.

All of the turning basin options and mitigation options are described in detail in the GRR REPORT.

This MCACES summarizes the costs associated with the selected options as follows

1. Construct one turning basin at Reach 3 (referred to as the Southern Turn Basin at Reach 3).
2. Construct rock rapids for fish passage at Lock & Dam 2 & 3.
3. Rehab Lock and Dam 2 to prevent deteriation.

A contingency of 20% was used for all areas of the cost estimate except for ACCOUNT 06. The 25% was used because of the current level of quantities and detail known to establish the costs.

LABOR ID: JC2006 EQUIP ID: JC2006 Currency in DOLLARS CREW ID: JC2006 UPB ID: JC2006

Wed 22 Aug 2007 U.S. Army Corps of Engineers TIME
17:06:51
Eff. Date 07/06/07 PROJECT WHMGRR: WILMINGTON HARBOR - GRR - ROCK RAPIDS & NEW TURNING BASIN
WILMINGTON HARBOR GRR SUMMARY
PAGE 1

** PROJECT OWNER SUMMARY - Contract (Rounded to 100's) **

		QUANTY	UOM	CONTRACT	CONTINGN	TOTAL COST	UNIT
06	FISH & WILDLIFE FACILITIES			6,125,900	1,531,500	7,657,400	
12	NAVIGATION, PORTS, AND HARBORS			10,861,100	2,172,200	13,033,400	
30	PLANNING, ENGINEERING, & DESIGN			2,000,000	400,000	2,400,000	
31	SUPERVISION AND ASSURANCE			2,000,000	400,000	2,400,000	
TOTAL	WILMINGTON HARBOR - GRR	1.00	PRJ	20,987,100	4,503,700	25,490,800	25490796

LABOR ID: JC2006 EQUIP ID: JC2006 Currency in DOLLARS CREW ID: JC2006 UPB ID: JC2006

** PROJECT OWNER SUMMARY - Scope (Rounded to 100's) **

	QUANTY	UOM	CONTRACT	CONTINGN	TOTAL COST	UNIT
06 FISH & WILDLIFE FACILITIES						
06.01 FISH FACILITIES AT DAMS						
06.01.01 FISH FACILITIES AT DAMS						
06.01.01.01 ROCK RAPIDS-LOCK & DAM 2			5,525,900	1,381,500	6,907,400	
06.01.01.04 MONITORING			600,000	150,000	750,000	
TOTAL FISH FACILITIES AT DAMS			6,125,900	1,531,500	7,657,400	
TOTAL FISH FACILITIES AT DAMS			6,125,900	1,531,500	7,657,400	
TOTAL FISH & WILDLIFE FACILITIES			6,125,900	1,531,500	7,657,400	
12 NAVIGATION, PORTS, AND HARBORS						
12.02 HARBORS						
12.02.01 SOUTHERN TURNING BASIN OPTION						
12.02.01.01 SOUTHERN TURN BASIN	633672	CY	10,861,100	2,172,200	13,033,400	20.57
TOTAL SOUTHERN TURNING BASIN OPTION	633672	CY	10,861,100	2,172,200	13,033,400	20.57
TOTAL HARBORS			10,861,100	2,172,200	13,033,400	
TOTAL NAVIGATION, PORTS, AND HARBORS			10,861,100	2,172,200	13,033,400	

30	PLANNING, ENGINEERING, & DESIGN		2,000,000	400,000	2,400,000
31	SUPERVISION AND ASSURANCE		2,000,000	400,000	2,400,000
	TOTAL WILMINGTON HARBOR - GRR	1.00 PRJ	20,987,100	4,503,700	

25,490,80025490796

LABOR ID: JC2006 EQUIP ID: JC2006 Currency in DOLLARS CREW ID: JC2006 UPB ID: JC2006

No errors detected...

* * * END OF ERROR REPORT * * *

LABOR ID: JC2006 EQUIP ID: JC2006 Currency in DOLLARS CREW ID: JC2006 UPB ID: JC2006

SUMMARY REPORTS SUMMARY PAGE

PROJECT OWNER SUMMARY - Contract.....1
PROJECT OWNER SUMMARY - Scope.....2

No Detailed Estimate...

No Backup Reports...

* * * END TABLE OF CONTENTS * * *

END