Fire Island Inlet to Montauk Point, NY
Final General Reevaluation Report

APPENDIX A

ENGINEERING

U.S. Army Corps of Engineers
New York District

February 2020

Updated April 2020

FIRE ISLAND TO MONTAUK POINT REFORMULATION STUDY – FINAL GRR
Appendix A

Engineering

TABLE OF CONTENTS

1.0 PROJECT DESCRIPTION 1
1.1 STUDY HISTORY 1
1.1.1 Study Authority 1
1.2 PURPOSE 2
1.3 VISION 2
1.4 PROBLEM IDENTIFICATION 4
2.0 SHORELINE HISTORY 5
2.1 HISTORICAL STORMS, BREACHING AND OVERWASH 5
2.2 HISTORICAL DEVELOPMENT AND MANAGEMENT OF PROJECT AREA 17
3.0 EXISTING SITE CONDITIONS 20
3.1 GEOLOGY 20
3.2 MAJOR MORPHOLOGICAL FEATURES 20
3.3 BEACH PROFILE CHARACTERISTICS AND MORPHOLOGICAL REACHES 21
3.3.1 Representative Profiles 21
3.3.2 Sediment Characteristics 25
3.3.3 Reach Delineation 25
3.3.4 Climate 29
3.3.5 Astronomical Tides 29
3.3.6 Sea Level Change 29
3.3.7 Storms 30
3.3.8 Winds 32
3.3.9 Storm Surge and Extreme Water Levels 35
3.3.10 Waves 36
3.4 SHORELINE CHANGES AND EROSION 42
3.5 SHORELINE UNDULATIONS 45
3.6 INLETS 45
3.6.1 Shinnecock Inlet 46
3.6.2 Moriches Inlet 46
3.6.3 Fire Island Inlet 47
3.6.4 The Wilderness Area Breach 47
3.7 BAYSIDE TIDAL HYDRODYNAMICS 47
4.0 SITE CONDITIONS, INTERIM PROJECTS AND PROBLEM IDENTIFICATION 49
4.1 INLET DREDGING AND BYPASSING 49
4.1.1 Fire Island Inlet 49
4.1.2 Moriches Inlet 49
4.1.3 Shinnecock Inlet 49
4.2 WESTHAMPTON INTERIM PROJECT 49
4.3 WEST OF SHINNECOCK INLET (WOSI) PROJECT 50

4.4 POST-SANDY ONE-TIME STABILIZATION EFFORTS 50
4.4.1 Fire Island Inlet to Moriches Inlet Stabilization Project (FIMI)...............................51
4.4.2 Downtown Montauk Stabilization Project 51
4.5 INTERIM BREACH CONTINGENCY PLAN (BCP) 51
4.6 WITHOUT PROJECT CONDITIONS 52
4.6.1 Pre-Sandy Baseline Conditions 52
4.6.2 Baseline Conditions (BLC)..52
4.6.3 Future Vulnerable Conditions (FVC) 52
4.6.4 Breach Closed Conditions (BCC)...52
4.6.5 Breach Open Conditions (BOC)..53

5.0 BORROW SOURCE INVESTIGATIONS..56

6.0 COASTAL PROCESSES INVESTIGATIONS 57

6.1 STORM SURGE AND STORM-INDUCED BARRIER ISLAND BREACHING MODELING................57
6.1.1 Numerical Models 63
6.1.2 Simulation Results 70
6.1.3 Stage-Frequency Methodology 92
6.1.4 Stage-Frequency and Numerical Model Uncertainty 93
6.1.5 Ocean Stage-Frequency Relationships..95
6.1.6 Without Project Bay Stage-Frequency Relationships...102
6.1.7 Breaching and Overwash Frequency 105
6.2 STORM-INDUCED BEACH EROSION 113
6.2.1 Storm Selection 113
6.2.2 Response-Frequency Methodology 114
6.2.3 Baseline Conditions Response-Frequency Relationships 115
6.3 SEDIMENT BUDGET 135
6.3.1 Previous Work 135
6.3.2 Recent Work...138
6.3.3 Methodology and Data Sources..140
6.3.4 Recent (1995-2001) Regional Sediment Budget...143
6.3.5 Existing (c. 2001) Regional Sediment Budget..149
6.4 INLET PROCESSES..152
6.4.1 Modeling System 153
6.4.2 Existing Condition 153
6.4.3 Consideration of Alternative Project Modification Conditions 175
6.5 LONG-TERM BREACH PROCESSES...176
6.5.1 Long-Term Breach Growth Estimates...176
6.5.2 Breach Sediment Transport Estimates 179
6.6 OVERWASH PROCESSES..188
6.6.1 Overwash Deposition...188
6.6.2 Estimated Reduction in Annual Overwash Areas...190
6.6.3 Average Reduction in Overwash Bare-sand Habitat over Project Life 193
6.6.4 Comparison to Historic Overwash..194

7.0 ALTERNATIVES DEVELOPMENT 196

7.1 INTRODUCTION 196
7.2 REACH DELINEATION 196
7.2.1 Project Reaches 197
7.2.2 Physical Reaches and Design Subreaches...197
7.2.3 New (Year 2000) Reach Designations 197

7.3 MEASURES CONSIDERED 199
7.4 INITIAL SCREENING 199
7.4.1 No Action..*199*
7.4.2 Coastal Process Restoration *200*
7.4.3 Sediment Management (Inlet Sand Modification)..............................*200*
7.4.4 Breach Response Measures...*201*
7.4.5 Beach Restoration *201*
7.4.6 Offshore Breakwaters..*202*
7.4.7 Seawalls *202*
7.4.8 Groins...*203*
7.4.9 Beach Restoration and Structures *203*
7.4.10 Removal/Modification of Groins *203*
7.4.11 Levees and Floodwalls *203*
7.4.12 Storm Closure Gates *204*
7.4.13 Results of Initial Screening *205*
7.5 SECONDARY SCREENING OF STORM DAMAGE REDUCTION MEASURES..........................205
7.5.1 Coastal Process Restoration *208*
7.5.2 Breach Response Measures...*210*
7.5.3 Beach Restoration *211*
7.5.4 Sediment Management (including Inlet Modifications) *212*
7.5.5 Removal/Modification of Groins *220*
7.5.6 Beach Restoration with Structures...*220*
7.5.7 Mainland Road Raising..*221*
7.6 CONCLUSIONS, ALTERNATIVE MEASURES SELECTION 222
7.7 DETAILED EVALUATION OF INDIVIDUAL STORM DAMAGE REDUCTION (SDR) MEASURES..223
7.7.1 Introduction..*223*
7.7.2 Non-Structural Measures..*224*
7.7.3 Breach Response Alternatives...*240*
7.7.4 Beachfill Alternatives...*260*
7.7.5 Sediment and Inlet Management Measures......................................*284*
7.7.6 Groin Modification Alternatives *300*
7.7.7 Land and Development Management *309*
7.7.8 Design and Evaluation of Alternative Findings *317*
7.8 ALTERNATIVE PLAN EVALUATION 319
7.8.1 Identification of Storm Damage Reduction Alternatives by Reach *319*
7.8.2 Plan 1 ....*319*
7.8.3 Plan 2 ....*326*
7.8.4 Plan 3 ....*336*
7.8.5 Summary of NED Alternative Integration *349*
7.9 INTEGRATION OF FEATURES FOR RECOMMENDED ALTERNATIVE 351
7.9.1 Integration of Groin Modification Alternatives...................................*352*
7.9.2 Integration of Restoration Alternatives *353*
7.9.3 Integration of Appropriate Land Use and Development Measures *354*
7.9.4 Consideration of the life cycle management of these plans *357*
7.9.5 Adaptive Management..*358*
7.9.6 Summary of Alternative Plan Comparison *359*

8.0 POST-SANDY TFSP MODIFICATIONS..361

8.1 FIMI BEACHFILL ALIGNMENT AND REAL ESTATE 363
8.1.1 Initial Construction Quantities *364*

8.1.2 Advance Fill and Renourishment *368*
8.1.3 Cost Basis Summary-MREI/MIDU Comparison *382*
8.2 BREACH RESPONSE COSTS 383
8.2.1 2007 Price Levels *383*
8.2.2 2013 Price Levels *384*
8.3 STORM SURGE MODELING 384

9.0 RECOMMENDED PLAN 385

9.1 OVERVIEW 385
9.1.1 Inlet Sand Bypassing *385*
9.1.2 Mainland and Non-Structural *385*
9.1.3 Barrier Islands *385*
9.1.4 Groin Modifications *386*
9.1.5 Coastal Process Features (CPFs) *386*
9.1.6 Adaptive Management *387*
9.1.7 Integration of Local Land Use Regulations and Management *387*
9.2 INLET SAND BYPASSING 392
9.2.1 Inlet Management – Initial Construction *392*
9.2.2 Inlet Management – Life Cycle *393*
9.3 NON-STRUCTURAL AND ROAD RAISING 394
9.4 BREACH RESPONSE PLANS 395
9.4.1 Proactive Breach Response Plan *395*
9.4.2 Reactive Breach Closure *397*
9.4.3 Conditional and Wilderness Conditional Breach Closure *397*
9.4.4 Breach Closure Costs *398*
9.5 BEACH FILL PLAN 401
9.5.1 Beach Fill Plan – Initial Construction *403*
9.5.2 Beach Fill Plan – Year 1 to Year 30 .... *403*
9.6 SEDIMENT MANAGEMENT AT DOWNTOWN MONTAUK 404
9.7 GROIN MODIFICATION PLAN 405
9.8 COASTAL PROCESSES FEATURES 405
9.9 INTEGRATION OF LOCAL LAND USE REGULATIONS AND MANAGEMENT 407

10.0 OPERATION, MAINTENANCE, AND MONITORING 408

10.1 OPERATIONS AND MAINTENANCE 408
10.2 MONITORING 408

11.0 REFERENCES 410

12.0 GLOSSARY (IMPORTANT DEFINITIONS) 413

12.1 BARRIER ISLAND PROCESSES 413
12.2 VERTICAL DATUMS 415
12.3 TIDAL CONSTITUENTS 416
12.4 OBSERVED AND MEASURED PEAK WATER LEVELS 416

SUB APPENDIX A1 – STORM SURGE MODELING AND STAGE-FREQUENCY RELATIONSHIPS

SUB APPENDIX A2: STORM-INDUCED BEACH EROSION RESPONSE-FREQUENCY RELATIONSHIPS

SUB APPENDIX A3: TIDAL INLET INVESTIGATIONS

SUB APPENDIX A4: NUMERICAL MODELING OF BREACH OPEN AT OLD INLET

SUB APPENDIX A5: TRIGGERS FOR PROACTIVE BREACH RESPONSE

LIST OF TABLES

Table 3-1. Typical Profile characteristics ... 24
Table 3-2. Reach stationing. ... 28
Table 3-3. ADCIRC-simulated average maximum annual astronomical tide elevation ... 30
Table 3-4. Historical storms selected for FIMP training set. ... 32
Table 3-5. Annual average wind directions. ... 33
Table 3-6. Annual percentage of wind direction by speed. ... 33
Table 3-7. Wind speed exceedance. ... 34
Table 3-8. Tropical (PBL) and extratropical (IKOA) maximum wind speed at Long Island for selected storms. ... 35
Table 3-9. Average Shoreline Rate of Change and Associated Standard Deviation ... 43
Table 4-1. Represenative Breach Locations. ... 53
Table 4-2. Breach Open Conditions for Numerical Simulation (Pre-Sandy). ... 54
Table 4-3. Post-Sandy Breach Open Conditions ... 55
Table 6-1. Storm water level output locations ... 57
Table 6-2. Storm Listing ... 71
Table 6-3. Combined breaching/overwash frequency values ... 107
Table 6-4. Regional Sediment Budget Cell Stations ... 139
Table 6-5. Shoreline Analysis Baseline Information ... 141
Table 6-6. Volume Change Rates by Reach and Data Source (1995 to 2001) ... 144
Table 6-7. Recent (1995-2001) Sediment Budget Equation Inputs ... 147
Table 6-8. Existing (c. 2001) Sediment Budget Equation Inputs ... 150
Table 6-9. Alternative Project Modification Condition Results ... 175
Table 6-10. More likely breach locations. ... 178
Table 6-11. Breach growth parameters. ... 178
Table 6-12. Estimated long-term potential breach cross-sectional areas. ... 178
Table 6-13. Estimated long-term potential breach widths. ... 179
Table 6-14. Historic breach sediment transport volumes. ... 180
Table 6-15. Estimated bay deposition during initial breach formation. ... 181
Table 6-16. Estimated bay deposition volumes during breach growth. ... 182
Table 6-17. Estimated total bay deposition areas during breach growth. ... 183
Table 6-18. Estimated bay deposition areas above msl during breach growth ... 184
Table 6-19. Estimated bay deposition areas below msl during breach growth ... 184
Table 6-20. Estimated Number of Breaches over 50-year Project Life ... 185
Table 6-21. Lifecycle Estimates of Cross-island Sediment Transport due to Breaching ... 186
Table 6-22. Storms Used for Significant Overwash Computation ... 189
Table 6-23. Overwash Deposition Estimates (in acres) for Small (10 yr) and Large (100 yr) Events ... 189
Table 6-24. FWOPC Annual Overwash Estimates ... 190
Table 6-25. FWPC Annual Overwash Estimates ... 191
Table 6-26. Estimated Reduction in Annual Overwash Areas ... 192
Table 6-27. Average Reduction in Overwash Bare-sand Habitat over 50-year Project Life ... 194
Table 7-1. Reach designations ... 198
Table 7-2 Summary Of Non-Structural Technique Evaluation ... 208
Table 7-3. Likely Breach Locations ... 211

Table 7-4. Approximate Beachfill Cost by Project Reach 212
Table 7-5. Preliminary List of Modification Alternatives Shinnecock Inlet 213
Table 7-6. Preliminary List of Modification Alternatives Moriches Inlet 214
Table 7-7. Preliminary List of Modification Alternatives Fire Island Inlet 214
Table 7-8 Screening Criteria – Inlet Modifications 216
Table 7-9 Screening Matrix Results – Shinnecock Inlet 217
Table 7-10 Screening Matrix Results – Moriches Inlet 218
Table 7-11 Screening Matrix Results – Fire Island Inlet 219
Table 7-12. Minimum Shoreline Erosion Rates for Structures to be Cost Effective (ft/yr) 221
Table 7-13. Assumptions inherent to the screening of back bay alternatives for representative buildings. 227
Table 7-14. Flood-proofing alternatives identified for back bay unit cost estimating. 228
Table 7-15. Comparison of Alternative Non-Structural First Costs 229
Table 7-16 –Structures Where Nonstructural Alternatives Reduce Risk of Damages 230
Table 7-17. Annual Damages: Nonstructural Alternatives 231
Table 7-18. Annual Benefits: Nonstructural Alternatives 231
Table 7-19. Annual Costs: Nonstructural Alternatives 232
Table 7-20. Net Benefits and BCRs: Nonstructural Alternatives 233
Table 7-21. Road Raising Areas 234
Table 7-22. Annual Damages: Nonstructural/Road Raising Alternatives 235
Table 7-23. Annual Benefits: Nonstructural/Road Raising Alternatives 235
Table 7-24. Annual Costs: Nonstructural/Road Raising Alternatives 236
Table 7-25. Net Benefits and BCRs: Nonstructural/Road Raising Alternatives 237
Table 7-26. Non-Structural Retrofit Alternatives 239
Table 7-27. Breach Closure Plan Design Template Cross Sectional Area (sq. ft.) 241
Table 7-28. Breach Closure Cost by BCP Location and Design Template 246
Table 7-29. Primary Input Data for Evaluating Breach Closure Template Maintenance Costs 248
Table 7-30. With Project Annual Damages: Breach Closure Alternatives 248
Table 7-31. Annual Benefits: Breach Closure Alternatives 249
Table 7-32. Annual Costs: Breach Closure Alternatives 250
Table 7-33. Net Benefits and BCRs: Breach Closure Alternatives 251
Table 7-34. Baseline Overwash and Breaching Risk (Return Period) Potential 252
Table 7-35. Summary of Proposed Proactive Breach Response Triggers 255
Table 7-36. Annual Damages: Proactive Breach Response Alternative 256
Table 7-37. Annual Benefits: Proactive Breach Response Alternative 256
Table 7-38. Proactive Breach Response Costs 257
Table 7-39. Breach Closure Alternatives 259
Table 7-40. Reaches where Beach Fill is Being Considered 262
Table 7-41. Required Berm and Dune Fill Lengths (MREI-Small Plan) 269
Table 7-42. Required Berm and Dune Fill Lengths (MREI-Medium Plan) 270
Table 7-43. Required Berm and Dune Fill Lengths (MREI-Large Plan) 270
Table 7-44. Required Fill Volume (MREI-Small Plan) 272
Table 7-45. Required Fill Volume (MREI-Medium Plan) 272
Table 7-46. Required Fill Volume (MREI-Large Plan) 273
Table 7-47. Annual Damages: Beach Fill Alternatives 275
Table 7-48. Annual Benefits: Beach Fill Alternatives 276

Table 7-49. Annual Costs: Beach Fill Alternatives.. 276
Table 7-50. Net Benefits and BCRs: Beach Fill Alternatives 277
Table 7-51. Volume and Fill Costs for Beachfll Alternatives 278
Table 7-52 Real Estate Impacts (number of structures) 280
Table 7-53 Real Estate Impacts (costs) 280
Table 7-54. Evaluations 281
Table 7-55. Inlet Modification Alternatives.. 285
Table 7-56. Summary of Costs for Shinnecock Inlet Alternatives 286
Table 7-57. Costs for SI Recommended Alternative: -16 ft MLW DB + Ebb Shoal Dredging 288
Table 7-58. Summary of Costs for Moriches Inlet Alternatives 290
Table 7-59 Costs for MI Recommended Alternative: AP + Ebb Shoal Dredging (1 yr cycle) 290
Table 7-60. Summary of Costs for Fire Island Inlet Alternatives... 292
Table 7-61. Costs for FII Recommended Alternative: AP + Ebb Shoal Dredging & DB Expansion 293
Table 7-62. Annual Damages: Inlet Management Alternative 295
Table 7-63. Annual Benefits: Inlet Management Alternative.. 295
Table 7-64. Incremental Annual Cost: Inlet Management Alternative 296
Table 7-65. Detailed Incremental Annual Cost: Inlet Management Alternative 296
Table 7-66. Inlet Management Measures... 299
Table 7-67. Modification of Westhampton Groins 306
Table 7-68 - Evaluations 307
Table 7-69. Combined Alternatives 318
Table 7-70. Annual Benefits Plan 1 – Management Only 320
Table 7-71. Annual Cost Plan1 – Management Only 322
Table 7-72. Net Benefits and BCR, By Project Reach Plan 1 – Management Only, 322
Table 7-73. Plan 1 (Plan 1.a and 1.b): Inlet Management Measures + BCP 324
Table 7-74: Annual Benefits Plan 2 – Management & Non-Structural Plans 329
Table 7-75. Annual Cost Plan 2 – Management & Non-Structural Plans........................... 330
Table 7-76. Net Benefits and BCR, By Project Reach Plan 2 – Management & Non-Structural Plans.. 331
Table 7-77. Plan 2 Alternatives (Inlet Management and BCP plus Non-Structural Retrofit)... 333
Table 7-78. Annual Benefits Plan 3 – Management, Non-Structural and Beach Nourishment Plans.. 339
Table 7-79. Annual Cost Plan 3 – Management, Non-Structural and Beach Nourishment Plans.. 340
Table 7-80. Net Benefits and BCR, By Project Reach Plan 3 – Management, Non-Structural and Beach Nourishment Plans 341
Table 7-81. Plan 3 Alternatives (Inlet Management and BCP, Non-Structural Retrofit, Beach Nourishment)... 344
Table 7-82. Summary of NED Alternative Integration Analysis 350
Table 8-1. Hurricane Sandy Peak Still Water Levels.. 361
Table 8-2. Barrier Island Elevation Conditions: Post Sandy vs. Model Cross-Sections 362
Table 8-3. MREI Design Fill Volumes... 365
Table 8-4. Previous (c. 2008) Representative Erosion Rates... 369
Table 8-5. Summary of 2009 Beachfill Project Performance 371

Table 8-6. Individual Beachfill Segments 377
Table 8-7. Diffusion Results for MIDU Baseline 379
Table 8-8. Diffusion Results for MREI Baseline........ 380
Table 8-9. Individual Beachfill Segments 382
Table 9-1. FIMP Recommended Plan Shorefront Reach Features – GSB to MB........ 388
Table 9-2. FIMP Recommended Plan Shorefront Reach Features – SB to M........ 389
Table 9-3. Number of Years between Last Inlet O&M Dredging Operation and FIMP Start........ 393
Table 9-4. Inlet Management (Initial Construction)........ 393
Table 9-5. Inlet Management (Life Cycle) 394
Table 9-6. Proactive BRP Initial Construction Quantities from Offshore Borrow Sources 397
Table 9-7. Breach Closure Cost by BCP Location and Design Template (Large & Standard Breach)........ 398
Table 9-8. Breach Closure Cost by BCP Location and Design Template (Small Breach)........ 399
Table 9-9. Beach Fill Locations........ 402
Table 9-10. Beach Fill Plan Initial Construction Quantities........ 403
Table 9-11. Beach Fill Plan - Renourishment Quantities Per Operation........ 404
Table 9-12. Sediment Management Fill Volumes at Downton Montauk 405
Table 9-13: Description of Coastal Process Features (CPF)........ 406

LIST OF FIGURES

Figure 1-1. Project site 2
Figure 2-1. Extensive overtopping of dunes and breaks through the barrier island east of Ponquogue Bridge during the hurricane of September 21, 1938. Shinnecock Inlet, which opened during the storm is shown in the photo. 7
Figure 2-2. Flooding of Main Street in Westhampton Beach during the September 21, 1938 hurricane 8
Figure 2-3. Flooding of Main Street in Westhampton Beach during the September 21, 1938 hurricane 8
Figure 2-4. Conditions in the area of Westhampton Beach before the Hurricane of 1938. Damages shown in the photo include significant damages to the west bay bridge, and a breach of the barrier island to the east of the bridge. 9
Figure 2-5. Conditions in the area of Westhampton Beach after the Hurricane of 1938. Damages shown in the photo include significant damages to the west bay bridge, and a breach of the barrier island to the east of the bridge. 9
Figure 2-6. Site of the Inlet breakthrough at Westhampton Beach during the Five High Nor'easter of 1962 11
Figure 2-7. Site of the Inlet breakthrough at Westhampton Beach during the Five High Nor'easter of 1962, following closure 12
Figure 2-8. Point O' Woods, dumage during the 1962 Five High Nor'easter 12
Figure 2-9. Fire Island Pines, dumage during the 1962 Five High Nor'easter. 13
Figure 2-10. Evolution of the 1992 breach at Westhampton from pre-breach to closure conditions. 15
Figure 2-11 Photos of Fire Island 2 to 4 days after Sandy made landfall: a) leveled beaches and scarped dunes in central Fire Island; b) houses undermined and destroyed at Davis Park; c) leveled dunes and large overwash sheets near Fire Island lighthouse; and d) the island breach at Old Inlet. 16
Figure 3-1. Representative profiles for the Fire Island subreaches. 22
Figure 3-2. Representative profiles for the Westhampton subreaches. 22
Figure 3-3. Representative profiles for the Ponds subreaches. 23
Figure 3-4. Representative profiles for the Montauk subreaches. 23
Figure 3-5. Reach delineation 27
Figure 3-6. Wind speed exceedance offshore of Long Island, New York (U.S. Naval Oceanographic Office, 1970). 34
Figure 3-7. NDBC significant wave height summary for buoy 44025 between 1991 and 2001 (1 m = 3.28 ft). 37
Figure 3-8. NDBC average wave period summary for buoy 44025 between 1991 and 2001 (1 m = 3.28 ft). 37
Figure 3-9. Computed mean wave direction summary for buoy 44025 betweeb 1991 and 2004 (1 m = 3.28 ft). 38
Figure 3-10. Long-term wave characteristics from the 1976-1994 WIS hindcast station 78, offshore of Westhampton. 38
Figure 3-11. Storm significant wave height frequency relationship for East of Fire Island Inlet design reach. 39
Figure 3-12. Storm significant wave height frequency relationship for Great South Bay design reach. 40
Figure 3-13. Storm significant wave height frequency relationship for West of Moriches Inlet design reach. 40

Figure 3-14. Storm significant wave height frequency relationship for West of Shinnecock Inlet design reach. *41*
Figure 3-15. Storm significant wave height frequency relationship for Ponds design reach. *41*
Figure 3-16. Storm significant wave height frequency relationship for Montauk Point design reach. *42*
Figure 6-1. Storm water level output locations *59*
Figure 6-2. Storm water level output locations (continued) *60*
Figure 6-3. Storm water level output locations (continued) *61*
Figure 6-4. Storm water level output locations (continued) *62*
Figure 6-5. Contributions to storm water level *64*
Figure 6-6. FIMP storm water level modeling and stage-frequency methodology *65*
Figure 6-7. WISWAVE 0.083° fine grid *65*
Figure 6-8. ADCIRC grid *66*
Figure 6-9. DELFT3D-FLOW grid (1 m = 3.28 ft) *67*
Figure 6-10. DELFT3D-WAVE (HISWA) grid (1 m = 3.28 ft) *68*
Figure 6-11. Simulated barrier island morphological response under Baseline Conditions (BLC) for historical storms. *73*
Figure 6-12. Simulated barrier island morphological response under Future Vulnerable Conditions (FVC) for historical storms. *74*
Figure 6-13. Simulated barrier island morphological response under Baseline Conditions (BLC) for additional alternate tide cases (A=Alternate tide where number is tide index, with 1.0 being high spring tide and 0.0 being low spring tide). 75
Figure 6-14. Simulated barrier island morphological response under Future Vulnerable Conditions (FVC) for additional alternate tide cases(A=Alternate tide where number is tide index, with 1.0 being high spring tide and 0.0 being low spring tide). (Note: For storms listed above and not listed here, FVC and BLC responses are classified identically.) *76*
Figure 6-15. Morphological response at Old Inlet under BLC for the historical 1938 hurricane. Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies. *77*
Figure 6-16. Morphological response at Smith Point County Park under BLC for the historical 1938 hurricane. Top pane is initial topography, center pane is topography several hours after peak surge, and bottom pane is elevation difference between the two topographies. *78*
Figure 6-17. Morphological response at Tiana Beach under BLC for the historical 1938 hurricane. Top pane is initial topography, center pane is topography several hours after peak surge, and bottom pane is elevation difference between the two topographies. *79*
Figure 6-18. Morphological response at West of Shinnecock Inlet under BLC for the historical 1938 hurricane. Top pane is initial topography, center pane is topography several hours after peak surge, and bottom pane is elevation difference between the two topographies. *80*
Figure 6-19. Morphological response at Old Inlet under BLC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies. *82*

Figure 6-20. Morphological response at Smith Point County Park under BLC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies. 83
Figure 6-21. Morphological response at Kismet to Corneille Estates under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies. 84
Figure 6-22. Morphological response at Old Inlet under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies..85
Figure 6-23. Morphological response at Smith Point County Park under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies. 86
Figure 6-24. Morphological response at Tiana Beach under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies. 87
Figure 6-25. Morphological response at WOSI under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies..88
Figure 6-26. Morphological response at Old Inlet under FVC for the historical 1992 Nor'easter. Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies...89
Figure 6-27. Morphological response at Smith Point County Park under FVC for the historical 1992 Nor'easter. Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies..90
Figure 6-28. Morphological response at Tiana Beach under FVC for the historical 1992 Nor'easter (significant overwash only). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies..91
Figure 6-29. Return period and spatial distribution of peak ocean water levels (without wave setup)...96
Figure 6-30. Ocean combined-storm stage-frequency relationships (without wave setup) in the vicinity of Fire Island (top), Westhampton (center) and the Ponds (bottom). 97
Figure 6-31. Stage-frequency relationships with and without ocean wave setup for Station 41, Great South Beach. 98
Figure 6-32. Stage-frequency relationships with and without ocean wave setup for Station 23, Great South Beach. 99
Figure 6-33. Stage-frequency relationships with and without ocean wave setup for Station 9, Old Inlet. 99
Figure 6-34. Stage-frequency relationships with and without ocean wave setup for Station 31, Post Lane. 100

Figure 6-35. Stage-frequency relationships with and without ocean wave setup for Station 63, Watermill Beach. 100
Figure 6-36. Stage-frequency relationships with and without ocean wave setup for Station 38, Apaquogue. 101
Figure 6-37. Stage-frequency relationships with and without ocean wave setup for Station 64, Napeaque Beach. 101
Figure 6-38. Stage-frequency relationships with and without ocean wave setup for Station 39, Ditch Plains. 102
Figure 6-39. Comparison between BLC, FVC, WP, and BCC stage-frequency curves. 103
Figure 6-40. Comparison between BLC, BOC4-3mo, BOC4-12mo, and Pre-Sandy BSL stage-frequency curves. 103
Figure 6-41. Breaching/overwash frequency relationships at Fire Island Lighthouse Tract for BLC, FVC, and BCC (BrCl). 108
Figure 6-42. Breaching/overwash frequency relationships at Kismet to Corneille Estates for BLC, FVC, and BCC (BrCl) 108
Figure 6-43. Breaching/overwash frequency relationships at Talisman to Blue Point Beach for BLC, FVC, and BCC (BrCl) 109
Figure 6-44. Breaching/overwash frequency relationships at Davis Park for BLC, FVC, and BCC (BrCl) 109
Figure 6-45. Breaching/overwash frequency relationships at Old Inlet, West for BLC, FVC, and BCC (BrCl) 110
Figure 6-46. Breaching/overwash frequency relationships at Old Inlet, East for BLC, FVC, and BCC (BrCl) 110
Figure 6-47. Breaching/overwash frequency relationships at Smith Point County Park for BLC, FVC, and BCC (BrCl). 111
Figure 6-48. Breaching/overwash frequency relationships at Sedge Island for BLC, FVC, and BCC (BrCl) 111
Figure 6-49. Breaching/overwash frequency relationships at Tiana Beach for BLC, FVC, and BCC (BrCl) 112
Figure 6-50. Breaching/overwash frequency relationships at West of Shinnecock Inlet for BLC, FVC, and BCC (BrCl) 112
Figure 6-51. Erosion distance vs. frequency for Fire Island subreach F-R2 (1 m = 3.28 ft). 116
Figure 6-52. Recession of 0-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R2 (1 m = 3.28 ft). 117
Figure 6-53. Eroded volume above 10-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R2 (1 m^3/m = 10.8 ft^3/ft). 118
Figure 6-54. Vertical erosion of dune crest vs. frequency for Fire Island subreach F-R2 (1 m = 3.28 ft). 119
Figure 6-55. Erosion distance vs. frequency for Fire Island subreach F-R5 (1 m = 3.28 ft). 120
Figure 6-56. Recession of 0-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R5 (1 m = 3.28 ft). 121
Figure 6-57. Eroded volume above 10-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R5 (1 m^3/m = 10.8 ft^3/ft). 122
Figure 6-58. Vertical erosion of dune crest vs. frequency for Fire Island subreach F-R5 (1 m = 3.28 ft). 123
Figure 6-59. Erosion distance vs. frequency for Westhampton subreach W-R5 (1 m = 3.28 ft). 124

Figure 6-60. Recession of 0-ft NGVD29 elevation vs. frequency for Westhampton subreach W-R5 (1 m = 3.28 ft). *125*
Figure 6-61. Eroded volume above 10-ft elevation vs. frequency for Westhampton subreach W-R5 (1 m^3/m=10.8 ft^3/ft). *126*
Figure 6-62. Vertical erosion of dune crest vs. frequency for Westhampton subreach W-R5 (1 m = 3.28 ft). *127*
Figure 6-63. Erosion distance vs. frequency for Ponds subreach P-R5 (1 m = 3.28 ft).128
Figure 6-64. Recession of 0-ft NGVD29 elevation vs. frequency for Ponds subreach P-R5 (1 m = 3.28 ft). *129*
Figure 6-65. Eroded volume above 10-ft NGVD29 elevation vs. frequency for Ponds subreach P-R5 (1 m3/m = 10.8 ft3/ft). *130*
Figure 6-66. Vertical erosion of dune crest vs. frequency for Ponds subreach P-R5 (1 m = 3.28 ft). *131*
Figure 6-67. Erosion distance vs. frequency for Montauk Point subreach M-R3 (1 m = 3.28 ft). *132*
Figure 6-68. Recession of 0-ft NGVD29 elevation vs. frequency for Montauk Point subreach M-R3 (1 m = 3.28 ft). *133*
Figure 6-69. Eroded volume above 10-ft elevation vs. frequency for Montauk Point subreach F-R2 (1 m^3/m = 10.8 ft^3/ft). *134*
Figure 6-70. Sediment Budget Cells *137*
Figure 6-71. Previous Sediment Budgets *138*
Figure 6-72. Volume Change Rates Computed from Long Profiles (1995-2001) *145*
Figure 6-73. Volume Change Rates Computed from Short and Long Profiles (1995-2001) *145*
Figure 6-74. Volume Change Rates Computed from Shoreline Changes (USACE data, 1995-2001) *146*
Figure 6-75. Change Rates Computed from Shoreline Changes (USGS data, 1995-2001) *146*
Figure 6-76. Shinnecock Inlet Peak Flood Tide *154*
Figure 6-77. Shinnecock Inlet Peak Ebb Tide *155*
Figure 6-78. Shinnecock Inlet Wave and Sediment Transport Patterns (105 deg) *157*
Figure 6-79. Shinnecock Inlet Wave and Sediment Transport Patterns (115 deg) *158*
Figure 6-80. Shinnecock Inlet Wave and Sediment Transport Patterns (145 deg) *159*
Figure 6-81. Shinnecock Inlet Wave and Sediment Transport Patterns (210 deg) *160*
Figure 6-82. Moriches Inlet Peak Flood Tide *161*
Figure 6-83. Moriches Inlet Peak Ebb Tide *162*
Figure 6-84. Moriches Inlet Wave and Sediment Transport Patterns (110 deg) *164*
Figure 6-85. Moriches Inlet Wave and Sediment Transport Patterns (135 deg) *165*
Figure 6-86. Moriches Inlet Wave and Sediment Transport Patterns (165 deg) *166*
Figure 6-87. Moriches Inlet Wave and Sediment Transport Patterns (210 deg) *167*
Figure 6-88. Fire Island Inlet Peak Flood Tide *168*
Figure 6-89. Fire Island Inlet Peak Ebb Tide *169*
Figure 6-90. Fire Island Inlet Wave and Sediment Transport Patterns (110 deg) *171*
Figure 6-91. Fire Island Inlet Wave and Sediment Transport Patterns (130 deg) *172*
Figure 6-92. Fire Island Inlet Wave and Sediment Transport Patterns (160 deg) *173*
Figure 6-93. Fire Island Inlet Wave and Sediment Transport Patterns (210 deg) *174*
Figure 7-1. Vulnerable Breach Locations *242*
Figure 7-2. Typical Breach Closure Sections *243*
Figure 7-3. Typical Breach Closure Plan Layout at Old Inlet West *244*

Figure 7-4. Typical Breach Closure Plan Layout at WOSI ... 245
Figure 7-5. Decision tree for Maintenance of Breach Closure Template ... 247
Figure 7-6 Reach Designation and Beach Restoration Locations ... 263
Figure 7-7. Typical Beachfill Section at GSB-1A, GSB-4B ... 265
Figure 7-8. Typical Beachfill Section at GSB-2B ... 266
Figure 7-9. Typical Beachfill Section at WOSI ... 267
Figure 7-10. Typical Beachfill Layout at WOSI: Medium Template, MID Dune Alignment ... 268
Figure 7-11. Recommended Alternative for SI: -16 ft MLW DB + Ebb Shoal Dredging ... 287
Figure 7-12. Recommended Alternative for MI: AP + Ebb Shoal Dredging ... 291
Figure 7-13. Recommended Alternative for FII: AP + Ebb Shoal Dredging & DB Expansion ... 294
Figure 7-14. Plan 1 Overview ... 321
Figure 7-15. Alternative Plan 2 Overview ... 328
Figure 7-16. Alternative Plan 3 Overview ... 338
Figure 8-1. Aerial Images from 2001, 2012 Pre-Sandy, 2012 Post-Sandy. MHW contour from 2000 LIDAR shown in Red, MHW contour from 2012 LIDAR shown in Cyan. ... 366
Figure 8-2: LIDAR Data from 2000, 2011, and 2012 (Fire Island Pines). ... 366
Figure 8-3. Comparison of Cut LIDAR Profiles at Fire Island Pines (Profile 68200) 367
Figure 8-4. Comparison of Cut LIDAR Profiles at Fire Island Pines (Profile 69000) 367
Figure 8-5. Spatial distribution of Dune and Berm Fill ... 368
Figure 8-6. 2009 Beachfill Project Location Map ... 370
Figure 8-7. 2009 Design Berm at Western Fire Island ... 371
Figure 8-8. 2009 Design Berm at Central Fire Island ... 372
Figure 8-9. 2009 Design Berm at Fire Island Pines ... 372
Figure 8-10. 2009 Design Berm at Davis Park ... 372
Figure 8-11. Solutions to Pelnard-Considere Equation ... 375
Figure 8-12. Nondimensional Beachfill Evolution Based on Diffusion Equation ... 375
Figure 8-13. Theoretical Longevity of Beachfill (Excluding Background Erosion) ... 376
Figure 8-14. Beachfill Evolution at Western Fire Island (MIDU) ... 379
Figure 8-15. Beachfill Evolution at Fire Island Pines (MIDU) ... 380
Figure 8-16: Beachfill Evolution at Western Fire Island (MREI) ... 381
Figure 8-17. Beachfill Evolution at Fire Island Pines (MREI) ... 381
Figure 9-1. Recommended Plan: Year 1 to Year 30 ... 390
Figure 9-2. Recommended Plan: Year 31 to Year 50 ... 391
Figure 9-3. Typical Proactive BRP Section (notes are included in the Plates Appendix) ... 395
Figure 9-4. Typical Breach Closure Sections (notes are included in the Plates Appendix) ... 400
Figure 9-5. Dune and Beach Fill Design Profile (notes are included in the Plates Appendix) ... 402
Figure 9-6: Typical Sediment Management Construction Template at Downtown Montauk (notes are included in the Plates Appendix) ... 405
Figure G-12-1. Definition of morphological responses used in this report. ... 414
Figure G-12-2. Water level contributions to HWM and WLG peak water level records. ... 417

1.0 PROJECT DESCRIPTION

The US Army Corps of Engineers, New York District (CENAN) is conducting a comprehensive feasibility-level reformulation of the coastal storm damage risk reduction project for the south shore of Long Island, New York from Fire Island Inlet to Montauk Point (FIMP), approximately 83-mile along the project area Atlantic shoreline. The Reformulation Study is a multi-year and multi-task effort, involving project planning and engineering, economic analyses and environmental studies.

1.1 Study History

The Federally authorized project area extends east from Fire Island Inlet to Montauk Point along the Atlantic Coast of Suffolk County, Long Island, New York as shown in Figure 1-1. The study area includes the barrier island chain from Fire Island Inlet to Southampton, the Atlantic Ocean shorelines from Southampton to Montauk Point, and the adjacent back-bay areas along Great South, Shinnecock and Moriches Bays. Total study length encompasses approximately 83 miles along the Atlantic Ocean and comprises approximately 70 percent of the total ocean frontage of Long Island, as well as hundreds of miles of bay shoreline.

1.1.1 Study Authority

The Fire Island Inlet to Montauk Point, New York, Combined Beach Erosion Control and Hurricane Protection Project was authorized by the River and Harbor Act of 14 July 1960 in accordance with House Document 425, 85th Congress, 2nd Session, dated 21 June 1960. The authorization was modified for the cost sharing of the beach erosion portion of the project in accordance with Section 103 of the River and Harbor Act of 12 October 1962. The project authorization was modified again by Section 31 of the Water Resources Development Act of 1986 (P.L. 99-662), which directed the Secretary of the Army to apply the cost sharing provisions of Section 31(1) of the Water Resources Development Act of 1974 (P.L. 93-251) to include periodic nourishment of the construction project at Westhampton Beach, New York, for a period of 20 years after the date of enactment of P.L. 99-662. The Water Resources Development Act of 1992 further modified the project to extend the period of renourishment for 30 years from the date of project completion for Westhampton Beach with the non-Federal share not to exceed 35 percent of the total project cost.

The authorized project provided for the dual purposes of beach erosion control and hurricane protection. Stated purposes of the authorized project, as described in House Document 425, were as follows: (1) the beach erosion control phase was to determine the most practicable economic method of restoring adequate recreational and protective beaches and to provide continued stability to the ocean shore from Fire Island Inlet to Montauk Point and (2) the hurricane study phase was to develop an adequate plan of protection against hurricane flooding for the same study area.

Elements of the authorized project included widening the beaches along the developed areas between Kismet and Mecox Bay to a minimum width of 100 feet at an elevation of 14 feet above Mean Sea Level (MSL). Dunes were to be raised to an elevation of 20 feet above MSL from Fire Island Inlet to Hither Hills State Park, and at Montauk and opposite Lake Montauk Harbor by artificial placement of suitable sand. Other elements of the authorized project included dune grass planting and interior drainage structures at Mecox Bay, Sagaponack Lake and Georgica Pond. The project authorized construction of up to 50 groins subject to future determination of the actual need based on experience.

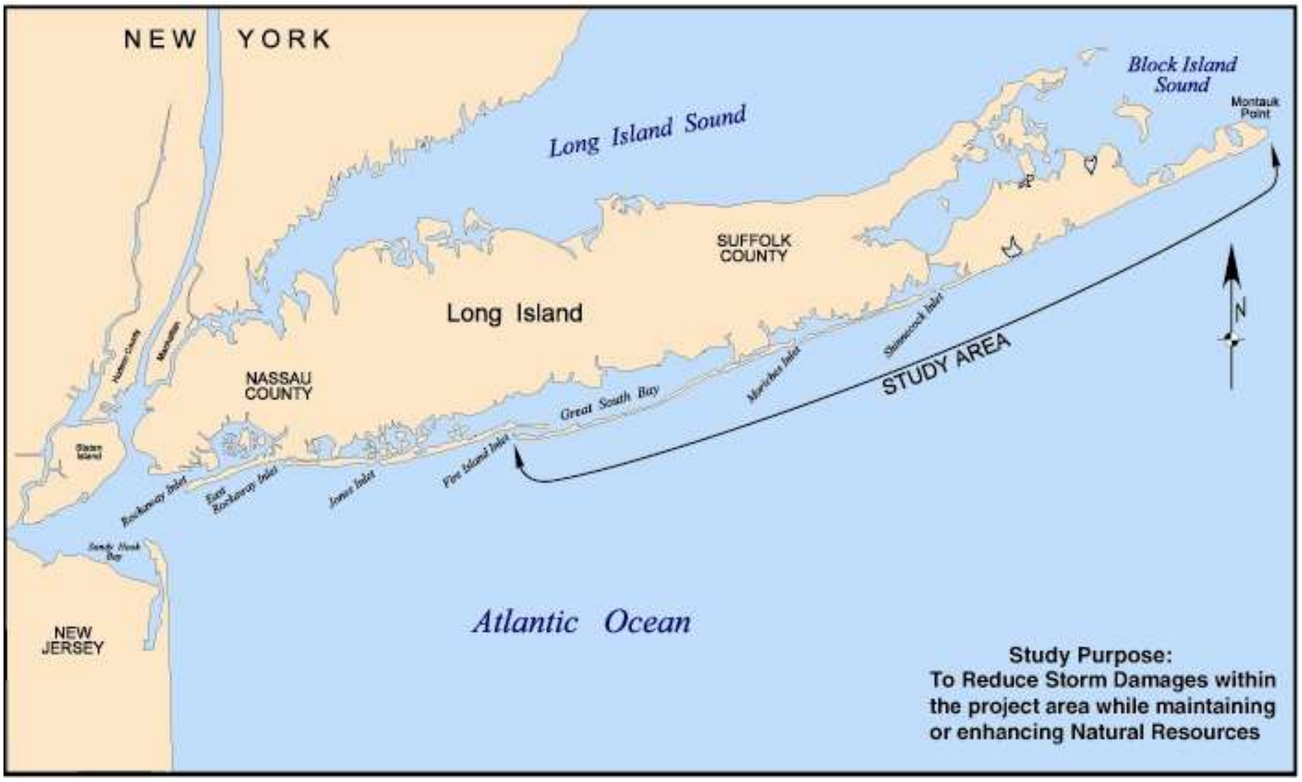


Figure 1-1. Project site

1.2 Purpose

The purpose of the FIMP project is to identify a long-term solution to reduce the risk of coastal storm damages in the study area in a manner which considers the risks to human life and property, while maintaining, enhancing, and restoring ecosystem integrity and coastal biodiversity.

1.3 Vision

The following vision statement was agreed upon by the U.S. Army Corps of Engineers, U.S. Fish and Wildlife Service, National Marine Fisheries Service, U.S. Department of the Interior, NY Department of State and Department of Environmental Conservation, The Nature Conservancy:

"The vision for the Fire Island to Montauk Point Reformulation Study is to prepare an implementable, comprehensive, and long-term regional strategy for the 83 mile portion of the south shore of Suffolk County, Long Island, New York that will reduce risks to human life and property while maintaining, enhancing, and restoring ecosystem integrity and coastal biodiversity. This will require an assessment of at risk properties, present and future sea level rise, restoration and protection of important coastal landforms and processes, and important public uses of the area. The Reformulation Study will lead to a project that provides New York State and its residents with lower storm damage risks and a full range of future options for coastal zone management.

The Reformulation Study is taking an innovative approach using a science-based model for addressing coastal storm risk reduction and pre- and post-storm shoreline management along both barrier and mainland shorelines. The U.S. Army Corps of Engineers and the State of New York, in their lead project planning and cost sharing roles, are developing innovative management and restoration measures working with a large range of stakeholders to establish comprehensive, consensus-based solutions. The final plan will recommend measures for Implementation by federal agencies, New York State, Suffolk County and local governments through the exercise of all applicable government authorities to the maximum extent practical to achieve national, state and local objectives.

- Priority will be given to non-structural measures that reduce risks and provide protection to human life and property, restore and enhance coastal processes and ecosystem integrity, and are environmentally sustainable.

- Measures that avoid or minimize adverse environmental impacts and adequately address long-term demands for public resources will be used wherever and whenever appropriate and required, while continuing to accept and embrace governmental responsibility and accountability under the law.

- Preference will be given to measures that protect and restore coastal landforms and natural habitats, aid in recovery of threatened and endangered species, improve water quality, enhance public recreation and use, and ensure perpetuation of essential physical and biological processes.

- Dune and beach replenishment will be minimized. Sand nourishment will be considered where it will create conditions suitable for restoration of natural processes and where appropriate to protect important uses. Active intervention will be considered where it is possible to achieve balance and synergy between human development, economic activities, and natural systems.

- Existing shore stabilization structures, inlet stabilization measures, dredging practices, and other coastal area modifications past and present, including bay and estuarine shorelines, will be assessed to examine their impacts and, as appropriate, recommended to be removed, altered, or mitigated to help restore important physical and biological processes.

- Efforts will be undertaken to reduce mainland and barrier flood risks and island flooding and erosion through site specific measures that address the variety of causes of flooding throughout the study area, consistent with applicable agency laws and missions.

- Collection, analysis, and independent technical review of scientific data will be conducted to improve understandings of complex and dynamic, regional hydrologic, geomorphic, and ecological factors and interrelationships while simultaneously facilitating the building and sharing of an integrated scientific, economic, and social knowledge base.

- No plan can reduce all risks. On-going monitoring will evaluate the effectiveness and impacts of implemented policies. The monitoring results will serve as the basis for adaptations and adjustments to improve the project's effectiveness. And respond to the dynamic nature of the FIMP study area."

1.4 Problem Identification

Although the area functions as a system, it can be delineated into three main problem areas. The three Problem Areas within the study area include: 1) the barrier islands segment, 2) the mainland behind the barrier islands, and 3) the Atlantic Ocean shoreline east of the barrier islands. Each area has distinct problems, as a result of its unique physical setting.

- **Barrier Island Segment**: Along the Barrier Island portion of the study area, development is dense and often located in high-hazard areas. Along the barrier island, buildings are vulnerable to storm damages due to wave attack, erosion and storm surge. The barrier islands are also vulnerable to overwash and breaching. An overwash or breach impacts the barrier island, as well as the backbay. Past breach events illustrate that a breach undermines and destroys houses on the barrier island as it grows.

- **Back Bay Segment**: Development in the backbay area is threatened by storm surge, which is made worse with a breach of the barrier islands, and increases in sea level rise.

- **Atlantic Ocean Shoreline**: The eastern portion of the study area is vulnerable to damages due to erosion, wave attack, and storm surge; similar to the problems along the barrier islands. Within this area, the damages are more localized, due to the nature of the existing development and physical conditions.

2.0 SHORELINE HISTORY

2.1 Historical Storms, Breaching and Overwash

The study area has a long history of storm damage. Prior to the 1930's the recorded history of storm impact is largely anecdotal, although references are available that describe the great storm of 1690, which opened Fire Island Inlet; the major hurricane of 1821 which made landfall near Jamaica Bay, and resulted in flooding 9.3 feet above average in New York City;, and the major hurricane in August 1893 which was labeled as "Long Island's Most Destructive Storm". Since 1930, the records are more detailed, and there have been a number of hurricanes and nor'easters that have impacted the area. The storm history indicates periods of time in which a series or cluster of storms have impacted the study area. It is these time periods where it appears that the storms had the greatest impact on the built environment, and where the consequences of the storms were greatest. It is also important to note that since the 1930s there is a history of human responses after storms to close breaches and restore the beaches and dune.

<u>1930's</u>

The 1930s had a number of significant storms, including the March 1931 nor'easter, and the "Long Island Express" hurricane in 1938, which is the storm of record in the area. The March 1931 nor'easter occurred during a full moon, and is the storm that created Moriches Inlet. It also resulted in widespread erosion along the study area. Prior to this storm, there was no inlet into either Moriches or Shinnecock Bays; only Fire Island Inlet prevailed. Prior to the 1938 hurricane, there were a number of low, narrow areas along the barrier beaches with several areas no higher than 6ft above MSL.

The 1938 hurricane, named the "Long Island Express" had wind gusts up to 135 MPH, and made landfall in the vicinity of Moriches Inlet, at a time nearly coinciding with a high tide. The results of this hurricane were devastating.

Waves 15-30 feet high swept the beaches along the entire south shore of Long Island. The storm surge and waves breached most of the dunes on Fire Island that were less than 16 feet in elevation. Dunes higher than 18 feet were generally left intact although they often showed evidence that they too had been overtopped. The ocean broke through the barrier island in hundreds of places inundating the normally dry land protected by the barrier and flooding the coastal bays and ponds. The storm resulted in 11 new openings of the barrier islands in the study area. The full storm, 200 to 300 miles across lasted only four hours but left 50 people dead and over 1,000 homes destroyed. Damages to property on Long Island was estimated at $87 million.

Coastal towns had water in the streets three to four feet high. A tidal wave six feet deep swept through Westhampton from the ocean to Main Street.

Westhampton Beach reported 28 deaths, the highest of the Long Island towns, and 157 of the 179 beach front homes were destroyed. In Saltaire, 127 houses were destroyed, at Fair Harbor 91 structures destroyed, at Oak Beach 29 homes were lost, Kismet Park lost more than 22 homes, Lonelyville lost 14 homes, and 300 homes were lost at Ocean Beach

In Southampton along Dune Road, only two homes remained after the storm waves swept the barrier beach. The landmark St Andrew's Church on the Dunes in Southampton was destroyed, pieces of the building and furnishings were found spread over a mile wide area. In Bridgehampton more than 50 barns were destroyed between Water Mill and Wainscott. Crops were buried beneath sand from the beach or washed away.

The fishing village at Montauk Point was swept away during the storm leaving about 150 people homeless, the residents having lost almost all their possessions. More than 80 fishing boats were destroyed or badly damaged. Nets and fish traps were also damaged. The Westhampton Yacht basin lost pleasure boats and work boats. At the Shinnecock Yacht Club the main floor of the club house was destroyed leaving the second story on the ground.

All the bridges in Westhampton and Quogue had been damaged during the storm. In Westhampton, the south end of the West Bay Bridge was destroyed. In Quogue, the Beach Lane Bridge was destroyed by flood waters and floating debris; the Ocean Avenue was damaged but not destroyed. The railroad tracks and highway at Napeague were washed out isolating the east end of the island. Railroad service between Amagansett and Montauk was disrupted for seven days.

Fire Island State Park was severely damaged by the storm, the beach dunes were damaged by the high waves, buildings were damaged beyond repair and more than two-thirds of the docks were destroyed. Three Coast Guard stations, including the Moriches and Potunk stations, were destroyed and the remaining fifteen stations from Jones Beach and to the west, were damaged to a lesser degree. Photos illustrating the overwash, breaching, shorefront damages, and back bay flooding are shown in Figure 2-1 to Figure 2-5.

The human response to the 1938 hurricane was extensive. The Superintendent of Highways for Suffolk County described the County's response as extensive debris removal, rebuilding dunes, rebuilding of public infrastructure, public facilities and the closure of breaches. Ten of the eleven breaches were reportedly closed using trucks and bulldozers. The 11th breach was at Shinnecock Inlet, where the County decided to stabilize the inlet with a timber crib structure on the western shoreline to create a permanent inlet. Robert Moses, in his 1938 report, described the other activities undertaken, including the placement of debris on the beach and in the dunes to act as sediment traps. The report also recommended an alternative to this practice, which included rebuilding a beach and dune, topped by a road, to be constructed with material from the back bay (much like Ocean Parkway on Jones Island). This plan was never implemented in the study area.

Figure 2-1. Extensive overtopping of dunes and breaks through the barrier island east of Ponquogue Bridge during the hurricane of September 21, 1938. Shinnecock Inlet, which opened during the storm is shown in the photo.

Figure 2-2. Flooding of Main Street in Westhampton Beach during the September 21, 1938 hurricane

Figure 2-3. Flooding of Main Street in Westhampton Beach during the September 21, 1938 hurricane

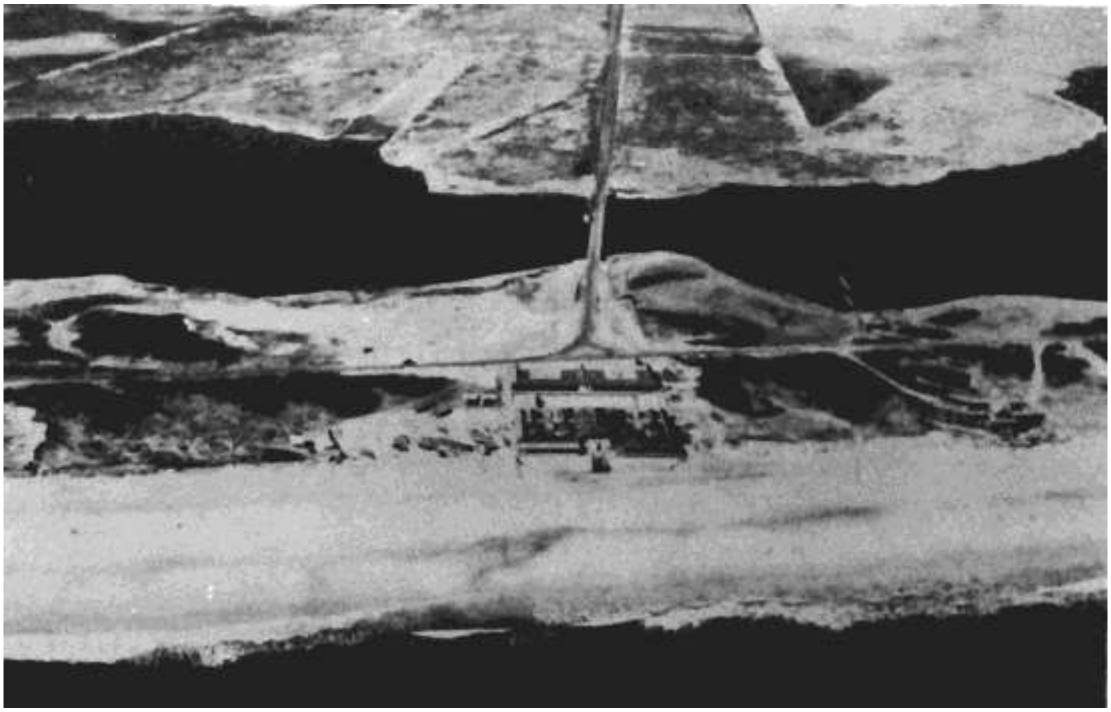


Figure 2-4. Conditions in the area of Westhampton Beach before the Hurricane of 1938. Damages shown in the photo include significant damages to the west bay bridge, and a breach of the barrier island to the east of the bridge.

Figure 2-5. Conditions in the area of Westhampton Beach after the Hurricane of 1938. Damages shown in the photo include significant damages to the west bay bridge, and a breach of the barrier island to the east of the bridge.

1950’s to 1980’s

The next period of intense storm activity was in the period of the mid 50’s and early 60’s. Notable storms impacting the area in the 50’s and 60’s include (1) the November 1950 Nor’easter, (2) the November 1953 nor’easter, (3) Hurricane Carol in 1954, (4) Hurricane Donna in 1960, and (5) the Ash Wednesday Nor’easter of 1962, also known as the “5-High Storm”, since the storm resulted in flooding over a period of five high tides. These storms had a considerable effect on the area and resulted in a continued human response to the problem.

1. The November 1950 nor’easter resulted in ocean tide 5.1 feet above mean sea level at Shinnecock Inlet, 5.2 feet above mean sea level at Montauk Point and 3.8 feet above mean sea level in Moriches Bay at Westhampton. The Coast Guard reported waves 20 feet high along the south shore. The Suffolk County authorities reported that barrier island dunes with an elevation less than 12 feet above mean sea level were overtopped. Dunes were cut through the at thirteen location between Fire Island Inlet and Moriches Inlet, and three locations east of Quogue. A major breach, 100 feet wide by 6 feet deep, joined the ocean with Moriches Bay at Westhampton Beach

2. During the November 1953 Nor’easter, the dunes at Westhampton Beach were destroyed by extremely high water levels as the storm arrived during high tide. Wave heights along the shore were estimated at 20 feet high. The ocean broke through the barrier island at five locations from Fire Island to an area 2.5 miles to the east. In the vicinity of Smith Point, the beach was breached contributing to the inundation of mainland structures one-quarter mile inland. The dunes between Democrat Point and Moriches inlet were cut back by the wave action a distance of 10 to 50 feet. The jetties at Moriches and Shinnecock Inlets were damaged by the storm, and Shinnecock Inlet was partially shoaled. At Westhampton Beach, the ocean broke through the barrier island in eight locations and resulted in the inundation of the mainland to a depth of two feet for one-half mile inland. In East Hampton, there were breaches into Georgica Pond, Hook Pond and near the east boundary of the village. There was water one- foot deep 150 feet inland. The high storm waves contributed to the severe structural damage to homes on Fire Island, where structures were inundated or undermined.

3. During Hurricane Carol, the ocean broke through the barrier beach between Montauk Point and Fire Island in 14 locations, including 10 at Westhampton Beach. A breach 200 feet wide was cut through the beach west of the West Bay Bridge at Westhampton Beach. The breach was filled and the roadway rebuilt only to be damaged again in the September 11 storm, Hurricane Edna. Deposition of sand from the damaged dunes along Beach Road between Quogue and Shinnecock Inlet isolated the area. Three homes were badly undermined and 100 beach front homes were evacuated. The dunes were also severely eroded at many locations along the barrier including Point O’Woods. In the vicinity of East Hampton, the dunes were breached at several locations into Mecox Bay, Sagaponack Lake, and Georgica and Hook Ponds. The waves broke through at Napeague between Amagansett and Montauk and damaged the railroad tracks disrupting service. The adjacent highway was flooded to a depth of three feet. The ocean broke through the dunes between Fort Pond Bay and Montauk. Severe erosion of the beach and cliffs east of Montauk was reported in addition to damage to the seawall at Montauk Point

4. The 1962 Ash Wednesday “Five High Storm” lasted through five consecutive high tides causing severe beach and dune erosion. Each successive high tide was able to reach

further inland or into back bay areas as the beaches and sand dunes eroded and were washed away. The storm destroyed 96 barrier beach homes; 53 of the homes at Westhampton Beach; 21 new built homes at Fire Island Pines. In the Town of Southampton, 45 houses were extensively damaged. Along Dune Road in Quogue, four houses were completed destroyed and several more were in danger of being swept into the ocean. Many houses not destroyed during the storm were left hanging on the edge of the eroded dunes

A new 300 foot wide inlet was formed through the barrier beach west of the Jessup Lane Bridge at Westhampton Beach. Dune Road was destroyed in several locations isolating unoccupied homes that weren't damaged in the storm. Additional smaller inlets in the barrier island were also formed. The local authorities worked quickly to repair the breaches, using two dredges provided by the county. It took approximately one week to close the major breach working 24 hours each day. Figures illustrating storm damages from the Five High Storm are shown in Figure 2-6 to Figure 2-9.

Figure 2-6. Site of the Inlet breakthrough at Westhampton Beach during the Five High Nor'easter of 1962

Figure 2-7. Site of the Inlet breakthrough at Westhampton Beach during the Five High Nor'easter of 1962, following closure

Figure 2-8. Point O' Woods, dumage during the 1962 Five High Nor'easter

Figure 2-9. Fire Island Pines, dumage during the 1962 Five High Nor'easter

The 1970s and 80s were a period of relative calm. That being said, a Nor'easter in January1980 resulted in a breach of the barrier island, just to the east of Moriches Inlet, which remained open for 13 months, until closed in February 1981 at a cost of $12 Million.

Hurricane Gloria impacted the study area in 1985, but made landfall at low-tide, sparing Long Island from severe flooding, and resulting in mostly wind damage. Still, 48 houses were reported as destroyed in the Study Area with peak wind gusts of 100 mph.

<u>1990's</u>

The next series of events impacting the project area included Hurricane Bob in 1991, the Halloween Nor'easter of 1991 (dubbed the "Perfect Storm"), the December 1992 Nor'easter, and the March 1993 "Storm of the Century". The eye of Hurricane Bob passed over Block Island to the east of Long Island, and resulted in a storm surge which caused widespread coastal flooding in low lying areas.

The 1991 October Halloween storm followed an unusual east to west track; when a northeaster joined with the remnants of a hurricane and began to move backwards. The storm circled several hundred miles offshore generating huge waves which battered the shoreline through three high tides. High winds and rough seas destroyed homes on Dune Road in Westhampton Beach as waves washed over the dunes. Along Dune Road in Westhampton and Quogue, 19 residences were destroyed, 17 homes seriously damaged and four homes were reported with minor damage. Approximately 4,000 feet of Dune Road required repair. Beach club facilities and hundreds of feet of beach were severely eroded. Dunes 15 feet high were washed away. The beach and dunes at Southampton suffered major erosion damage, the remains of several buildings destroyed in the 1938 hurricane were exposed. Breaches in the barrier island in front of Georgica Pond, Mecox Bay and Sagaponack Lake exposed the waters to the ocean. Near Mecox Bay, the dunes were washed over and two houses were damaged. At East Hampton, there was severe erosion to the beaches and the dunes, as well as major erosion at the Montauk Lighthouse.

The December 1992 Nor'easter resulted in significant damages along Long Island's ocean shoreline and in the back bays. The most severe damage was along the Westhampton Barrier where 36 houses were lost, and where there were 2 breaches at Westhampton (Pikes Inlet and Little Pikes Inlet). Overwashes of the island were also observed along western Fire Island, at Smith Point County Park, Old Inlet, and in the area just west of Shinnecock Inlet. The dune area, with dunes 15 to 20 feet high, west of the jetties at Shinnecock Inlet in Hampton Bays was leveled and Dune Road was covered with sand 6 to 8 feet deep. Several homes on the bayside were covered with sand up to the roof tops. Homes on the ocean side stood on their wood piles as waves rolled underneath. In Mastic Beach, the water reached 2 to 4 feet deep in the streets.

Pikes Inlet, initially the larger of the two breaches at Westhampton was closed quickly, while Little Pikes Inlet was left open to possibly close on its own. However, Little Pikes inlet instead grew to 3,000 ft wide and 20 ft deep by April 1993. The widening breach had caused damage to an additional 80 homes along Dune Road. The breaches in the barrier island caused an increase in the bay side tidal range which in turn caused an increase in flooding on the mainland. Eventual emergency closure of the inlet was undertaken in October 1993 at the cost of $10,000,000. Photos illustrating the growth and closure of the breach at Westhampton are shown in Figure 2-10.

The March 1993 resulted in severe wave action that scoured the beaches along the entire barrier island. The dunes were overtopped, lowering the height of the dunes 15 to 20 feet. It was reported that homes were destroyed or severely damaged at Kismet- 7 houses, Saltaire- 18 houses, Fair Harbor- 39 houses, Lonelyville- 2 houses. Extensive flooding was also reported in the area of Remsemburg along Moriches Bay. The severity of the flooding was linked to the breach of the barrier island in Westhampton that had opened in December 1992.

September 1992

December 1992

January 1993

March 1993

June 1993

October 1993

Figure 2-10. Evolution of the 1992 breach at Westhampton from pre-breach to closure conditions.

Hurricane Sandy

According to the National Hurricane Center, Hurricane Sandy, at nearly 2,000 kilometers (km) in diameter, is the largest storm on historical record in the Atlantic basin. The storm, which made landfall coincident with astronomical high tides, affected an extensive area of the east coast of the United States. The highest waves and storm surge were focused along the heavily populated New York and New Jersey coasts. The storm made landfall near Atlantic City, New Jersey, the evening of October 29, 2012. At the height of the storm, a record significant wave height of 9.6 m (m) was recorded at the wave buoy offshore of Fire Island, New York. During the storm, beaches were severely eroded and dunes extensively overwashed. Fire Island was breached in three locations, and the coastal infrastructure, including many private residences, was heavily damaged (Figure 2-11). Summaries of the damage are shown below:

Figure 2-11 Photos of Fire Island 2 to 4 days after Sandy made landfall: a) leveled beaches and scarped dunes in central Fire Island; b) houses undermined and destroyed at Davis Park; c) leveled dunes and large overwash sheets near Fire Island lighthouse; and d) the island breach at Old Inlet.

West of Shinnecock Inlet (WOSI), NY

The beach berm eroded approximately 50 to 100 ft to a width of approximately 50 ft from toe of the dune. Approximately 100,000 CY were lost from the berm. Dune erosion resulted in a loss of 3-5 ft of dune height and 25-30 ft of dune width. The eroded seaward dune face was nearly vertical as is common during severe erosion events. Some 50 to 80% of the dune volume (approximately 30,000 CY) was lost during the storm. While there were no breaches of the dune, a significant portion of the eastern project area was overtopped with sediment overwashing into leeward roads and buildings. As for the beach berm, it was lowered 1-3 feet by the storm and eroded 50 to 100 feet. The volumetric loss of the berm has been estimated at 100,000 CY.

Westhampton, NY

Storm impacts to the beach cross-section consist of lowering and flattening of the berm above the mean tide line, reduction of berm width, and damage to the dune cross-section. Although no ocean water level data were available at this location, measured ocean storm tide elevations to the west (Ocean Beach, Fire Island) and east (Easthampton) suggest that the beach berm was inundated with at least 0.5 ft of still water plus waves at the peak of the storm. Lowering and flattening of the berm occurred over the entire project length (Groin 7 through to the park facility at Cupsogue) with an estimated average drop in beach elevation of 5-8 feet. Berm widths decreased along the entire project shoreline. The primary dune, initially constructed in 1996 and located most landward, suffered at least 50% to almost 80% volume loss for 4,100 feet, out of the 10,000 ft of the dune from Groin 15 to the western limit of the project within Cupsogue Park. Secondary lower dunes, more oceanward, were destroyed along 9,300 feet of the project length. Within the groin field from groin 7 through groin 15, the beaches lowered and receded, and there were considerable impacts to the most-oceanward dunes. There was evidence of wave runup over the primary landward dune and overwash of ocean water in some project locations. Overwash of sand over the existing dune occurred at Pike Beach in the area of the vehicle cross-over, which had been consistently at a lower dune elevation than the surrounding dunes. Total beach and dune volume lost due to Hurricane Sandy has been estimated to be 450,000 cubic yards (CY).

Fire Island, NY

The beaches and dunes on Fire Island were severely eroded during Hurricane Sandy, and the island breached in three locations on the eastern segment of the island. Landward shift of the upper portion of the beach averaged 19.7 meters (m) but varied substantially along the coast. Shoreline change was also highly variable, but the shoreline prograded during the storm by an average of 11.4 m, due to the deposition of material eroded from the upper beach and dunes onto the lower portion of the beach. The beaches and dunes lost over 50 percent of their pre-storm volume, and the dunes experienced overwash along almost 50 percent of the island. The inland overwash deposits account for 14 percent of the volume lost from the beaches and dunes, indicating a majority of material moved offshore.

2.2 Historical Development and Management of Project Area

In the years following the 1938 hurricane, there was increased human investment along the shoreline. In 1941, Fire Island Inlet was stabilized with the east jetty to improve the navigability of the inlet. In the early 1950's Suffolk County and New York State further stabilized Moriches Inlet and Shinnecock Inlet with stone jetties and dredged the inlets for improved navigation access. For Moriches Inlet, these improvements were also intended to improve water quality in the bay. This period also saw an increase in development in the Study Area. Building after World War II resulted in extensive development along the western bay shorelines. NPS documents also indicate 1,260 houses and businesses were located on Fire Island in 1955, with an increase to approximately 2,400 by 1962.

The storm activity in the mid-50s was also the impetus for the original FIMP Study. The study concluded with the 1958 Survey Report which was endorsed by Congress. Construction of elements of the project followed in the 60s, including the partially constructed groinfield in Westhampton and two groins in Easthampton near Georgica Pond. This time period also saw continued development along the shoreline and additional hard structures built. Groins were constructed by State and local interests in the areas of Ocean Beach on Fire Island and in Easthampton, which were a precursor to the Federal groins. Numerous local and homeowner projects were also constructed, as evidenced by the small groins, bulkheads, and dunes sometimes reinforced with stone, concrete and cars, which are intermittently exposed today.

Following the 1962 Ash Wednesday storm, the Federal Government responded with "Operation Five High" which undertook efforts to rebuild beaches and dunes along the entire Atlantic Ocean shoreline from Virginia to New York. Within the study area there was significant Federal dune and beach rebuilding as part of this program, and a number of smaller efforts undertaken by local governments. As part of Operation Five-High, approximately 2,220,000 CY of sand was placed along 14.7 miles of shoreline in the Study Area. Additional local efforts undertaken included dune rebuilding and emergency protective measures at Cherry Grove, Point O' Woods, Village of Saltaire, Village of Ocean Beach, and the Village of East Hampton.

During the 1960's and 70's, emphasis was placed on improved decision-making regarding the coastal zone, and a greater consideration of the environment in decision-making. This period included the introduction of the National Flood Insurance Program in 1968, the introduction of NEPA in 1969, the introduction of the CZMA in 1972, and the authorization of the Fire Island National Seashore in 1964. Within New York State, this period also saw the introduction of the New York State Coastal Erosion Hazard Act Regulations (CEHA). Collectively, these policy guidelines, jurisdictions, and land use regulations govern largely what is in place today. Of these, it is important to particularly note the creation of the Fire Island National Seashore. This requires that any beach nourishment plan within the boundaries of Fire Island, arising from this study, must be mutually agreeable to both the Secretary of the Army and the Secretary of the Interior.

Storms in the early 1990's served as the basis for re-convening the Governor's Coastal Erosion Task Force, which in 1994 established both short-term and long-term policies for the State of New York,, and recommended specific actions that included: 1) initiate sand bypassing at the inlets and at the Westhampton groin field; 2) maintain barrier island landform integrity by filling highly vulnerable washover fans and new inlet breaches, and maintaining longshore sand transport; 3) establish a reserve of funds to enable rapid response to critical erosion problems caused by coastal storms, such as breaches in the barrier island; 4) press federal, state, and local governments to elevate or provide protection for key evacuation routes; 5) initiate an erosion monitoring program to provide scientific information to design future projects, modify existing ones as necessary, and refine management practices; and 6) use the Corps of Engineers to expedite the Fire Island Inlet to Montauk Point Reformulation Study.

There have also been additional actions undertaken, since the early 1990's, to protect infrastructure along the shoreline. This includes the Federal, State, and County project to construct an interim beach and dune project in the area of the Village of Westhampton Dunes, and the similar project to protect the area immediately west of Shinnecock Inlet. Consistent with the Task Force findings, there has been a renewed emphasis on bypassing material dredged from the inlets for navigation. A Breach Contingency Plan was also developed to reduce the time to close future breaches, based upon the 11 months it took to close the breach at Westhampton. In the absence of government-led response in other locations along the shoreline, there also have been a number of community-funded and County-funded beachfill and beach scraping projects on Fire Island, and a number of localized stone, steel and geotextile structures constructed throughout the study area.

3.0 EXISTING SITE CONDITIONS

Existing site conditions within the study area are summarized to provide the basis of design evaluation for structural coastal storm damage risk reduction measures. These site conditions include geology, major morphological features, climate, winds, waves, tides, storm records, and sea level rise estimates.

3.1 Geology

Long Island is part of the Atlantic and Gulf Coastal physiographic province which lies along the eastern border of the United States and lays at the southern boundary of the late Pleistocene glacial advance in the eastern part of North America (Taney, 1961). The Ronkonkoma and Roanoke Point moraine deposits (i.e., mounds of unstratified glacial drift chiefly consisting of boulders, gravel, sand and clay) characterize the topography along the northern side of Long Island, while a gentler southward dipping gradient on the outwash plains makes up much of the southern side of the island (Schwab et al., 1999).

From Montauk Point west to Southampton (approximately 33 miles) headlands formed by Ronkonkoma moraine and outwash deposits are eroded forming a narrow beach and a series of small bays (i.e., ponds). Eroded sediments along this reach are transported westward by wave action. West of Southampton reworked glaciological outwash has formed low-relief, sandy (fine- to medium-grained sand) barrier islands enclosing shallow back-barrier bays. The barrier islands were formed by a combination of spit extension (westward from Southampton) and offshore bar development. The larger bays have historically been intermittently connected to the ocean by tidal inlets. In the normal course of events, inlets would be cut through the barrier island during storms, migrate over time to the west, and eventually close by natural processes (Taney, 1961).

The principal geologic features of the inner continental shelf offshore of Fire Island are summarized by Schwab et al. (2013):

(1) a regional unconformity separating Cretaceous-age coastal plain strata from overlying Quaternary sediment; (2) a Pleistocene glaciofluvial sedimentary deposit exposed at the seafloor over much of the inner continental shelf at water depths between ~15 and ~32 m, the seaward limit of the study area; and (3) a series of Holocene sand ridges on the inner continental shelf W of Watch Hill extending across the study area.

West of Watch Hill, the Holocene (modern) sedimentary deposit is organized into a series of shoreface-connected sand ridges oriented at angles of 30° to 40° to the coast (Schwab et al., 2013). Seismic reflection data collected in 1996 and 2011 by the USGS (Schwab et al. 2013) indicate that the thickness of the Holocene sediment thickness is between 1 and 6 meters. The thickness of the sand ridges is greatest (approximately 6 meters) offshore of central Fire Island and gradually thins to the west (approximately 1 meter thick offshore of Fire Island Inlet).

3.2 Major Morphological Features

Taney's 1961 physiographic delineation of the FIMP project area morphology divides the area into two major geologic sections. The easternmost 53 km (33 miles), from Montauk Point to Southampton, is the headland section, which is followed to the west by an 80-km-long reach characterized by barrier beaches and barrier islands. The headland section is further subdivided into three units. Bluffs that rise to 18 m (60 ft) or more above sea level and narrow beaches of coarse sand and gravel characterize the shoreline from Montauk Point westward for a distance of approximately 16 km (10 miles). The next unit, which includes Napeague Beach, is considered a connecting beach that provides a link between two areas of deposition of the Ronkonkoma moraine. This unit is approximately 6.4 km (4 miles) long. A low sandy beach backed by dunes characterizes the shoreline within this unit. The third unit of the headland section is 30.6-km long and extends from just west of Promised Land (in the second unit) to Southampton. Sandy beaches and long continuous dunes that rise to an elevation of +6 m (20 ft) above sea level characterize this unit. Lying just north of the shoreline are several small ponds or bays that have been cut off from the ocean by baymouth bars and narrow barrier beaches which are periodically breached during and after storms. To the north of the ponds the Ronkonkoma morainal ridge provides the dominant topographic relief of the area.

The project reach extending from Southampton westward to Fire Island Inlet, a distance of 80 km (50 miles), is delineated as the barrier beach section. The barrier beach section is presently segmented by two tidal inlets (Shinnecock and Moriches inlets) and is bounded by Fire Island Inlet at its western end. The Westhampton barrier island is approximately 25 km (15.5 miles) in length and is bounded on the east by Shinnecock Inlet and on the west by Moriches Inlet. The Fire Island barrier island is approximately 50 km (31 miles) in length and is bounded on the east by Moriches Inlet and on the west by Fire Island Inlet.

3.3 Beach Profile Characteristics and Morphological Reaches

3.3.1 Representative Profiles

To determine the morphological response for each of the morphological subreaches, representative initial beach profiles were constructed for each subreach by evaluating measured profiles. The measured profile database included 213 conventional beach profile survey lines measured in March 1995, October 1995, March 1996, and October 1996. In addition, more than 200 subaerial cross-sections were extracted from the September 2000 lidar survey. These profiles were sorted by morphological reach for analysis.

For each subreach, the submerged portion of the representative profile was taken as the average of all conventional profiles, from the 1990s, available for that subreach (Gravens *et al*., 1999). The subaerial portion of the representative profile was defined by a specific lidar (September 2000) cross-section. The lidar cross-section selected for each subreach was selected to be characteristic of all lidar cross-sections within that subreach. The representative profiles for all subreaches are shown in Figure 3-1 through Figure 3-4, and Table 3-1 summarizes the characteristics of each representative profile.

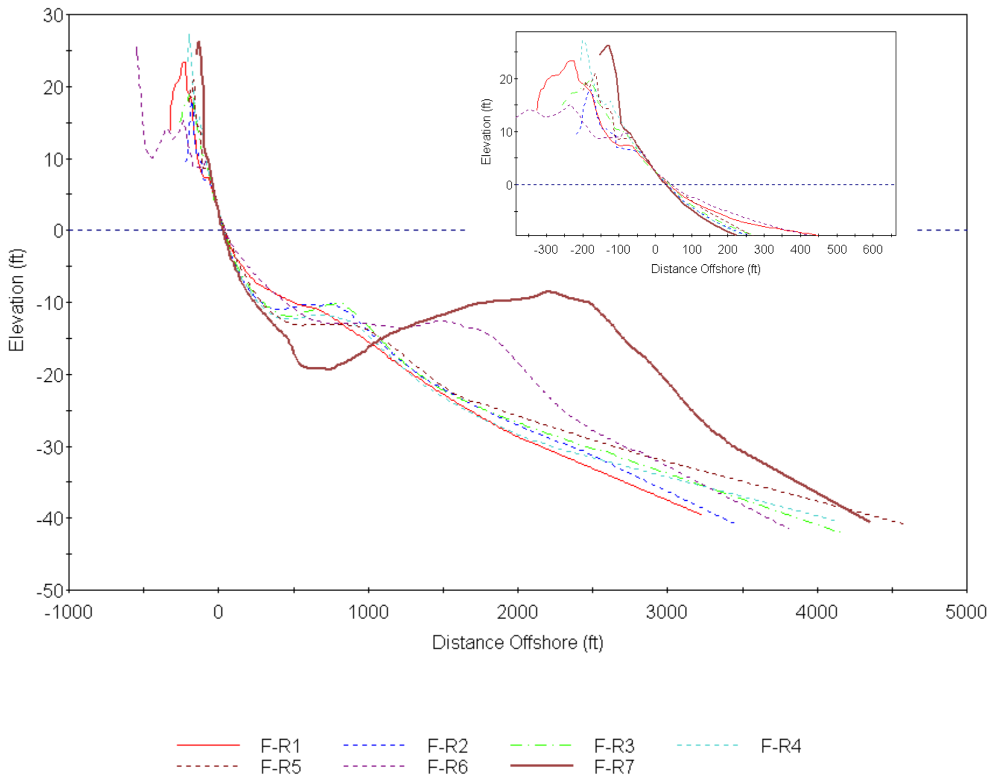


Figure 3-1. Representative profiles for the Fire Island subreaches.

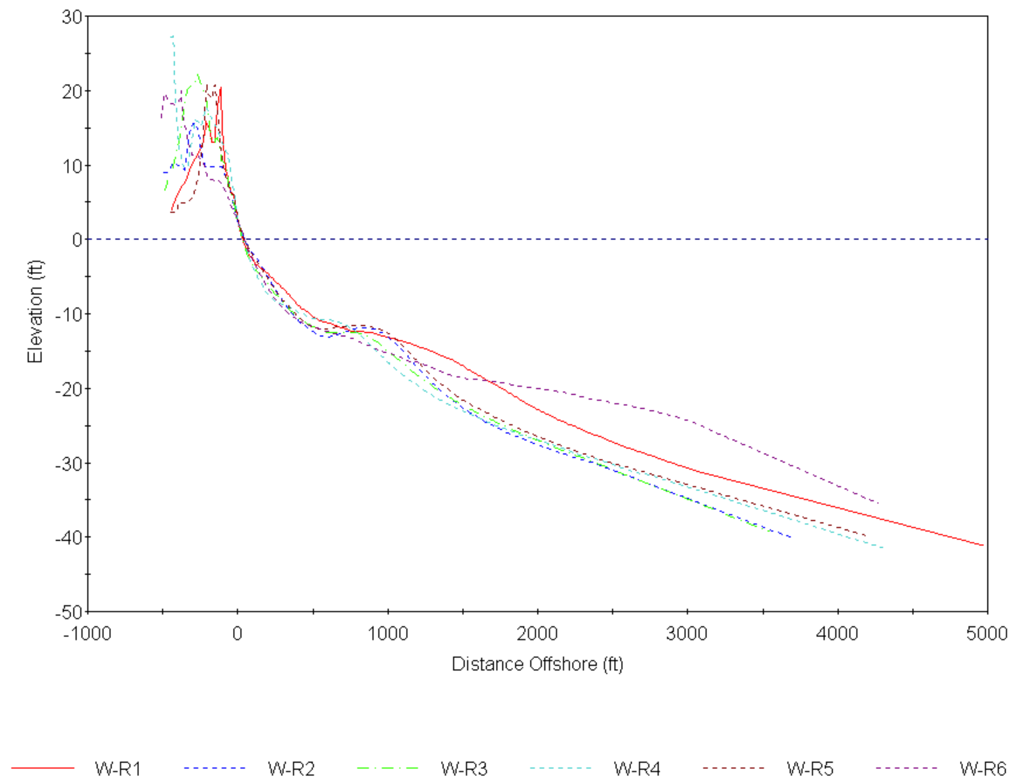


Figure 3-2. Representative profiles for the Westhampton subreaches.

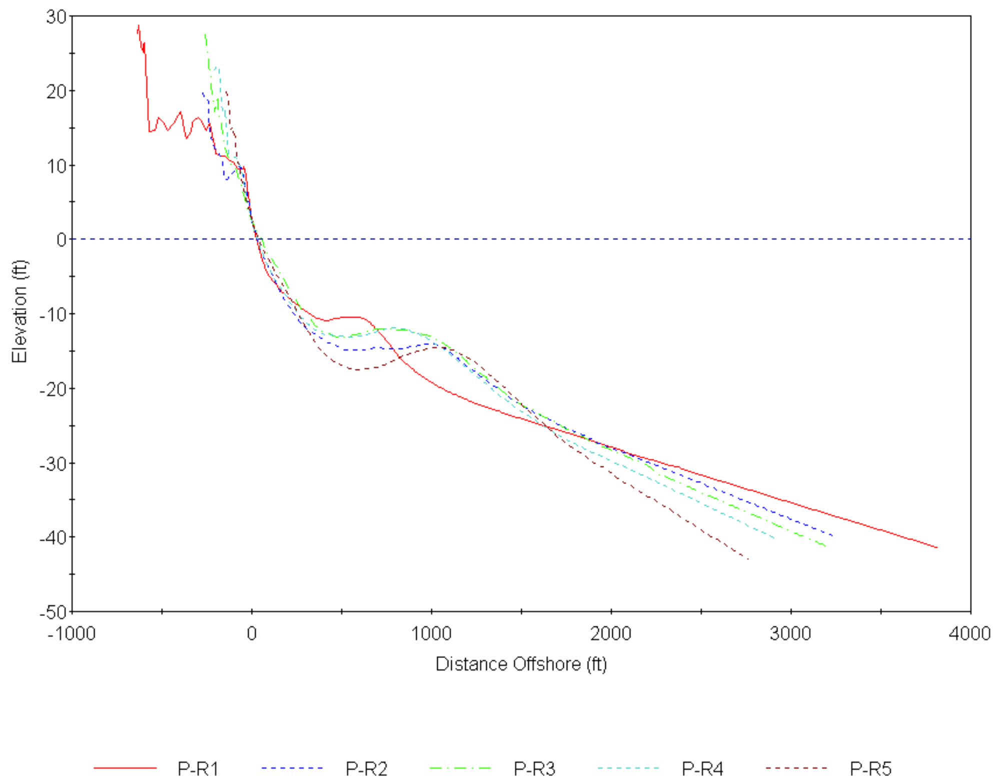


Figure 3-3. Representative profiles for the Ponds subreaches.

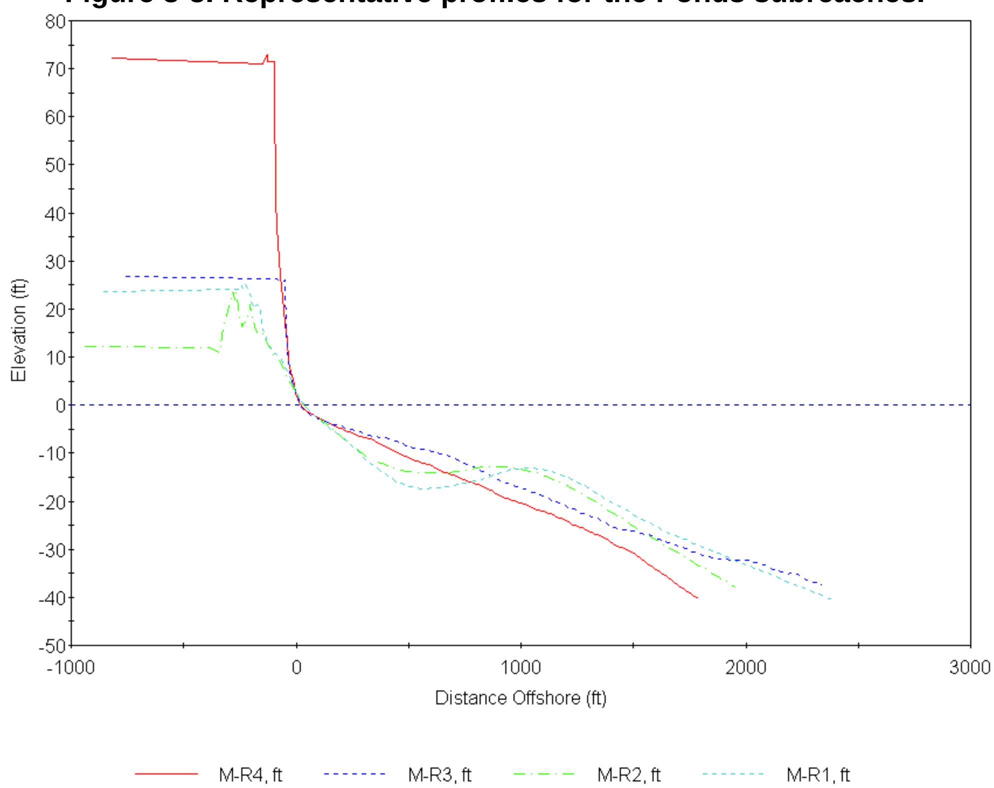


Figure 3-4. Representative profiles for the Montauk subreaches.

Table 3-1. Typical Profile characteristics

Project Reach	Design Reach	Name	Morphologic Reach	Dune El. (ft NGVD)	Dune Slope (1 on)	Berm Height (ft NGVD)	Berm Width (ft)	Beach Slope (1 on)
GSB	GSB-D1	Fire Island Inlet -East	F-R1	20.3	7.6	8.9	160.8	14.3
		Robert Moses - West						
		Robert Moses - East						
		Coast Guard Station						
	GSB-D2	Saltaire	F-R2	16.4	2.4	9.2	78.7	12.7
		Atlantique						
		Ocean Beach						
		Ocean Bay Park						
		Sailors Haven	F-R4	25.3	3.5	14.1	39.4	14.3
		Fire Island Pines	F-R3	18.4	3.7	9.8	52.5	14.3
		Water Island						
		Davis Park						
	GSB-D3	Wilderness Area - West	F-R4	25.3	3.5	14.1	39.4	14.3
		Old Inlet	F-R5	19.0	3.3	9.8	108.3	16.3
		Wilderness Area - East						
MB	MB-D1	Smith Point - West						
		Smith Point - East						
	MB-D2	Great Gun	F-R6	24.6	5.7	13.5	285.4	22.9
		Moriches Inlet - West	F-R7	26.2	5.7	12.1	91.9	11.4
		Moriches Inlet - East	W-R1	18.7	9.5	8.2	36.1	16.3
		Pikes	W-R2	15.1	5.7	4.3	23.0	14.3
		Westhampton	W-R3	22.3	11.4	12.5	49.2	14.3
SB	SB-D1	Hampton Beach	W-R4	17.7	5.7	10.5	88.6	16.3
		Sedge Island	W-R5	21.0	7.1	-	-	16.3
		Tiana Beach						
	SB-D2	Ponquogue						
		Shinnecock Inlet - West	W-R6	18.7	11.4	5.9	49.2	16.3
		Shinnecock Inlet - East	P-R1,2	27.6	4.0	13.1	157.5	12.7
	SB-D3	Southampton Beach	P-R2	19.7	2.1	8.2	52.5	19.1
		Southampton						
		Agawam						
P	P-D1	Wickapogue						
		Watermill	P-R3	27.6	4.7	16.4	26.2	28.6
		Mecox Bay	P-R4	23.0	3.5	15.7	16.4	16.3
		Dune Road						
		Surfside Drive						
		Sapaponack Lake						
		Peters Lane	P-R5	-	-	17.1	-	14.3
		Wainscott						
		Georgica Pond						
		Apaquogue						
M	M-D1	Beach Hampton	M-R1	23.3	6.3	24.3	29.5	16.3
		East Hampton Beach	M-R2	24.6	5.1	21.0	52.5	22.9
		Hither Hills						
		Montauk Beach						
		Ditch Plains	M-R3	-	-	-	-	7.1

3.3.2 Sediment Characteristics

3.3.2.1 Beach Sediment

Along the study area, the grain size distribution of the beach material varies. Typically, grain size increases from west to east, with mean grain size of 0.39 mm at Robert Moses State Park to 0.52 mm at Montauk Point. However, there are some exceptions with the mean grain size west and east of Westhampton groin field, being 0.45 mm and 0.40 mm, respectively.

3.3.2.2 Offshore Sediment

The inner continental shelf south and offshore of the Study Area is characterized by ridge and swale morphology. Surficial sediments are predominantly fine to medium grained sands. Fine-grained sediment outcrops exist in isolated areas of the inner shelf and shoreface. The geology of this area is complex and is characterized by Holocene sediments of variable thickness. These sediments generally consist of either organic-rich muds (backbarrier deposits typically found in the sheltered waters leeward of a barrier island) or modern marine and inlet-filling sands. The area west of Moriches Inlet is typified by a seaward-sloping wedge-shaped deposit of backbarrier sediments underlying marine sand. The maximum thickness of these Holocene sediments is 10 ft along the western portion of Fire Island. This sedimentary layer thins towards Moriches Inlet. Although there are some isolated pockets of backbarrier sediments, marine sands generally lie directly over Pleistocene sediments in the area between Moriches and Shinnecock Inlets with maximum thicknesses of 1 m. The Holocene sediments east of Shinnecock Inlet typically consist of a thin layer of sand and gravel overlying Pliestocene sediments.

Since the 1960’s, efforts have been undertaken in the Study Area to identify locations offshore which contain sediment (sand) which would be a suitable source for beach nourishment, including considerations for compatibility to native beach grain size, the amount of volume available, environmental considerations, and distance to the project site. A number of borrow sites were investigated based on existing and recent collection of boring logs, seismic maps, and samples collected at various upland sites. These borrow sites are described in detail in the Borrow Appendix.

3.3.3 Reach Delineation

3.3.3.1 Project Reaches

Due to its large size and the physical diversity within its borders, the FIMP study area has been divided into smaller reaches to facilitate study efforts, and for improvement design. Five project reaches subdividing the FIMP study area have been established based on major morphological features. Project reaches are large in scale and are defined by common physical characteristics that reflect site conditions such as waves and underlying geology, and which may influence the design of structural works. The study shoreline has been divided into five project reaches, as follows:

Project Reach 1 – Great South Bay (GSB)
Project Reach 2 – Moriches Bay (MB)

Project Reach 3 – Shinnecock Bay (SB)
Project Reach 4 – Ponds (P)
Project Reach 5 – Montauk (M)

Each of the project reaches is identified by a letter abbreviation, as shown above. Some site conditions including astronomical tides vary over the length of the FIMP study area and project reaches are used to differentiate parameters that change over distance. Project reaches, physical reaches and design sub-reaches are shown in Figure 3-5.

3.3.3.2 Physical Reaches and Design Sub-Reaches

Project reaches are further subdivided into physical reaches and design sub-reaches, for the purpose of conceptual screening of alternatives and design of improvements. Physical reaches (Figure 3-5) were defined as continuous shore segments having similar geomorphic features and constraints. As stated above, physical reaches are subdivisions of project reaches. Project features would be consistent within a physical reach, but may vary between neighboring physical reaches. Consequently, alternatives for a given project reach include the design features of each applicable physical reach. Design sub reaches (Figure 3-5) correspond to those areas where storm damage problems and economic development may provide economic justification for coastal storm damage risk reduction plans, but were primarily selected based upon identified storm damage problems. Each of the designated reaches and sub reaches start with a letter abbreviation representing its location so that reach locations may be readily identified. These are summarized in Table 3-2.

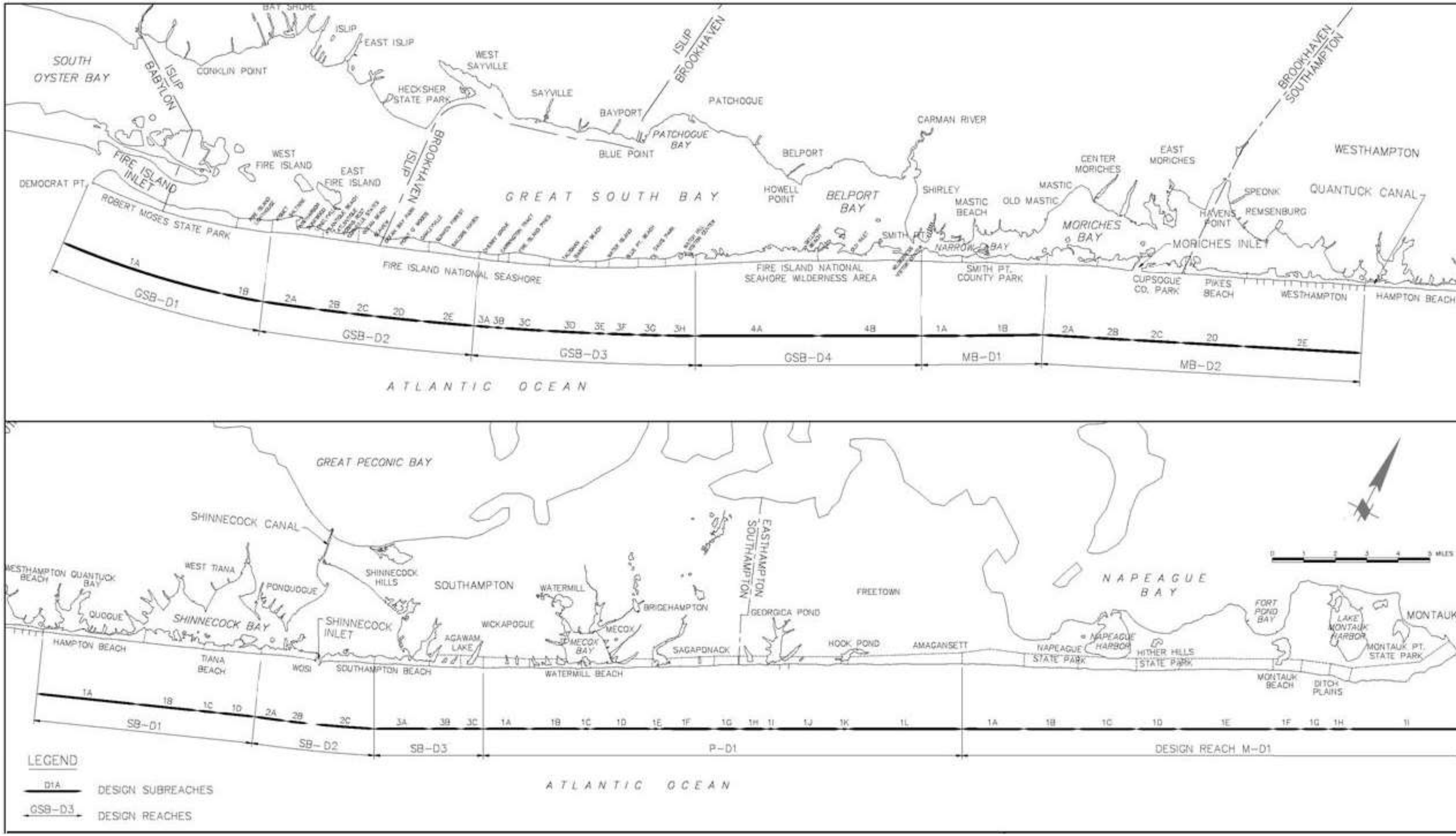


Figure 3-5. Reach delineation

Table 3-2. Reach stationing.

Project Reach	Design Subreach	Design Subreach	length (feet)	Beginning Station	Ending Station	Distance from Fire Island Inlet (miles)
GSB	GSB-D1	GSB-D1A	5,942	0+00	59+42	1.1
		GSB-D1B	5,102	59+42	110+44	2.1
		GSB-D1C	10,825	110+44	218+68	4.1
		GSB-D1D	7,545	218+68	294+14	5.6
	GSB-D2	GSB-D2A	5,231	294+14	346+44	6.6
		GSB-D2B	5,800	346+44	404+45	7.7
		GSB-D2C	5,377	404+45	458+22	8.7
		GSB-D2D	6,831	458+22	526+53	10.0
		GSB-D2E	7,876	526+53	605+30	11.5
		GSB-D2F	11,859	605+30	723+89	13.7
		GSB-D2G	13,323	723+89	857+12	16.2
		GSB-D2H	8,582	857+12	942+94	17.9
	GSB-D3	GSB-D3A	21,305	942+94	1155+99	21.9
		GSB-D3B	5,918	1155+99	1215+17	23.0
		GSB-D3C	8,433	1215+17	1299+50	24.6
MB	MB-D1	MB-D1A	4,824	1299+50	1347+74	25.5
		MB-D1D	18,059	1347+74	1528+32	28.9
	MB-D2	MB-D2A	4,213	1528+32	1570+45	29.7
		MB-D2B	5,455	1570+45	1625+00	30.8
		MB-D2C	10,285	1631+30	1734+15	32.8
		MB-D2D	5,448	1734+15	1788+63	33.9
		MB-D2E	17,715	1788+63	1965+78	37.2
SB	SB-D1	SB-D1A	19,028	1965+78	2156+06	40.8
		SB-D1B	9,514	2156+06	2251+20	42.6
		SB-D1C	9,633	2251+20	2347+53	44.5
	SB-D2	SB-D2A	5,946	2347+53	2407+00	45.6
		SB-D2B	3,735	2407+00	2444+34	46.3
		SB-D2C	10,509	2451+00	2556+09	48.4
	SB-D3	SB-D3A	8,366	2556+09	2639+75	50.0
		SB-D3B	4,426	2639+75	2684+01	50.8
		SB-D3C	3,822	2684+01	2722+23	51.6
P	P-D1	P-D1A	9,090	2722+23	2813+13	53.3
		P-D1B	8,497	2813+13	2898+10	54.9
		P-D1C	1,877	2898+10	2916+87	55.2
		P-D1D	5,434	2916+87	2971+21	56.3
		P-D1E	3,359	2971+21	3004+80	56.9
		P-D1F	2,075	3004+80	3025+55	57.3
		P-D1G	12,449	3025+55	3150+04	59.7
		P-D1H	3,854	3150+04	3188+58	60.4
		P-D1I	1,976	3188+58	3208+34	60.8
		P-D1J	11,264	3208+34	3320+97	62.9
M	M-D1	M-D1A	24,954	3320+97	3570+52	67.6
		M-D1B	15,216	3570+52	3722+68	70.5
		M-D1C	14,121	3722+68	3863+89	73.2
		M-D1D	22,033	3863+89	4084+21	77.4
		M-D1E	28,929	4084+21	4373+51	82.8

Notes: Reach baseline stationing is based on the most recent topographic maps available (Fire Island-1999, Moriches Inlet to Montauk Point-1995)

3.3.4 Climate

Mild winters and relatively cool summers characterize the climate of Long Island. Extreme fluctuations of temperature are relatively infrequent due to the moderating effects of the Atlantic Ocean. The mean annual temperature in the project area is approximately 50°F. The coldest months (i.e., January and February) average about 30°F, while the warmest month (July) averages about 70°F. Extreme temperatures range from about -10°F to 100°F. Annual precipitation averages approximately 45 inches, with lower amounts in the summer months. According to USACE (1958), the heaviest precipitation recorded on Long Island for a 6-hour period was 5.6 inches, recorded on 7 August 1946 at Riverhead.

3.3.5 Astronomical Tides

Astronomical tides on the south shore of Long Island are semi-diurnal, rising and falling twice daily. For storm damage assessment, understanding the expected range of astronomical tide along the project length and within the three bays is required. For this study, the ADCIRC long-wave hydrodynamic numerical model was employed to determine astronomical tide amplitudes throughout the project and to determine the maximum expected annual water level associated with astronomical tides (Table 3-3). Additional details on the ADCIRC model are provided in Chapter 6.1.1.

3.3.6 Sea Level Change

By definition, sea level change is a change (increase or decrease) in the mean level of the ocean. Eustatic sea level rise is an increase in global average sea level brought about by an increase to the volume of the world's oceans (thermal expansion). Relative sea level change takes into consideration the eustatic increases in sea level as well as local land movements of subsidence or lifting. Long Island is one of many areas in which the land is subsiding. This Reformulation effort considers a range of future sea level rise projections, including the historic rate as the low boundary, and accelerated rates of sea level rise, as described below.

Historic information and local MSL trends used for the Study Area are provided by the NOAA/NOS Center for Operational Oceanographic Products and Services (CO-OPS) using the tidal gauge at Sandy Hook, New Jersey. The historic sea level change rate (1935-2013) is approximately 0.0128 ft. per year or about 1.3 ft. per century.

Recent climate research has documented observed global warming for the 20th century and has predicted either continued or accelerated global warming for the 21st century and possibly beyond (IPCC 2013). One impact of continued or accelerated climate warming is continued or accelerated rise of eustatic sea level due to continued thermal expansion of ocean waters and increased volume due to the melting of the Greenland and Antarctic ice masses (IPCC, 2013). A significant increase in relative sea level could result in extensive shoreline erosion and dune erosion. Higher relative sea level elevates flood levels which may result in smaller, more frequent storms that could result in dune erosion and flooding equivalent to larger, less frequent storms.

The current guidance (ETL 1100-2-1 dated 30 Jun 2014) from the Corps states that proposed alternatives should be formulated and evaluated for a range of possible future local relative sea level change rates. The relative sea level rates shall consider as a minimum a low rate based on an extrapolation of the historic rate, and intermediate and high rates which include future acceleration of the eustatic sea level change rate. These rates of rise correspond to 0.7 ft., 1.1 ft., and 2.4 – 6 ft. over 50 years for the low, medium and high rates of relative sea level rise, respectively. It is noted that the cited Corps guidance includes an assessment of some of the most relevant work on sea level rise including the 2012 National Climate Assessment, Rahmstorf et al. (2012), or Kopp et al., (2014).

New York State has also recently adopted sea level rise scenarios as part of the Community Risk and Resiliency Act. As part of this Act, NYSDEC has identified 5 different projections of sea level rise for three different regions within N.Y. that are tidally influenced. The projections for the Long Island Region are as follows. The 2050s projections are: 8 in. (low), 11 in. (low-medium), 16 in. (medium), 21 in. (high-medium), and 30 in (high). The 2080s projections are: 13 in. (low), 18 in. (low-medium), 29 in. (medium), 39 in. (high-medium), and 58 in. (high).

Most of the analysis contained within this report applies the historic (low) rate of sea level rise. The use of the historical rate of sea level rise for planning purposes is acknowledged to be a conservative approach. Including a higher rate of sea-level rise would result in a larger amount of damages, and could show the need for plans that would only be required under higher accelerated sea level rise conditions. Consistent with Corps guidance, the alternative evaluation was conducted using the historic rate of RSLC in order to select a plan. Following selection of the plan, the TSP has been evaluated to show the effectiveness of the plan under the intermediate and high rate of RSLC.

Table 3-3. ADCIRC-simulated average maximum annual astronomical tide elevation

Location		Average Maximum Annual Astronomical Tide Elevation (ft, NGVD29)
Ocean	Great South Beach (41)	3.9
	Old Inlet (9)	3.7
	Post Lane (31)	3.5
	Watermill Beach (63)	3.3
	Ditch Plains (39)	3.0
Great South Bay	West of Fire Island Inlet	2.3 – 2.7
	East of Fire Island Inlet	1.6 – 1.8
Moriches Bay		2.3 – 2.8
Shinnecock Bay		3.2 – 3.4

3.3.7 Storms

Two types of storms are of primary significance along the south shore of Long Island: (1) tropical storms which typically impact the New York area from July to October, and (2) extratropical storms which are primarily winter storms occurring from October to March. Extratropical storms (northeasters) are usually less intense than hurricanes, but tend to have a much longer duration. These storms often cause high water levels and intense wave conditions, and are responsible for significant damages and flooding throughout the Long Island coastal region.

Hurricanes are the most powerful tropical storms to reach the New York area with wind speeds in excess of 74 mph (by definition). Records are available for 24 hurricanes having impacted the New York Area in the past century. Heavy storm damage usually occurs when high astronomical tides and storm surge coincide for storms approaching the project area from the south-southwest. The combined water levels allow large waves to penetrate inland resulting in extreme erosion and flooding.

Extratropical storms originate outside of the tropics, usually in the mid- to upper-latitudes during winter months. In the New York region, these storms are referred to as "northeasters" due to the predominate direction from which the winds originate. Northeasters are less intense than hurricanes with sustained wind speeds generally below 50 knots. Localized winds may, however, reach hurricane strength. Extratropical storms cover large areas and are slow moving with typical storm duration lasting for a period of days thus persisting through several periods of high astronomical tide. The long duration greatly enhances the ability of northeasters to cause damages. USACE (1969) states that 65 moderate to severe northeasters have impacted the New York coastal region over the 100 year period preceding 1965. More recently, a series of severe northeasters has impacted the New York coastal region in October 1991, December 1992, and March 1993.

3.3.7.1 Storm Training Set Selection

To evaluate storm surge, storm profile response, and storm wave conditions, a set of storms was identified for use in statistical analysis. This storm set, or training set, was selected using the peak-over-threshold method and is representative of the expected tropical and extratropical storm climate within the study area. The training set includes 14 historical tropical and 22 historical extratropical storms (Table 3-4). The 14 tropical storms include all tropical storms whose track came within 500 nautical miles of Long Island between 1930 and 2001 and whose surge (water level minus predicted astronomical tide) in the vicinity of Long Island exceeded 2.23 ft. The 22 extratropical storms include all significant extratropical storms impacting the Long Island area between 1950 and 1998 whose surge in the vicinity of Long Island exceeded 3.3 ft. It is noted that the FIMP storm training set was developed in the 2000's, a decade before the NACCS storm suite became available. However, a comparison of the FIMP stage frequency curves and NACCS stage frequency curves showed that the two sets of curves matched along the open ocean coastline.

Table 3-4. Historical storms selected for FIMP training set.

Tropical Events (1930 – 2001)			Extratropical Events (1950 – 1998)	
Name	**Start Date (based on NHC database)**	**Duration** (hours)**	**Start Date**	**Duration** (hours)**
not named	10-Sep-1938*s	15	22-Nov-1950	34s
not named	9-Sep-1944s	10	04-Nov-1953	26s
Carol	25-Aug-1954s	5	11-Oct-1955	43
Edna	2-Sep-1954	7	25-Sep-1956	34
Hazel	5-Oct-1954	6	03-Mar-1962*	56s
Connie	3-Aug-1955	0	05-Nov-1977	28
Donna	29-Aug-1960*s	13	17-Jan-1978	16
Esther	10-Sep-1961	14	04-Feb-1978	27
Doria	20-Aug-1971	2	22-Jan-1979	19
Agnes	14-Jun-1972	18	22-Oct-1980*	17s
Belle	6-Aug-1976*s	7	26-Mar-1984	31
Gloria	16-Sep-1985*s	5	09-Feb-1985	17
Bob	16-Aug-1991*	4	28-Oct-1991	50+
Floyd	7-Sep-1999*	3	01-Jan-1992	18
			08-Dec-1992*	78s
			02-Mar-1993	12
			10-Mar-1993*	25s
			28-Feb-1994*	22
			21-Dec-1994*	23
			05-Jan-1996	25
			6-Oct-1996	12
			02-Feb-1998	24

* Indicates storm is included in the calibration set.
s Indicates storm included in supplemental set with alternate astronomical tide condition.
** Storm durations represent duration that storm surge exceeded 1 ft (0.3 m), based on ADCIRC simulations at Station 31.
\+ Storm duration for this storm based on measured storm surge at Sandy Hook, NJ.

3.3.8 Winds

3.3.8.1 Long-Term Average Annual Wind Conditions

Records of the US Coast Guard and Suffolk County Highway Department available for the south shore of Long Island from 1940 to 1959 were compiled (Table 3-5). Predominant wind directions are from the southwest, west and northwest with percent-occurrences of 22, 17 and 17 percent, respectively. Given the orientation of the study area shoreline, winds from the southeastern quadrant have a marked influence on study area coastal processes. These winds, which blow over practically unlimited fetch distances, account for nearly 25 percent of all wind occurrences. Wind speeds in the project vicinity were also described in USACE (1958). It was reported that over 50 percent of winds exceeding 38 miles per hour (mph) were from the west and northwest, with similar winds from the east, southeast and south totaling about 20 percent. Wind data extracted from Hubertz *et al.* (1993), while not directly applicable to the study area, represent the offshore wind environment and indicate that predominate wind speeds range from 5.5 to 28 mph totaling about 90 percent of all recorded wind speeds. Furthermore, approximately 70 percent of recorded wind records were less than 16.5 mph.

Table 3-5. Annual average wind directions.

Wind Direction	Percent-Occurrence
North	10
Northeast	9
East	9
Southeast	6
South	9
Southwest	22
West	17
Northwest	17
Calm	1

Source: U.S. Coast Guard and Suffolk County Highway Department

Additional wind speed/direction data for the study area were available from the U.S. Naval Oceanographic Office (1970). Annual percent-occurrence statistics for wind direction/speed data were separated into eight direction bands as shown in Table 3-6. As shown in this table, predominant wind directions are from the south, southwest, and west, which occur approximately 18, 16 and 17 percent of the time, respectively. Winds from the south and southeast account for nearly 26 percent of all wind occurrences. Wind speed-exceedance relationships for the study area, based on data in Table 3-6, are shown in Table 3-7 and Figure 3-6. It is evident that wind speeds are typically less than 27 knots, accounting for approximately 95 percent of all observations. The dominant wind speed range is from 7 to 16 knots, which occurs nearly 49 percent of the time. Wind speeds exceeding 27 knots (strong breeze) are less frequent with a total occurrence percentage of approximately 5 percent.

Table 3-6. Annual percentage of wind direction by speed.

Wind Speed		Direction									
Knots	Description	Ind.	North	NE	East	SE	South	SW	West	NW	Total
0-6	Calm	3.2	2.3	1.8	2.4	2.9	4.8	3.9	2.9	2.0	**26.2**
7-16	Gentle Breeze		4.1	4.1	4.2	4.2	9.7	8.8	7.9	5.8	**48.8**
17-27	Fresh Breeze		2.0	2.2	1.3	1.0	2.7	2.8	4.5	3.7	**20.2**
28-40	Strong Breeze		0.5	0.7	0.2	0.1	0.3	0.4	1.3	0.9	**4.4**
>41	Gale		*	0.1	*	*	0.0	*	0.1	0.1	**0.3**
TOTAL		**3.2**	**8.9**	**8.9**	**8.1**	**8.2**	**17.5**	**15.9**	**16.7**	**12.5**	**99.9**

Source: U.S. Naval Oceanographic Office (1970).

Table 3-7. Wind speed exceedance.

Wind Speed (knots)	Description	Mean Wind Speed (Knots)	Percent Occurrence	Percent Exceedence
<0.1	Calm	0.1	0.1	99.9
0.1-6	Calm	3	26.1	73.8
7-16	Gentle Breeze	11.5	48.8	25.0
17-27	Fresh Breeze	22	20.2	4.8
28-40	Strong Breeze	34	4.4	0.4
>41	Gale	41	0.3	0.1

Source: U.S. Naval Oceanographic Office (1970).

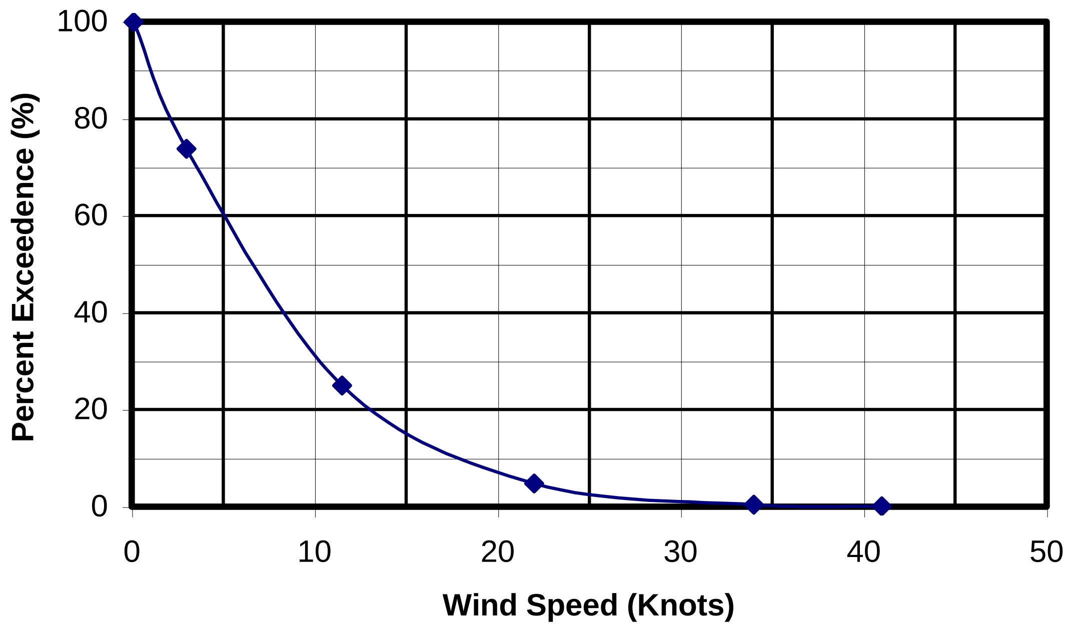


Figure 3-6. Wind speed exceedance offshore of Long Island, New York (U.S. Naval Oceanographic Office, 1970).

3.3.8.2 Storm Wind Conditions

For this study, Oceanweather, Inc. developed meteorological forcing for 36 tropical and extratropical storms. All wind velocity fields represent the 30-minute average velocity[1] at 10-m above the water surface. Tropical wind and barometric pressure fields were developed using a Planetary Boundary Layer (PBL) model, a tropical cyclone model (Thompson and Cardone, 1996). PBL describes the vortex pressure field using existing historical information on storm track, central pressure deficit, and other parameters.

Storm tracks and initial estimates of intensity for each of the 14 tropical storms were taken, with some modification, from the NOAA Tropical Prediction Center's database (Jarvinen et al., 1984). Surface winds generated from PBL were then imported into a

[1] Wind speeds used with the Saffir-Simpson Hurricane Scale are 1-min. The 3-min to 1-min conversion is 1.2.

graphical interface at 6-hourly intervals and evaluated against available surface data and aircraft reconnaissance wind observations adjusted to the surface as described by Powell and Black (1989). This process was iterated until a solution for the surface wind fields that is most consistent with all of the available data was achieved. The final wind field is this best fit model solution. Maximum PBL wind speeds near landfall on Long Island are given in Table 3-8.

Table 3-8. Tropical (PBL) and extratropical (IKOA) maximum wind speed at Long Island for selected storms.

Tropical Events			Extratropical Events	
Name	**Start Date**	**Maximum PBL Wind Speed (kt, 30-min, 10-m)**	**Start Date**	**Maximum IKOA Wind Speed at NDBC Buoy 44025 (kt, 30-min, 10-m)**
not named	10-Sep-1938	57 (68, 1-min)	08-Dec-1992	47
Carol	25-Aug-1954	66 (79, 1-min)	10-Mar-1993	45
Belle	6-Aug-1976	64 (77, 1-min)		
Gloria	16-Sep-1985	76 (91, 1-min)		

Extratropical storm wind fields were developed using the Interactive Kinematic Objective Analysis (IKOA) system. The benefits of IKOA enhancement to the performance of ocean response modeling over wind fields produced by strictly automated methods for extratropical storms are well established (e.g., Cardone et al., 1995). The IKOA starts from a first-guess background wind field and then proceeds to assimilate observations of surface winds from ships, buoys, coastal stations, and remote sensing sources. If available, background winds were taken from the AES40 hindcast (Swail and Cox, 1999). Maximum IKOA wind speeds at the NDBC Buoy 44025 location are given in Table 3-8 for the December 1992 and the 9 March 1993 Nor'easters.

For extratropical events, barometric pressure fields were taken directly from NOAA's NCEP (National Center for Environmental Prediction) database (www.ncep.noaa.gov).

Tropical and extratropical wind and pressure fields were produced on a grid domain extending from 30° N to 47° N and from 64° W to 82° W to capture far-field surge and wave field generation (Figure 3). Wind fields were reported at a high-resolution grid spacing of 0.0625° latitude by 0.0625° longitude (about 7 km) for tropical events to resolve the details of the cyclonic structure. A coarser grid spacing of 0.625° latitude by 0.833° longitude was used to report wind fields for extratropical events. Temporal resolution for tropical and extratropical events was 30 minutes and 3 hours, respectively.

No land effects were considered during wind field development. Therefore, a 30 percent reduction in wind speed for all offshore-directed winds in nearshore areas was adopted for this study (Resio, personal communications).

3.3.9 Storm Surge and Extreme Water Levels

Storm effects (i.e., storm surge and wave setup) combine with astronomical tides to produce extreme water levels in the study area. Storm surge is a temporary rise in water level generated during the passage of a major extratropical or tropical storm. The rise in water level results from wind action, low pressure of the storm disturbance and a Coriolis force. Wind stress is an important factor in coastal areas fronted by a shallow, broad continental shelf. Strong onshore winds drive ocean waters towards the coast. Water levels rise at the shoreline when the motion of wind driven water is arrested by the coastal landmass. A rise in water level also attends the low barometric pressure near the center of the storm. Wave setup is a term used to describe the rise in water level attending wave breaking. Specifically, the change in momentum associated with the breaking of waves propagating towards shore results in a surf zone force raising water levels at the shoreline. Using the storm training set given in Table 3-4, storm surge was simulated using a numerical modeling suite to provide a database of extreme water levels at multiple ocean and bay locations within the study area. A full discussion of the modeling suite, simulation results, and stage-frequency relationships is in Chapter 6.1.

3.3.10 Waves

3.3.10.1 Long-term Wave Conditions

Both measured and hindcast wave information is available for the FIMP study domain. The measured wave information is available from two sources; the Westhampton nearshore wave gage (NY001), and National Data Buoy Center (NDBC) buoy 44025. The USACE Westhampton nearshore directional wave gage (DWG-1 type) indexed as NY001 provides directional wave information at hourly intervals. This instrument was installed in June of 1994 and data are generally available until November 2000, except for an approximate 5-month period between mid-January and mid-June 1997. The Westhampton wave gage is positioned approximately 0.8 km (0.5 miles) east of the Westhampton groin field and 1 km (0.6 miles) offshore in a nominal water depth of 10 m. NDBC buoy 44025 is a 3 m discus buoy with a meteorographic payload that measures directional wave information in addition to a suite of other meteorological elements. Directional wave information from NDBC buoy 44025 is available from April 1991 to the present. NDBC buoy 44025 is positioned approximately 43 km (26.7 miles) south-southeast of Fire Island Inlet in a nominal water depth of 40 m. Figure 3-7 through Figure 3-9 summarize wave characteristics at NDBC Buoy 44025. Gravens *et al.* (1999) provide additional information on the measured wave data and data analysis.

The Wave Information Study (WIS) wave hindcast for the Atlantic Ocean provides hindcast estimates of the wave climatology at a total of seven Stations within the FIMP project domain. The WIS provides two hindcast databases for the Atlantic Ocean: 1956-1975 and 1976-1994. The more recent hindcast database was exclusively used in this study because it includes hurricane storm events and was generated using an upgraded hindcast model and wind information as compared to the 1956 to 1975 hindcast database. Figure 3-10 illustrates representative long-term wave characteristics within the FIMP area. Gravens et al. (1999) provides additional information on the WIS hindcast and data analysis.

Long-term wave characteristics determined from the WIS hindcast were used primarily to determine longshore transport potential, to drive the shoreline evolution model GENESIS, and as input to long-term sediment budgets. Long-term wave characteristics determined from the shorter-duration measurements, namely NDBC Buoy 44025 and USACE NY001, were used primarily for input to inlet morphology modeling and for shorter-term sediment budgets spanning the 1990s.

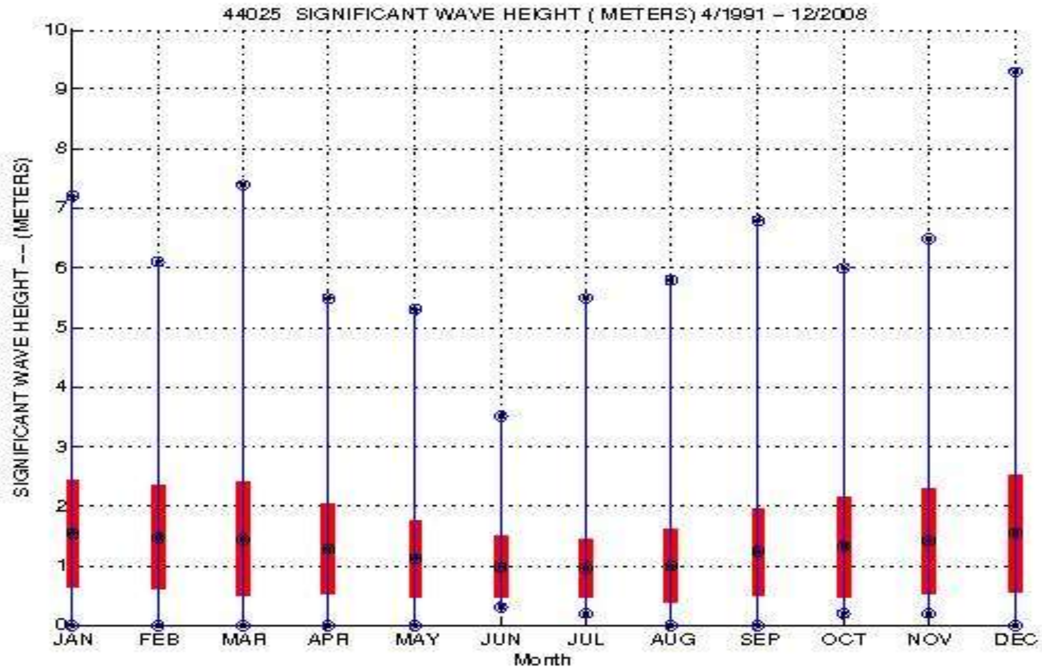


Figure 3-7. NDBC significant wave height summary for buoy 44025 between 1991 and 2001 (1 m = 3.28 ft).

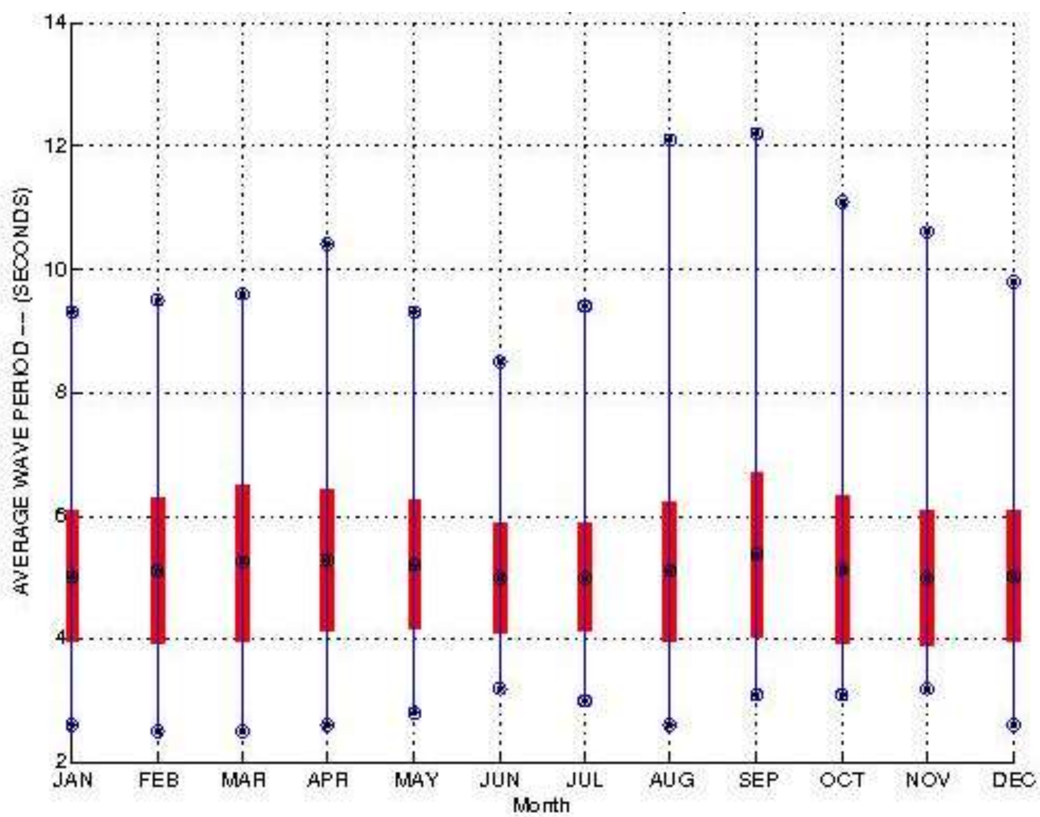


Figure 3-8. NDBC average wave period summary for buoy 44025 between 1991 and 2001 (1 m = 3.28 ft).

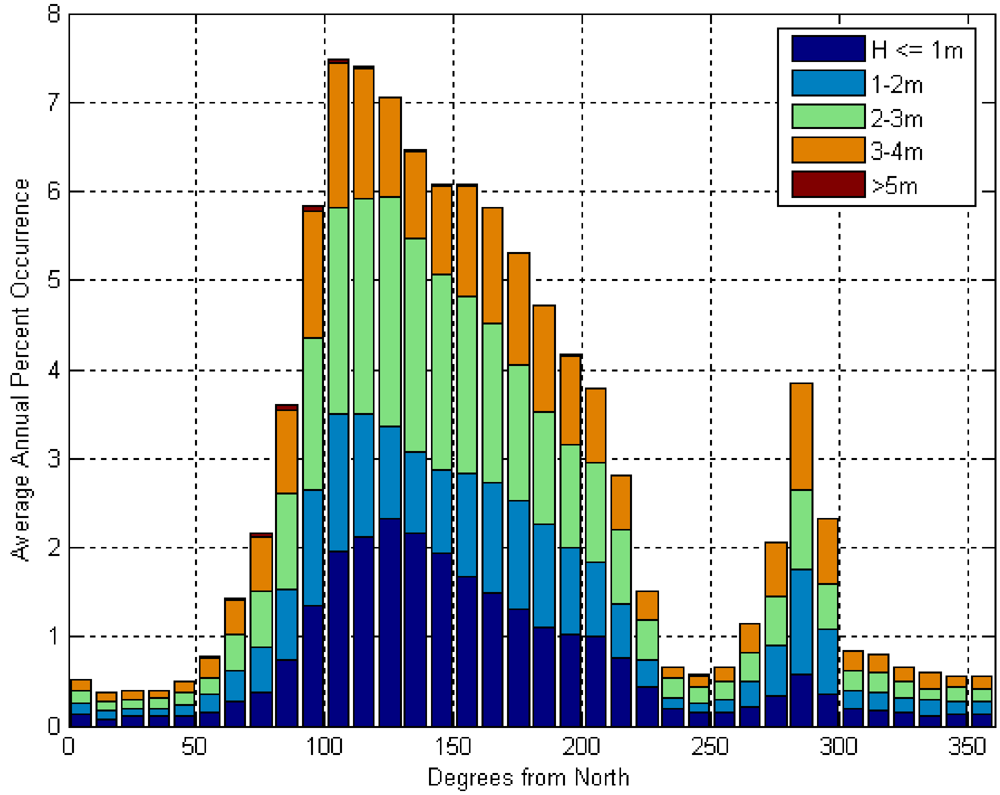


Figure 3-9. Computed mean wave direction summary for buoy 44025 betweeb 1991 and 2004 (1 m = 3.28 ft).

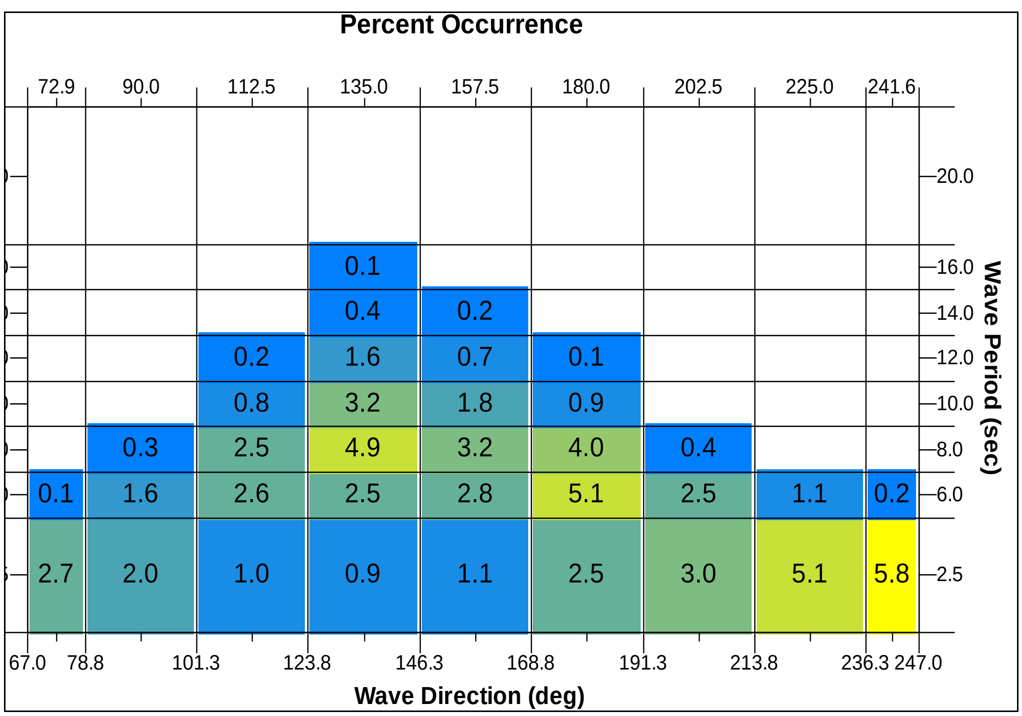


Figure 3-10. Long-term wave characteristics from the 1976-1994 WIS hindcast station 78, offshore of Westhampton.

3.3.10.2 *Storm Wave Conditions*

WISWAVE/WAVEAD (Resio and Perrie, 1989; Hubertz, 1992), a directional spectral wave model, was used to simulate bulk directional spectra, at hourly intervals, at 30-m depths for each of the 36 storms in the FIMP storm training set. WISWAVE solves the time-dependent wave action balance equation and simulates wave growth from wind following the combined Phillips and Miles mechanism. The model includes weak nonlinear wave-wave interaction and accounts for refraction, shoaling, and dissipation by using linear theory.

For this study, WISWAVE was forced with the hindcasted storm wind fields discussed in Chapter 3.1.4. WISWAVE was configured to compute directional wave spectra using 15 frequency bands, 0.03 to 0.31 Hz, and 16 direction bands. To capture both far-field generation and the spatial resolution desired inshore, a nested-grid approach was adopted. The coarsest grid, at 1° resolution, extended from 50° to 80° west longitude and from 20° to 45° north latitude while the finest grids, at 0.0083° resolution, cover inshore areas from west of Fire Island inlet to Montauk Point. For the FIMP study, the directional wave spectra output from WISWAVE were post-processed and used to force both the SBEACH cross-shore profile change model and the DELFT3D nearshore wave model. Additionally, bulk wave characteristics at 10-m water depth were determined using the WIS Phase III transformation technique in order to develop storm significant wave height-frequency relationships. Figure 3-11 through Figure 3-16 present these frequency relationships, as determined using the Empirical Simulation Technique (EST) in multivariate mode, for the seven FIMP design reaches.

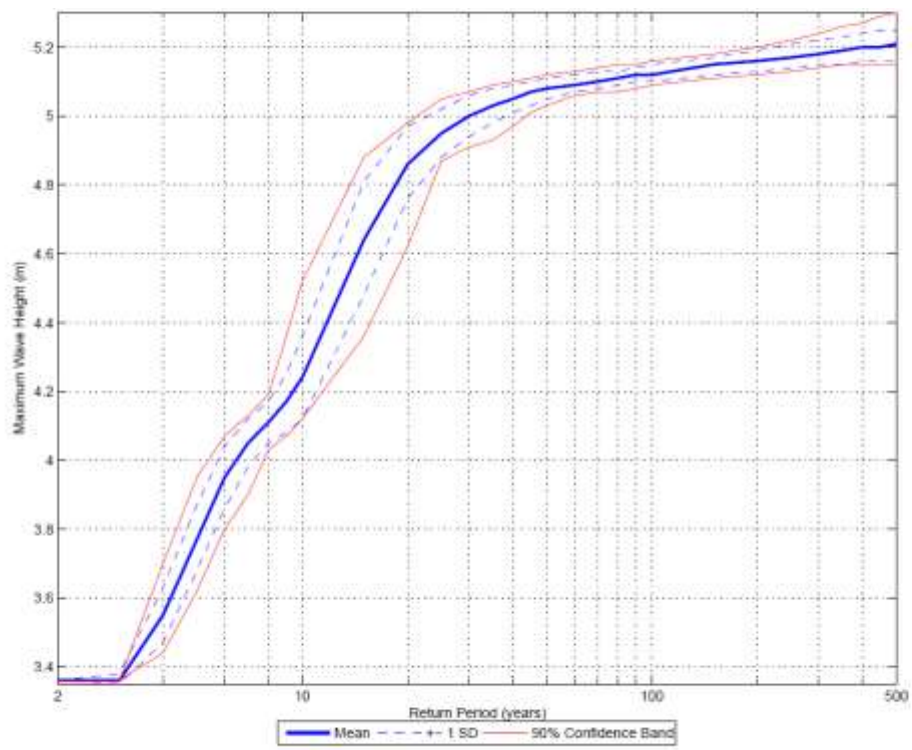


Figure 3-11. Storm significant wave height frequency relationship for East of Fire Island Inlet design reach.

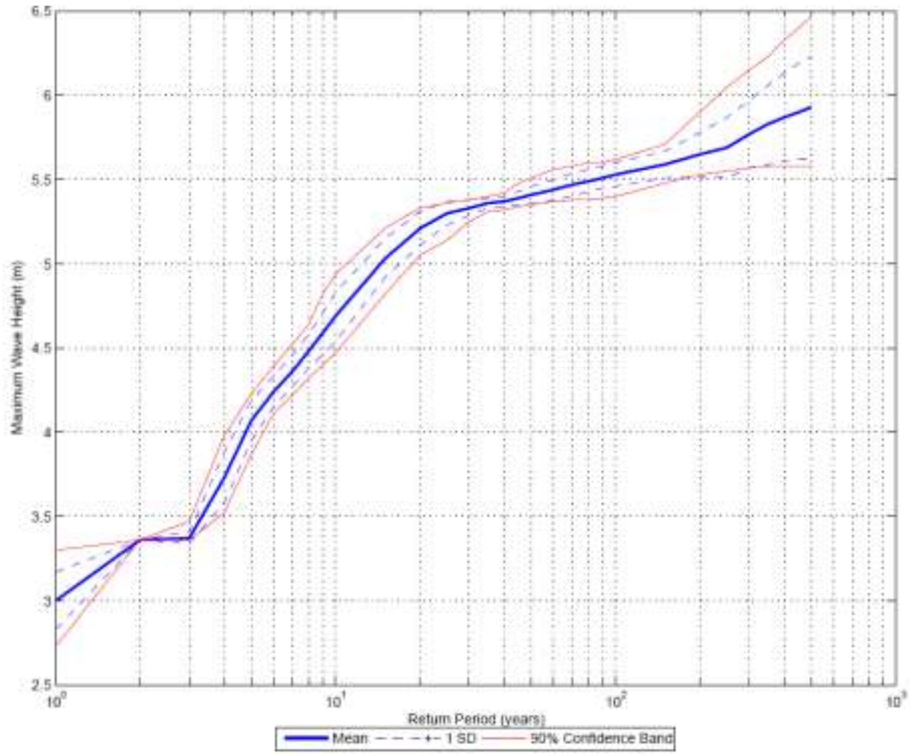


Figure 3-12. Storm significant wave height frequency relationship for Great South Bay design reach.

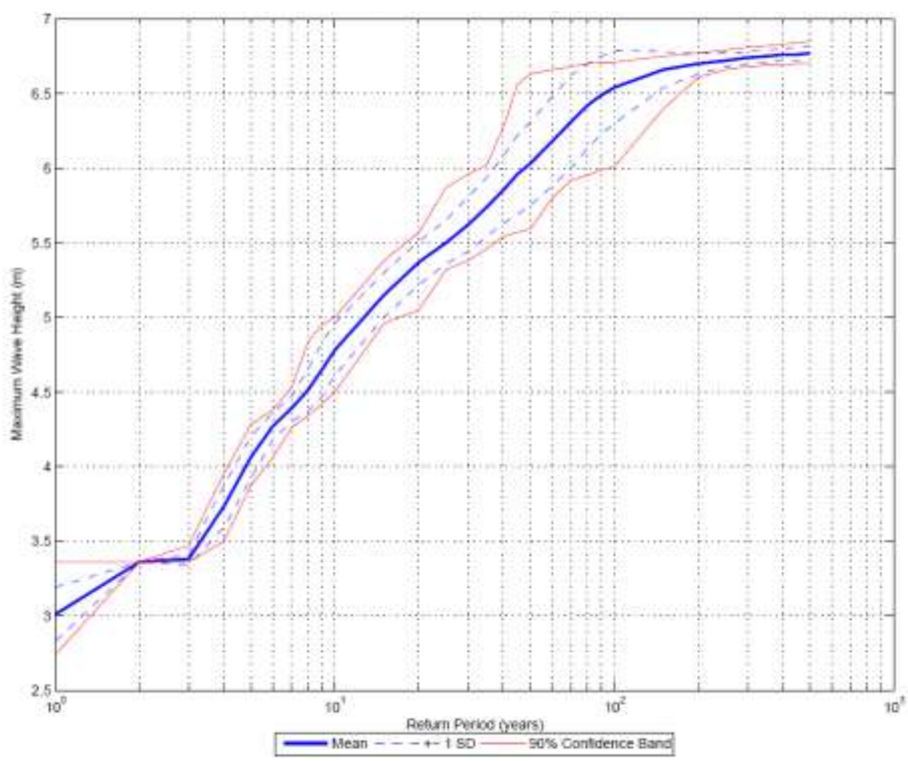


Figure 3-13. Storm significant wave height frequency relationship for West of Moriches Inlet design reach.

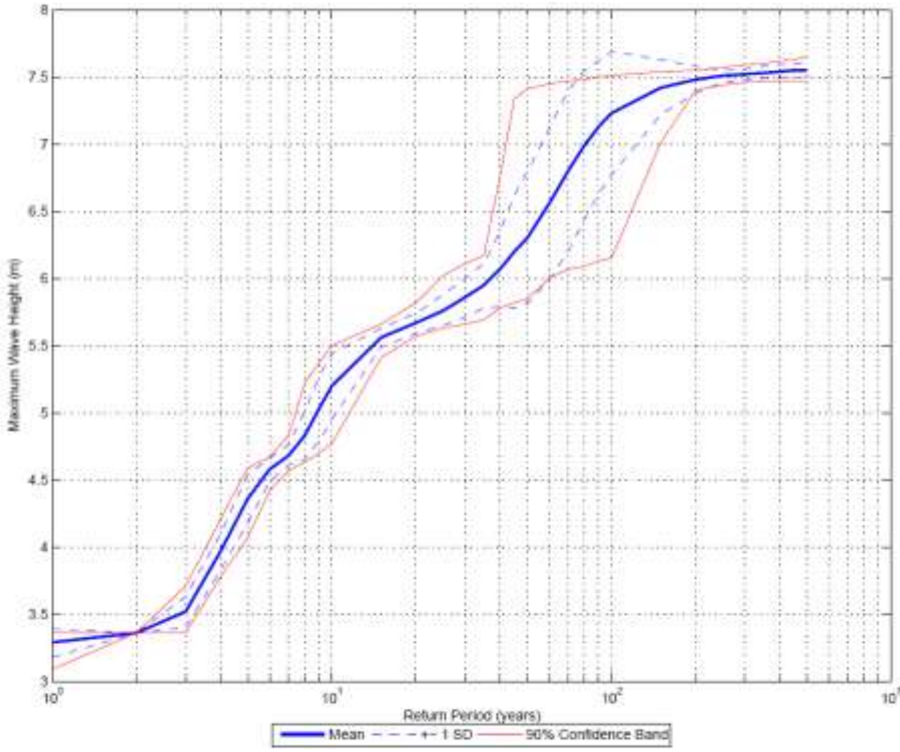


Figure 3-14. Storm significant wave height frequency relationship for West of Shinnecock Inlet design reach.

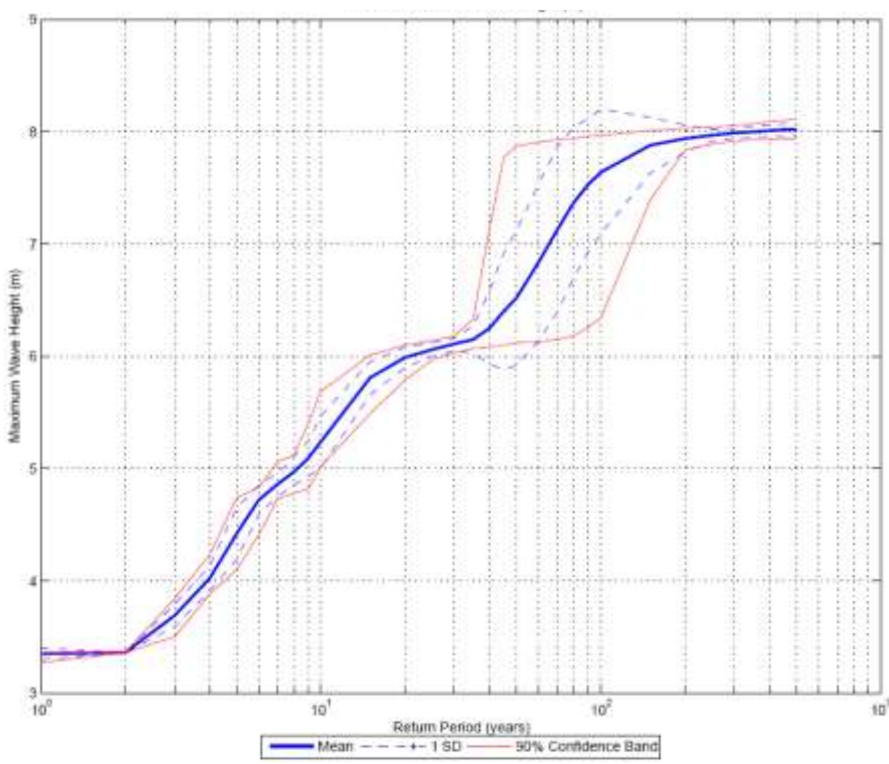


Figure 3-15. Storm significant wave height frequency relationship for Ponds design reach.

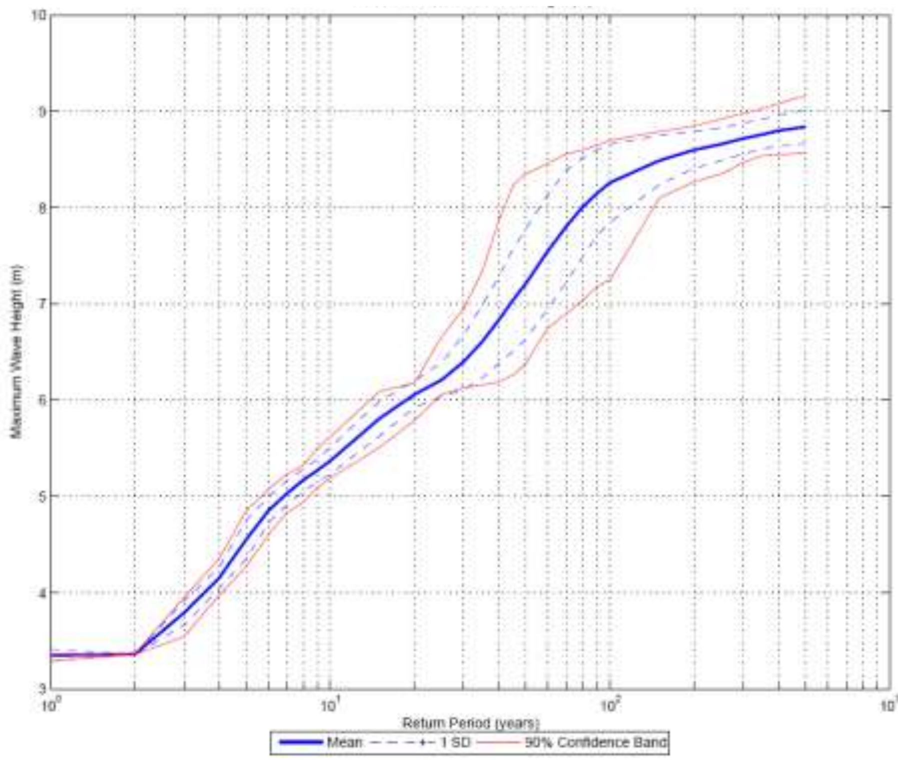


Figure 3-16. Storm significant wave height frequency relationship for Montauk Point design reach.

3.4 Shoreline Changes and Erosion

Beach and dune systems are exposed to three types of erosion, namely, long-term erosion resulting from day to day wave conditions, short-term storm-induced erosion, and erosion resulting from long-term sea level rise. Long-term erosion is associated with gradients and/or interruptions in littoral drift (i.e. long-shore sediment transport). Storms and sea level rise, on the other hand produce cross-shore sediment transport that erodes the shoreface, beach berm and dunes. Storms can dramatically alter the shoreline geometry in a matter of hours or days. The beach profile, however, tends to recover after storm passage and, with sufficient supplies of sediment, can eventually build back to pre-storm geometry. Net shoreline retreat may occur if there is not enough sand available for a full recovery, particularly when longshore sediment transport is interrupted.

Historic Shoreline Rate-of-Change (SRC) values in the FIMP study are documented in Gravens *et al.* (1999), which examined three non-overlapping time intervals using available shoreline data sets. The first period, representative of the epoch prior to significant human influence on the barriers, is 63 years long (1870 to 1933). The second period, representative of initial development on the barriers and the initiation of human intervention with natural processes including inlet stabilization and significant beach fill placements, is approximately 46 years long (1933 to 1979). The third period, representative of modern times and reflecting the most recent beach nourishment practices, is approximately 15 years long (1979 to 1995). Computed average SRC and associated standard deviation values are summarized in Table 3-9 for each of three barrier island-scale analysis domains in the study.

Table 3-9. Average Shoreline Rate of Change and Associated Standard Deviation

Time Period	Analysis Reach		
	Fire Island (i.e., Fire Island to Moriches Inlet)	Westhampton (i.e., Moriches to Shinnecock Inlet)	Montauk (i.e., Shinnecock Inlet to Montauk Point)
1870-1933	-0.4 (1.1)	+0.1 (0.6)	+0.2 (0.3)
1933-1979	-0.4 (1.8)	-1.1 (1.1)	-0.4 (0.6)
1979-1995	-0.7 (1.9)	-0.8 (2.8)	0.0 (1.3)
NOTES: Table adapted from Gravens *et al.*, (1999) Standard Deviation in parenthesis All values in meters/year All values adjusted to account for beach fill placement			

The SRC quantities in Table 3-9 indicate that the Fire Island barrier has, in general, been eroding at a historically consistent rate of about 0.4 m/year (1.3 ft/year). Average shoreline recession has increased to 0.7 m/year (2.3 ft/year) over the most recent 15-year time interval on Fire Island. It is important to note that these SRC values are average values for the entire 30-mile barrier island and that the standard deviation in the SRC is between 3 and 4 times larger than the mean. The comparatively large SRC standard deviation indicates significant variation in the shoreline change signal along Fire Island.

The computed historic SRC within the Westhampton analysis reach varies from nearly stable for the earliest time interval to notably erosive for the intervals since significant development began on the Westhampton barrier. The modern evolution (1933 to 1995) of the Westhampton barrier can generally be characterized as being erosional at an average rate of about 1.0 m/year (3.3 ft/year). Again, the large SRC standard deviation indicates that segments of the barrier are considerably more or less erosive than indicated by the average SRC.

The SRC within the Montauk analysis reach indicates that this analysis reach is the least erosive, or conversely, the most stable of the three domains. Like the Fire Island and Westhampton reaches, it appears that on an overall average basis the Montauk reach has become more erosive in the modern eras compared to the more historic era represented by the 1870 to 1933 time period. The modern evolution (1933 to 1995) of the Montauk reach is generally characterized as being slightly erosional with an average erosion rate of about 0.3 m/year (1.0 ft/year). As noted for the other analysis domains the SRC standard deviation is large indicating considerable alongshore variability in the shoreline rate-of-change.

More recent shoreline change values area given below and in Section 8.1.2.3.

Table 3.10 shows updated shoreline change rates based additional shoreline and beach profile data through 2001 at the design sub-reach level of detail. These updated estimates, which were also adjusted to remove the effects of beach fill, and refined level of detail were used to evaluate life cycle vulnerability.

Lentz, et al., (2013) analyzed three historical data sets (topography derived from 1969 aerial photography and LIDAR data from October 1999 and December 2009) to extract shoreline change data along Fire Island for three time periods: 1969-1999, 1999-2009 and 1969-2009. Shoreline change results, which include the positive (i.e., accretional) effect of beach fill activity show a mean accretional trend between 1969 and 1999 of +2.15 feet/year along Fire Island. The period from 1999 to 2009 is dominated by erosion (-0.62 feet/year) particularly in the eastern reach of the island. Total change results from 1969 to 2009 are more similar to the 1969 to 1999 period (+1 foot/year).

Table 3.10. Shoreline Rate of Change (1979-2001) by Design Subreach

Design Subreach	Shoreline Change Rate (ft./yr.)
Great South Bay	
GSB-D1A	1
GSB-D1B	4
GSB-D2A	4
GSB-D2B	4
GSB-D2C	1
GSB-D2D	1
GSB-D2E	1
GSB-D3A	1
GSB-D3B	1
GSB-D3C	1
GSB-D3D	1
GSB-D3E	1
GSB-D3F	1
GSB-D3G	1
GSB-D3H	1
GSB-D4A	1
GSB-D4B	2
Moriches Bay	
MB-D1A	2
MB-D1B	2

Design Subreach	Shoreline Change Rate (ft./yr.)
Moriches Bay (continued)	
MB-D2A	2
MB-D2B	0
MB-D2C	1
MB-D2D	0
MB-D2E	0
Shinnecock Bay	
SB-D1A	1
SB-D1B	3
SB-D1C	3
SB-D1D	3
SB-D2A	0
SB-D2B	0
SB-D2C	0
SB-D3A	1
SB-D3B	1
SB-D3C	1
Ponds	
P-D1A	1
P-D1B	1
P-D1C	2

Design Subreach	Shoreline Change Rate (ft./yr.)
Ponds (continued)	
P-D1D	2
P-D1E	2
P-D1F	2
P-D1G	4
P-D1H	1
P-D1I	1
P-D1J	1
P-D1K	1
P-D1L	1
Montauk	
M-D1A	1
M-D1B	1
M-D1C	1
M-D1D	1
M-D1E	2
M-D1F	3
M-D1G	3
M-D1H	3
M-D1I	3

3.5 Shoreline Undulations

At least part of the alongshore variability in the observed shoreline rate-of-change owes to undulating shoreline features that are locally referred to as longshore sand waves or erosion waves (Gravens *et al.,* 1999). The presence of these features should be considered in the formulation of a project within Fire Island. Gravens *et al.* (1999) showed that the wavelength of the shoreline undulations generally ranges between 1 and 2 km (0.6 and 1.2 miles). The total root mean square (rms) shoreline undulation height was determined to be about 32 m (104 ft). The landward and seaward rms amplitudes were both quantified at about 16 m (52 ft). Gravens *et al.* (1999) also showed that the shoreline undulations do not appear to propagate from one end of the barrier to the other, although limited alongshore propagation (1 to 2 km or 0.6 to 1.2 miles) of the shoreline undulations is possible. An important finding of the study was that the seaward and landward bulges of the shoreline undulations were preferentially positioned along the shoreline. That is, based on the data sets examined, certain locations along the shoreline can be expected to periodically develop large erosion or accretion cusps but not likely both. This finding indicates that the shoreline undulations may be excited by specific forcing conditions (waves from a particular direction) and their location controlled by irregularities in the offshore bathymetry. In support of the assertion that specific forcing excites the shoreline undulations is the finding from the spatial analysis that the shoreline undulations are intermittent features that are more prominent in some data sets than in others. Nonetheless, the data also suggests that undulations may occur at any location along the project shoreline.

The impact of shoreline undulations on a typical beach fill design configuration was shown to be significant and could lead to greater than anticipated maintenance costs or a reduced level of protection at areas of erosional cusps. Explicit consideration of the presence of shoreline undulations in the development of alternative design configurations and the assessment of baseline and future without project conditions is essential for a successful project.

3.6 Inlets

As presented previously, there are three inlets in the Study Area: Fire Island Inlet, Moriches Inlet, and Shinnecock Inlet, all of which are Federal navigation projects. A fourth inlet has formed at Old Inlet within the Wilderness Area of the Fire Island National Seashore as a result of a breach in the barrier island during Hurricane Sandy. Coastal inlets play an important role in nearshore processes. Inlets are the openings in coastal barriers through which water, sediments, nutrients, planktonic organisms, and pollutants are exchanged between the open sea and the protected embayments behind the barriers. In addition, inlets are important economically because harbors are often located in the back bays, requiring that the inlets be maintained for commercial navigation. At many inlets, the greatest maintenance cost is incurred by periodic dredging of the navigation channel.

Tidal inlets experience diurnal or semidiurnal flow reversals and are characterized by large sand bodies that are deposited and shaped by tidal currents and waves. The ebb shoal is a sand mass that accumulates seaward of the mouth of the inlet. It is formed by ebb tidal currents and is modified by wave action. The flood shoal is an accumulation of sand at the bayward opening of an inlet that is mainly shaped by flood currents (USACE, 2002). However, not all of the sediment in the littoral transport stream is trapped at these shoals; a large proportion may be bypassed by a variety of mechanisms, particularly at inlets that have already developed shoals with a volume approaching equilibrium.

Typically, jetties are built to stabilize a migrating inlet, to protect a navigation channel from waves, or to reduce the amount of dredging required to maintain a specified channel depth. However, jetties can profoundly affect sand bypassing and other processes at inlets and adjacent shorelines (USACE, 2002). The FIMP inlets do not function as natural inlets in several respects. First, the FIMP inlets are stabilized by jetties (only one jetty in the case of Fire Island), are periodically dredged, and do not migrate as natural inlets do. Second, the stabilized FIMP inlets are judged to be more of a sand sink than natural inlets. Natural inlets tend to facilitate bypassing of littoral drift over a series of shallow shoals relatively close to the shore. The jetties act to confine flows within a relatively narrow area compared to natural inlets; they also act to deepen the inlet throat and shift the ebb tidal delta further offshore than a natural inlet. Accordingly, the inlets have acted to trap sand at least during their formative stages. The following paragraphs provide an overview of the most relevant coastal processes at each FIMP area inlet.

3.6.1 Shinnecock Inlet

Shinnecock Inlet was formed in 1938, and has since been stabilized with jetties at its present location and geometry since 1953. The presence and continued evolution of Shinnecock Inlet has strongly influenced adjacent shoreline conditions, particularly west of the inlet. Historic interruption of westerly-directed sediment transport has created a large offset in the shoreline position across the inlet from east to west. Beach material is distributed throughout the inlet and is generally confined to three primary locations: (1) east of the east jetty in a large accretional fillet, (2) ebb-tidal shoal, including updrift and downdrift lobes or bars, (3) flood-tidal shoal. Nevertheless, Shinnecock Inlet has, albeit intermittently, permitted natural bypassing that serves to re-establish littoral transport to the downdrift shoreline. This effect is apparent in the shoreline near Ponquogue where a bulge in the shoreline points to the location where ebb shoal materials are bypassed to shore.

3.6.2 Moriches Inlet

Moriches Inlet is located along the Atlantic Coast in the Town of Brookhaven and connects the Atlantic Ocean with Moriches Bay through the narrow barrier island. Available maps and records indicate that numerous inlets to Moriches Bay have existed during the last several centuries. The present Moriches Inlet was opened during a storm on 4 March 1931, and the existing jetties were constructed in 1954. In 1983, the USACE completed a General Design Memorandum for Moriches Inlet Navigation, which recommended Federal participation in inlet improvements including the following: (1) a 100-foot wide by 6-foot deep inner channel extending from the Intercoastal Waterway to Moriches Inlet, (2) an outer channel extending from the ocean to the inner channel with a width of 200 feet, a low water depth of 10 feet and an advanced maintenance deposition basin. Construction activities were completed by 1986, and since this time the inlet has been maintained as a Federal Navigation Channel.

A notable offset in the shoreline progressing east to west across the Moriches Inlet reflects shoreline impacts associated with the westerly-directed littoral drift. Nonetheless, shoreline conditions immediately west of Moriches Inlet are generally characterized by a relatively robust barrier system with wide beaches and high dunes. Beach widths increase notably approximately 4,000 feet west of inlet, and reflect dredged material placement and natural bypassing of Moriches Inlet. It should also be noted that the historic updrift sediment accumulation (fillet) east of Moriches Inlet appears to be less than at Shinnecock Inlet. This condition is likely to have arisen due to four primary factors, namely: (1) the Westhampton groin field reduces transport reaching Moriches Inlet, (2) historical migration of Moriches Inlet left a narrow barrier segment, (3) tidal currents have scoured the bayside shoreline, (4) a shorter updrift (east) jetty.

3.6.3 Fire Island Inlet

Fire Island Inlet is located at the western end of Fire Island and connects the Atlantic Ocean with Great South Bay. Available records indicate that Fire Island Inlet has existed continuously since the early 1700's. The position of the inlet, however, has varied significantly over time and has migrated a total distance of about 5 miles from a point east of its present position between 1825 and 1940. Federal jetty construction at Democrat Point in 1941, as part of the Fire Island Inlet Navigation Project halted this westward migration. Due to chronic erosion on the western shore, modification of the Federal project was authorized in 1971 to provide for a sand bypassing system at Fire Island Inlet. Since this time, continued dredging of the inlet has been performed to both maintain a navigable channel, and to provide shore protection on the westerly, downdrift beaches and to protect the Ocean Parkway. Dredged material has also been placed in Robert Moses State Park to alleviate chronic erosion.

3.6.4 The Wilderness Area Breach

Hurricane Sandy resulted in three barrier island breaches within the Study Area. One of the breaches within the Wilderness Area of the Fire Island National Seashore was not closed immediately following the storm. After the initial formation of the breach during Hurricane Sandy the breach grew rapidly for several months before breach growth slowed. DOI has been monitoring the Wilderness Area Breach and is preparing an Environmental Impact Statement (Plan/EIS) to determine how best to manage the breach that was created in Fire Island's federally-designated wilderness area. The planning process will include opportunities for public input as well as consultation with federal, state, and local agencies with a regulatory interest or special expertise related to proposed actions.

Observations and modeling results have shown that, at its current size, the breach at the Wilderness Area has not significantly altered tidal elevations in Great South Bay or Moriches Bay. However, the model simulations show that the breach at Wilderness Area will increase storm tide elevations within Great South Bay and Moriches Bay during storm events.

3.7 Bayside Tidal Hydrodynamics

The study area estuarial system, comprised of Great South, Moriches and Shinnecock Bays, are respectively connected to the Atlantic Ocean through Fire Island, Moriches and Shinnecock Inlets. The bays are also connected to each other through narrow tidal waterways of the Long Island Intracoastal Waterway (ICW). A summary of hydrodynamic conditions is presented in the following paragraphs. The description is largely based on previous study references (USACE-NAN, DRAFT, 1998; USACE-NAN, 1999a; USACE-NAN, INTERIM DRAFT, 2002; and USACE-NAN, DRAFT, 2004).

Bay water levels are controlled by tidal elevations at Fire Island, Moriches, and Shinnecock Inlets. The uniformity of tide ranges throughout Great South, Moriches, and Shinnecock Bays is a characteristic of the so-called "pumping mode" of inlet-bay hydraulics where water levels within an embayment remain nearly horizontal during ebb and flood tide phases. Bay tides are generally less than and lag the ocean tides. The difference between ocean and bay tides is particularly significant within eastern Great South Bay. The tidal range at the ocean end of Fire Island Inlet is approximately 4.3 ft. However, the ocean tidal signal is significantly muted along the long inlet throat. Recent monitoring at the Fire Island Coast Guard Station suggests a tidal range of 1.6 ft at this location (i.e., a 50% reduction in approximately 3 miles) compared to bay waters in most of Great South Bay away from the inlet that have an average tidal range on the order of 1 ft, i.e., a 70% reduction. Tidal prism discharge through Fire Island Inlet is the order of 2,300 million cubic feet. The average tidal range in the bay is approximately 1 ft.

The tidal range at the ocean side of Moriches Inlet is approximately 3.6 ft; the range is decreased to 2.5 ft across the inlet in the vicinity of the Coast Guard Station. In areas removed from the inlet, such as Potunk Point and Mastic Beach at the eastern and western limits of Moriches Bay, respectively, the range is decreased to 1.6-2 ft. The estimated average tidal range in Moriches Bay obtained using recent available tidal records is on the order of 2 ft. Tidal prism is estimated as on the order of 1,300 million cubic feet.

The reduction in tidal range within Shinnecock Bay is less pronounced due to the configuration of the inlet and flood shoals. The range goes from approximately 3.3 ft at the ocean side of the inlet to 2.5 ft in the vicinity of Ponquogue Point. The tide range in the bay averages approximately 2.9 ft. The estimated tidal prism is on the order of 1,300 million cubic feet.

At the three inlet-bay systems, maximum current velocities are always at the inlet mouth, where values exceed 4 ft/sec. Peak velocities in the bays away from the inlets are typically less than 1 ft/sec.

Freshwater enters the estuaries primarily through adjoining tributaries and groundwater seepage. Drainage areas for each bay were estimated as: (1) Great South Bay – 378 square miles, (2) Moriches Bay – 75 square miles, and (3) Shinnecock Bay – 25 square miles. Information concerning freshwater sources is relatively sparse. However, the U.S. Geological Survey (USGS) monitors several tributaries at locations far removed from the bays (the available average daily flow rates for major tributaries). Estimates indicate that nearly 25% of all freshwater entering the estuaries can be attributed to groundwater seepage.

4.0 SITE CONDITIONS, INTERIM PROJECTS AND PROBLEM IDENTIFICATION

4.1 Inlet Dredging and Bypassing

4.1.1 Fire Island Inlet

The most recent dredging of Fire Island Inlet was undertaken in August 2013 through March 2014, as borrow material to repair and restore the Fire Island Inlet to Shores Westerly project from erosion due to Hurricane Sandy. Approximately 2,032,000 cy of inlet material was placed at Gilgo, Tobay and Overlook beaches, in both dune and beach areas. Navigation Channel Condition surveys of Fire Island Inlet taken in April 2016 show significant shoaling across the channel, with both spot shoaling and shoaling across the entire channel, resulting in minimum depths of 2.4 feet below Mean Lower Low Water (MLLW)[2] in the left inside quarter of the channel, and shoaling reaching a maximum height of +6.3 feet above MLLW encroaching on the outer right (south) side of the channel.

4.1.2 Moriches Inlet

The most recent dredging of Moriches Inlet was in 2013 as part of the Interim Breach Contingency Plan (BCP) efforts (see below) at Cupsogue Beach. Navigation Channel Condition surveys of Moriches Inlet taken in April 2016 show shoaling across the outer channel width to a depth of approximately 3 feet below MLLW, beginning at the seaward entrance of the channel and continuing until 420 feet off the end of the east jetty, when it tapers to the east, but some shoaling still exists at the seaward end of the east jetty. In the inner channel, channel-wide shoaling begins approximately 415 feet landward of green can 3E and continues to the end.

4.1.3 Shinnecock Inlet

Shinnecock Inlet was formed as a result of a barrier island breach during the “Long Island The most recent dredging of Shinnecock Inlet was undertaken in December 2012 through February 2013, as borrow material to repair the West of Shinnecock Inlet (WOSI) Project (see below) from erosion due to Hurricane Sandy. Navigation Channel Condition surveys of Shinnecock Inlet taken in April 2016 show that the entire channel is deeper than design depths, with the minimum depth being 13.4 feet below MLLW in the middle of the channel.

4.2 Westhampton Interim Project

A plan to provide interim storm risk management to the Westhampton Beach area west of Groin 15 and the affected mainland communities north of Moriches Bay was completed in December 1997. The plan provides for a beach berm 90 feet wide and a dune of +15 ft NGVD[3], tapering of the western two existing groins (groins 14 and 15) and construction of an intermediate groin (groin 14a) between these two. The project also includes periodic

---

[2] Mean Lower Low Water (MLLW) is 1.9 – 2.5 feet lower than North American Vertical Datum of 1988 (NAVD88 or NAVD) at Fire Island Inlet, as determined using vdatum (ver 3.0). Therefore, the shoaling on the right side of the inlet reaches approximately +4 feet NAVD88.

[3] National Geodetic Vertical Datum of 1929 (NGVD29 or NGVD) is approximately 1.06 feet lower than North American Vertical Datum of 1988 (NAVD88 or NAVD) within the FIMP study area. Therefore, the crest elevation the dune is +13.94 feet NAVD88.

nourishment, as necessary to ensure the integrity of the project design, for up to 30 years (2027).

Beachfill for this interim project also includes placement within the existing groin field to fill the groin compartments and encourage sand transport to the areas west of groin 15. The interim plan was determined to be in the Federal interest to provide storm risk management until the findings of the reformulation effort are available.

Initial construction of the project was completed in December 1997. The interim project was subsequently renourished in 2001 (961,000 cubic yards), 2004 (759,000 cubic yards) and 2009 (627,000 cubic yards), requiring less sand at longer intervals than was estimated when designed. Due to severe erosion experienced due to Hurricane Irene in 2011 and Hurricane Sandy in 2012, approval was received from HQUSACE to restore and repair the project to design conditions. A contract was awarded in Sept 2014 and completed in March 2015 with 740,000 cubic yards of sand placed.

4.3 West of Shinnecock Inlet (WOSI) Project

The West of Shinnecock Interim Project study was initiated in 1995 and was approved in May 2002. The recommendations include beach nourishment along the 4000 ft. shoreline immediately west of the inlet, and renourishment every 2 years for a period of 6 years, to provide storm risk management for the area until the completion of the Reformulation Study. The project was constructed in March 2005 with placement of 610,000 cubic yards of sand. The project received limited placement of sand as part of the maintenance dredging of Shinnecock Inlet, but no renourishment during the authorized period of renourishment between 2005 and 2011.

Due to severe erosion experienced due to Hurricane Irene in 2011 and Hurricane Sandy in 2012, approval was received from HQUSACE to repair the beach from the impacts of the two hurricanes. A contract was awarded to place 301,000 cubic yards of sand in the 4000 feet west of the west jetty, from December 2012 to February 2013. All of the 2013 material came from the Shinnecock Inlet authorized navigation channel and deposition basin. A second contract was award to restore the beach to its design condition. Approximately 450,000 cy of material was placed in the WOSI area from February to March 2014. In conjunction with these contracts, a Memorandum of Agreement was executed between the Corps and NYS for placement of an additional 24,000 cubic yards of material at Tiana beach as a betterment. The 2014 material was taken from the Shinnecock Borrow Area, east of the inlet.

In January 2016, Suffolk County placed an additional 70,000 cubic yards of material on the WOSI project area, which was dredged from a Shinnecock Bay interior channel.

4.4 Post-Sandy One-Time Stabilization Efforts

The Corps, State of New York and U.S. Department of Interior have developed a mutually acceptable one-time stabilization plan along the Fire Island barrier island to provide coastal storm damage risk reduction until implementation of the recommendations of the overall Reformulation Study. These stabilization efforts are one-time placement projects and include no nourishment cycles. The efforts are meant to provide coastal storm damage risk reduction until the implementation and construction of final recommendations of the overall Reformulation Study. An interim stabilization project has also been constructed at Downtown Montauk.

4.4.1 Fire Island Inlet to Moriches Inlet Stabilization Project (FIMI)

Following Hurricane Sandy, the beach and dune condition along Fire Island was heavily impacted, and there was the need to take action since the barrier island condition was vulnerable to subsequent storms. In response to this need, the Corps in partnerships with New York State initiated a stabilization project, under P.L. 113-2 to reestablish the beach and dune condition, as a one-time action. The Corps developed a plan that was supported by NYS and DOI that included a beach and dune at elevation +15 ft NGVD29 that is located in the post-Sandy dune alignment and includes the acquisition or relocation of approximately 40 homes. The report and NEPA documents (USACE, 2014a) for this project were approved in July 2014, and a Project Partnership Agreement was executed in August 2014.

Construction was initiated in September 2014 on Contract 1, Smith Point County Park, which placed total of 2,731,000 cubic yards over 24,500 feet of shoreline and was competed in April 2016. Beachfill material for Contract 1 was taken from Great Gunn Beach, just west of Moriches Inlet (519,000 cy) and Borrow Area 4C offshore of Westhampton (2,212,000 cy). Two environmental enhancement features were included in Contract 1. Construction on Contract 2, Robert Moses State Park, Lighthouse Beach (NPS) and the communities of Kismet and Saltaire (13,000 ft.) began in October 2015 and was completed in March 2016. The beachfill quantity of 1,470,000 cy was taken from a portion of Borrow Area 2C, offshore of Fire Island. Contract 3A, Fair Harbor to Seaview will be awarded in July 2016. It is estimated that 1,800,000 cy of material will be placed over 18,400 ft of shoreline, and the borrow area is a portion of Borrow Area 2C. Plans for the remainder of the placement areas under the FIMI study, Contract 3B, Ocean Bay Park to Davis Park are under development. It is expected that 2,500,000c y of material will be placed, also taken from Borrow Area 2C. All beachfill sections will be have annual beach profile surveys and subsequent coastal processes analyses, and annual condition surveys will be taken of all borrow areas utilized for the FIMI project.

4.4.2 Downtown Montauk Stabilization Project

The Downtown Montauk Hurricane Sandy Limited Reevaluation Report and Environmental Assessment were approved in November 2014. A Project Partnership Agreement was executed with the State of New York in March 2015. Contract Award occurred in March 2015, with project completion expected in the summer of 2016. The project will provide coastal storm risk management to over 3200 ft. of shoreline in the eastern Long Island hamlet of Montauk. The project consists of a reinforced dune created with approximately 11,000 geotextile bags, covered with three to six feet of native sand, a fronting beach berm, planting of dune grass, four pedestrian access cross-overs, vehicle cross-overs and two drainage structures.

4.5 Interim Breach Contingency Plan (BCP)

As a result of the experience in the closure of the Little Pikes Inlet, a BCP was prepared and approved in 1996 by Corps Headquarters (HQUSACE) that provides for a rapid response to close breaches along the barrier islands within the authorized project area. This plan provides for a limited response action to restore the barrier island to an elevation of +9 feet NGVD and provides limited risk management (a 20% Annual Chance Exceedance (ACE)) for low-lying areas likely to be overwashed and subsequently breached again during relatively minor events.

The interim Breach Contingency Plan (BCP), that included a process to close breaches within three months and which was approved as an interim action pending the outcome of the Reformulation study, will not continue. It should be noted that a Breach Response Plan is among the possible alternatives in the Reformulation Study.

The Interim BCP was enacted in two locations in the wake of breaches developing at Cupsogue Beach County Park and Smith Point County Park due to impacts of Hurricane Sandy. At Cupsogue Beach County Park, 262,000 cy of material was taken from Moriches Inlet as borrow sources and the breach was closed in November 2012. At Smith Point County Park, approximately 60,000 cy of material was taken from the Long Island Intracoastal Waterway as borrow source, and the breach was closed in December 2012.

4.6 Without Project Conditions

4.6.1 Pre-Sandy Baseline Conditions

Prior to Hurricane Sandy and the breach at the Wilderness Area, the baseline conditions were defined by three inlets and the barrier island topography captured by the September 2000 LIDAR. Dune height, berm, and barrier island width vary along the barrier island system. The 2000 LIDAR indicated lowest dune heights at Old Inlet, where the dune is about 8.5 ft NGVD29 and at Smith Point County Park, where the dune is about 10 ft NGVD29. Vulnerable areas in eastern and central Fire Island were characterized by dune heights around 11 to 12 ft NGVD29 and 15 ft NGVD29, respectively. Vulnerable areas along Shinnecock Bay were characterized by dune heights ranging from 11 to 13 ft NGVD29.

4.6.2 Baseline Conditions (BLC)

The BLC conditions reflect the presence of the Wilderness Area Breach formed during Hurricane Sandy. The barrier island topography is based on the conditions captured by the 2000 LIDAR. The 2000 LIDAR captured a relatively healthy dune and berm along many much of the barrier island. These conditions are representative of today's existing conditions, which have been improved by Post-Sandy beach fill projects.

4.6.3 Future Vulnerable Conditions (FVC)

The Future Vulnerable Conditions (FVC) barrier island topography represents a topography that is more vulnerable than the BLC. However, the FVC represents a topography that is reasonably expected to occur at some point during a 50-year project life, taking into account historic trends and current engineering activities. In the vulnerable locations, dune height, berm width, and barrier island width are smaller than that under BLC. In most vulnerable areas, the FVC dune height lies between the BLC and Breach Closed Condition (BCC) topographies. However, in the vicinity of Old Inlet (Old Inlet West), the FVC dune height is about 8 ft NGVD29, lower than the BLC and BCC.

4.6.4 Breach Closed Conditions (BCC)

The Breach Closed Conditions (BCC) barrier island topography is defined as the minimum breach closure section under consideration for the FIMP study. This breach closure section is defined by a 9.5 ft NGVD29 dune height and a barrier island width that matches the pre-breach condition. Here, the pre-breach barrier island width is taken as that on the BLC.

4.6.5 Breach Open Conditions (BOC)

4.6.5.1 Pre-Sandy

The BOC conditions represented a range of possible breach open conditions for each of the three bays. Prior to Hurricane Sandy, a total of 12 different BOC scenarios were considered representing 4 different breach location combinations and 3 different breach sizes (3, 6, and 12 month from breach formation).

A total of 6 breach locations were considered to be representative of the range of possible breach open conditions for each of the three bays in the FIMP area, Table 4-1. These 6 locations were arranged into 4 different combinations that would be modeled (Table 4-2). These combinations were selected to cover the range of possible breach conditions under the following assumptions:

1. Two neighboring breaches cannot coexist in Great South Bay; it is assumed that one of them will remain open and the other one will close.

2. Only one open breach can be supported at Shinnecock Bay; therefore, the combination of a breach open at Tiana Beach and West of Shinnecock simultaneously was not considered.

Table 4-1. Represenative Breach Locations.

Breach Location	Area of Direct Influence
Kismet to Corneille States	Western GSB
Talisman to Blue Pt. Beach	Central GSB
Old Inlet	Eastern GSB
Smith Point CP – East	Eastern Moriches Bay
Tiana Beach	Western Shinnecock Bay
West Shinnecock	Shinnecock Bay

Historical evidence, hydrodynamic modeling, and inlet/breach stability analyses support the assumption that two breaches cannot coexist within the same reach in addition to the existing Fire Island Inlet; the tidal prism of one breach would become dominant, and the other breach would naturally close.

Table 4-2. Breach Open Conditions for Numerical Simulation (Pre-Sandy).

Breach Open Scenario	Western GSB	Central GSB	Eastern GSB	Eastern Moriches Bay	Western Shinnecock Bay	Shinnecock Bay
BOC-1						
BOC-2						
BOC-3						
BOC-4						

4.6.5.2 Post-Sandy

This section describes the approach used to redefine the stage frequency curves for the set of BOC with the Old Inlet Breach. The important differences between the pre-Sandy approach and the post-Sandy approach is describe below.

In the BLC the Old Inlet Breach (Eastern GSB) is assumed to remain open. Therefore, the BOC-1 scenario in GSB and Moriches Bay, is now essentially the same as the baseline condition. Since BOC-2 must now be combined with the breach at Old Inlet it becomes equivalent to BOC-4.

BOC-3, breach in Central GSB, must be combined with the new breach at Old Inlet. No model simulations have ever been performed to estimate bay water levels with simultaneous breach open conditions at Central and Eastern GSB. In the past it was assumed that GSB could not support and maintain two stable inlets at Central and Eastern GSB simultaneously, and that one of them would tend to naturally close. In the absence of any suitable modeling scenarios to define the bay water levels for BOC-3, the water levels will be taken as the maximum of the original BOC-3 and new BLC.

The top half of Table 6 shows the revised 2015 BOC scenario matrix. The bottom half of the table shows additional BOC used in the life-cycle simulations following the same approach used in 2006. It is noted that the bay system of Great South Bay-Moriches Bay is considered independent of Shinnecock Bay. The right half of the table shows the stage frequency curves to be used for the additional BOC-5, BOC-6, BOC-7/BOC-8 scenarios which better approximate the expected values under those breach open conditions.

Table 4-3. Post-Sandy Breach Open Conditions

Breach Open Scenario	WGS B	CGS B	EGS B	EMB	1-2-3-4-17-20-42	5-6-7-21-22	8-24-25	10-11-12-13-26-27-29-30-43-44
BOC-1 / BLC					BLC	BLC	BLC	BLC
BOC-2 / BOC-4					BOC-4	BOC-4	BOC-4	BOC-4
BOC-3					Max (BLC, BOC-3)	Max (BLC, BOC-3)	Max (BLC, BOC-3)	Max (BLC, BOC-3)
BOC-5					BOC-3	BOC-3	Max(BOC-3, BOC-4)	BOC-4
BOC-6					BOC-4	BOC-4	BLC	BLC
BOC-7 / BOC-8					BLC	BLC	BOC-4	BOC-4

In addition, the 3, 6, and 12 month breach sizes in Great South Bay were modified based on cross sectional area measurements following the breach at the Wilderness Area. The measurements from C. Flagg (No. 9) include data thru May 30, 2013 and show a fairly stable cross section since the end of February 2013 of approximately 4,300 ft^2. In the previous BCP analysis for Great South Bay, a maximum breach cross section of 36,200 ft^2 was assumed. In order to reflect the recent observations at the Wilderness Area Breach an additional cost estimate was developed at all Great South Bay breach locations for a smaller breach with a maximum breach cross sectional area, A0, of 6,500 ft^2.

5.0 BORROW SOURCE INVESTIGATIONS

Complete detail regarding the Borrow Areas is shown in the Borrow Source Appendix. However, a short synopsis is presented in the following paragraph.

Suitability between native beach sediments and borrow sediments was evaluated using the 1984 Shore Protection Manual Overfill Method. Fourteen borrow areas were delineated surrounding the suitable cores (Borrow Areas 1A, 2A, 2B, 2C, 2D, 2F, 2G, 2H, 3A, 3B, 4C, 5A, 5B and 5B Expanded). The proposed usage of these borrow areas was modified to minimize adverse impact to potential onshore sediment transport processes proposed by data collection efforts of the USGS. Deeper borrow areas were proposed to be used first, along with pre and post-dredging monitoring and adaptive management. Towards this, Borrow Areas 1A, 2A, 2B, 2D, 2F, 2G, 2H, 3A and 3B were deferred until renourishment. And Borrow Area 2C is being used for initial construction in fill areas between Fire Island Inlet and Davis Park, and Borrow Area 4C has been used for initial construction in fill areas between Smith Point County Park and Moriches Inlet.

6.0 COASTAL PROCESSES INVESTIGATIONS

FIMP consists of several smaller projects. However, when examined from a scientific and engineering aspect, it is a very large interconnected system. A project initiated in one area may have positive, neutral or detrimental effects on another portion of study area. To account for this, an approach was adopted that included hydrodynamic modeling, beach erosion modeling, breach analysis, a sediment budget and inlet morphological analysis. The results are summarized in the subsequent sections.

6.1 Storm Surge and Storm-Induced Barrier Island Breaching Modeling

For input to stage-frequency development, storm-surge numerical modeling was performed to produce peak storm water levels at 49 locations throughout the study area. These 49 locations were selected to capture the variability in storm water levels along the open coast and within the three bays. Model output was also saved at 31 additional locations within the region, but outside the study area, to support other New York District projects. Table 6-1 and Figure 6-1 through Figure 6-4 give these 80 output locations. Stations within the three bays influenced by storm-induced barrier island overwash and breaching are marked in red.

Table 6-1. Storm water level output locations

Station Number	Latitude (deg)	Longitude (deg)	Location Description
1	-73.4288736260	40.6550903730	Unqua Point
2	-73.4614488710	40.6317107990	South Oyster Bay
3	-73.2269947500	40.7144234420	Great Cove
4	-73.1581093300	40.6525715130	Ocean Beach
5	-73.1239263700	40.7222901470	Connetquot River
6	-72.9890471930	40.6963460930	Watch Hill
7	-73.0111938060	40.7452313470	Patchogue
8	-72.8909393410	40.7570645950	Long/Sandy Point
9	-72.8946815280	40.7176628460	Old Inlet (ocean)
10	-72.8424683520	40.7500541100	Mastic Beach
11	-72.7477013760	40.7989795920	Hart Cove
12	-72.7267709190	40.8070857950	Seatuck Cove
13	-72.6696602340	40.8009574760	Apacuck Point
14	-72.5827236370	40.8185131460	Quogue Canal
15	-72.5367418400	40.8579335080	Tiana Bay
16	-72.4921723890	40.8797241020	Cormorant Point
17	-73.3121000000	40.6816000000	Sampawams Point
18	-73.3100000000	40.6300000000	Fire Island Mouth
19	-73.2700000000	40.6300000000	Fire Island Bridge
20	-73.1868000000	40.6986000000	Heckshire State Park
21	-73.0736000000	40.7162000000	Brown Point
22	-73.0728000000	40.6754000000	Great South Beach (bay)
23	-73.0707000000	40.6564000000	Great South Beach (ocean)
24	-72.9477000000	40.7313000000	Narrow Bay
25	-72.8849000000	40.7383000000	Smith Point
26	-72.8040000000	40.7778000000	Masury Point
27	-72.7533000000	40.7699000000	Moriches Inlet (bay)
28	-72.7556000000	40.7620000000	Moriches Inlet (ocean)
29	-72.7484000000	40.7846000000	Moriches CGS
30	-72.7000000000	40.7950000000	Westhampton Beach
31	-72.5900000000	40.8000000000	Post Lane
32	-72.5553000000	40.8392000000	Pine Neck Point
33	-72.5200000000	40.8500000000	Shinnecock CGS
34	-72.5000000000	40.8420000000	Shinnecock Bridge

Station Number	Latitude (deg)	Longitude (deg)	Location Description
35	-72.4770000000	40.8355000000	Shinnecock Inlet (ocean)
36	-72.4789000000	40.8479000000	Shinnecock Inlet (bay)
37	-72.4423085900	40.8707413300	Shinnecock Indian Reservation
38	-72.2069981610	40.9279511080	Apaquogue (ocean)
39	-71.9135552390	41.0334139410	Ditch Plains (ocean)
40	-71.9342158830	41.0741126610	Montauk Harbor
41	-73.1905700000	40.6295300000	Great South Beach (ocean)
42	-73.3581600000	40.6561300000	Great South Bay
43	-72.8045800000	40.7705000000	Moriches Bay
44	-72.6851100000	40.7893200000	Moriches Bay (Gunning Point)
45	-72.5342900000	40.8292500000	Shinnecock Bay (opposite Tiana Beach)
46	-74.0136700000	40.5735800000	Coney Island Lighthouse
47	-73.9469200000	40.5731200000	Manhattan Beach Park
48	-73.8850900000	40.6176400000	Island Channel (Jamaica Bay)
49	-73.8358300000	40.6428100000	Channel Bridge (Jamaica Bay)
50	-73.7964500000	40.6324100000	Grassy Bay (JFK airport)
51	-73.8849200000	40.5735300000	Marine Parkway Bridge
52	-73.8359600000	40.5735500000	Rockaway Park (ocean)
53	-73.7567300000	40.5879000000	East Rockaway Inlet (ocean)
54	-73.5802400000	40.5783200000	Jones Inlet (ocean)
55	-73.6673900000	40.5794900000	Long Beach (ocean)
56	-73.5685000000	40.5926800000	Jones Inlet (bay)
57	-73.4565400000	40.5986400000	Tobay Beach (ocean)
58	-74.0084400000	40.5836300000	Gravesend Bay Entrance (Rockaway Point)
59	-73.9230500000	40.5470900000	Rockaway Beach (ocean)
60	-73.7861300000	40.6067700000	Grass Hassock Channel
61	-73.8199000000	40.5926300000	Cross Bay Bridge
62	-73.6724700000	40.5946200000	Reynolds Channel (Long Beach)
63	-72.3429900000	40.8817900000	Watermill Beach (ocean)
64	-72.0528200000	40.9868200000	Napeaque Beach (ocean)
65	-71.8483500000	41.0745500000	Montauk Point (ocean)
66	-73.2994000000	40.6167000000	Fire Island - Democrat Point (ocean)
67	-74.0176460000	40.4628330000	Sandy Hook, NJ (also NOAA)
68	-74.0214290000	40.6991740000	The Battery, NY (also NOAA)
405	-73.8046779576	40.5978451856	Jamaica Bay (kad 1)
407	-73.7741085619	40.6151637767	Jamaica Bay (kad 3)
426	-73.8783340000	40.6310540000	Jamaica Bay (kad 6a and 7a)
429	-73.8705460000	40.6323550000	Jamaica Bay (kad 6b and 7b)
435	-73.8845624808	40.6283545569	Jamaica Bay (kad 8)
436	-73.8904190000	40.6165130000	Jamaica Bay (kad 9a and 10a)
442	-73.8910250000	40.6222360000	Jamaica Bay (kad 9a and 10a)
446	-73.9068350000	40.5843110000	Jamaica Bay (kad 11)
452	-73.9036478695	40.5866923940	Jamaica Bay (kad 12)
453	-74.0826343800	40.5027055600	Sandy Hook/Raritan Bays (cr 1)
454	-74.1679275400	40.4742975100	Sandy Hook/Raritan Bays (cr 2)
520	-74.2708333300	40.4900000000	Raritan Bay (Stu)

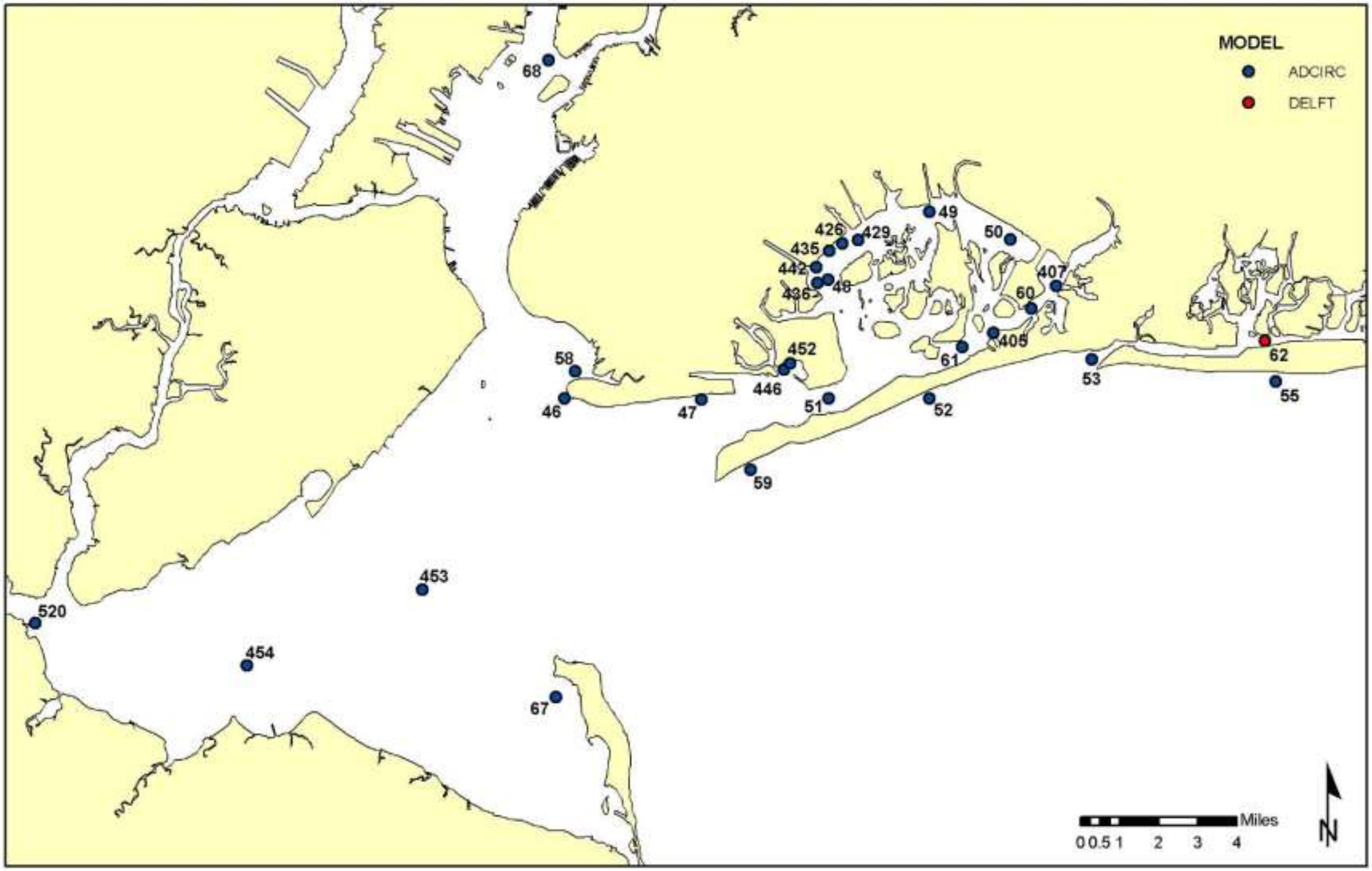


Figure 6-1. Storm water level output locations

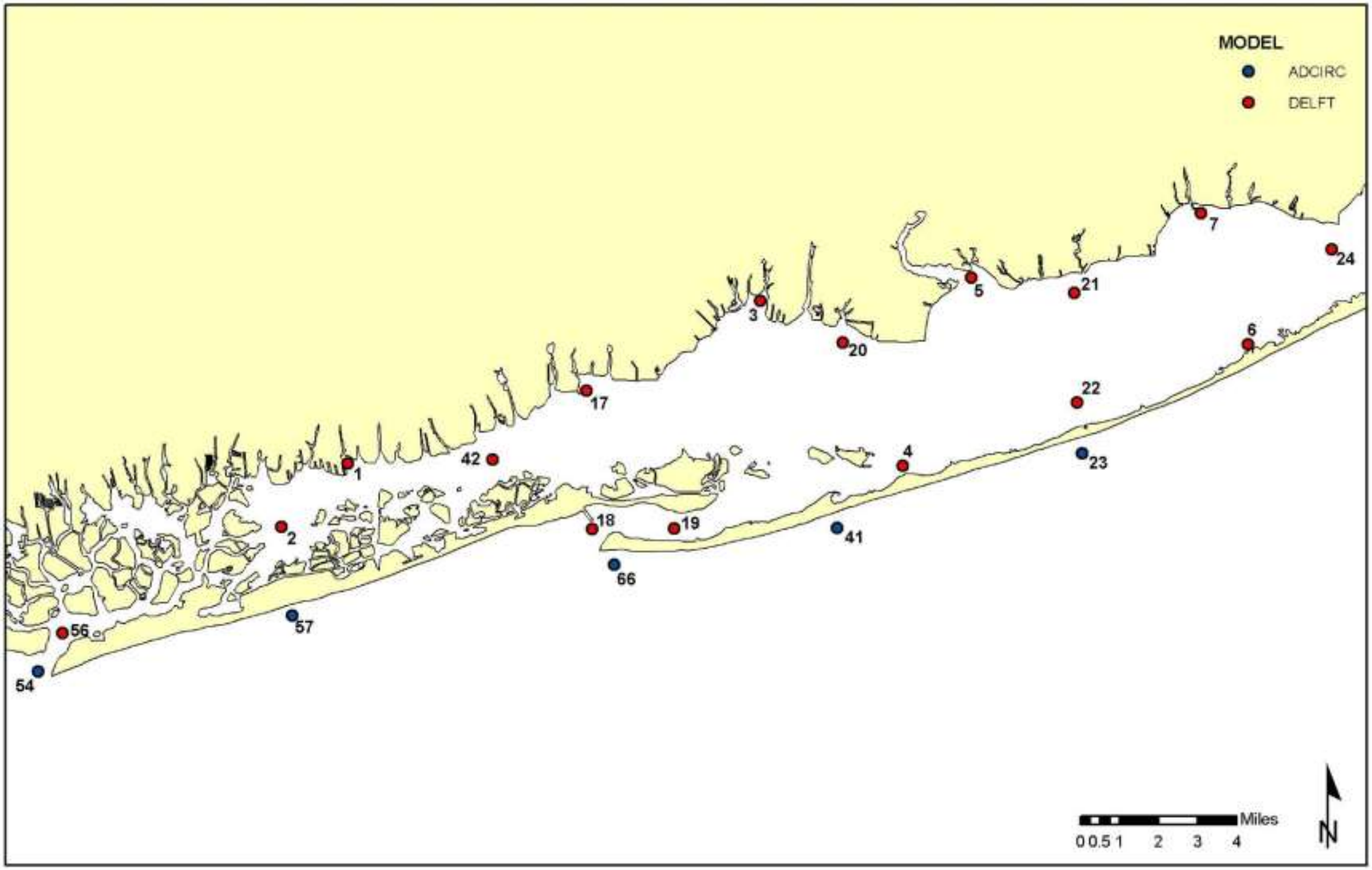


Figure 6-2. Storm water level output locations (continued)

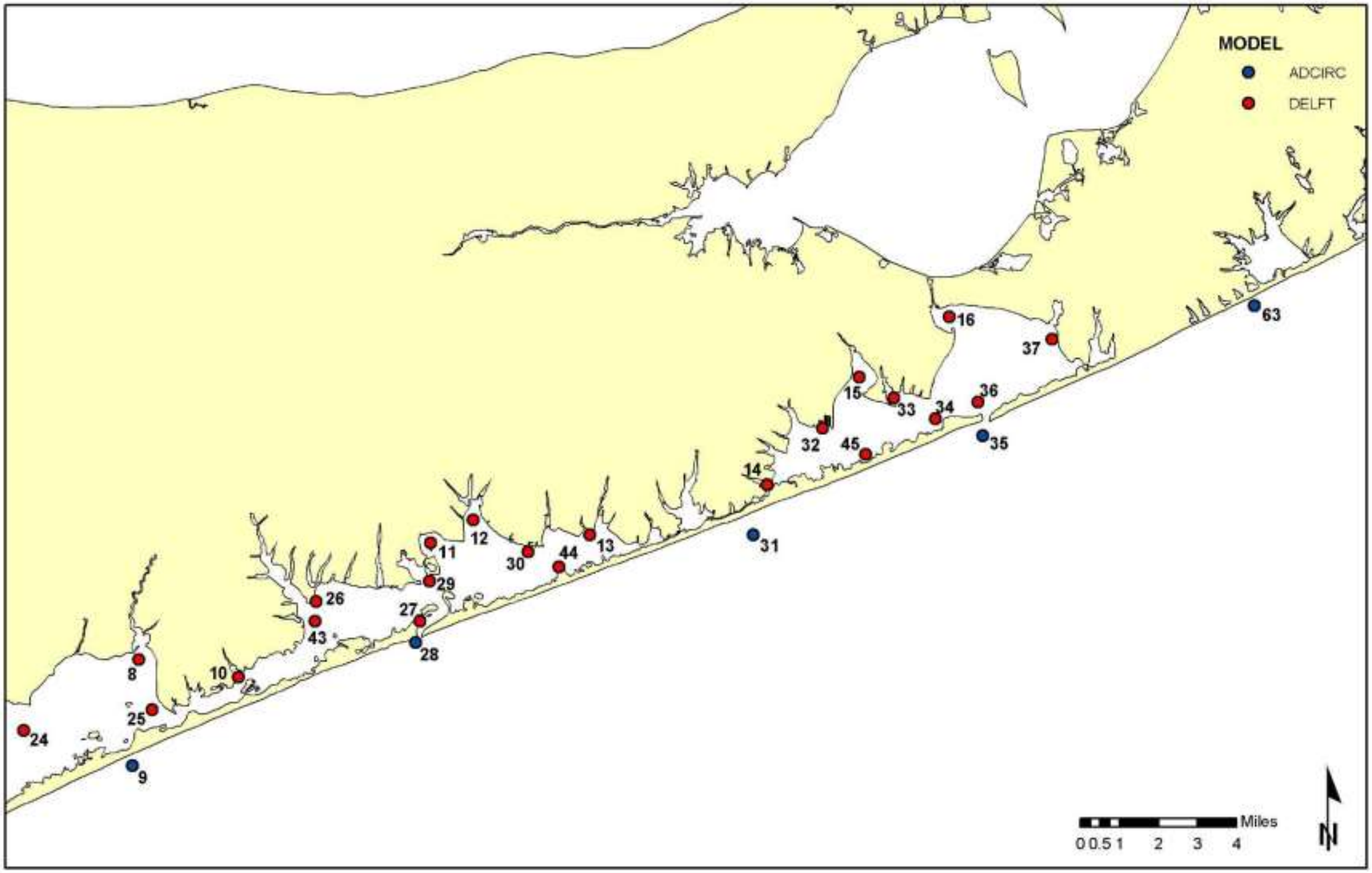


Figure 6-3. Storm water level output locations (continued)

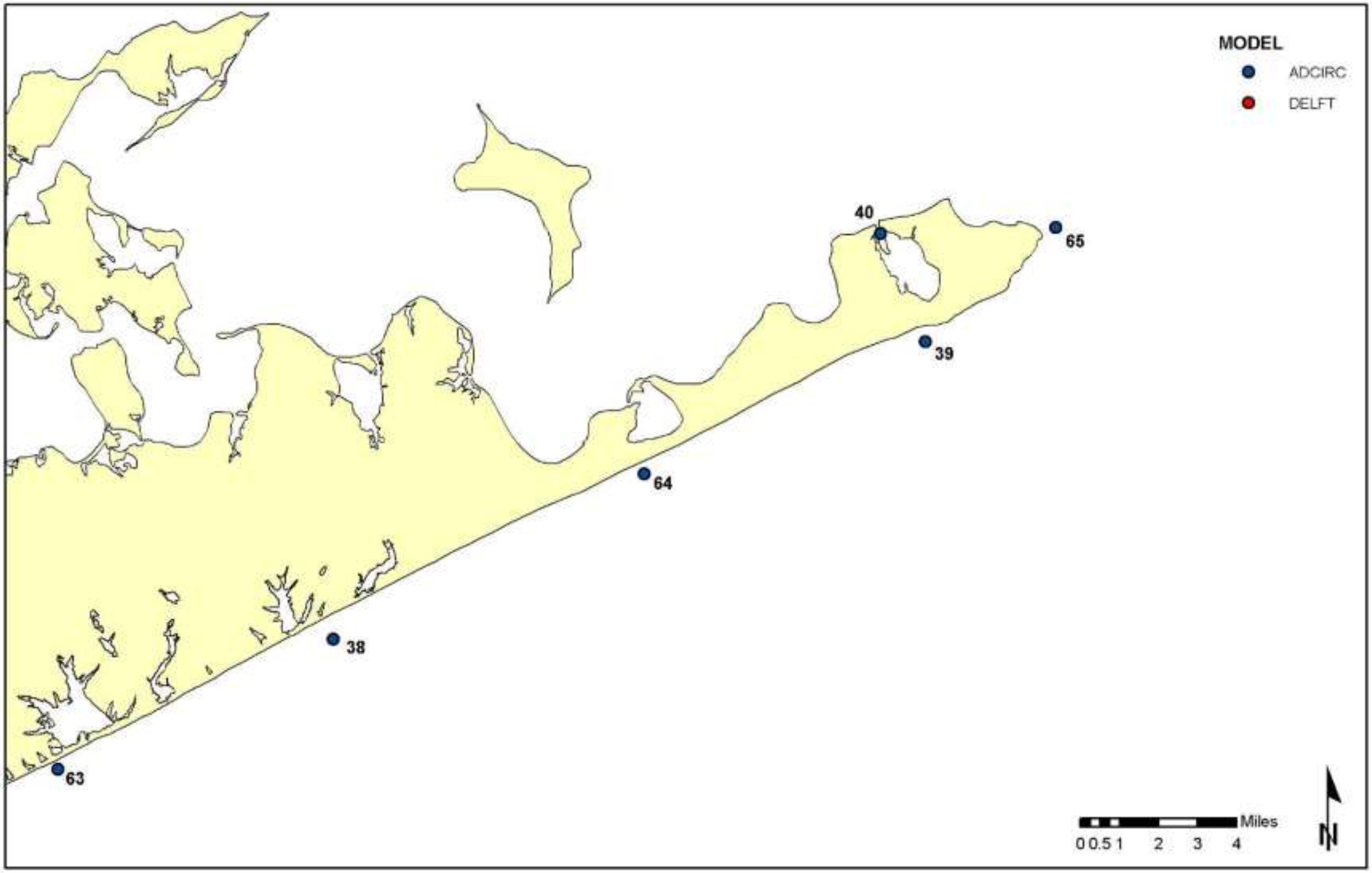


Figure 6-4. Storm water level output locations (continued)

The storm-surge numerical modeling strategy for FIMP addresses a comprehensive list of physical processes (wind conditions, barometric pressure, astronomic tide, wave conditions, morphologic response, namely barrier island overwash and breaching, and localized wind and wave setup) by merging hydrodynamic, wave, and sediment transport models (Figure 6-5).

6.1.1 Numerical Models

The modeling method consisted of four numerical models (Figure 6-6). Of the four models presented below, two models are preferred for use by the HH&C Community of Practice (CoP) (ADCIRC and SBEACH), and one model is allowed for use by the HH&C CoP (WISWAVE) (see the HH&C CoP Sharepoint site for model software list and Enterprise Standard (ES -08101) Software Validation for the HH&C CoP). At the time of the original modeling study, the DEFLT 3D Modeling Suite was the leading modeling package available to allow the simulation of cross-island topographic changes which contribute to barrier island variations, overwash and breaching potential. The complete storm modeling suite architecture was approved by the Coastal and Hydraulic Laboratory, and further reviewed and accepted by the Technical Review Panel.

6.1.1.1 WISWAVE

WISWAVE (also WAVAD) was applied to determine extreme storm wave conditions. Model theory, assumptions, and application for this study are summarized in Chapter 3.1.10 and high resolution grid domain, relative to project location, is illustrated in Figure 6-7. WISWAVE output was used as input forcing for the DELFT3D modeling suite and for SBEACH.

6.1.1.2 ADCIRC

ADCIRC was used to simulate the ocean and nearshore, outside the surf zone, storm water levels (Luettich et al., 1992). ADCIRC is a long-wave hydrodynamic finite-element model that simulates water surface elevations and currents from astronomic tides, wind, and barometric pressure by solving the two-dimensional, depth-integrated momentum and continuity equations.

Grid resolution varies from very coarse at the open ocean boundaries to 50-m in some nearshore locations (Figure 6-8). ADCIRC was forced with the hindcasted storm wind and barometric pressure fields discussed in Chapter 3.1.4 to capture meteorological effects on water levels. ADCIRC was also forced with astronomic tidal constituents from the ADCIRC East Coast 2001 Tidal Constituent Database for seven main tidal constituents (Mukai et al., 2002). Water level time series were output, at 6-minute intervals, at 20-m depths offshore of the study area. These time series were used to as input forcing for the DELFT3D modeling suite and for SBEACH.

6.1.1.3 SBEACH

SBEACH was used to estimate pre-inundation dune lowering which is likely to occur early in the storm because of wave-induced overtopping. SBEACH (Larson and Kraus 1989a; Larson, Kraus, and Byrnes 1990) is a numerical model for predicting beach, berm, and dune erosion due to storm waves and water levels. A basic assumption of the SBEACH model is that profile change is produced solely by cross-shore processes, resulting in a redistribution of sediment across the profile with no net gain or loss of material. Longshore transport processes are assumed to be uniform and therefore can be neglected in the calculation of beach profile change. These assumptions are expected to be valid for short-term storm-induced profile responses on open coasts sufficiently removed from the influence of tidal inlets and coastal structures. These assumptions are valid for this project. SBEACH was initially formulated using data from prototype-scale laboratory experiments and further developed and verified based on field measurements and sensitivity testing from four sites (CHL's Field Research Facility (FRF) at Duck, North Carolina; Manasquan and Point Pleasant Beach, New Jersey; and Torrey Pines, California).

SBEACH is an empirically-based model of beach profile change developed to replicate dynamics of dune and berm erosion using standard data (topography, beach profiles, etc.) available in most engineering applications. In model simulations, the beach profile progresses to an equilibrium state as a function of the initial profile condition (including median grain size and shoreward boundary conditions) and storm conditions (wave height, period, and direction; wind speed and direction; and water level). The model predicts profile response to storms including wave overtopping and dune lowering (Kraus and Wise 1993, Wise and Kraus 1993). Model improvements including the implementation of a random wave model for wave transformation and sediment transport and the dune overwash algorithm are documented in SBEACH Report 4 (Wise, Smith, and Larson 1996) together with extensive model validation with data collected in both the laboratory and the field.

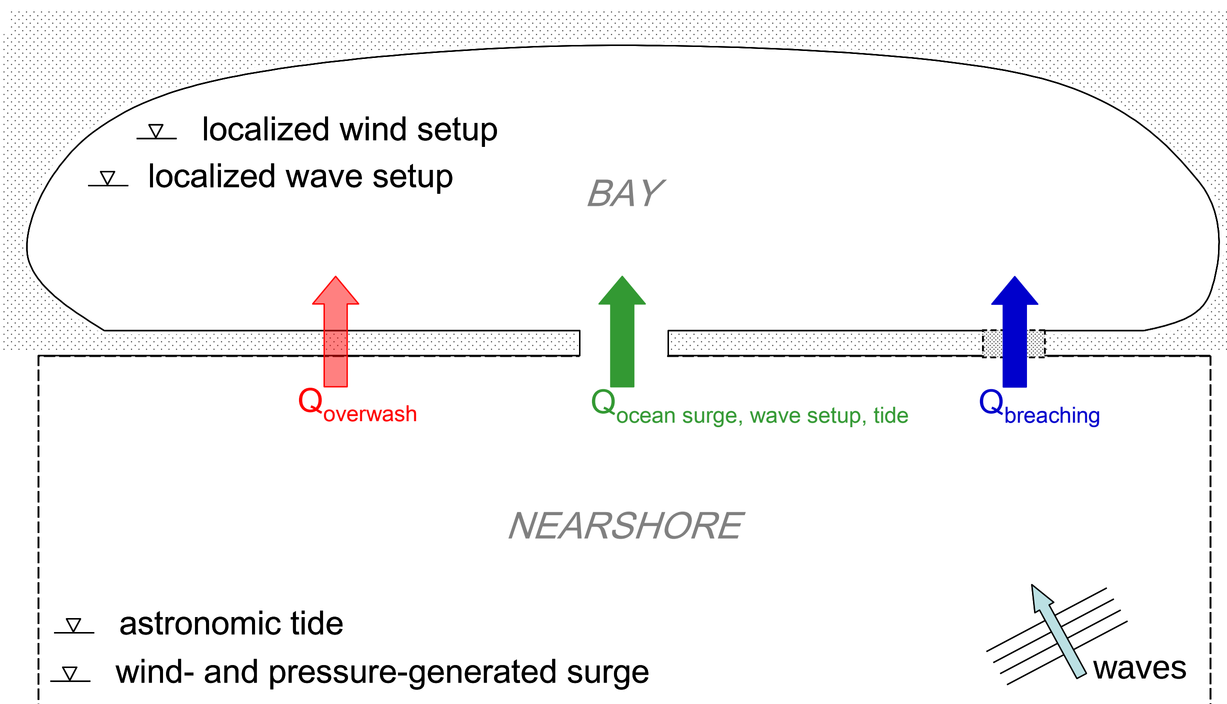


Figure 6-5. Contributions to storm water level

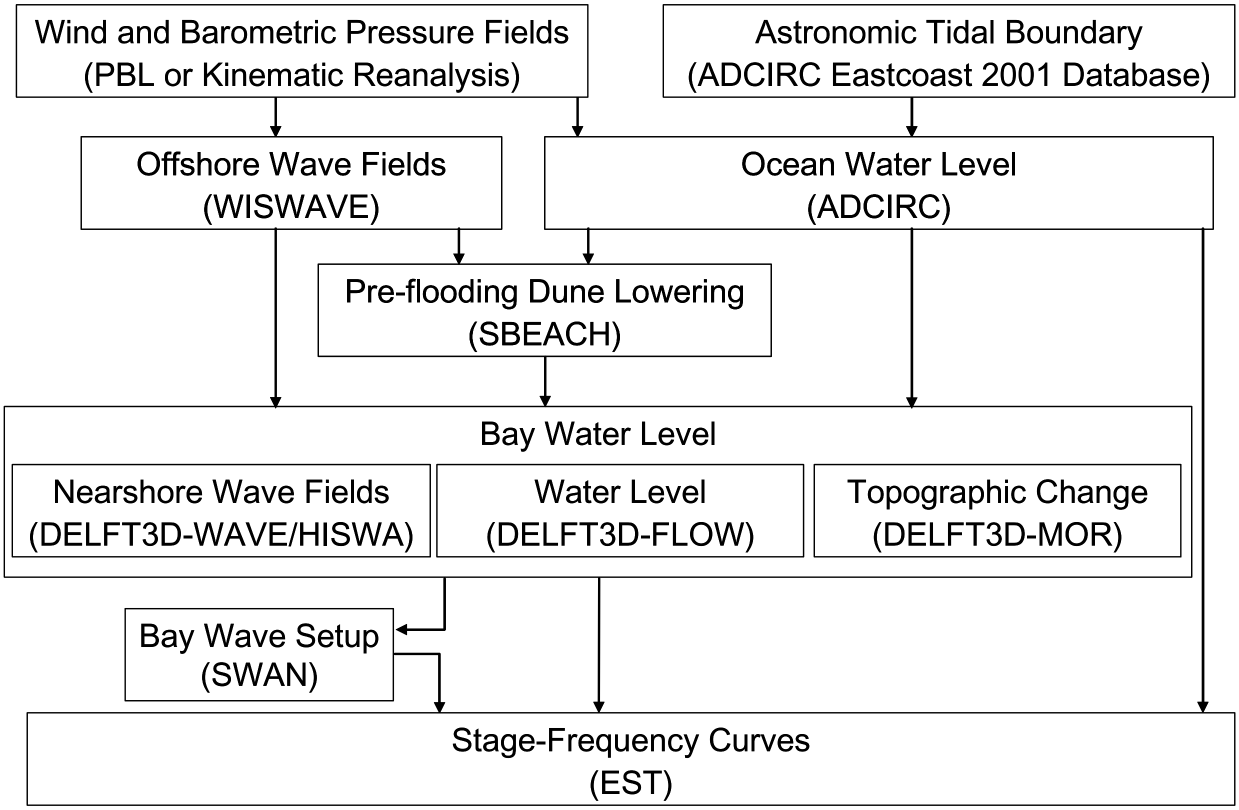


Figure 6-6. FIMP storm water level modeling and stage-frequency methodology

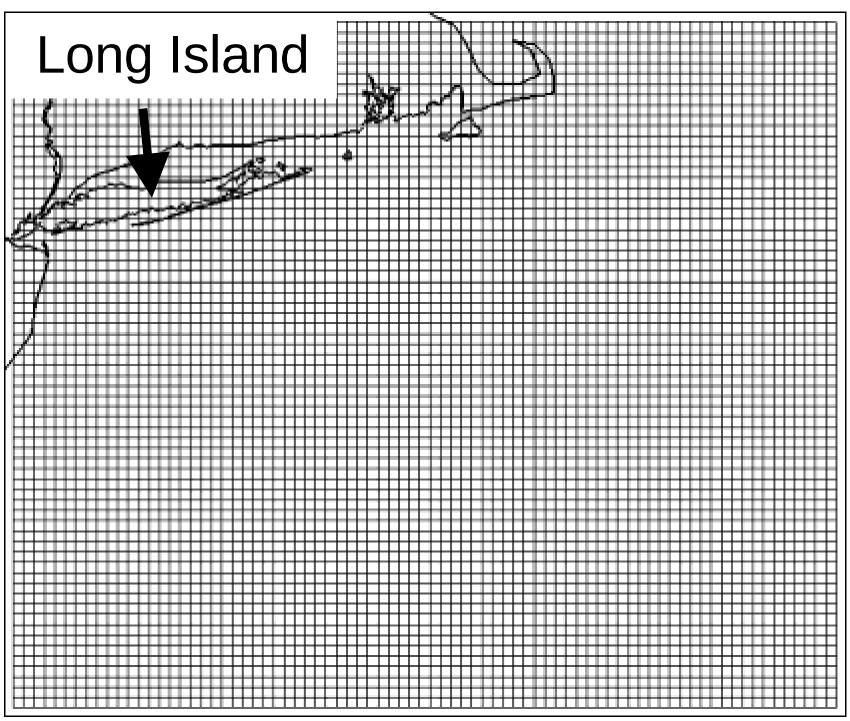


Figure 6-7. WISWAVE 0.083° fine grid

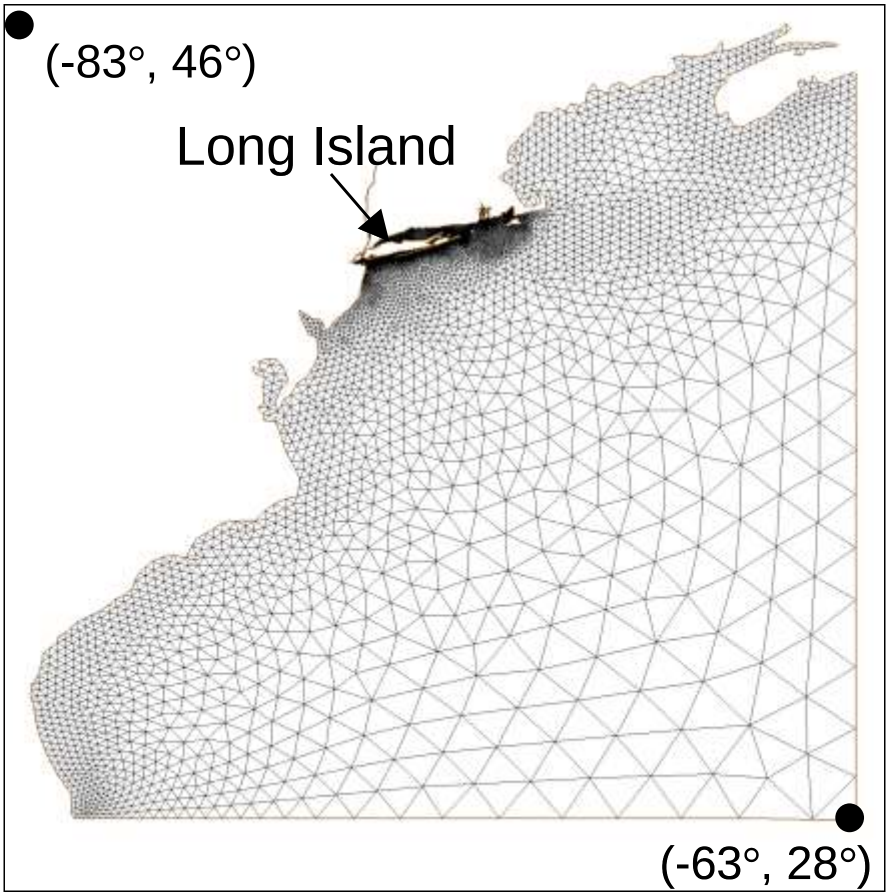


Figure 6-8. ADCIRC grid

For storm surge modeling, SBEACH storm simulations were performed for more than 200 beach profiles cut from the 2000 LIDAR topography. Dune crest elevation change just prior to inundation was extracted from the SBEACH simulation results to pre-condition the DELFT3D topography grid to improve estimates of potential breaching and overwash processes.

6.1.1.4 DELFT3D Modeling Suite

The DELFT3D modeling suite (FLOW, WAVE, MOR) was used to compute the bay water levels under storm conditions, taking into account the contribution of storm surge, waves, winds and the contribution of overwash and/or breaching (Figure 6-5).

DELFT3D-FLOW simulates water level and currents from tidal, meteorological, and wave forcing by solving either the two-dimensional depth-integrated or three-dimensional flow and transport phenomena. The two-dimensional mode was adopted for this study.

The DELFT3D-FLOW orthogonal curvilinear grid for this study extends from East Rockaway Inlet eastward to the east side of Shinnecock Bay (Figure 6-9). The model grid includes Great South, Moriches, and Shinnecock Bays, and their inlets, and extends up to 5 km from across the nearshore. The model grid has variable resolution throughout the domain. The cross-shore resolution varies from values of 15-20 m at the barrier island and the intertidal zone, to around 350 m at the offshore boundary. The typical model's longshore resolution is around 200-300 m. At Moriches and Shinnecock inlets the grid size is in the order of 30 m. Grid resolution is on the order of 75 m at Fire Island inlet. To simulate storm water levels, DELFT3D-FLOW was forced along its offshore boundary with water level time series from ADCIRC, throughout its domain with the storm wind and pressure fields, and with wave radiation stress fields simulated with HISWA

The stationary wave model HISWA (DELFT3D-WAVE) was used to compute nearshore wave climate and resulting surf-zone radiation stresses (Holthuijsen et al., 1989). HISWA is a second generation wave model that computes wave propagation; wave generation by wind; non-linear wave-wave interactions and dissipation for a given bottom topography; and stationary wind, water level, and current field in waters of deep, intermediate and finite depth. The model accounts for the following physics: wave refraction over a bottom of variable depth and/or spatially varying ambient current; depth and current induced shoaling; wave generation by wind; dissipation by depth-induced breaking and/or bottom friction; and wave blocking by strong counter currents. HISWA is based on the action balance equation and wave propagation is based on linear wave theory (including the effect of currents).

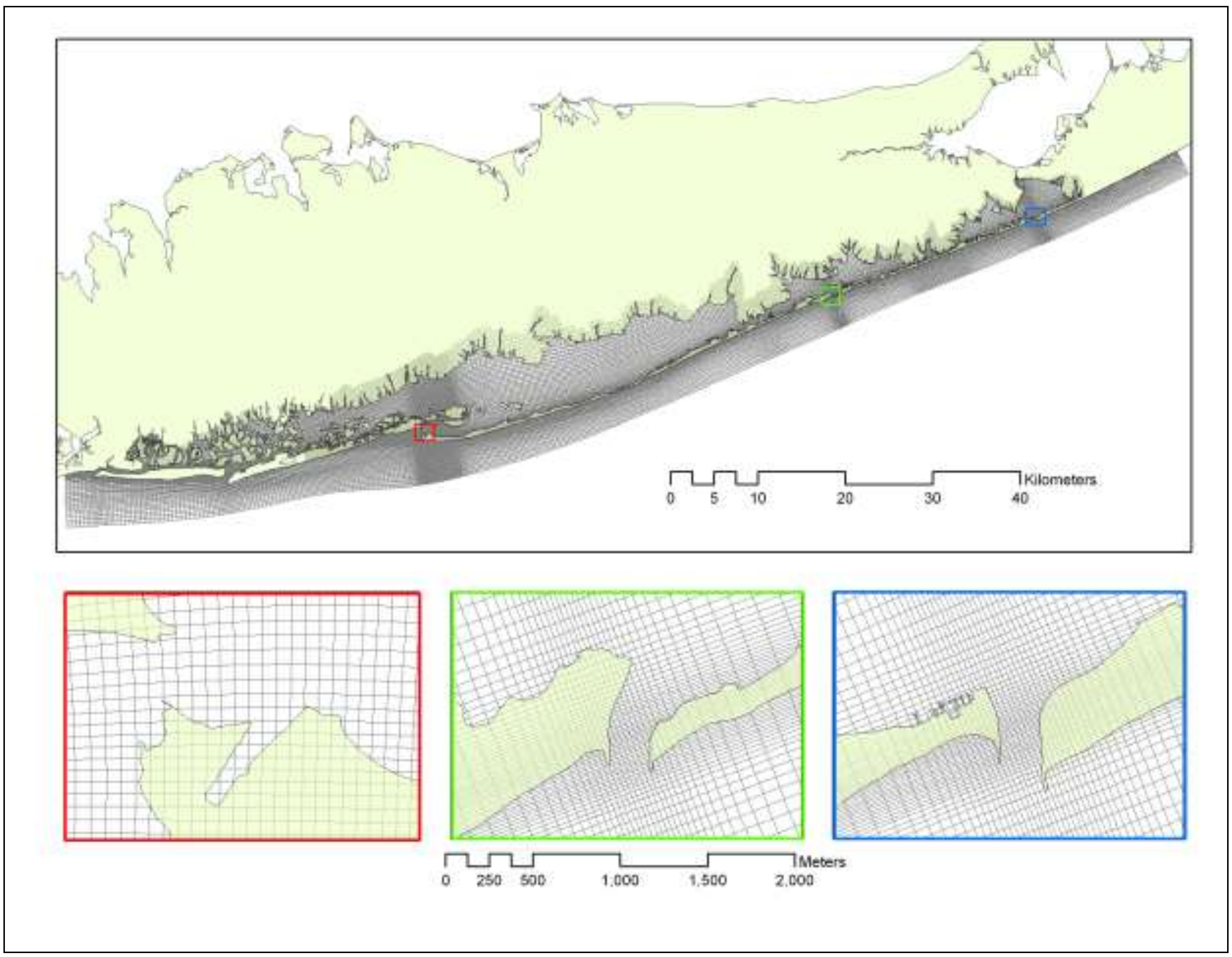


Figure 6-9. DELFT3D-FLOW grid (1 m = 3.28 ft)

HISWA wave computations are carried out on a rectangular grid (Figure 6-10). A nested grid approach was also used for nearshore wave modeling and spans from East Rockaway Inlet to Montauk Point. The offshore grid, with 250 m alongshore by 50 m across-shore resolution, was forced on its offshore boundary with significant wave height, peak period, and mean wave direction. These inputs were computed from the bulk spectra from WISWAVE simulations.

Non-stationary conditions (i.e. those conditions that change with time) may be simulated with HISWA as quasi-stationary with repeated model runs. For this study, HISWA simulated wave conditions for each hourly input condition from WISWAVE. The HISWA model has a dynamic interaction with DELFT3D-FLOW (i.e. two way wave-current interaction). By this, the effect of waves on current and the effect of flow on waves, including wave setup, are accounted for. The resulting radiation stresses obtained from the HISWA local rectangular grids are automatically transferred to DELFT3D-FLOW, which simulates the flow on a curvilinear grid. This process allows direct simulation of the impacts of wave setup on hydrodynamics, specifically water level at the coastline and in the estuarial bays.

Morphological change, namely barrier island overwash and breaching, were simulated using DELFT3D-MOR. Three-dimensional transport of suspended sediment is calculated in DELFT3D by solving the three-dimensional advection-diffusion (mass-balance) equation for the suspended sediment, including both bed load and suspended load. Based on available data for this barrier island system, it was assumed that all sediment was non-cohesive (sand) and with a constant median grain size (D50) throughout the whole model domain. The local flow velocities and eddy diffusivities are based on the results of the hydrodynamic computations. Computationally, the three-dimensional transport of sediment is computed in exactly the same way as the transport of any other conservative constituent, such as salinity and heat. Van Rijn (1993) deals with initiation of motion, suspension and settlement of non-cohesive sediments associated with the effect of currents and waves. Based on these sediment transport calculations, the elevation of the bed is dynamically updated at each computational time-step.

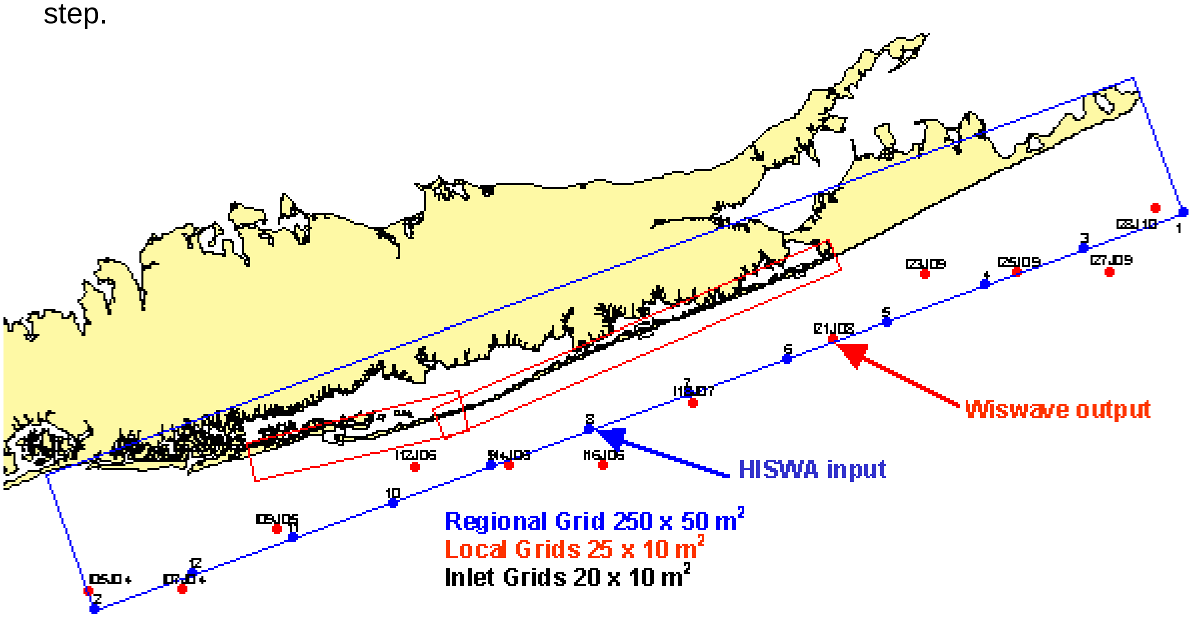


Figure 6-10. DELFT3D-WAVE (HISWA) grid (1 m = 3.28 ft)

The hydrodynamic model implementation used in the sediment transport and morphology model includes the effects of the waves on both nearshore hydrodynamics (i.e., longshore currents and wave setup) and sediment transport (i.e., increased bottom shear stresses and turbulence). It should be noted, however, that the model does not include all of the physics affecting beach profile changes during storm conditions, such as the three-dimensional wave and hydrodynamic processes that generate undertow and offshore sand transport. Nonetheless, this model implementation is particularly suitable for simulating barrier island inundation and sediment overwash processes.

6.1.1.5 Numerical Model Calibration and Verification

Both hydrodynamic models, ADCIRC and Delft3D, underwent extensive calibration before the models were used to simulate historical storm events.

The ADCIRC model was calibrated to match measured tidal water levels by simulating a 30-day record and comparing model output with measurements at four NOAA stations and one Long Island SHORE (LISHORE) station. To match measured tidal water levels in ADCIRC, the bottom friction values were adjusted within reasonable ranges. Ocean storm surge modeling with ADCIRC requires wind stress and barometric pressure for each node within the grid as well as tidal constituent forcing. Significant efforts were put forth to ensure that the wind and pressure inputs were the best available. In addition, research into the drag coefficient formulation for wind stress calculation led to changes from the default ADCIRC drag coefficients, which resulted in better water level comparisons to available measured data (See Sub Appendix A1 for a more detailed description of the drag coefficient calibration). To assess ADCIRC's calibration for storm surge due to wind and barometric pressure, 12 historical tropical and extratropical events were modeled, and the results were compared with NOAA measured hydrographs at four nearshore locations: Sandy Hook, NJ; The Battery, NY; Montauk Fort Pond, NY; and Newport, RI. This rigorous calibration verified that ADCIRC reliably and accurately simulates both tide and storm surges over a regional domain that spans from New Jersey to Rhode Island.

Calibration of the SBEACH Model was performed for the FIMP region using available data describing storm-induced beach change. Details of the calibration can be found in Gravens et al (1999).

As with the ocean tidal calibration, the Delft3D model was calibrated for bay tide by simulating a 30-day record and comparing model output with measurements at 13 measurement locations (6 in Great South Bay, 4 in Moriches Bay, and 3 in Shinnecock Bay). To match measured tidal water levels in Delft3D, the bottom friction values in this model were also adjusted within reasonable ranges. A February 2003 field investigation, including water level gages at six locations in Great South and Moriches Bays, provided reliable information for calibration of the Delft3D model in the bays under storm conditions. The simulation water levels were compared with the measured water levels at the six bay locations and simulated results compare well with measured, showing that Delft3D performs well for this small winter storm.

The Delft3D model skill for simulating barrier island overwash and breaching was assessed by comparing model results with available high water marks (HWM) and overwash and breaching data for two of the most significant storms on record: the September 1938 Hurricane and the December 1992 Nor'easter (See Appendix A1 for storm parameters). The intent of the test was specifically to qualitatively validate the ability of the model to reproduce observed overwash and breaching. Overall, the model simulations for these historic storms provide very realistic results, particularly when considering the uncertainty in the input hydrodynamic conditions and, more importantly, the pre-storm topography. The simulation results are particularly realistic in the case of the 1938 storm, for which more comprehensive topographic data in the vicinity of some of the damaged areas were available. The agreement between simulated peak water levels for both storms and the reported measurements can be considered excellent considering the uncertainty associated with this type of data.

6.1.2 Simulation Results

In this section, water level and morphological response results for a few storm simulations under BLC and FVC are presented. The storm listing was divided into 3 subsets to ensure proper coverage of actual meteorological events: Tropical, Extratropical and Supplemental (Table 6-2). Supplemental storms represent one or more alternate tide variations to the historical storms listed. For example, the high spring tide, near-high tide and mid-range tide cases were simulated. These supplemental simulations provided information needed to better capture the surge level variation with tide whereas linear superposition of tide and storm surge was not adequate. The storm listing is consistent with the training set shown in Table 3-4. Figure 6-11 through Figure 6-14 summarize barrier island response in the 10 vulnerable areas for all storm simulations for both BLC and FVC.

Table 6-2. Storm Listing

Storm Type	Name	Start Date	Duration (hours)**
Tropical Storms	N/A	10-Sep-1938*	15
	N/A	9-Sep-1944	10
	Carol	25-Aug-1954	5
	Edna	2-Sep-1954	7
	Hazel	5-Oct-1954	6
	Connie	3-Aug-1955	0
	Donna	29-Aug-1960*	13
	Esther	10-Sep-1961	14
	Doria	20-Aug-1971	2
	Agnes	14-Jun-1972	18
	Belle	6-Aug-1976*	7
	Gloria	16-Sep-1985*	5
	Bob	16-Aug-1991*	4
	Floyd	7-Sep-1999*	3
Extratropical Storms	N/A	22-Nov-1950	34
	N/A	04-Nov-1953	26
	N/A	11-Oct-1955	43
	N/A	25-Sep-1956	34
	N/A	03-Mar-1962*	56
	N/A	05-Nov-1977	28
	N/A	17-Jan-1978	16
	N/A	04-Feb-1978	27
	N/A	22-Jan-1979	19
	N/A	22-Oct-1980*	17
	N/A	26-Mar-1984	31
	N/A	09-Feb-1985	17
	N/A	28-Oct-1991	50+
	N/A	01-Jan-1992	18
	N/A	08-Dec-1992*	78
	N/A	02-Mar-1993	12
	N/A	10-Mar-1993*	25
	N/A	28-Feb-1994*	23
	N/A	21-Dec-1994*	23
	N/A	05-Jan-1996	12
	N/A	06-Oct-1996	12
	N/A	02-Feb-1998	24
Supplemental Storms	N/A	10-Sep-1938*	15
	N/A	9-Sep-1944	10
	Carol	25-Aug-1954	5
	Donna	29-Aug-1960*	13
	Belle	6-Aug-1976*	7
	Gloria	16-Sep-1985*	5
	N/A	22-Nov-1950	34
	N/A	04-Nov-1953	26
	N/A	03-Mar-1962*	56

Storm Type	Name	Start Date	Duration (hours)**
	N/A	22-Oct-1980*	17
	N/A	08-Dec-1992*	78
	N/A	09-Mar-1993*	25

*Indicates storm is included in calibration set
**Storm durations represent duration that storm surge exceeded 1 ft (0.3 m), based on ADCIRC simulations at Station 31.
+Storm duration for this storm based on measured storm surge at Sandy Hook, NJ

6.1.2.1 Tropical Storms

Peak water levels were determined for the historical 1938 Hurricane and Hurricane Gloria (1985) for BLC and FVC. The historical 1938 hurricane produced much higher water levels than the historical Hurricane Gloria. While both of these storms are similar in intensity, they occurred on distinctly different phases of the tide: the 1938 hurricane's peak surge coincided with high spring tide while Hurricane Gloria's peak surge coincided with low spring tide.

Under BLC, the historical 1938 Hurricane results in some barrier island overwash and breaching (Figure 6-15 through Figure 6-18). In particular, the simulations predict a full breach (to elevations below MLW) occurs at Old Inlet while the simulations predict partial breaches (to elevations between MHW and MLW) at Smith Point County Park, Tiana Beach, and West of Shinnecock Inlet. Widespread overwash is also predicted in all 10 vulnerable areas.

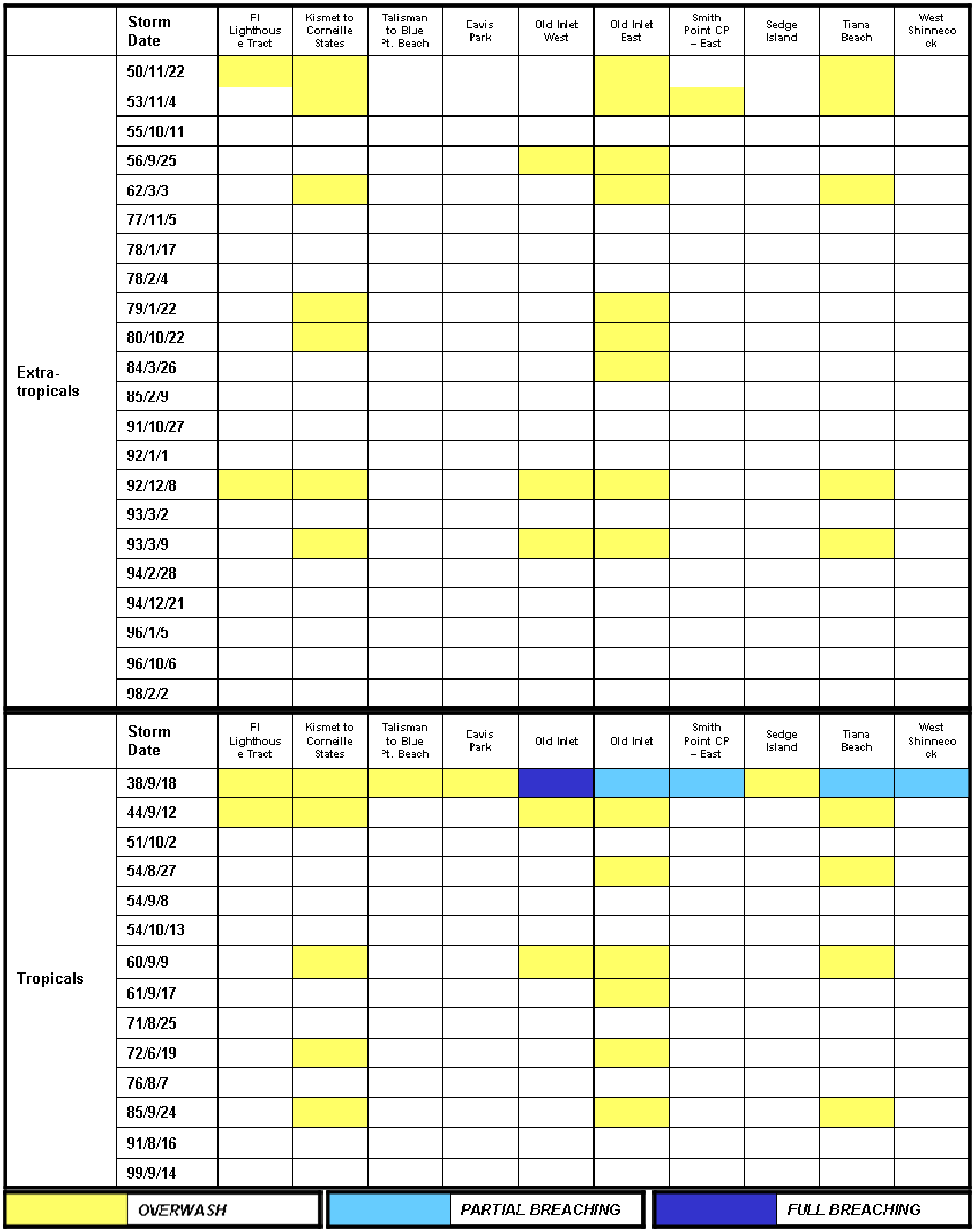


Figure 6-11. Simulated barrier island morphological response under Baseline Conditions (BLC) for historical storms.

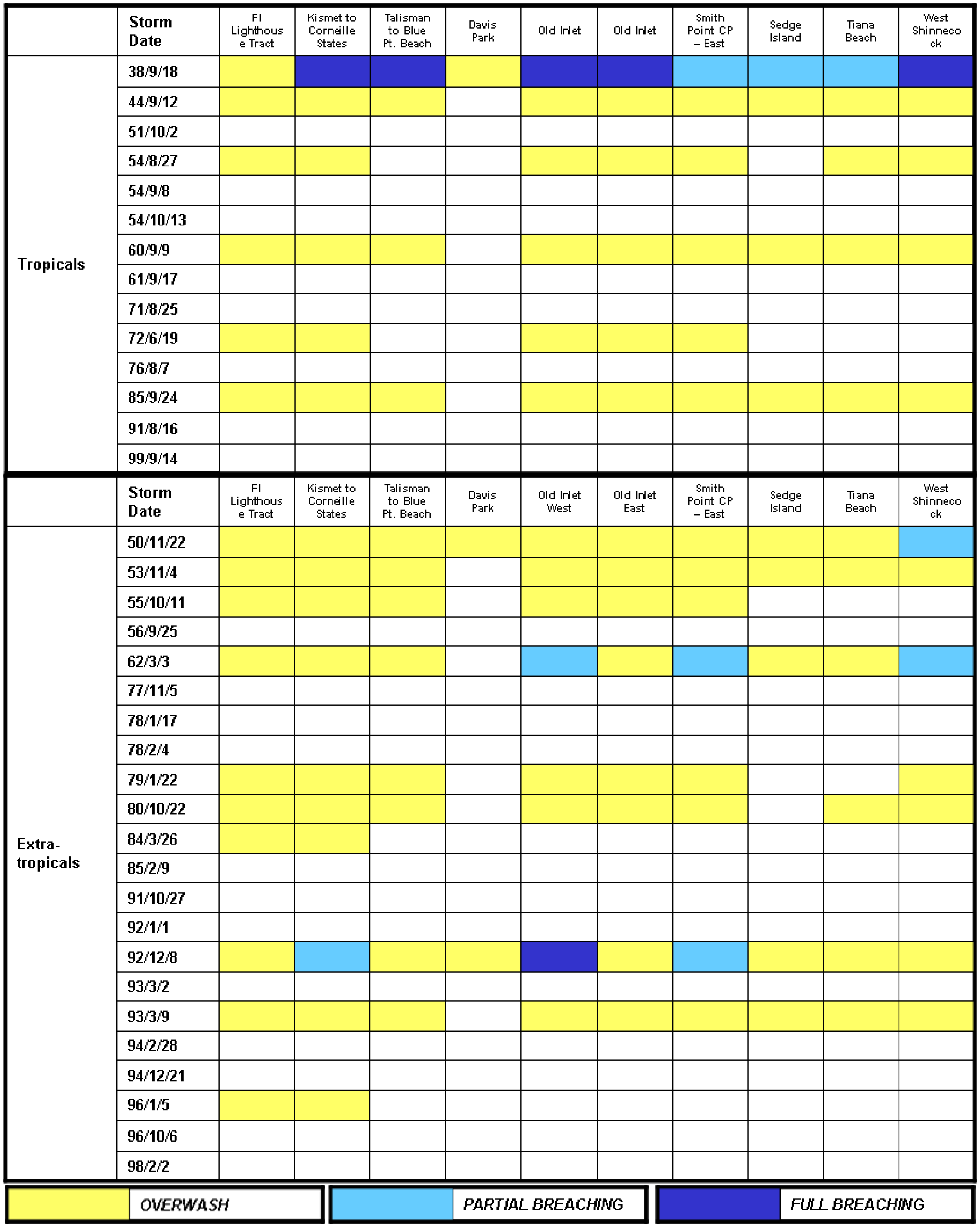


Figure 6-12. Simulated barrier island morphological response under Future Vulnerable Conditions (FVC) for historical storms.

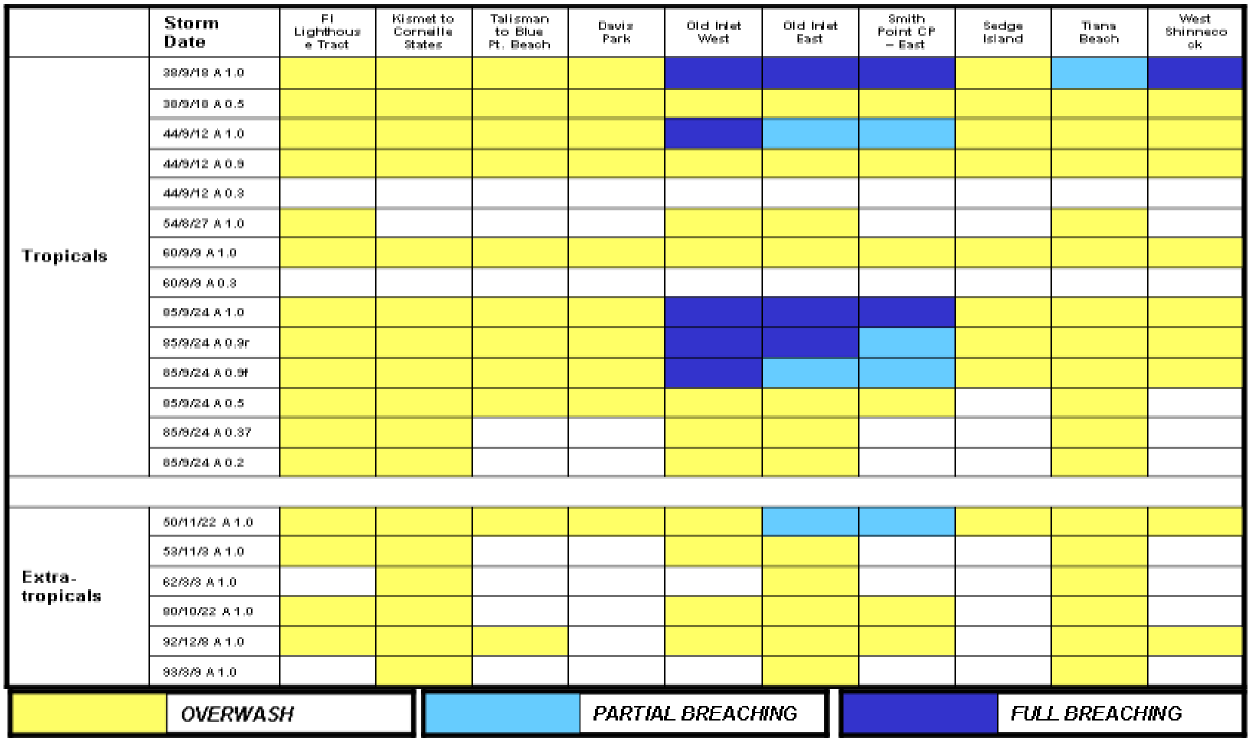


Figure 6-13. Simulated barrier island morphological response under Baseline Conditions (BLC) for additional alternate tide cases (A=Alternate tide where number is tide index, with 1.0 being high spring tide and 0.0 being low spring tide).

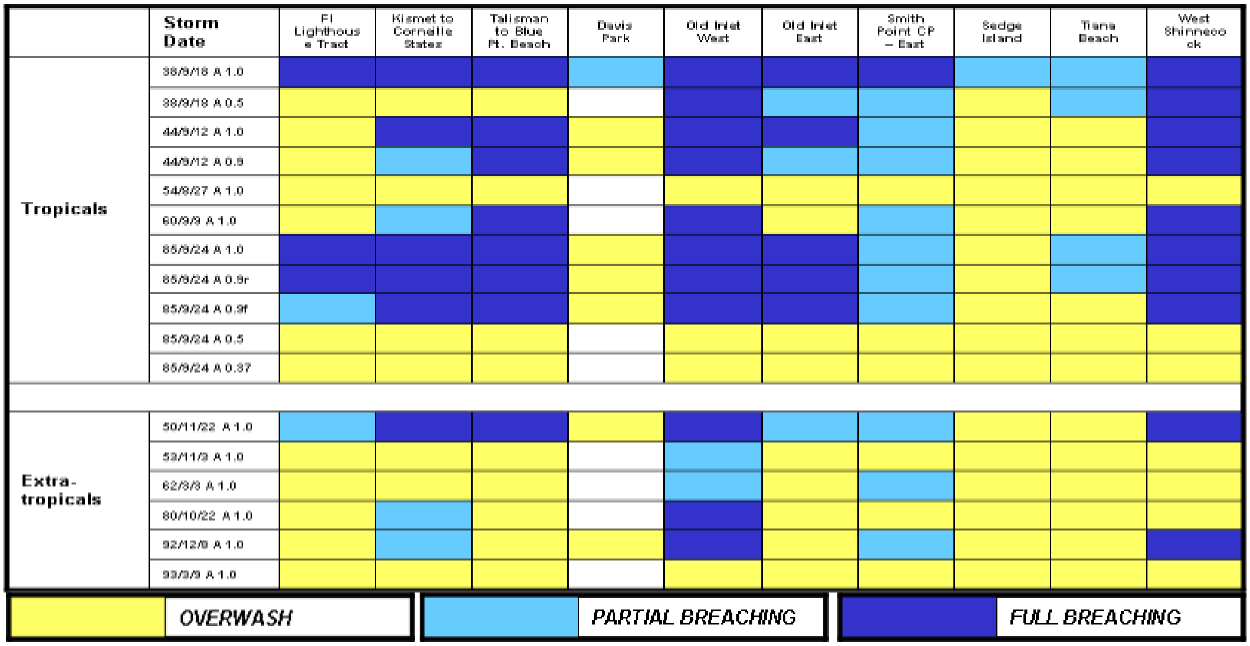


Figure 6-14. Simulated barrier island morphological response under Future Vulnerable Conditions (FVC) for additional alternate tide cases(A=Alternate tide where number is tide index, with 1.0 being high spring tide and 0.0 being low spring tide). (Note: For storms listed above and not listed here, FVC and BLC responses are classified identically.)

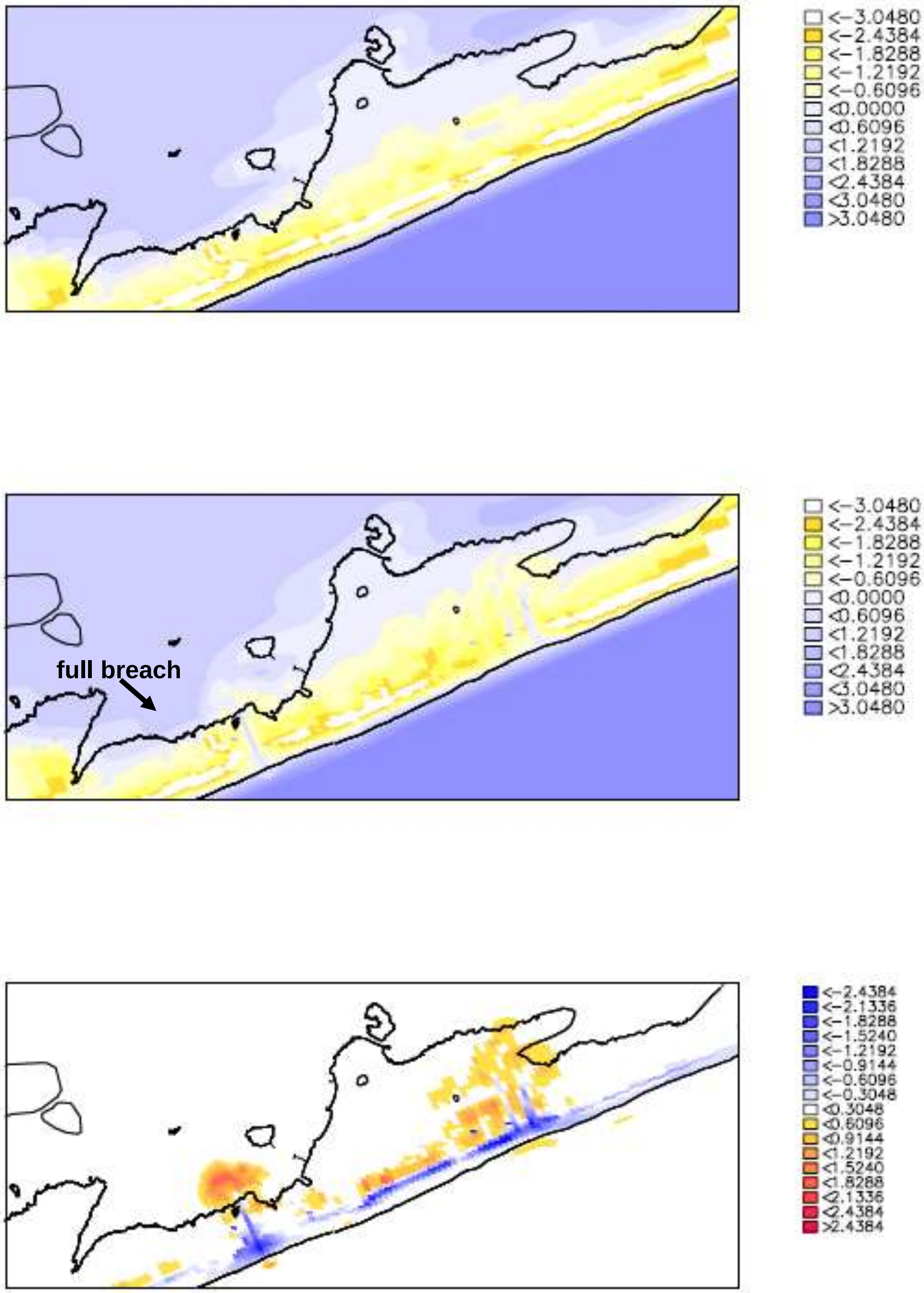


Figure 6-15. Morphological response at Old Inlet under BLC for the historical 1938 hurricane. Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

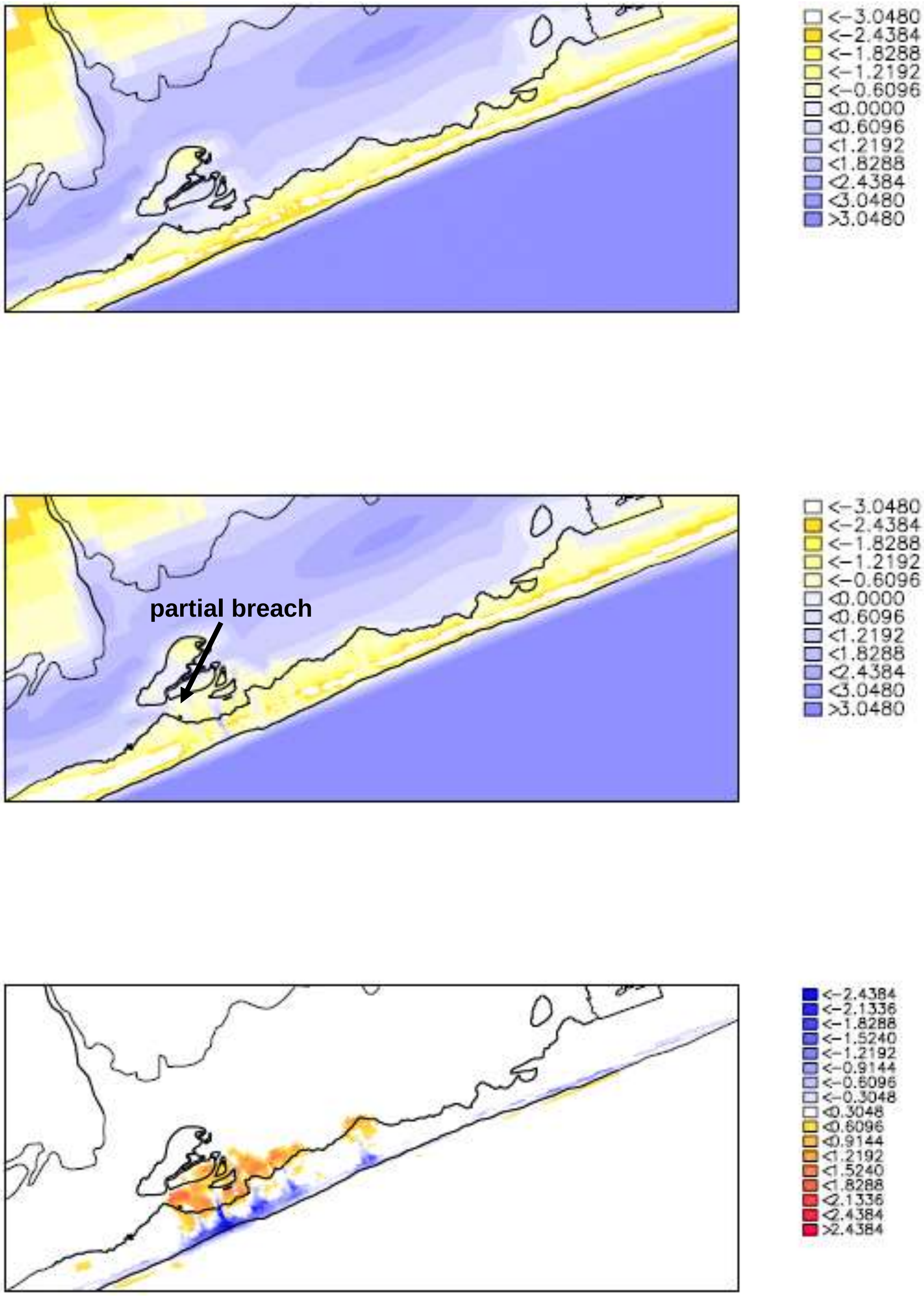


Figure 6-16. Morphological response at Smith Point County Park under BLC for the historical 1938 hurricane. Top pane is initial topography, center pane is topography several hours after peak surge, and bottom pane is elevation difference between the two topographies.

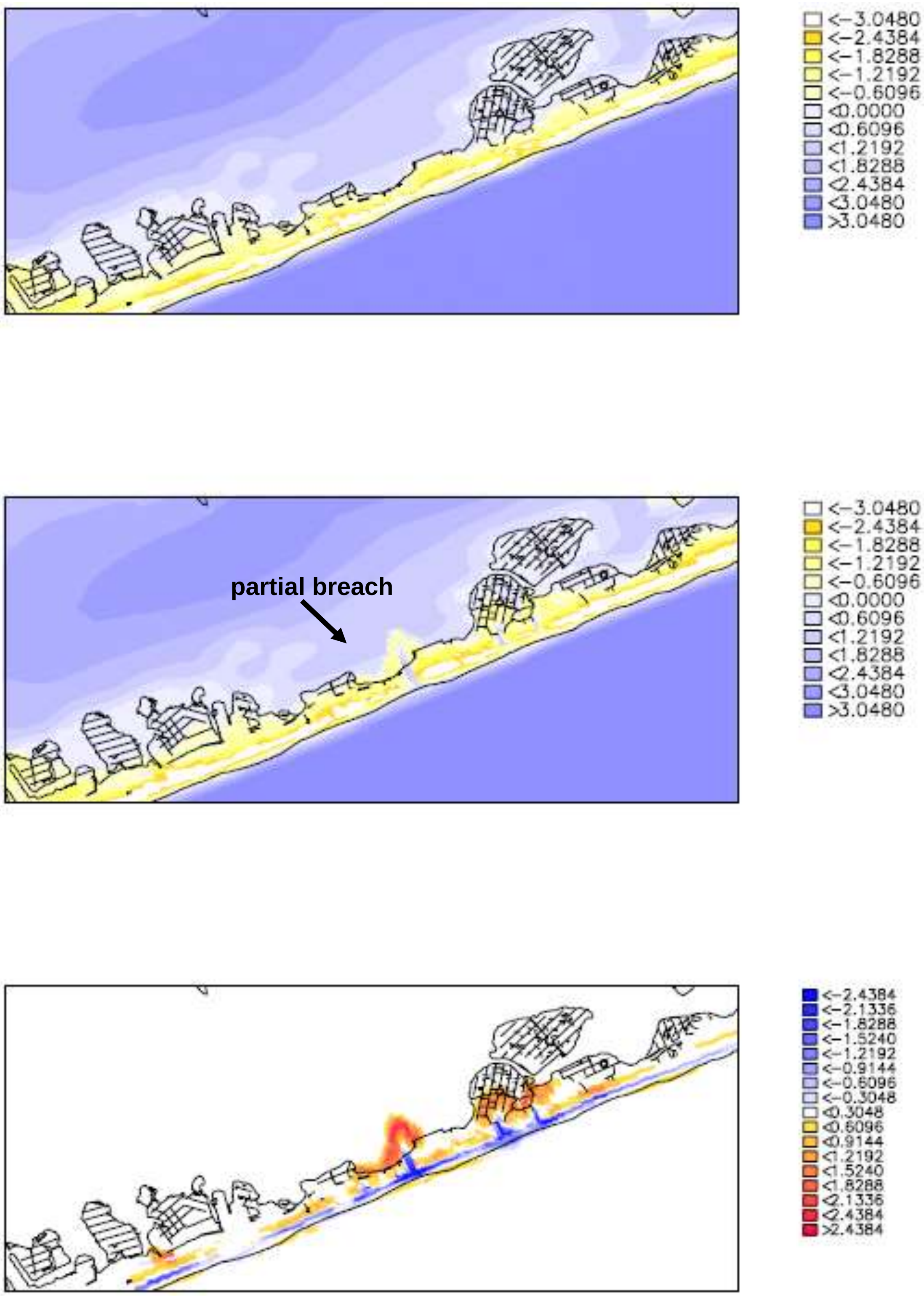


Figure 6-17. Morphological response at Tiana Beach under BLC for the historical 1938 hurricane. Top pane is initial topography, center pane is topography several hours after peak surge, and bottom pane is elevation difference between the two topographies.

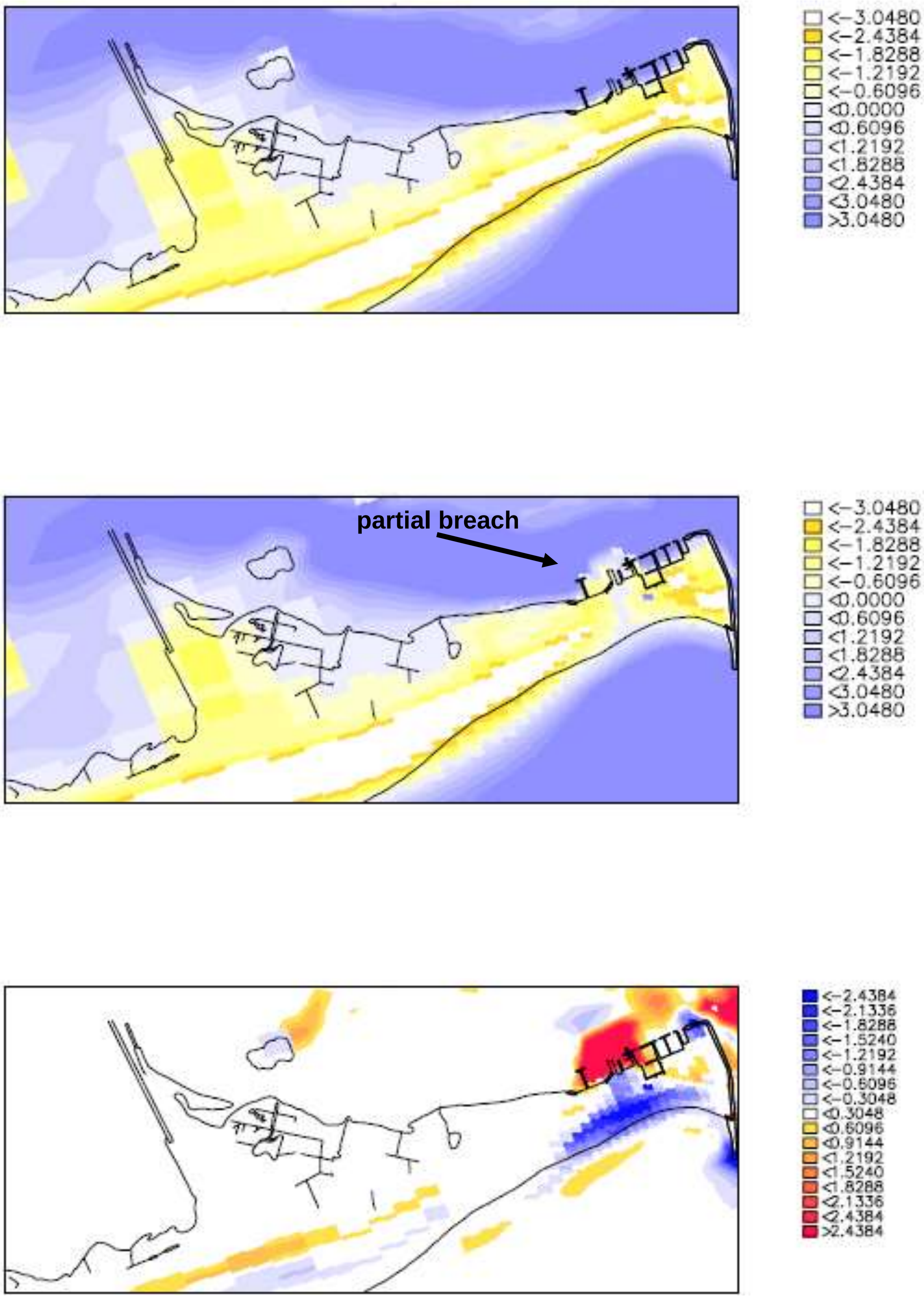


Figure 6-18. Morphological response at West of Shinnecock Inlet under BLC for the historical 1938 hurricane. Top pane is initial topography, center pane is topography several hours after peak surge, and bottom pane is elevation difference between the two topographies.

While similarly intense, the simulated barrier island response to historical Hurricane Gloria is much less severe because total water levels were much lower owing to peak surge coinciding with low tide. Here, the numerical simulations only predict overwash (to elevations as low as MHW) at Old Inlet and Tiana Beach.

Under FVC conditions, morphological response for both the historical 1938 hurricane and the historical Hurricane Gloria are more widespread. Under FVC, the historical 1938 hurricane simulation predicts full breaching at Kismet to Corneille Estates, Talisman to Blue Point Beach, and West of Shinnecock Inlet, in addition to Old Inlet. Partial breaching is also predicted at Sedge Island in addition to Smith Point County Park and Tiana Beach. Owing to the fact that the storm made landfall during low tide, the historical Hurricane Gloria simulation under FVC still predicts overwash (to elevations as low as MHW) only. However, overwash is much more widespread under FVC than under BLC occurring at all but one of the 10 identified vulnerable areas.

Peak water levels were determined for the alternate high spring tide case of Hurricane Gloria for BLC and FVC. The coincidence of peak surge with high spring tide for this alternate storm case shows how dramatically different the hydrodynamic and barrier island response can be because of a tropical storm's timing relative to astronomical tide. For the case of Hurricane Gloria, the high spring tide water levels are significantly higher throughout the project under both FVC and BLC. In fact, in many locations, these water levels are higher than those simulated for the historical occurrence of the 1938 hurricane. Barrier island response to this high spring tide case of Hurricane Gloria is also much more severe than for the historical tide case. Under BLC, breaching is predicted at Old Inlet and Smith Point County Park (Figure 6-19 and Figure 6-20). In addition to these breaching locations, under FVC breaching is also predicted at Fire Island Lighthouse Tract, Kismet to Corneille Estates, Talisman to Blue Point Beach, Tiana Beach, and West of Shinnecock (Figure 6-21 through Figure 6-25).

6.1.2.2 Extratropical Storms

Peak water levels were determined for the historical March 1962 and December 1992 Nor'easters for BLC and FVC. While the March 1962 storm is often considered a storm-of-record along parts of Long Island, simulation results under BLC and FVC indicate that the December 1992 storm was slightly more severe. While there are similarities between these two storms, peaks water level and wave height were slightly larger for the 1992 storm. As such peak storm water levels are slightly higher for the 1992 storm under both BLC and FVC.

Simulations of both the 1962 and 1992 Nor'easters predict widespread overwash along the barrier island system under BLC. Morphological response under FVC is more dramatic. Here, breaches are predicted at Old Inlet and Smith Point County Park for both storms. In addition, a breach is predicted at Kismet to Corneille Estates during the 1992 storm while a breach is predicted at West of Shinnecock during the 1962 storm. Figure 6-26 through Figure 6-28 present morphological responses simulated during the historical 1992 Nor'easter under FVC.

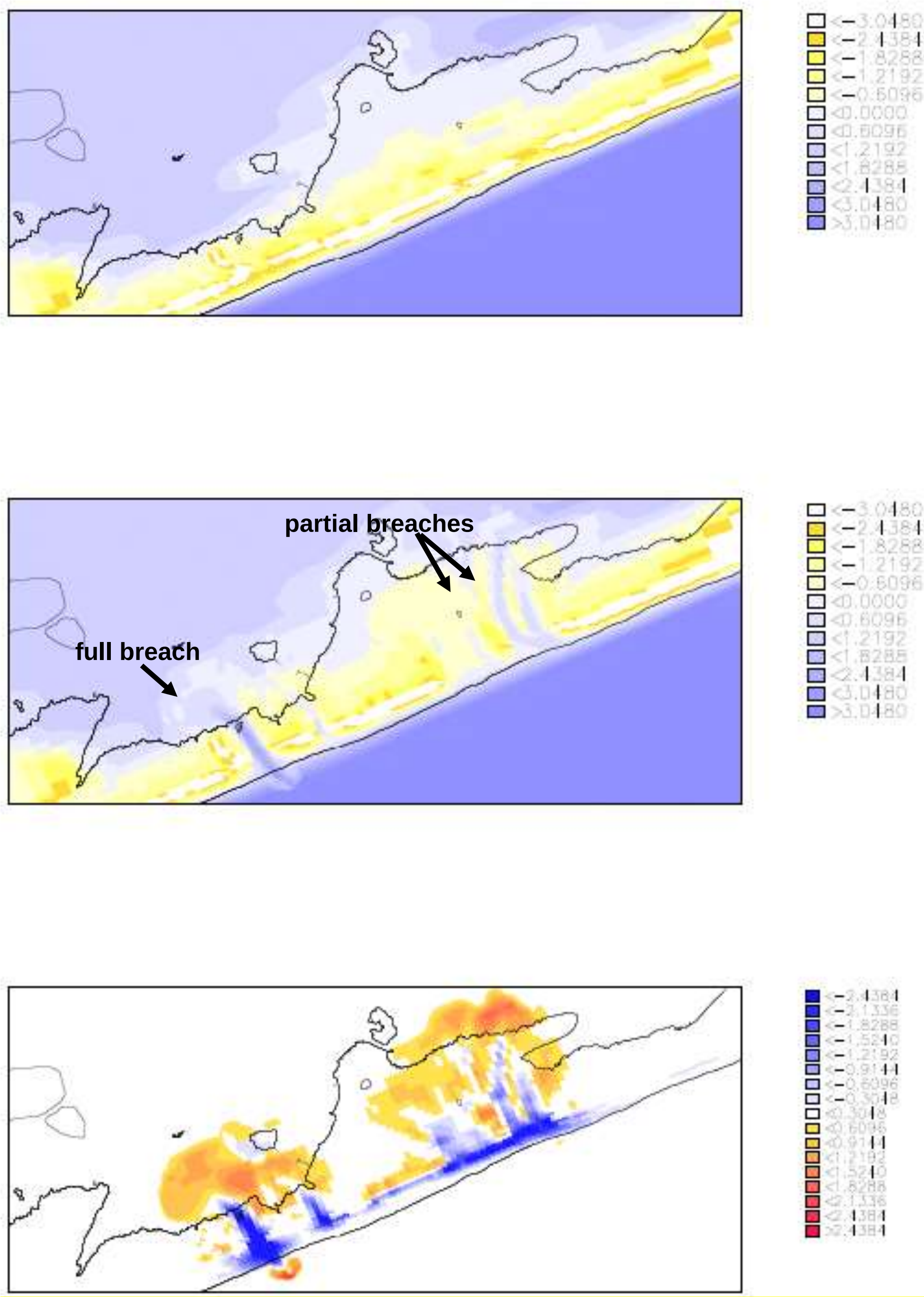


Figure 6-19. Morphological response at Old Inlet under BLC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

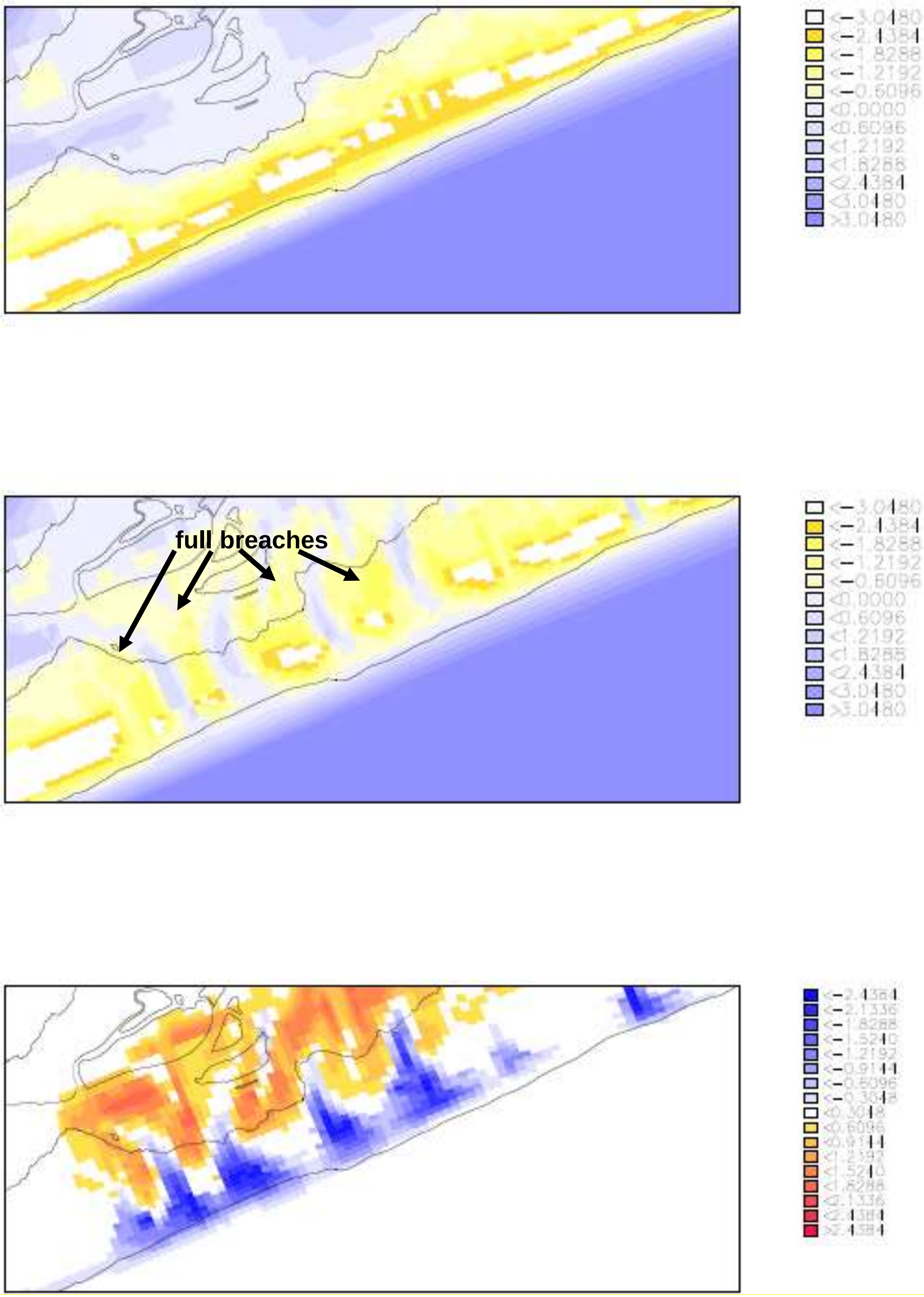


Figure 6-20. Morphological response at Smith Point County Park under BLC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

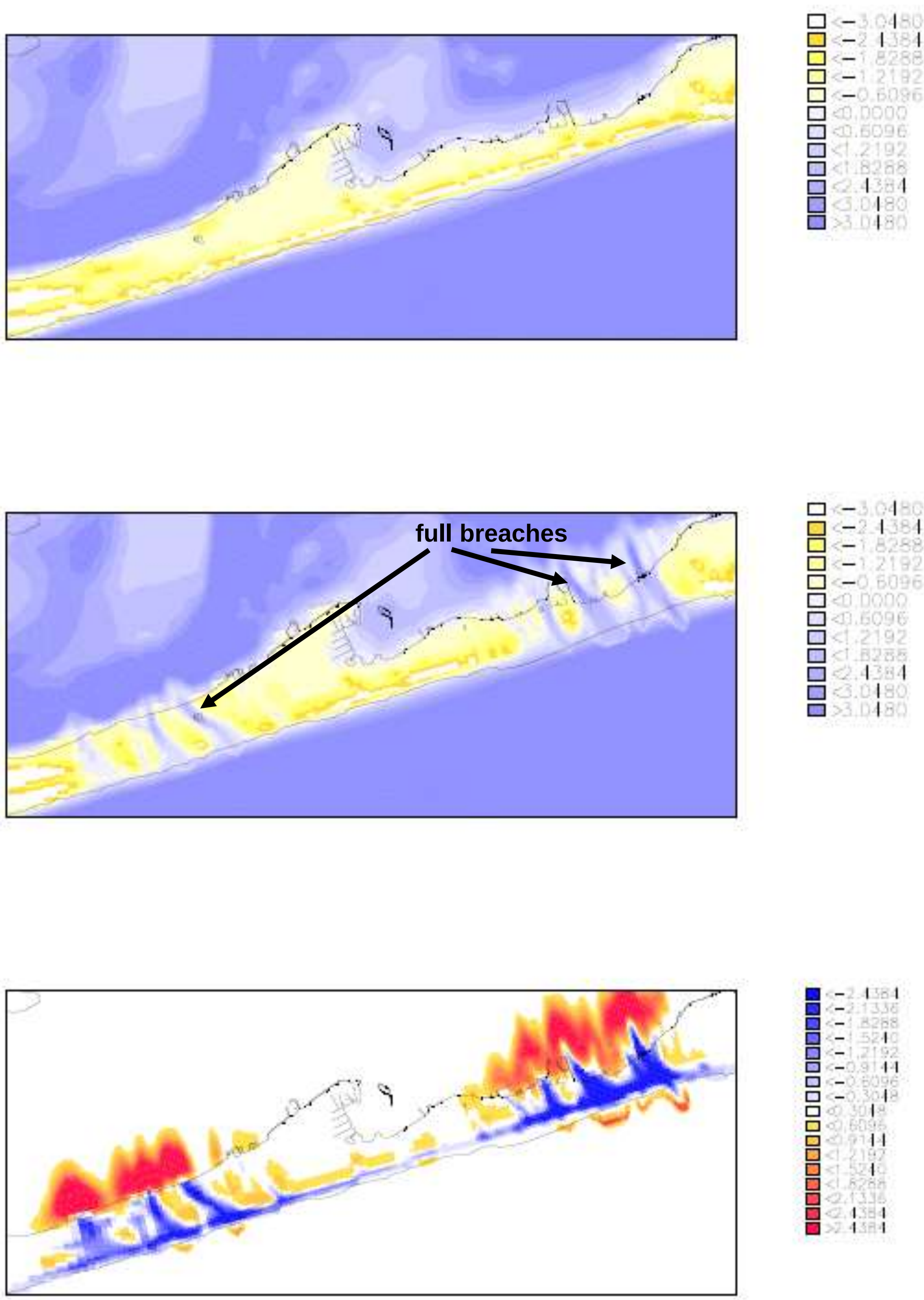


Figure 6-21. Morphological response at Kismet to Corneille Estates under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

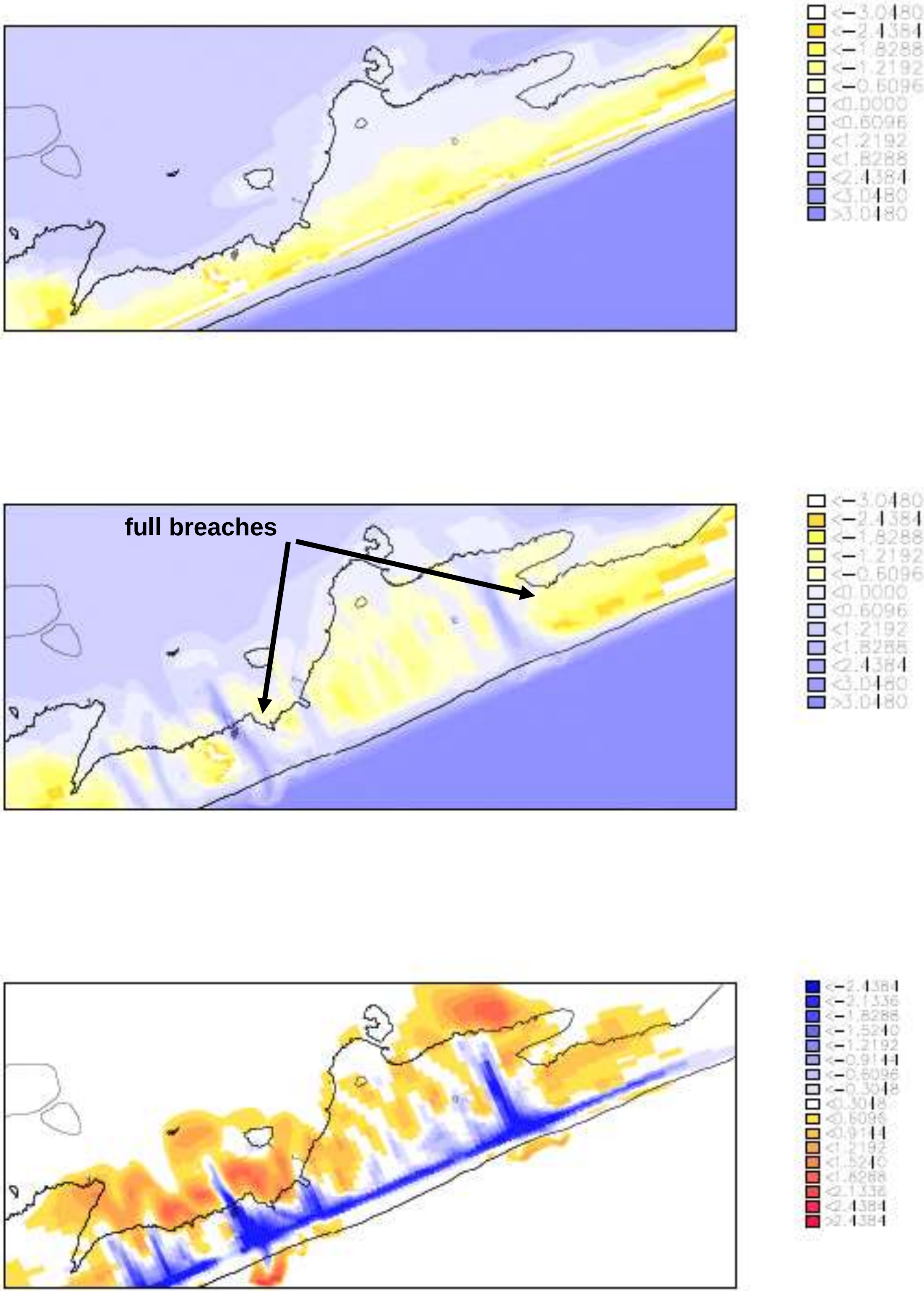


Figure 6-22. Morphological response at Old Inlet under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

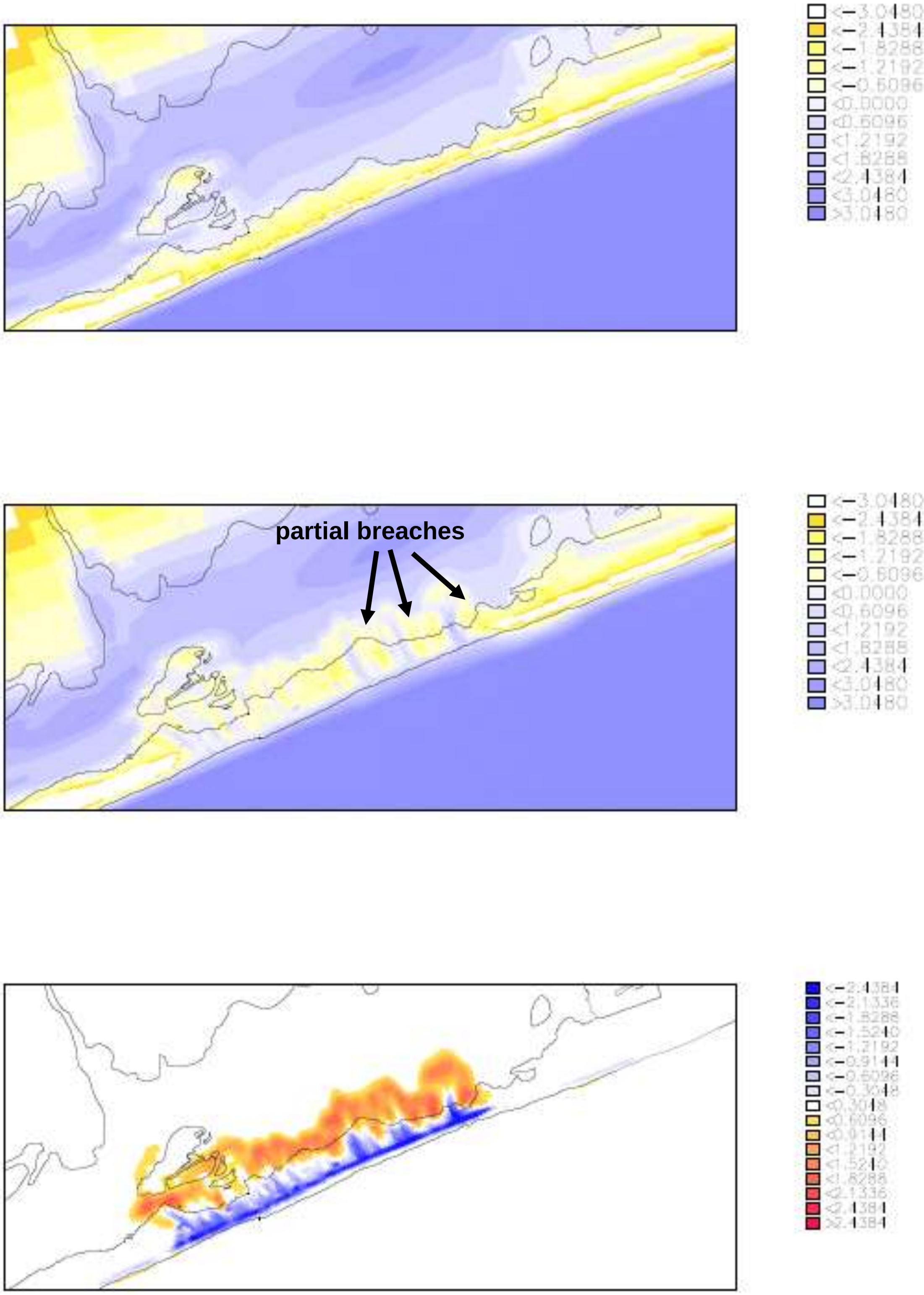


Figure 6-23. Morphological response at Smith Point County Park under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

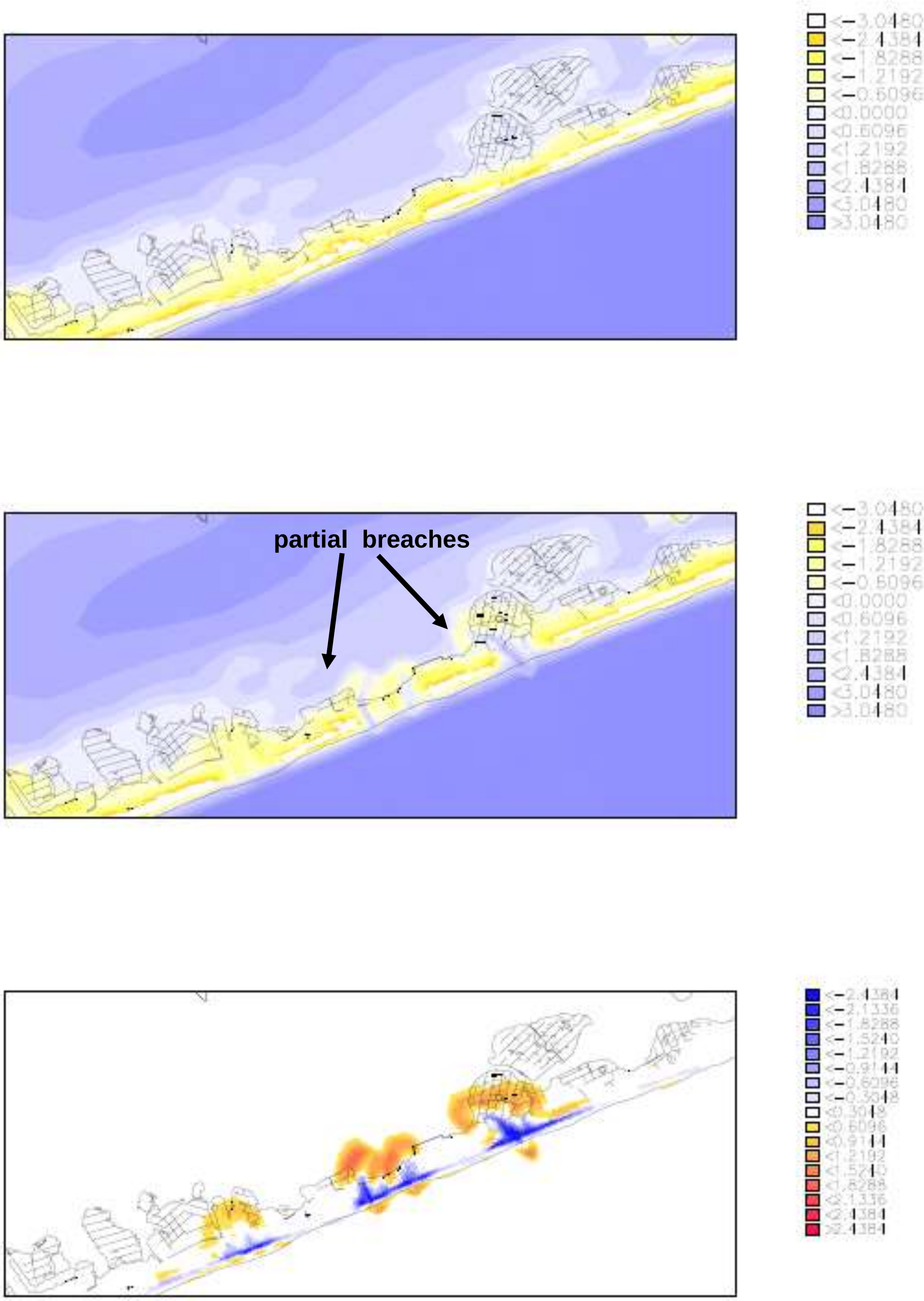


Figure 6-24. Morphological response at Tiana Beach under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

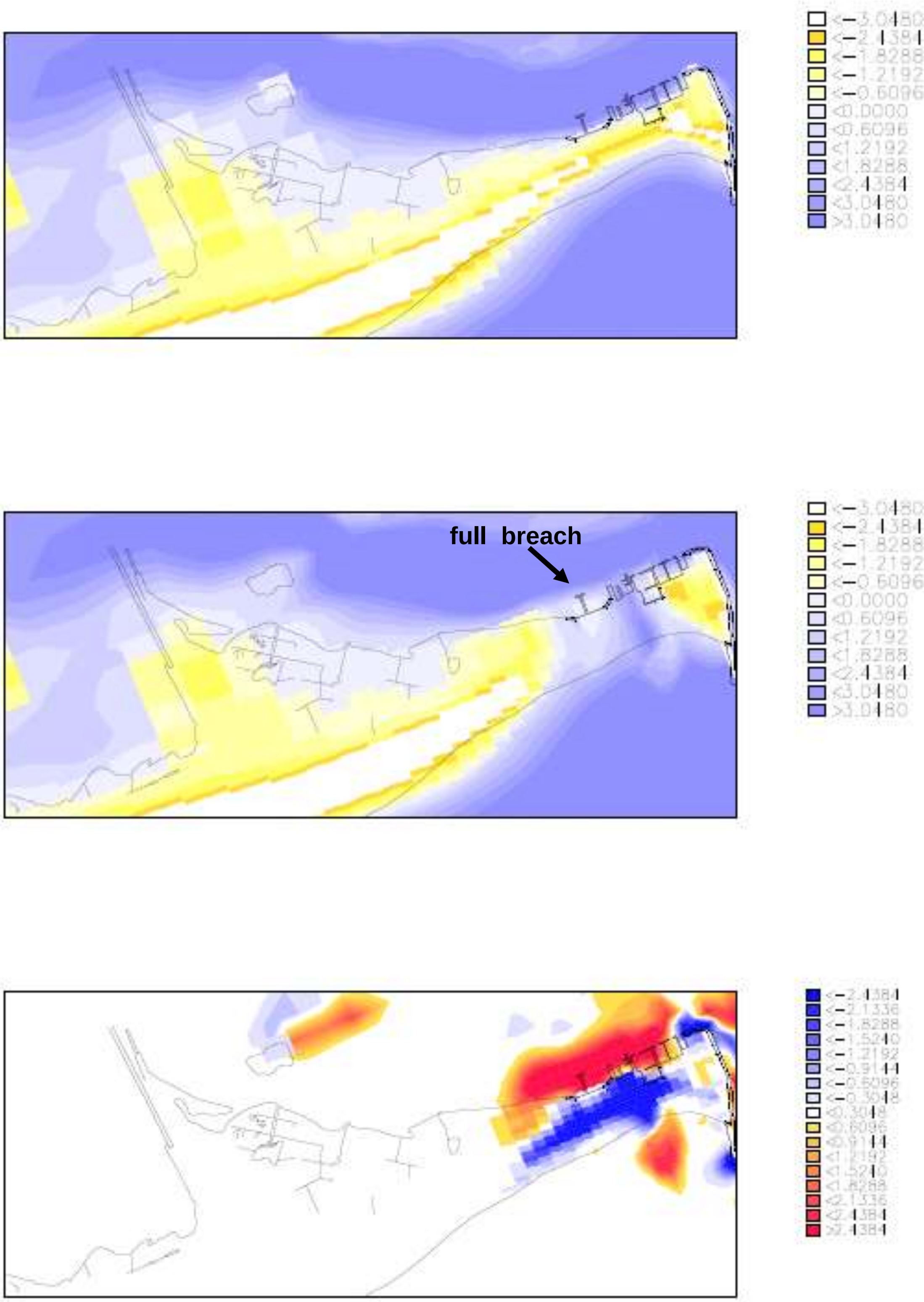


Figure 6-25. Morphological response at WOSI under FVC for the alternate high spring tide case of Hurricane Gloria (1985). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

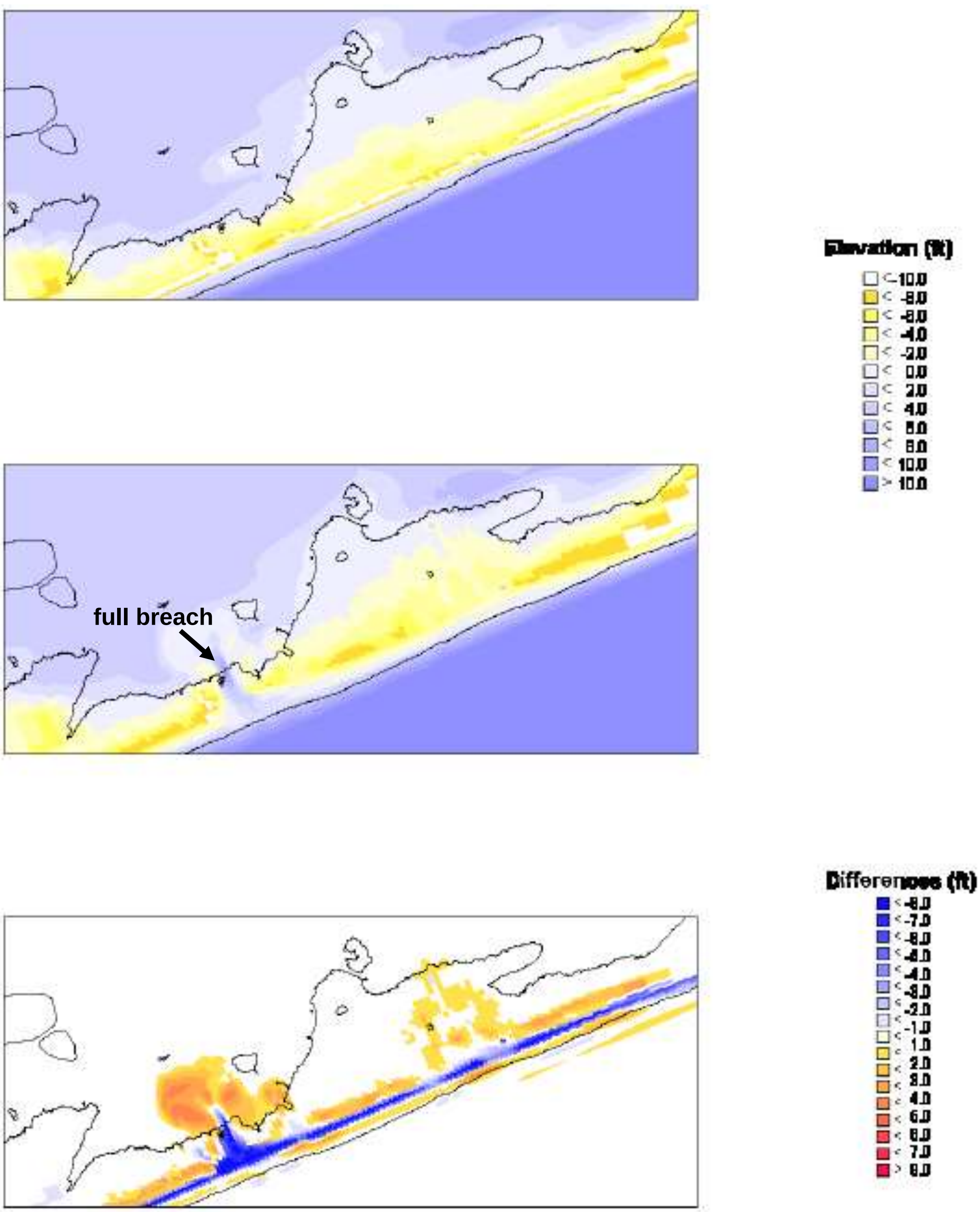


Figure 6-26. Morphological response at Old Inlet under FVC for the historical 1992 Nor'easter. Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

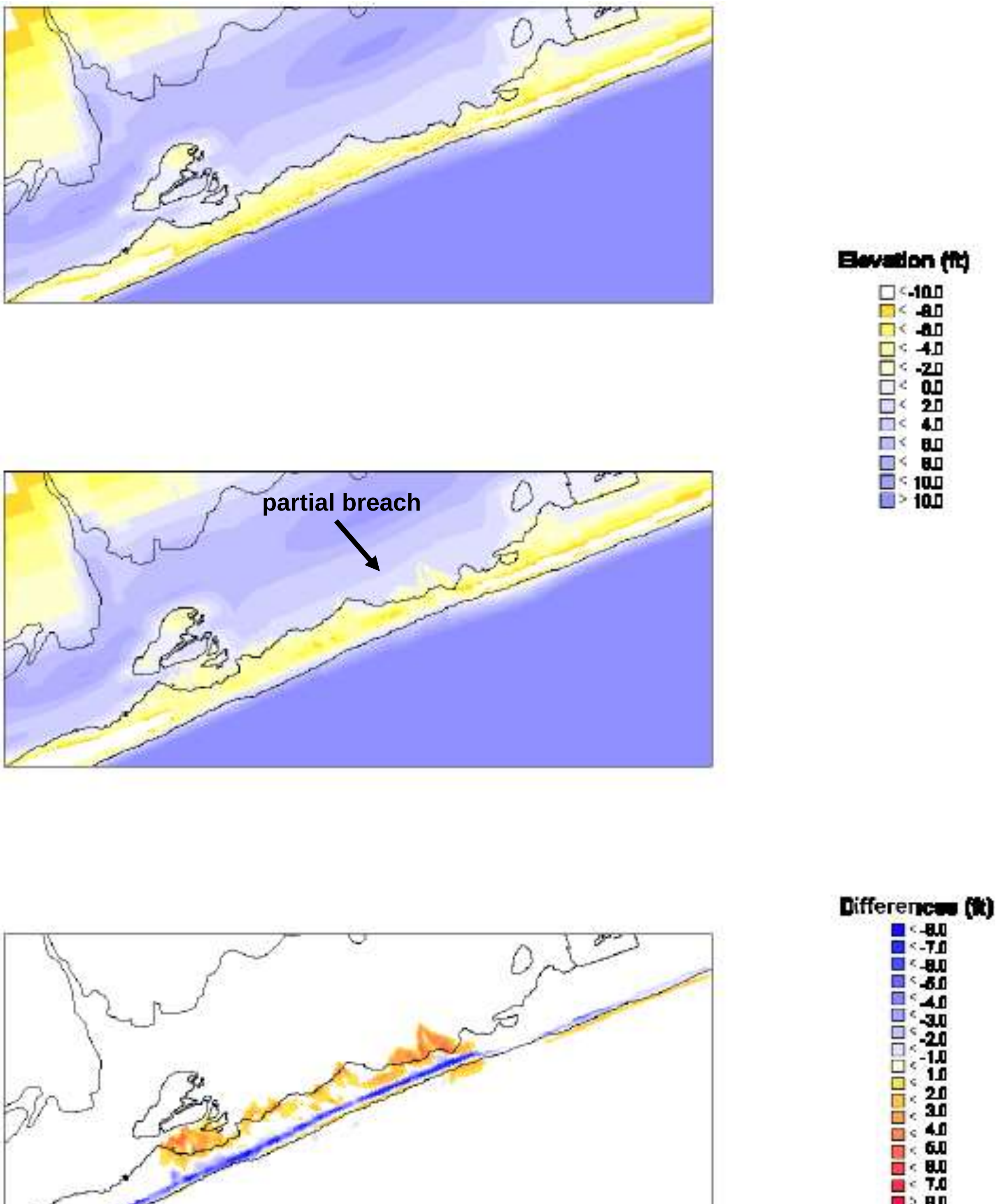


Figure 6-27. Morphological response at Smith Point County Park under FVC for the historical 1992 Nor'easter. Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

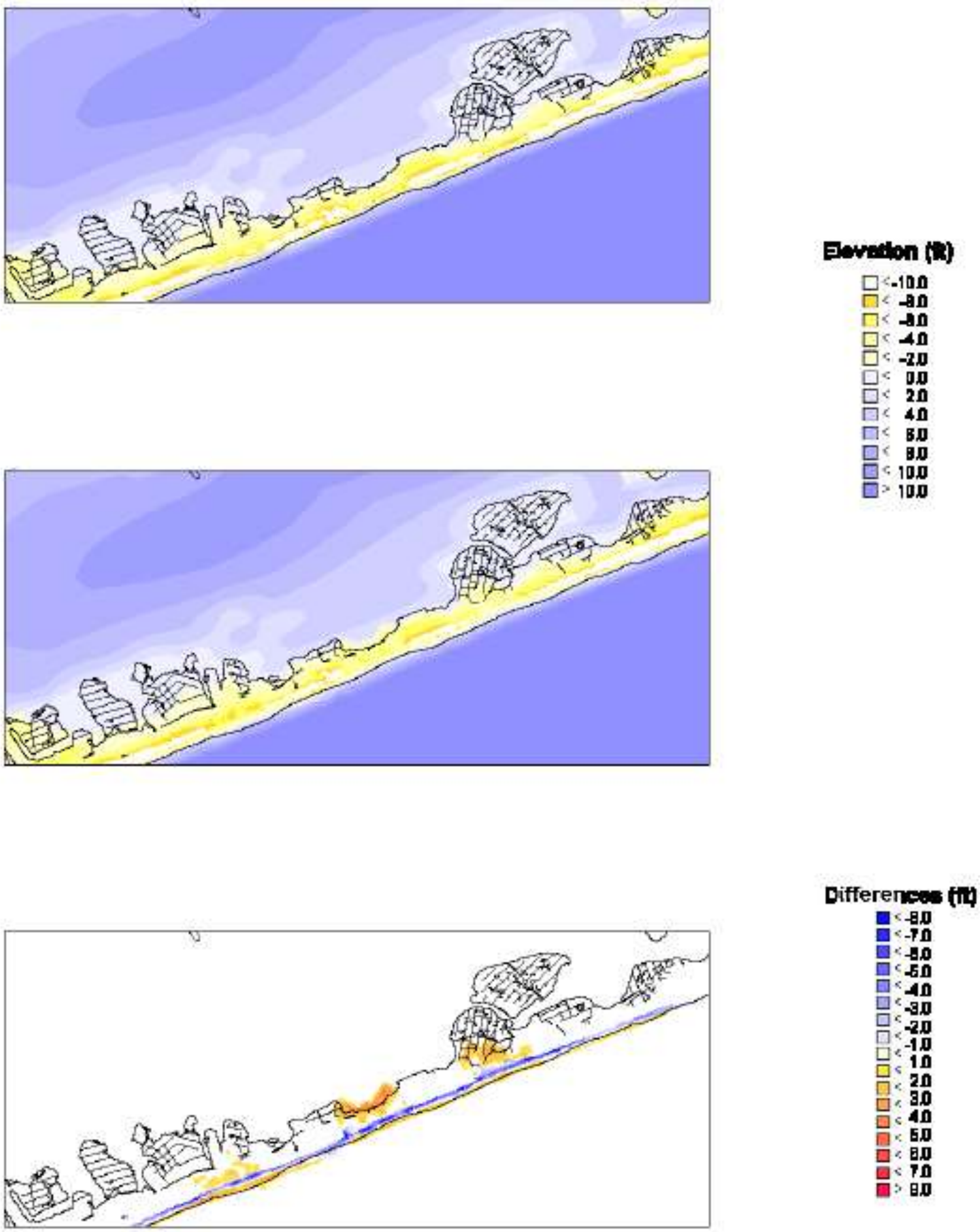


Figure 6-28. Morphological response at Tiana Beach under FVC for the historical 1992 Nor'easter (significant overwash only). Top pane is initial topography, center pane is topography several hours afterpeak surge, and bottom pane is elevation difference between the two topographies.

6.1.3 Stage-Frequency Methodology

Parametric and nonparametric methods may be used to determine probability distributions. Parametric methods assume that the storm population follows some prescribed probability distribution, for example a normal (Gaussian) distribution. In contrast, nonparametric methods do not presume a distribution; instead the distribution is computed from the available data. When selecting a method for use with a particular data set, it is important to realize that nonparametric methods are more appropriate when the population distribution is unknown, while parametric methods are more appropriate if the distribution is known beforehand. As such, nonparametric methods are more appropriate for the storm water levels in the FIMP study.

Empirical Simulation Techniques (EST) are a group of nonparametric methods for proceeding directly from hydrometeorological storm data to simulations of future storm activity and coastal impact, without introducing parametric assumptions concerning the probability law formulas and related parameters of the data (Scheffner *et al.*, 1999).

Two EST procedures, one univariate (1-D) and the other multivariate, were used in the FIMP studies. The 1-D EST methodology, using water level as the one dimension, was employed for stage-frequency development for the FIMP study. The multivariate EST was used in conjunction with SBEACH for modeling of beach profile response and estimation of storm-induced coastal changes, primarily for economic life-cycle analysis (see Gravens *et al*., 1999).

For the FIMP study, the 1-D EST methodology was improved to account for other, equally probable, astronomical tide timings relative to each individual storm's timing. Along the open coast, linear superposition of surge and tide gives a realistic estimate of storm stage.

However, in the bays, linear superposition of surge and tide does not provide as good of an estimate to total water level. This is due to bay and inlet effects as well as barrier island overwash and breaching. In order to implement this EST method, several supplemental non-historical storms were also selected for numerical modeling. In all, 14 historical tropical, 22 historical extratropical, and 21 supplemental (alternate tide) storms were simulated. The 21 supplemental storms represent one or more alternate tide variations of 12 historical storms (6 tropical and 6 extratropical). The storms were shown previously in Table 6-2. Specifically, the high spring tide case was simulated for each of these 12 storms. In addition, near-high tide and mid-range tide cases were simulated for 3 of the tropical storms. These supplemental simulations provided information needed to better capture the surge level variation with tide where linear superposition was not adequate.

As a result of including alternate tide scenarios, final stage-frequency curves demonstrate gradual alongshore variability in ocean station peak water levels, at all return periods, as a result of accounting for variation in astronomical tide scenarios. Furthermore, stage-frequency relationships within each of the three bays reflect spatial variations that are consistent with each bay's geometry and inlet configuration as well as with each bay's corresponding ocean stage-frequency relationship.

6.1.3.1 Special Treatment of the Lower Return Period Distribution

The FIMP storm training set was selected using the peak-over-threshold method. Namely, only storms exceeding a prescribed surge level were included. Such a statistical approach produces reliable stage-frequency estimates for moderate to large return periods. However, stage for smaller return periods (less than 10 years) may not be adequately represented. For the FIMP study, the peaks-over-threshold method significantly underestimated water level for small return periods (See Appendix A1 Section 10 for additional details). Because very small events, 1- to 2-year return period, play an important role in the economic analyses, an alternate approach was adopted for the FIMP study to determine stages for small return periods.

By evaluating long-term NOAA gage records, the stage associated with a 1-year return period, at these NOAA locations, may easily be determined. Because tropical events are less frequent, peak water levels for return periods less than 10 years are best defined by extratropical events. Therefore, this analysis considered only peak gage water levels associated with extratropical events. Annual maximum extratropical water levels were extracted from historical gage measurements at each of 3 NOAA gages (Sandy Hook, The Battery, and Montauk Fort Pond). By ranking these maximum annual water levels, by magnitude, an estimate of stage-frequency for lower return periods at these locations was determined. These estimates of stage-frequency for lower return periods were used to develop a lower cutoff threshold criterion for stage-frequency output locations for FIMP. Specifically, the analysis of the measured data was used to select a small extratropical event for the FIMP training set to represent the minimum expected annual peak water level. This peak water level associated with this small event was used to truncate the peak-over-threshold stage-frequency relationships for all FIMP station locations.

6.1.4 Stage-Frequency and Numerical Model Uncertainty

Sources of uncertainty in the final stage-frequency results come from several sources:

- Topographic and bathymetric survey accuracy and topographic assumptions
- Vertical datum conversion accuracy
- Input metrology accuracy
- Wave model accuracy
- Hydrodynamic model accuracy
- Morphology model accuracy
- Statistical assumptions and extrapolation

Uncertainty in the initial topographic and bathymetric conditions leads to uncertainty in numerically simulated flow volumes through the three tidal inlet and to uncertainty in initiation of barrier island overwash and overflow. The uncertainty in both of these flow processes due to bathymetric and topographic uncertainty also leads to uncertainty in bay water levels. For the FIMP study, bathymetric and topographic uncertainty is related to survey measurement accuracy and vertical datum conversion accuracy. Furthermore, the Future Without Project Conditions (FWOPC) topographies also have uncertainties associated with the assumptions required to estimate these unknown conditions. Bathymetric survey error for the measured data sets used in the FIMP study is generally about 0.5 ft, while variation in the vertical datum conversion throughout the project is also about 0.5 ft (based on NOAA reported values for NGVD29 and MSL). Uncertainty associated with the assumptions made in developing the FWOPC topographies is less well defined. However, the manner in which these topographies are used in economics lifecycle analyses allows for consideration of all topographic scenarios that lie between the FWOPC and the more accurately defined BLC thereby accounting for uncertainty in the FWOPC.

Uncertainty in meteorological input leads to uncertainty in simulated wave fields and water levels. Storm wind and pressure field uncertainty is related to uncertainty in storm wind and pressure measurements and reported storm parameters. While the meteorological fields developed for FIMP used state-of-the-art methods, they still contain some inaccuracies. Comparisons between the FIMP storm wind fields and NDBC buoy 44025 measurements indicate that wind magnitude error is about 3 ft/s.

The numerical wave models WISWAVE and HISWA both produce simulation errors that contribute to uncertainty in water level predictions. Comparisons between simulation output and NDBC Buoy 44025 measurements indicate error in spectral wave height near the peak of the storm to be about 3 ft. This error in wave height prediction likely transfers to an error in nearshore wave setup prediction of about 0.5 ft.

Both the ADCIRC and DELFT3D hydrodynamic models were rigorously calibrated for astronomical tides and storm water levels (see Sub Appendix A1). In comparing simulated output with nearshore NOAA measurements, error in tidal amplitude is about 0.1 ft while RMS error in surge is about 0.8 ft. Within the three bays, errors in tidal amplitude are less than 0.3 ft. Comparisons between simulation results and measurements collected during the small February 2003 Nor'easter indicate that total water level (surge + tide + ocean setup contributions + local wind setup/setdown) prediction errors within the bays are less than 0.3 ft.

Uncertainty in pre-inundation dune lowering simulations with SBEACH and in post-inundation morphology change simulations with DELFT3D both contribute to bay water level uncertainty. Without quantitative measurements of dune lowering and storm-induced barrier island breaching, it is difficult to quantify the error associated with these simulated processes. However, realism tests performed as part of model verification demonstrate that these models perform admirably in qualitatively replicating historical overwash and breaching events and associated bay water levels (see Sub Appendix A1).

Finally, there is uncertainty associated with the statistical approach adopted for the FIMP study. At both extremes of the stage-frequency distribution, uncertainty is introduced. For return periods below 10 years, the approach introduced in Chapter 6.1.3 for truncating the distribution does introduce some uncertainty. However, this truncation approach provides a result that is much improved over curve-fitting with the 36 storm set alone. For return periods above 150 years, data extrapolation techniques are employed. Therefore, the stage-frequency relationship in this region is based on the trend of the simulated data below 150 years rather on data in this region. Statistical uncertainty in the stage-frequency relationships is represented by the quartile bands (or standard deviation) about the median result.

The life cycle model, used to compute the damages and economic benefits for the various alternatives discussed in Section 7.0, accounts for the uncertainty in inputs such as stage-damage curves, breaching, erosion, sea level rise, timing of storms, etc. by assuming a range of variability for each of these parameters and using Monte Carlo sampling techniques.

6.1.5 Ocean Stage-Frequency Relationships

Storm-surge modeling was performed with ADCIRC to determine ocean-side stage-frequency relationships, where the stage represents astronomical tide and surge generated by winds and barometric pressure. See Chapter 6.1.4 for a discussion on stage-frequency uncertainty. Figure 6-29 shows the spatial distributions of tropical, extratropical, and combined peak water levels along the open coast and within the three bays for the 6-year, 10-year, 25-year, 50-year, 73-year, and 100-year return periods. Figure 6-30 shows combined-storm stage frequency offshore of Fire Island, Westhampton, and the Ponds, while a full set of ocean stage-frequency relationships for this study are in Sub Appendix A1.2. As Figure 6-29 shows, extratropical peak water levels increase from east to west. Because the New York and New Jersey land masses effectively funnel water to the west as winds are typically from the east. This trend is expected for extratropical events. For return periods smaller than 50 years, the tropical peak water levels also decrease from west to east. However, peak tropical water levels for return periods greater than 50 years increase alongshore to the east of Shinnecock Inlet.

Peak 6-year combined ocean water level slowly varies from about 5 ft to 7 ft, increasing from east to west. The 6-year water level is dominated by extratropical events, whose peak water level also varies within the same range. Around the 25-year return period at eastern stations and around the 50-year return period at stations in the western FIMP area, extratropical and tropical events nearly equally contribute to the combined ocean peak water level along the project length. Peak combined water level for the 50-year return period varies from about 7.5 ft in the eastern project area to about 9 ft in the western project area. At the 100-year return period, the contributions to the combined stage-frequency estimate for extratropical and tropical events are still nearly equal for stations west of Moriches Inlet. In contrast, combined peak water levels are dominated by tropical events to the east of Moriches Inlet. In this region, tropical peak 100-year water levels are about 2 ft to 3 ft higher than extratropical peak 100-year water levels. Combined peak 100-year water levels vary from 9 ft to 10.5 ft in the project area, where the water level slowly increases easterly and westerly about Moriches Inlet.

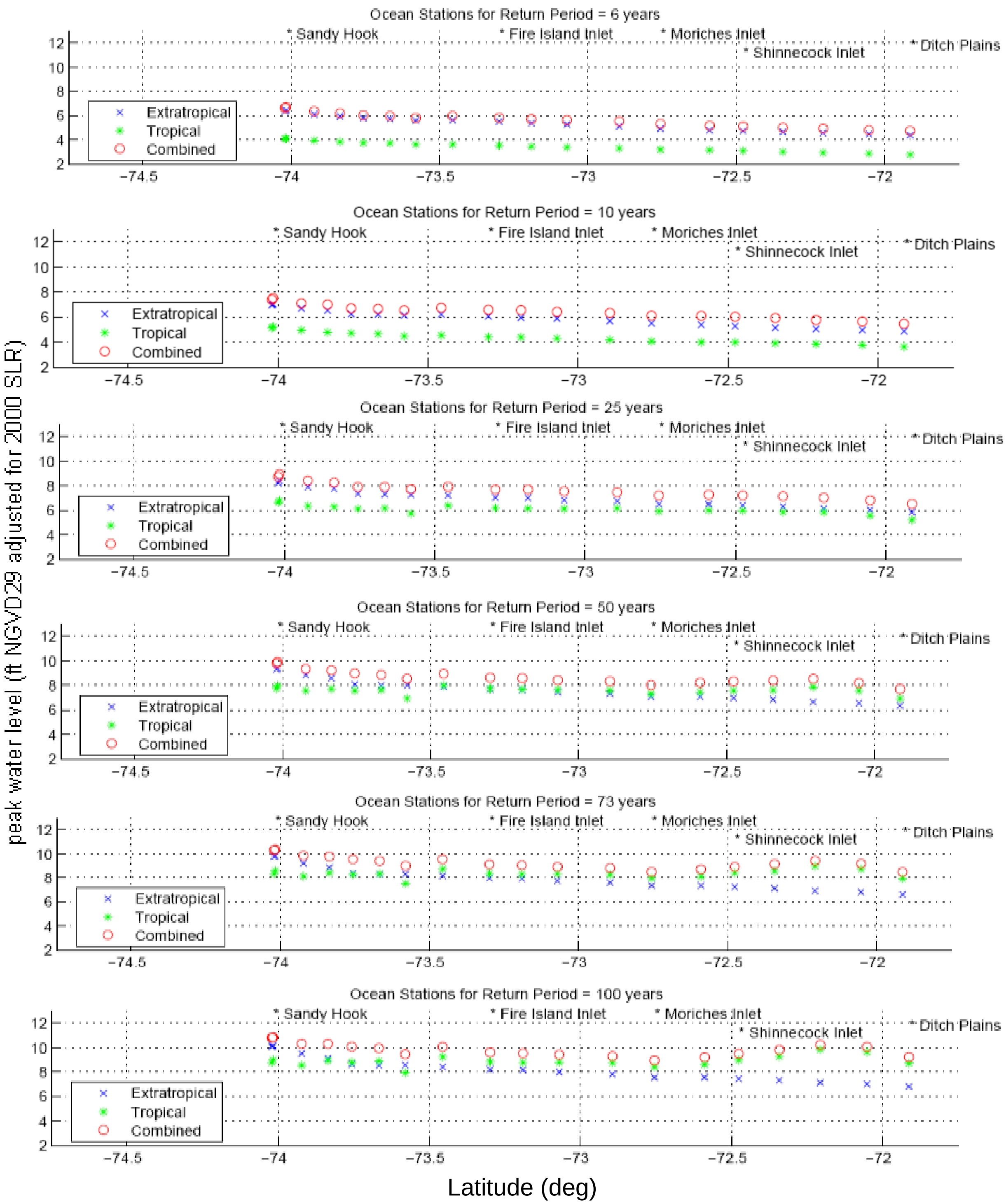


Figure 6-29. Return period and spatial distribution of peak ocean water levels (without wave setup).

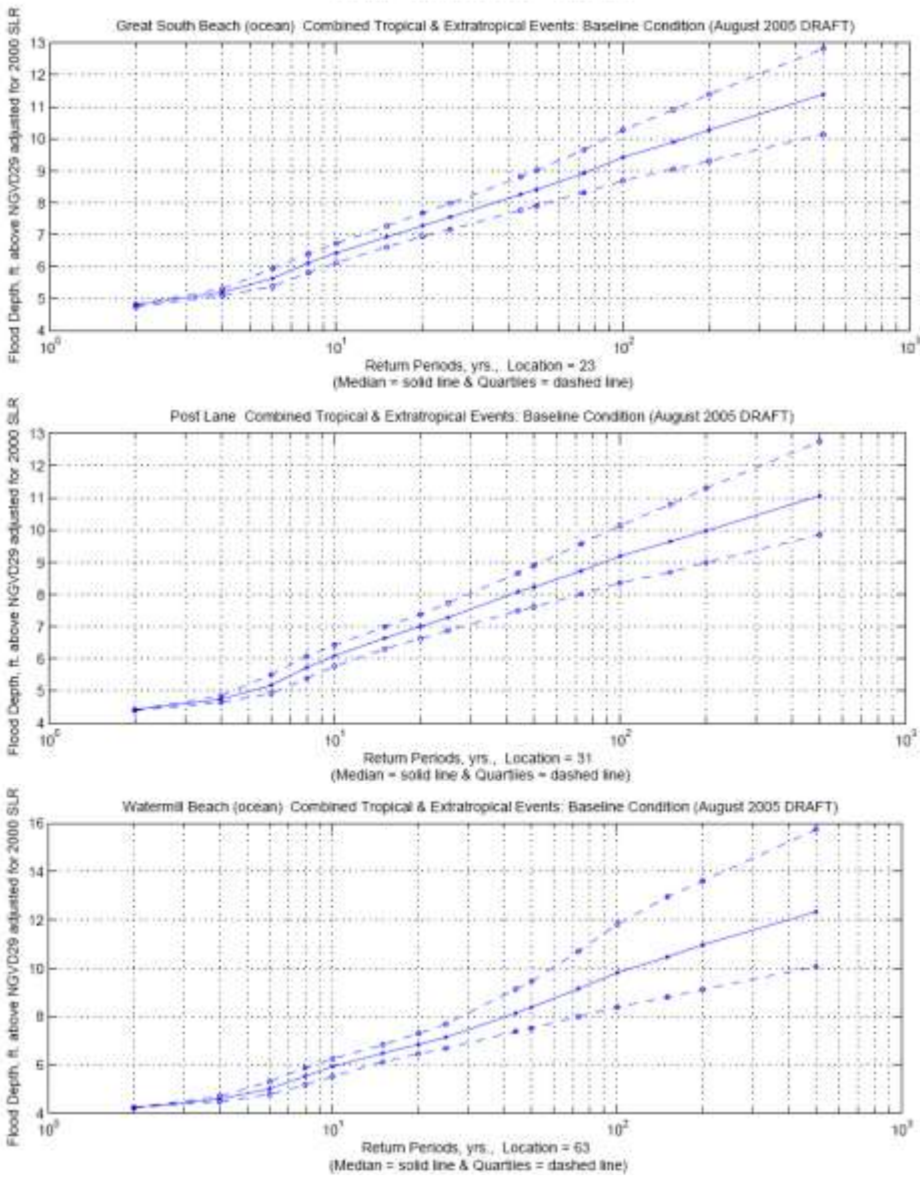


Figure 6-30. Ocean combined-storm stage-frequency relationships (without wave setup) in the vicinity of Fire Island (top), Westhampton (center) and the Ponds (bottom).

6.1.5.1 Ocean Wave Setup

Ocean wave setup is an important physical process for simulating storm water level and barrier island morphology during storm events. The additional contribution to total water level at the shoreline from wave setup is on the order of 20% of the nearshore wave height. This additional contribution is sizable for major storms impacting the south shore of Long Island. The contribution to total water level at the ocean shoreline due to wave setup was estimated using SBEACH.

Because wave setup varies with profile shape, the peak wave setup at the instantaneous shoreline for a particular storm varies with alongshore location. As such, ocean wave setup for a given alongshore region is presented here as a range. Figure

6-31 through Figure 6-38 show stage-frequency relationships with and without ocean wave setup for 8 ocean stations along the project length.

On average, ocean wave setup adds 2 to 3 ft to the entire stage-frequency curve. Variability in the ocean wave setup contribution due to profile shape increases as return period increases. For most cases, this variability is 0.5 to 1 ft about the average ocean wave setup result for return periods greater than 50 years.

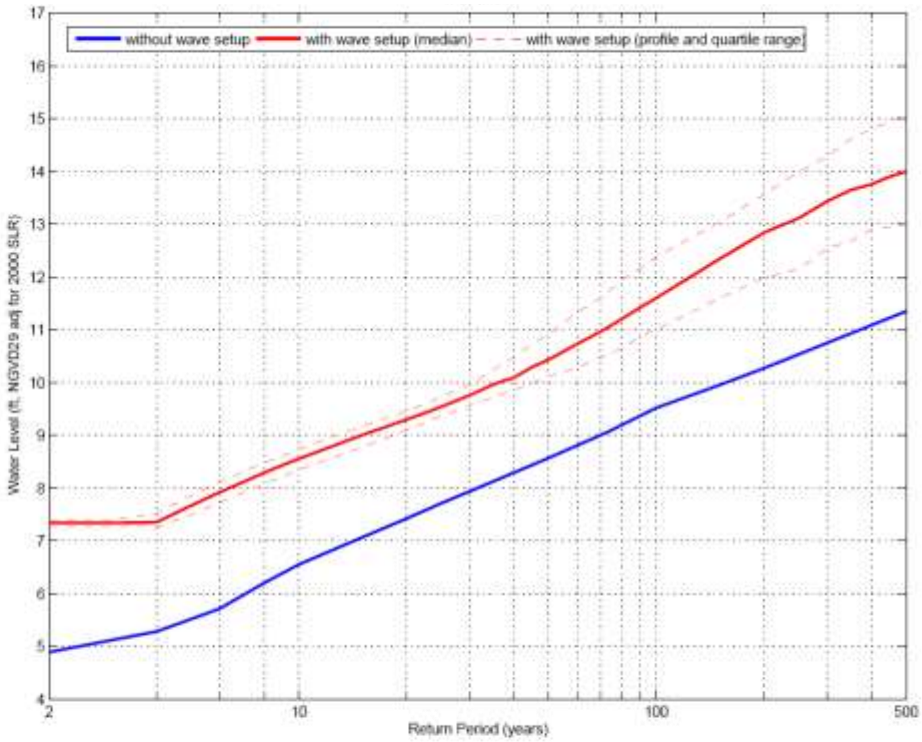


Figure 6-31. Stage-frequency relationships with and without ocean wave setup for Station 41, Great South Beach.

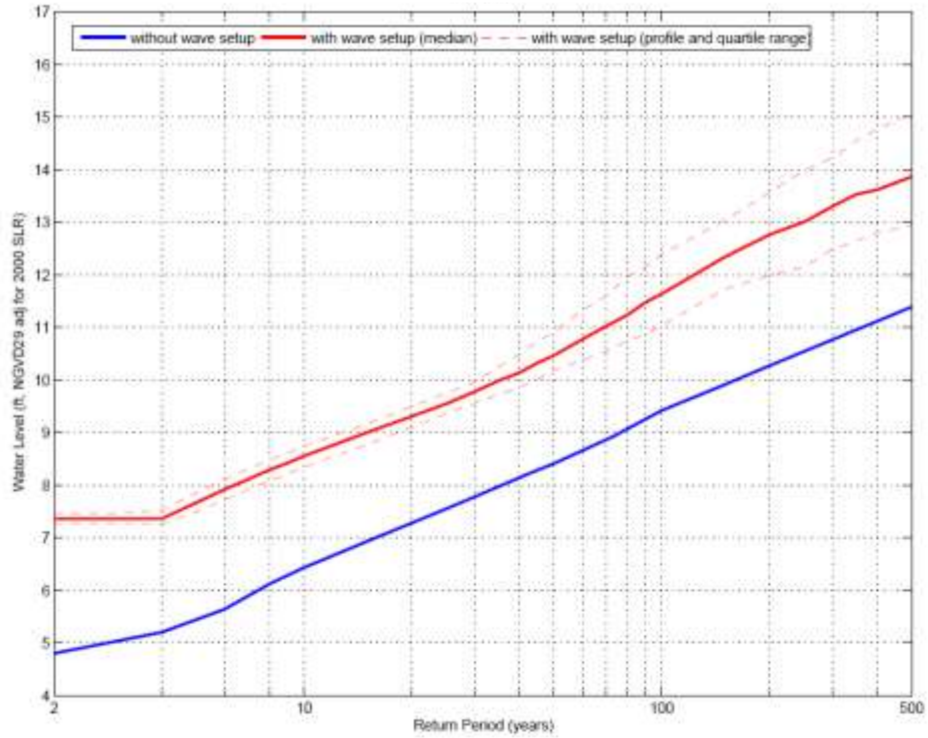


Figure 6-32. Stage-frequency relationships with and without ocean wave setup for Station 23, Great South Beach.

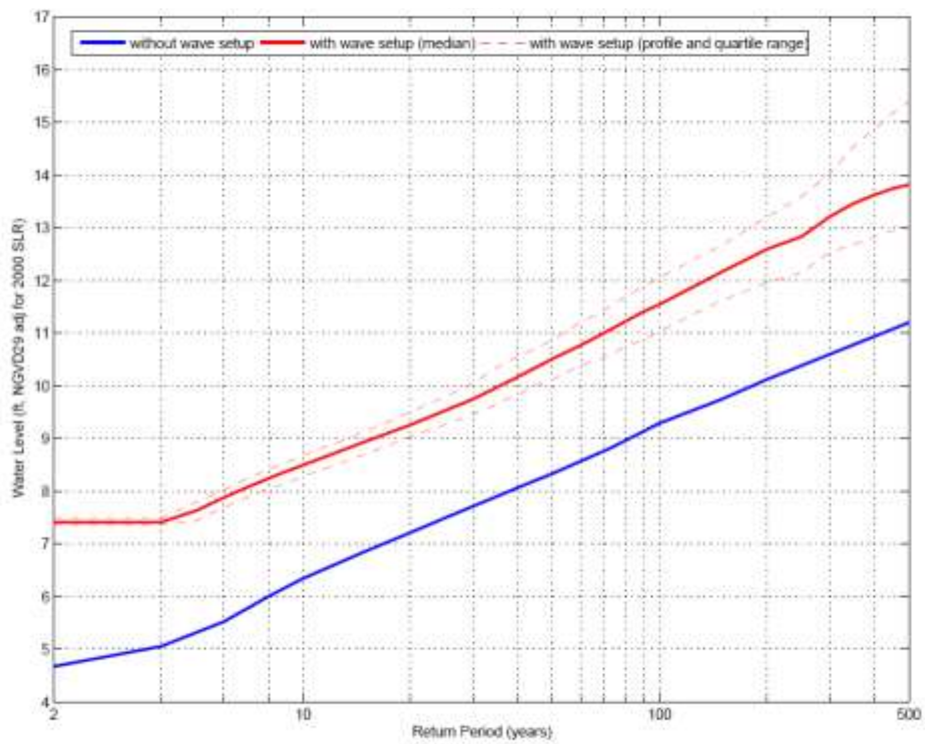


Figure 6-33. Stage-frequency relationships with and without ocean wave setup for Station 9, Old Inlet.

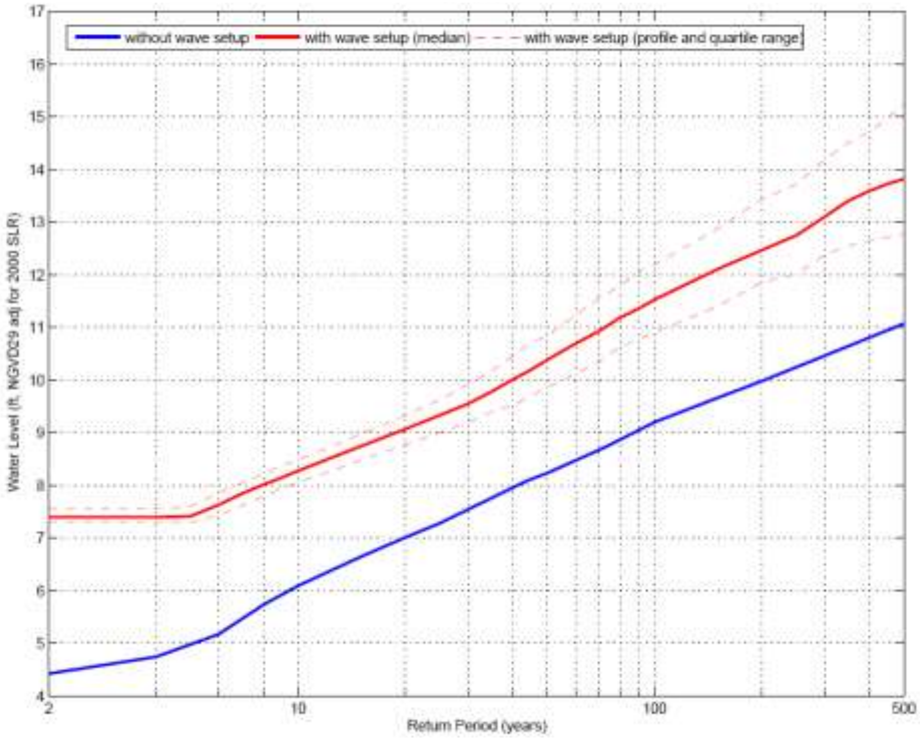


Figure 6-34. Stage-frequency relationships with and without ocean wave setup for Station 31, Post Lane.

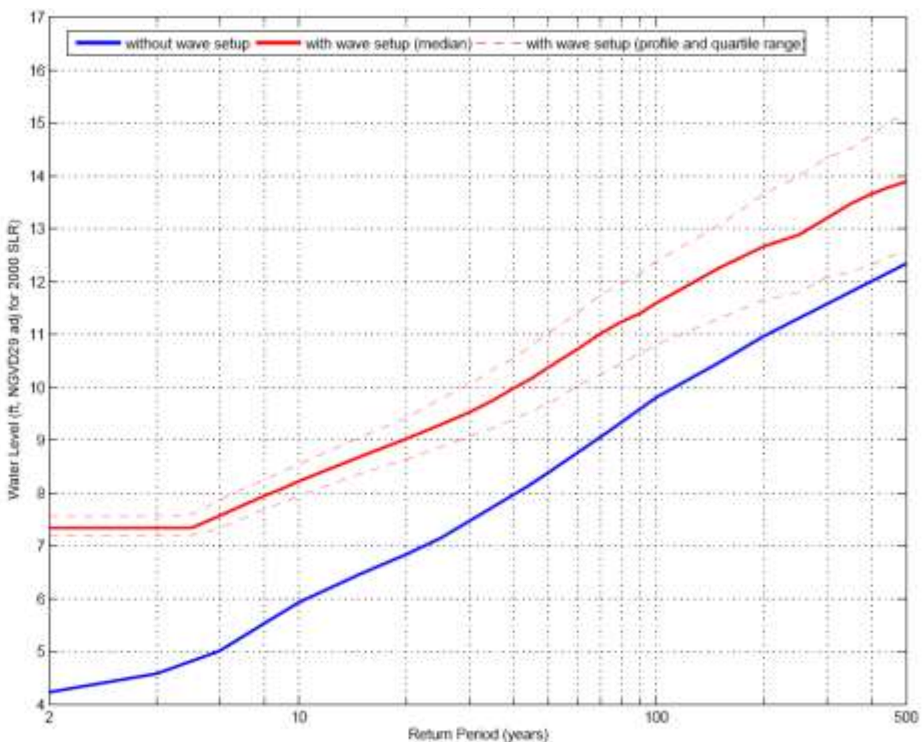


Figure 6-35. Stage-frequency relationships with and without ocean wave setup for Station 63, Watermill Beach.

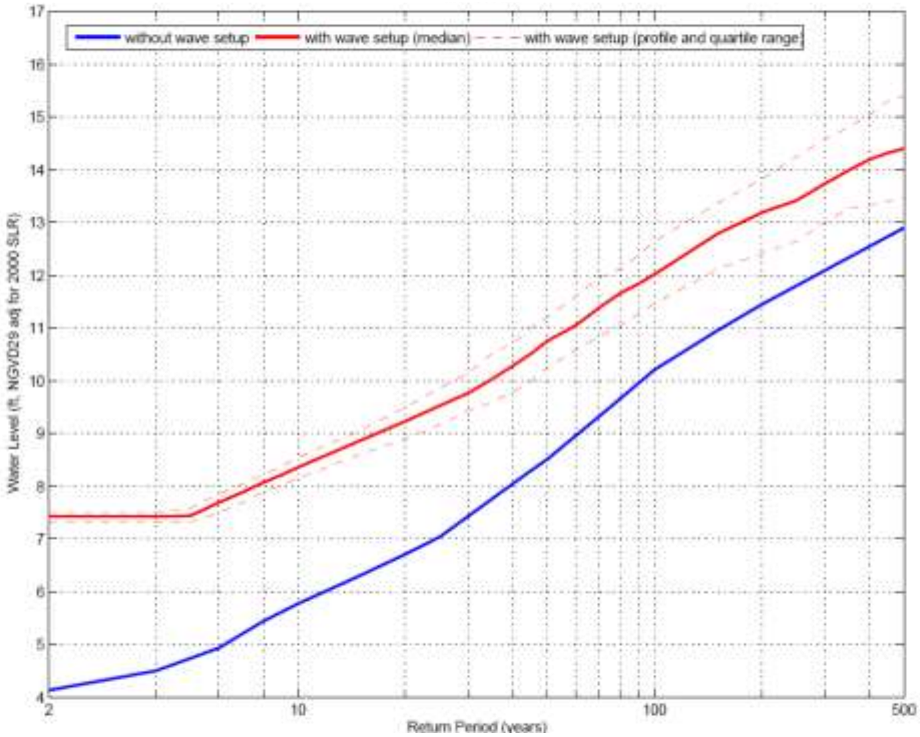


Figure 6-36. Stage-frequency relationships with and without ocean wave setup for Station 38, Apaquogue.

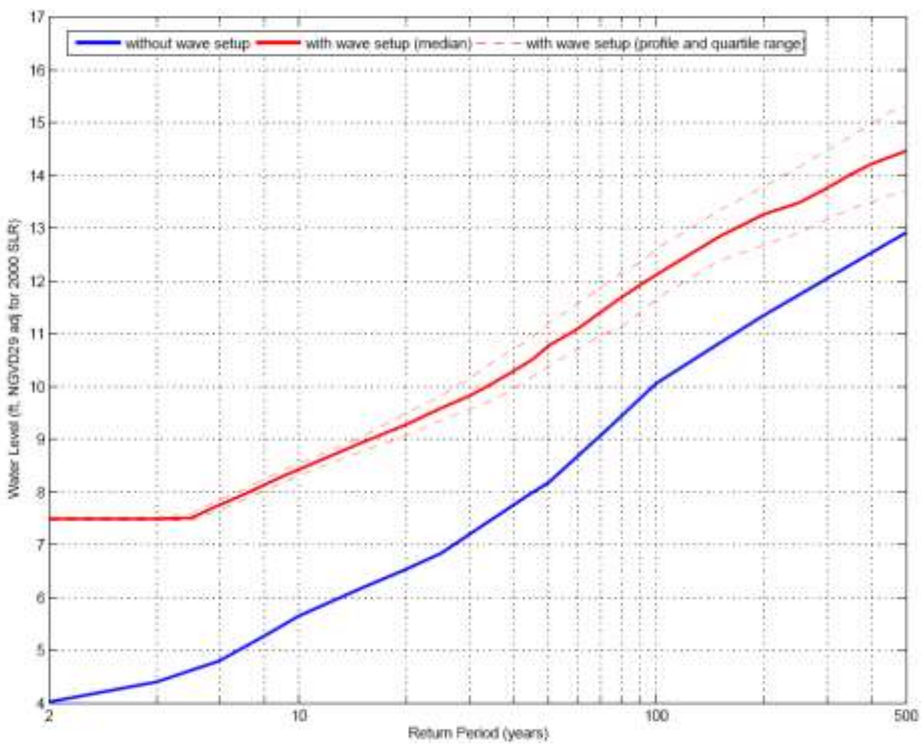


Figure 6-37. Stage-frequency relationships with and without ocean wave setup for Station 64, Napeaque Beach.

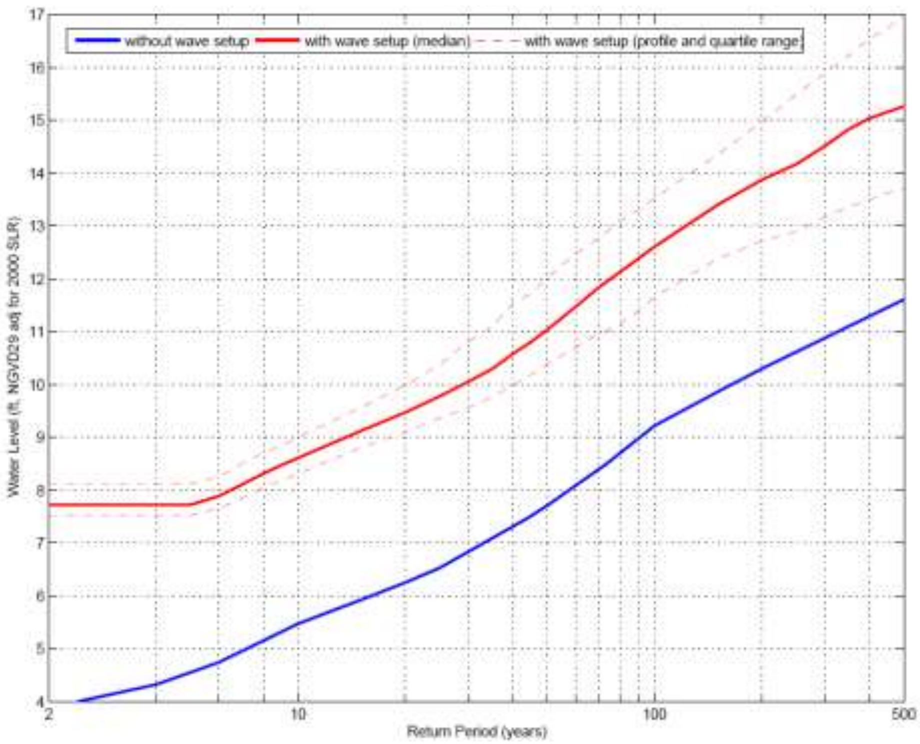


Figure 6-38. Stage-frequency relationships with and without ocean wave setup for Station 39, Ditch Plains.

6.1.5.2 Bay Wave Setup

Locally-generated bay wave setup for each storm in the FIMP training set was estimated from wave characteristics simulated with the Delft model SWAN. The average ratio of wave setup to significant wave height for all backbay locations and storm events is approximately 15%. In Great South Bay, bay wave setup ranges from 0.04 to 0.7 ft for all historical storms. In Moriches and Shinnecock Bays, bay wave setup ranges from 0.04 to 1.0 ft and 0.08 to 0.5 ft for historical tropical and extratropical storms, respectively. For this study, the sum of bay stage and bay wave setup is assumed to represent all contributions to the quasi-steady-state total water level.

6.1.6 Without Project Bay Stage-Frequency Relationships

Storm surge and storm-induced barrier island overwash and breaching were simulated with the surge modeling suite to develop Without Project Conditions stage-frequency relationships. Without Project Conditions, as described in Chapter 4.6, comprises the following topographic scenarios: Baseline (BLC, represented by 2000 lidar), Future Vulnerable (FVC), Breach Open (BOC), and Breach Closed (BCC). Numerical simulations of storm surge were performed and stage-frequency relationships were developed separately for each scenario. The BLC, FVC, and BCC stage-frequency relationships quantify the range of stages possible within the FIMP area during a 50-year period that allows for a Breach Contingency Plan but does not allow for a preemptive storm-damage reduction project. Additionally, the BOC results capture the range of expected water levels within the project area in the absence of a breach closure plan that would allow for immediate breach closure. See Section 6.1.4 for a discussion on stage-frequency uncertainty.

Figures 6-39 and 6-40 show the differences in the stage frequency curves for a representative location in Great South Bay, Moriches Bay and Shinnecock Bay. Two sets of curves are provided for each station. The first set compares baseline conditions, with project, future vulnerable conditions, and breach closed conditions illustrating the impact the pre-storm barrier island topography has on bay water levels. The second set compares the baseline condition, pre-Sandy baseline condition, and various breach open conditions illustrating the impact unclosed breaches on bay water levels. The subsections below summarize the stage-frequency relationship results for each topographic scenario. Additional details on each scenario may be found in Sub Appendix A1.

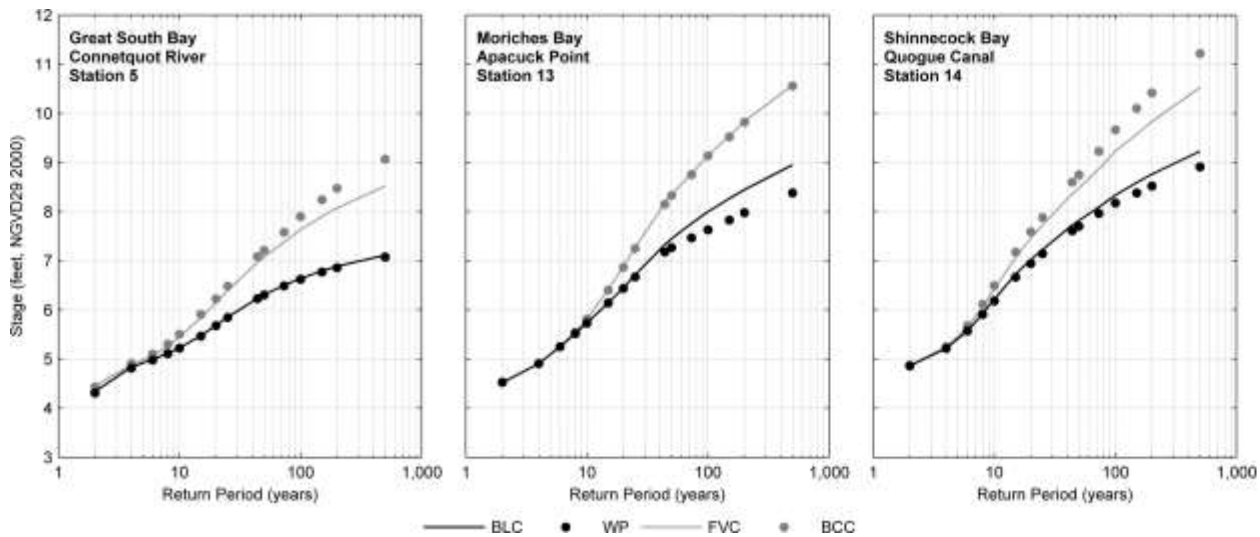


Figure 6-39. Comparison between BLC, FVC, WP, and BCC stage-frequency curves.

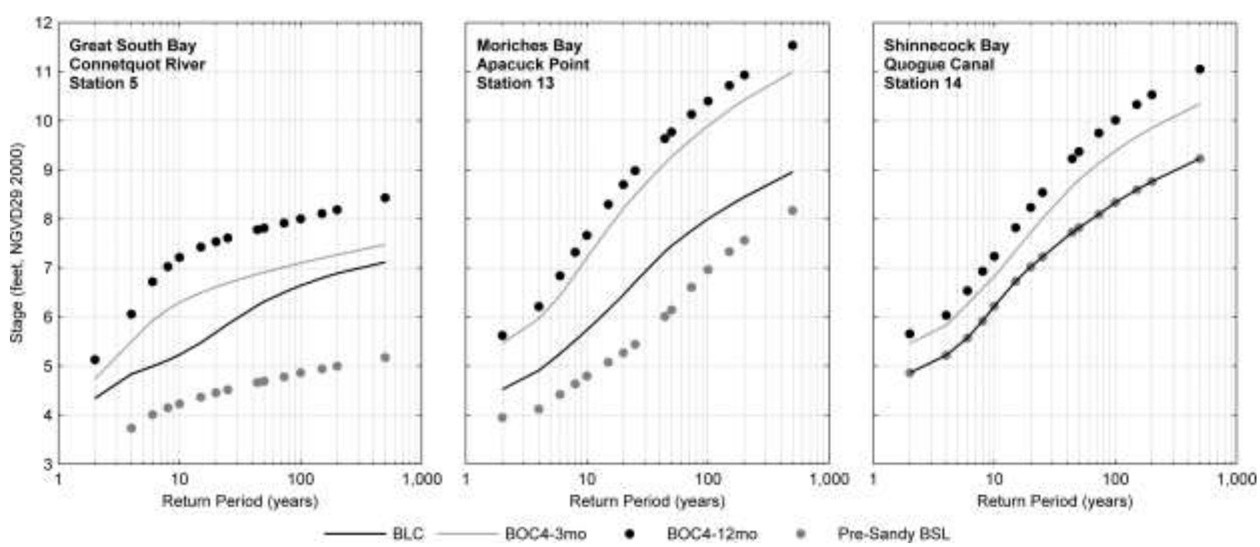


Figure 6-40. Comparison between BLC, BOC4-3mo, BOC4-12mo, and Pre-Sandy BSL stage-frequency curves.

6.1.6.1 Baseline Conditions (BLC)

In Great South Bay, peak water levels at all return periods are spatially consistent in that the values slowly vary from east to west. For all return periods, extratropical events are the dominating contributor to the combined stage-frequency estimate at all Great South Bay locations except stations 8 and 25 at the far eastern end of the bay. This is indicative of the hydraulic inefficiency of Fire Island Inlet. Numerical modeling simulations for this study show that Great South Bay is slow to respond to water level changes in the ocean. Consequently, water levels in this bay do not respond as dramatically to faster-moving tropical events as they do to the longer-duration extratropical events. The peak water levels in this bay are generally much lower than those computed for the same return period at ocean stations.

Stage-frequency results in Moriches Bay are generally higher than those in Great South Bay as this bay more readily responds to ocean conditions. The combined stage-frequency curves are dominated by extratropical events for return periods below 25 years. However, extratropical and tropical events more equally contribute to the combined relationships for return periods of 50 years and larger. This demonstrates that Moriches Bay responds more quickly to fast changes in ocean water level.

Of the three bays within the FIMP area, Shinnecock Bay is characterized by the highest peak water levels. Furthermore, Shinnecock Bay is more influenced by tropical events for larger return periods. This is a direct consequence of the relative efficiency of Shinnecock Inlet and the stage-frequency trends along the ocean. Near Shinnecock Inlet, and at eastward ocean locations, the ocean combined stage-frequency relationships are dominated by tropical events for return periods larger than 50 years. This trend is carried through to the Shinnecock Bay combined stage-frequency relationships.

6.1.6.2 Future Vulnerable Conditions (FVC)

Bay storm stages are sensitive to pre-storm barrier island topography with measurable differences between BLC and FVC ranging from 0.5 ft and 1.5 ft. These differences are directly attributed to the additional volume of water entering the bays during storm-induced barrier island overwash and breaching. Furthermore, these differences make up a significant portion (15% to more than 50%) of the observed differences between ocean and bay stage-frequency relationships under BLC.

The first influence of additional flow contributions over the barrier island generally occurs around the 10- to 20-year return period. Combined stage differences between FVC and BLC are most dramatic in western Moriches Bay. Here, combined stages are increased under FVC by 1.0 ft to 1.5 ft between the 18- and 100-year return periods.

6.1.6.3 Breach Closed Conditions (BCC)

Bay storm stages are generally higher, by 0.25 ft to 0.75 ft, under BCC than under FVC in Great South and Shinnecock Bays for tropical storms. BCC bay stages for tropical events vary little from those for FVC in Moriches Bay. In all bays, BCC extratropical stages are similar to those for FVC.

Differences between the BCC and FVC tropical stage-frequency relationships are most significant in Great South Bay. In Great South Bay, the BCC tropical relationship is as much as 0.75 ft higher than that for FVC for return periods between 25 years and 100 years for all stations except 1 and 2, at the far western end of the bay. Combined relationships in Great South Bay accordingly reflect the tropical relationships.

6.1.6.4 Breach Open Conditions (BOC)

Bay storm water levels are sensitive to open barrier island breaches with measurable differences between 0.5 ft and 3 ft for the simulated breach open scenarios, even when the breach opening is small (e.g. 3-month cases). Furthermore, these differences make up nearly all (15% to more than 100%) of the observed differences between ocean (without ocean wave setup) and bay stage-frequency relationships under BLC.

In Great South Bay, open breaches increase storm water levels even for small storm events. BOC 1 and BOC 3, representing one open breach in eastern and central Great South Bay, respectively, similarly increase storm water levels throughout the bay, relative to Baseline Conditions. These 3-month openings result in storm water levels that are about 0.5 ft to 1.5 ft higher than under BLC, for all return periods.

When a breach is open directly into Moriches Bay, storm water levels are significantly higher than BLC. Under BOC 2 with a 3-month opening, storm water levels are 1 to 2 ft higher than BLC for all return periods at all locations within Moriches Bay. Moriches Bay water levels are also measurably increased when a breach is open in Great South Bay due to flow through Narrow Bay. Under BOC 1 and BOC 3, with a 3-month opening, storm water levels in Moriches Bay are about 0.25 to 1 ft higher than BLC.

In Shinnecock Bay, when there is a breach open west of Shinnecock Inlet (BOC 3 and BOC 4), under the 3-month scenario, storm water levels are 0.5 to 2 ft higher than BLC water levels at all Shinnecock Bay stations for all return periods. Shinnecock Bay is most sensitive to a breach into the western part of the bay, as depicted by BOC 1 and BOC 2. For the 3-month opening, both BOC 1 and BOC 2 storm water levels are about 0.5 to 2.5 ft above those for BLC.

Storm simulations using 6-month breach openings demonstrated that peak water level varies linearly with respect to the alongshore length of the breach. As such, stage-frequency relationships for the 6-month BOC scenarios is represented by the average of the 3- and 6-month scenarios.

Stage-frequency relationships with bay wave setup may be found in Sub Appendix A1.

6.1.7 Breaching and Overwash Frequency

Breaching/overwash-frequency relationships for the ten areas most vulnerable to overwash and breaching are presented in Figure 6-41 through Figure 6-50 and combined frequency results are tabulated in Table 6-3. These relationships reflect the storm morphological responses simulated by Delft3D-MOR under the BLC, FVC, and BCC topographies. Based on these morphological modeling results, BLC overwash is expected to be a very frequent occurrence under all topographies at all vulnerable locations except Talisman to Blue Point Beach, Davis Park, and Sedge Island. Under FVC and BCC, overwash is also expected to be frequent at Talisman to Blue Point Beach and Sedge Island. With respect to all vulnerable areas and topographic conditions, partial and full breaching, both of which have the potential for permanent breach formation following a storm, is expected to be more frequent at Old Inlet (East and West), Smith Point County Park (SPCP), Tiana Beach, and West of Shinnecock Inlet (WOSI). The expected frequency of partial and full breaching at Kismet to Corneille Estates and Talisman to Blue Point Beach measurably increases under FVC and BCC. However, it is important to note that all ten vulnerable locations exhibit some risk of partial and full breaching under FVC and BCC scenarios.

Table 6-3. Combined breaching/overwash frequency values

	Fire Island Lighthouse Tract	Kismet to Corneille Estates	Talisman to Blue Point Beach	Davis Park	Old Inlet, West	Old Inlet, East	Smith Point County Park	Sedge Island	Tiana Beach	West of Shinnecock Inlet
Baseline Conditions (return period in years)										
Overwash	14 - 184	9 – 141	20 – 213	22 – 145	10 – 45	5 – 24	8 – 26	25 – 251	7 – 72	18 - 74
Partial Breaching	184 – 500	141 – 500	213 – 500	145 – 500	45 – 82	24 – 118	26 – 145	251 – 500	72 – 336	74 – 326
Full Breaching	> 500	> 500	> 500	> 500	> 82	> 118	> 145	> 500	> 336	> 326

FVC (return period in years)										
Overwash	3 – 34	5 – 15	5 – 12	15 – 73	4 -7	5 – 19	4 – 9	4 – 48	4 – 30	4 - 8
Partial Breaching	34 – 106	15 – 34	12 – 31	73 – 288	7 – 22	19 – 84	9 – 141	48 - 291	30 - 266	8 - 25
Full Breaching	> 106	> 34	> 31	> 288	> 22	> 84	> 141	> 291	> 266	> 25

Breach Closed (return period in years)										
Overwash	5 – 21	5 – 17	5 – 39	12 – 26	4 – 12	5 – 34	5 – 20	4 – 66	4 – 44	5 - 18
Partial Breaching	21 – 43	17 – 37	39 -80	26 – 108	12 – 67	34 – 191	20 – 139	66 – 291	44 – 264	18 - 60
Full Breaching	> 43	> 37	> 80	> 108	> 67	> 191	> 139	> 291	> 264	> 60

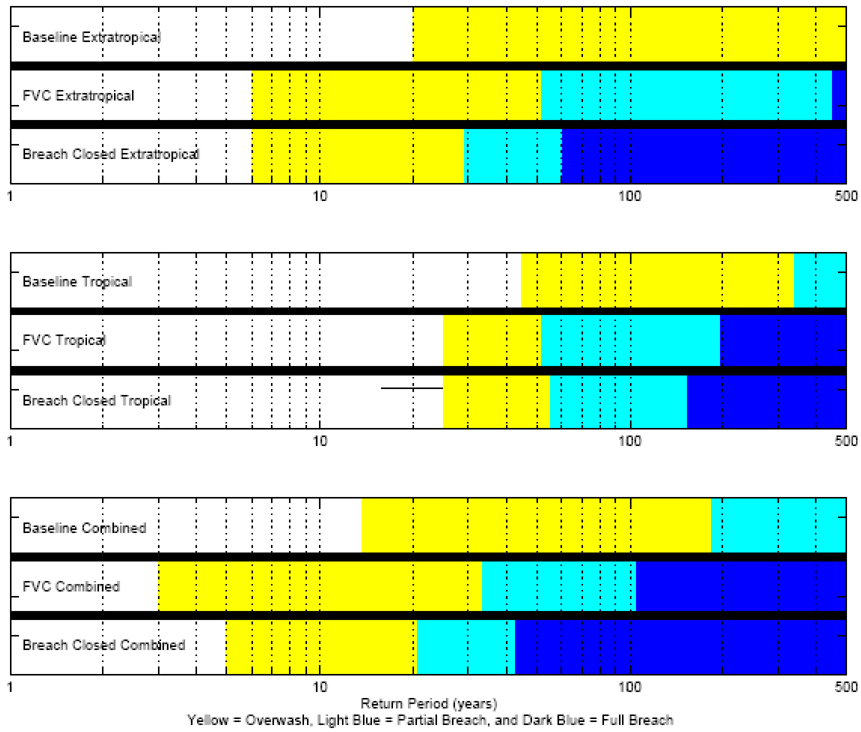


Figure 6-41. Breaching/overwash frequency relationships at Fire Island Lighthouse Tract for BLC, FVC, and BCC (BrCl)

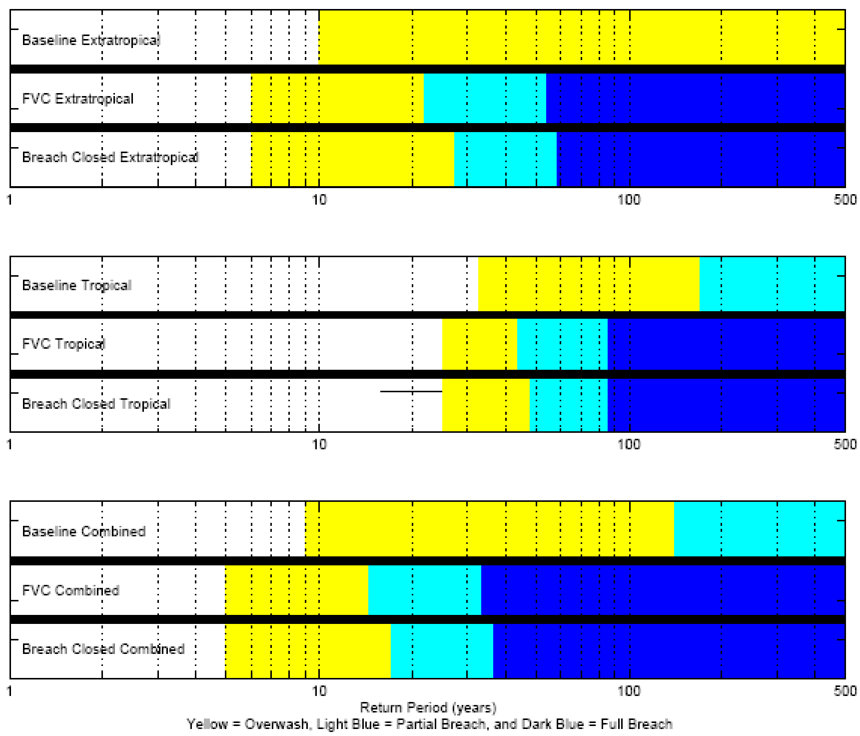


Figure 6-42. Breaching/overwash frequency relationships at Kismet to Corneille Estates for BLC, FVC, and BCC (BrCl)

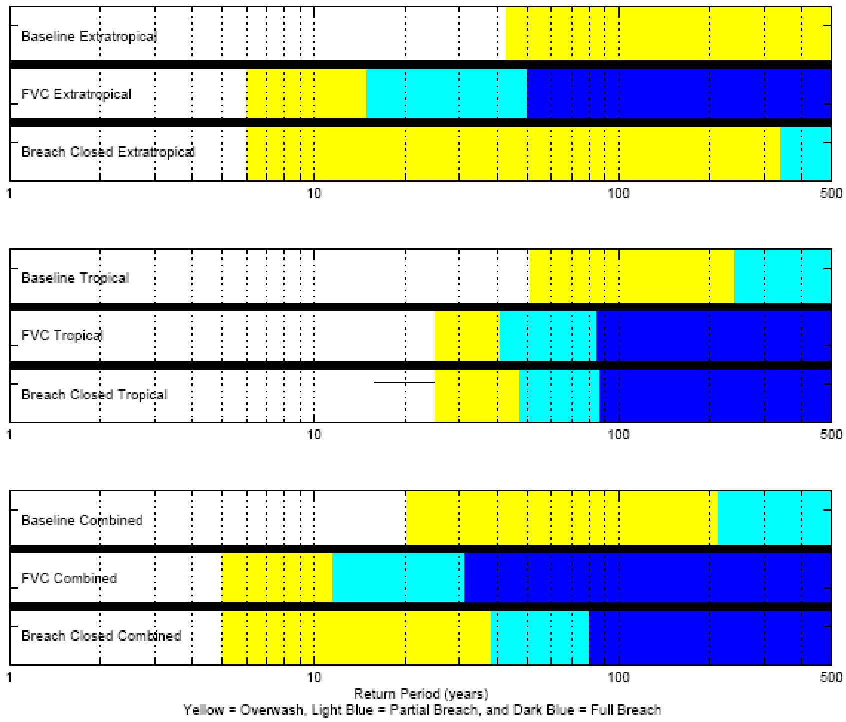


Figure 6-43. Breaching/overwash frequency relationships at Talisman to Blue Point Beach for BLC, FVC, and BCC (BrCl)

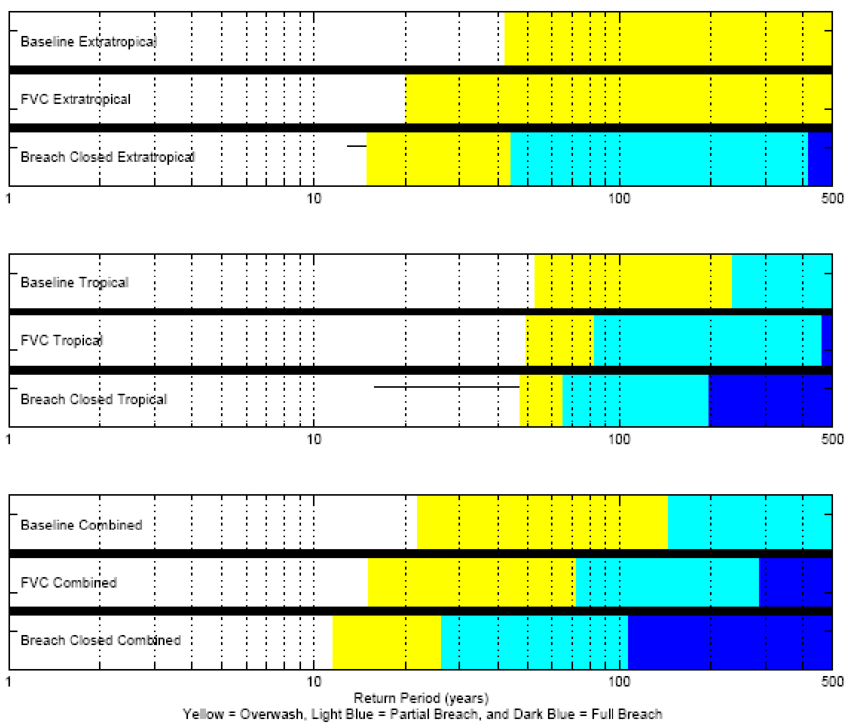


Figure 6-44. Breaching/overwash frequency relationships at Davis Park for BLC, FVC, and BCC (BrCl)

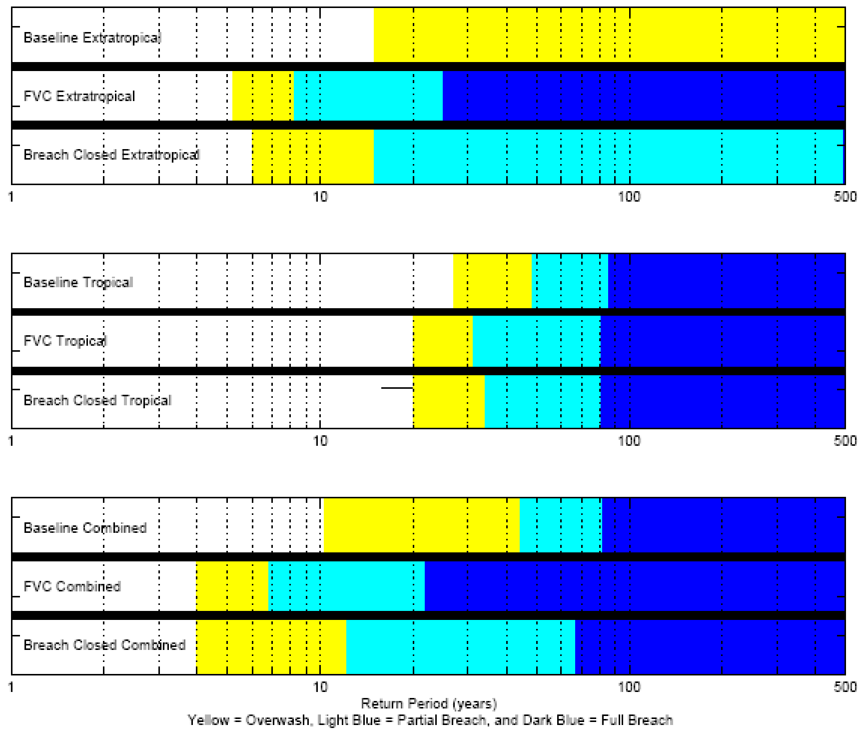


Figure 6-45. Breaching/overwash frequency relationships at Old Inlet, West for BLC, FVC, and BCC (BrCl)

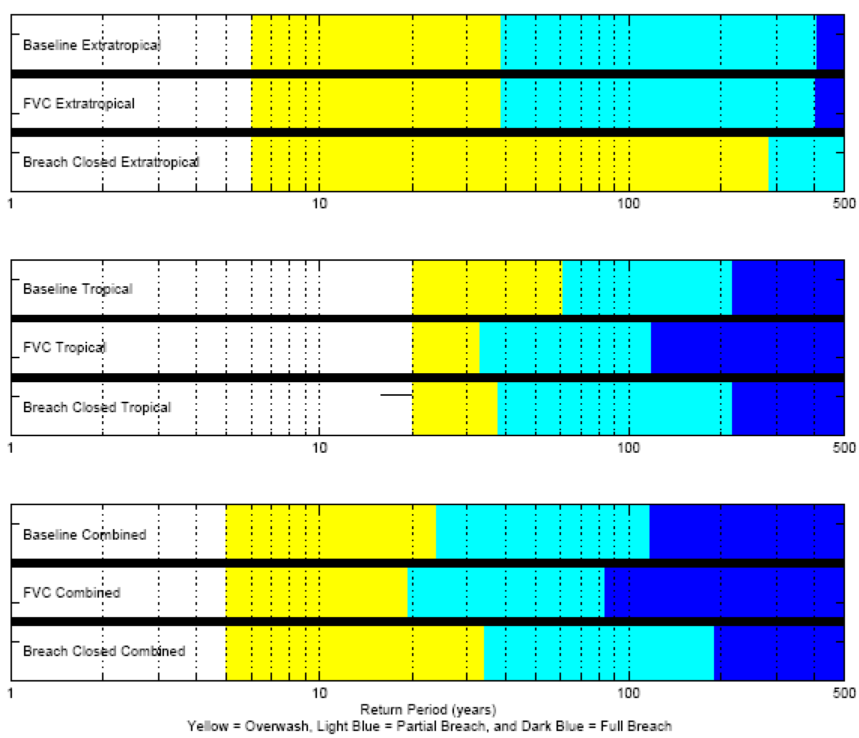


Figure 6-46. Breaching/overwash frequency relationships at Old Inlet, East for BLC, FVC, and BCC (BrCl)

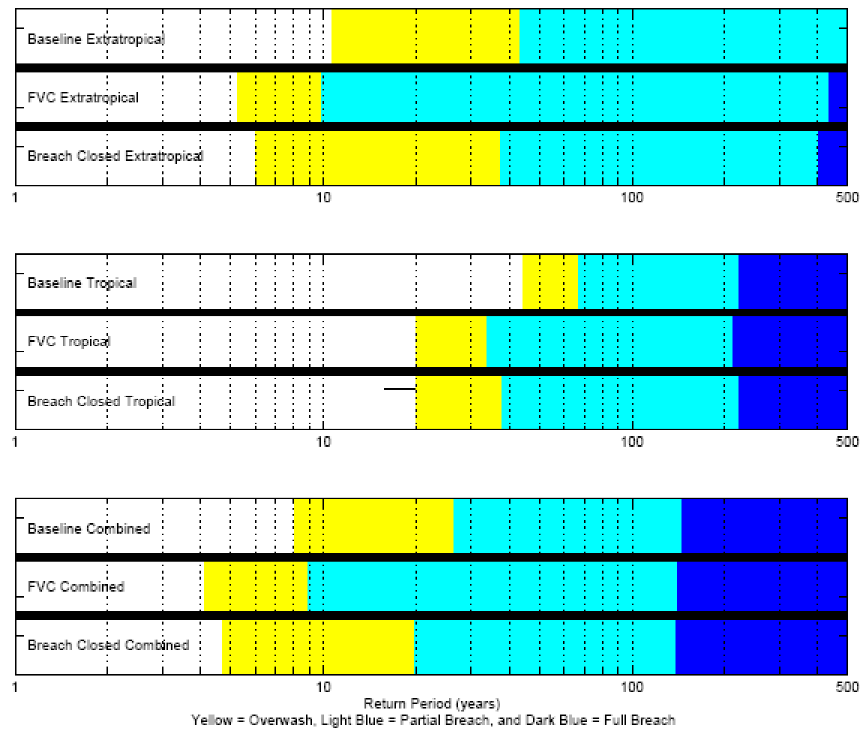


Figure 6-47. Breaching/overwash frequency relationships at Smith Point County Park for BLC, FVC, and BCC (BrCl)

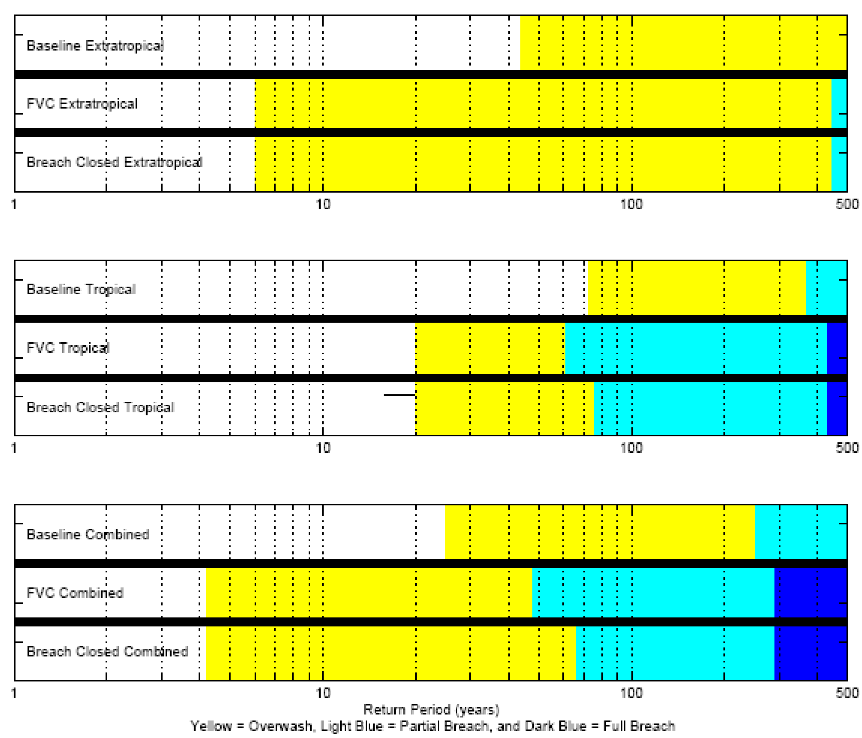


Figure 6-48. Breaching/overwash frequency relationships at Sedge Island for BLC, FVC, and BCC (BrCl)

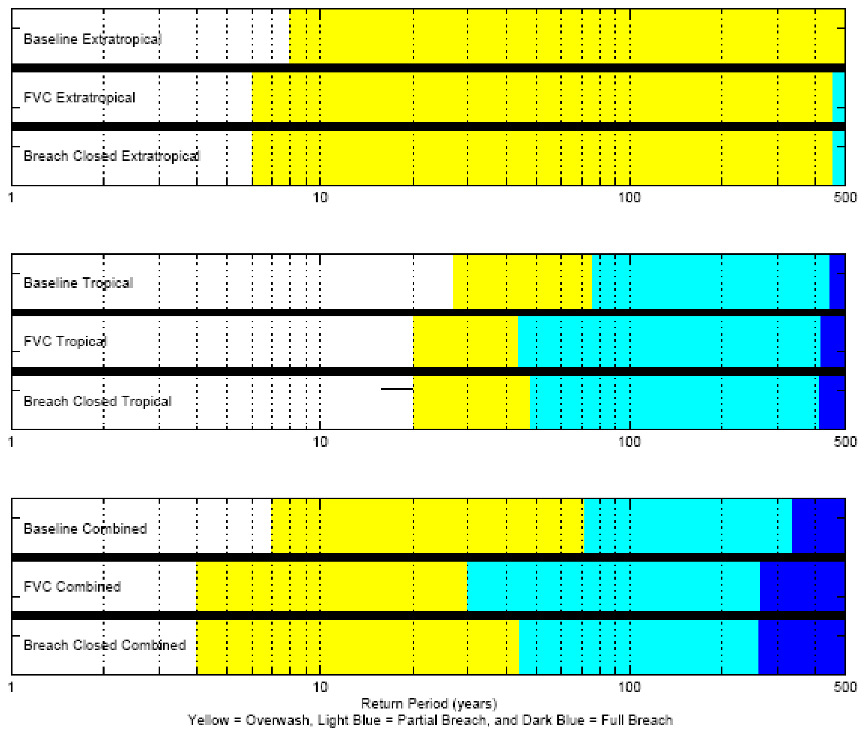


Figure 6-49. Breaching/overwash frequency relationships at Tiana Beach for BLC, FVC, and BCC (BrCl)

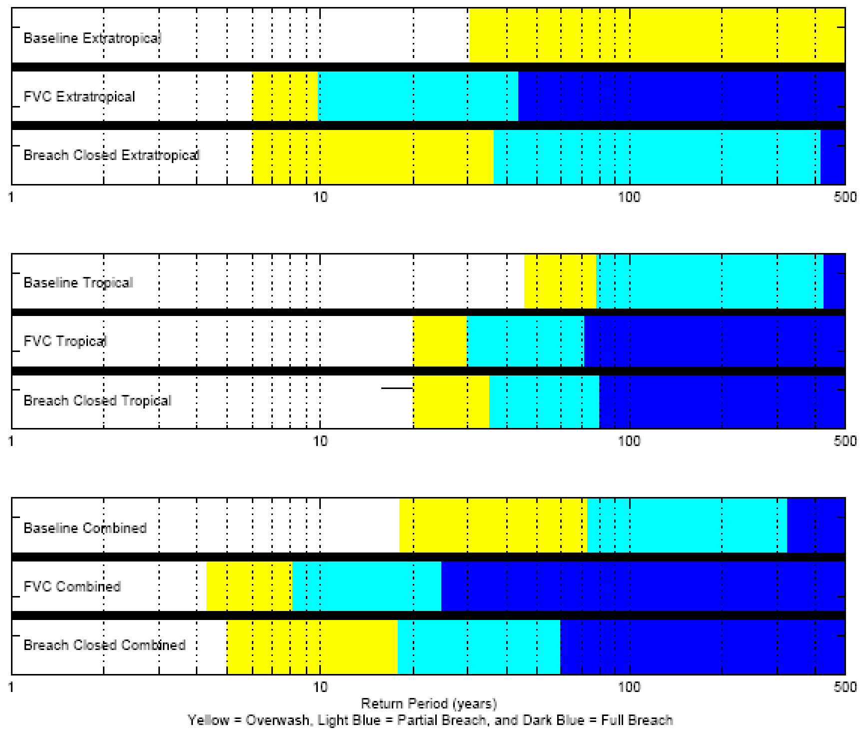


Figure 6-50. Breaching/overwash frequency relationships at West of Shinnecock Inlet for BLC, FVC, and BCC (BrCl)

6.2 Storm-Induced Beach Erosion

In addition to its use in storm water level numerical modeling, SBEACH was also used to evaluate storm-induced beach profile response throughout the study area on both Baseline Conditions and Design conditions. These simulated beach profile responses were required as input to the economics model for evaluating shorefront damages. A discussion of SBEACH model theory and set up for FIMP is provided in Chapter 6.1.

This SBEACH work was performed to develop response-frequency relationships for the following 10 morphological responses:

- Erosion distance from the 0 ft NGVD29 location on the initial (pre-storm) profile to the landward-most point of 1 ft of vertical erosion or accretion.
- Eroded volume above 0 ft NGVD29
- Eroded volume above +10 ft NGVD29
- Vertical erosion distance of dune crest
- Landward translation of dune crest
- Active profile distance taken as the distance between the 0 ft NGVD29 location on the initial profile to the landward limit of profile change.
- Recession of the 0 ft NGVD29 location.
- Recession of the +5 ft NGVD29 location.
- Recession of the +10 ft NGVD29 location.
- Recession of the +15 ft NGVD29 location.

These response parameters are dependent on profile shape; therefore, they vary with morphological subreach (Figure 3-5). To determine the morphological response for each of the morphological subreaches, the representative beach profiles discussed in Chapter 3.1.5 and presented in Figure 3-1 through Figure 3-4 were used to represent the initial pre-storm condition.

6.2.1 Storm Selection

To develop response-frequency relationships that capture the range of expected morphological responses due to hurricanes and Nor'easters, the storm training set for the SBEACH simulations included multiple alternate tide scenarios for each historical storm listed in Table 3-4. Like storm water level, morphological response for a given storm varies considerably depending on the tide conditions. As discussed in Chapter 6.1.2, storm water level variation with tide conditions may be reasonably estimated by linear superposition of tide and surge for most scenarios. However, there is no such direct way to estimate morphological response for all possible surge-tide combinations. As such, 16 discrete tide conditions, for each storm, were simulated with SBEACH to capture the range of morphological response variability.

The spring, neap, and mean tidal ranges, determined from a 20-year equilibrium tide record and generated from simulated ADCIRC tidal constituents, are on average 3.05 ft, 1.58 ft, and 2.26 ft, respectively. During the 28-day lunar cycle, spring and neap conditions each occur once. The mean condition occurs twice, once during the transition from spring to neap conditions and once again during the transition from neap back to spring conditions. As such, in the response-frequency analysis the mean conditions listed above are given twice as much weight (i.e. counted 2 times each) as is given to the spring or neap conditions. Four equally weighted phases of the tide (high tide, low tide, rising tide, and falling tide) were simulated for each of the three tide ranges. In summary, the surge and wave input hydrographs for each storm were aligned so that peak surge coincided with the following 12 tide and lunar phases:

High spring tide (weight = 1/16)
Low spring tide (weight = 1/16)
Rising spring tide (weight = 1/16)
Falling spring tide (weight = 1/16)
High neap tide (weight = 1/16)
Low neap tide (weight = 1/16)
Rising neap tide (weight = 1/16)
Falling neap tide (weight = 1/16)
High mean tide (weight = 2/16)
Low mean tide (weight = 2/16)
Rising mean tide (weight = 2/16)
Falling mean tide (weight = 2/16)

As with the simulations made for storm surge modeling (Chapter 6.1), hydrodynamic input surge hydrographs were specified using ADCIRC output while wave height and period hydrographs were specified using WISWAVE output transformed to a 10-m depth.

6.2.2 Response-Frequency Methodology

The multivariate EST (Empirical Simulation Technique) was used to develop response-frequency relationships from the SBEACH output (Scheffner et al., 1996). Unlike the univariate (1-D) EST used for stage-frequency development (see Chapter 6.1.2) that accounts only for the expected variability of one parameter (i.e. water level), the multivariate EST accounts for the expected variability of more than one variable. In the multivariate EST application here, joint probability relationships inherent in the multidimensional dataset are used to simulate multiple future storm response histories, or lifecycles. These lifecycles, containing both tropical and extratropical events, were used directly as inputs to the economic models and as the basis for response-frequency generation. As with the univariate EST, the multivariate EST does not presuppose a parametric probability distribution. Instead it used nonparametric methods (see discussion on parametric and nonparametric methods). The multidimensional dataset, and the corresponding lifecycle analyses, include the morphological response variables listed in Chapter 6.2 plus input hydrodynamic variables such as wave height and period, storm surge, and tide phasing plus response hydrodynamic variables such as wave setup and wave runup.

6.2.2.1 Special Treatment of the Lower Return Period Distribution

As with stage-frequency development, the peaks-over-threshold storm training set alone does not adequately capture the expected morphological responses associated with small storms (i.e. return periods less than 10 years). As such, a set of representative “annual events” were added to the analysis to better define the response-frequency relationships for small return periods. In all, four small extratropical events occurring in the 1990s were selected by analyzing NOAA water level records at Sandy Hook and Montauk and NDBC wave measurements at NDBC Buoy 44025: January 1998, October 2002, February 2003, and March 2004. Based on data analyses, each of these four storms is a representation of the average annual condition as determined by ranking of measurements of annual maximum events. Following SBEACH simulation of these four storms, each morphological response was averaged to produce a single estimate of the expected annual (1-year return period) responses. These values were then inserted into each of the future lifecycles for each time zero events in a year were predicted by the multivariate EST.

6.2.3 Baseline Conditions Response-Frequency Relationships

A subset of morphological response-frequency relationships, for combined (tropical and extratropical) events, for subreaches F-R2, F-R5, W-R5, P-R5, and M-R3 are presented in Figure 6-51 through Figure 6-69. Specifically, these figures show erosion distance, recession of the 0-ft NGVD location, eroded volume above the 0-ft NGVD elevation, and dune lowering for Baseline Conditions. Figure 6-51 through Figure 6-54 in comparisons with Figure 6-55 through Figure 6-58 show that morphological response for subreach F-R2 is much more dramatic than that for subreach F-R5. This highlights the variability in morphological response along Fire Island. Furthermore, it illustrates how variable the response is with respect to initial profile shape. As the inset in Figure 3-1 shows, Profile F-R2 has a narrower and lower dune with respect to the Profile F-R5 dune. As a result, Profile F-R2 is more susceptible to dune lowering.

Profile W-R5 (Figure 6-59 through Figure 6-62), within the Westhampton morphological reach, shows lower erosion distance and shoreline recession values for all return periods, relative to Profiles F-R2 and F-R5, indicating that this profile provides slightly more protection against landward translation of the shoreline during a storm. In addition, the higher, wider dune at this subreach, with respect to the two Fire Island subreaches, affords more protection against dune lowering, where no dune lowering occurs for annual chance exceedance of less than 1%.

Profile P-R5 (Figure 6-63 through Figure 6-66), in the Ponds morphological reach, responses indicate an increase in shoreline erosion and dune lowering with storm return periods higher than 30 years. With respect to the F-R2, F-R5, and W-R5, Profile P-R5 is characterized by a more steeply sloped offshore, while the dune height is comparable with W-R5. However, shorefront properties within this morphological subreach are significantly set back from the shoreline. As such, property damage vulnerability is small relative to other project locations.

Unlike other morphological reaches, the Montauk reach is characterized by high bluffs. Storm-induced erosion response to a profile characterized by a bluff varies somewhat from that of a dune. Specifically, the primary subaerial erosion mechanisms are undercutting of the bluff by waves and surge followed by bluff failure, which could best be described as avalanching (USACE, 2005). The failure is typically catastrophic instead of a slow erosion. Profile M-R3 is characterized by a high bluff (no dune) and a uniformly sloping offshore profile. Eroded volume loss for this profile varies linearly with the log of return period for return periods greater than 5 years (Figure 6-69). Similarly, shoreline recession varies linearly with the log of return period for return periods greater than 20 years (Figure 6-64). However, no shoreline recession is expected for return periods less than 20 years. This volume loss and shoreline recession may be interpreted as the volume lost from the bluff and bluff recession, respectively.

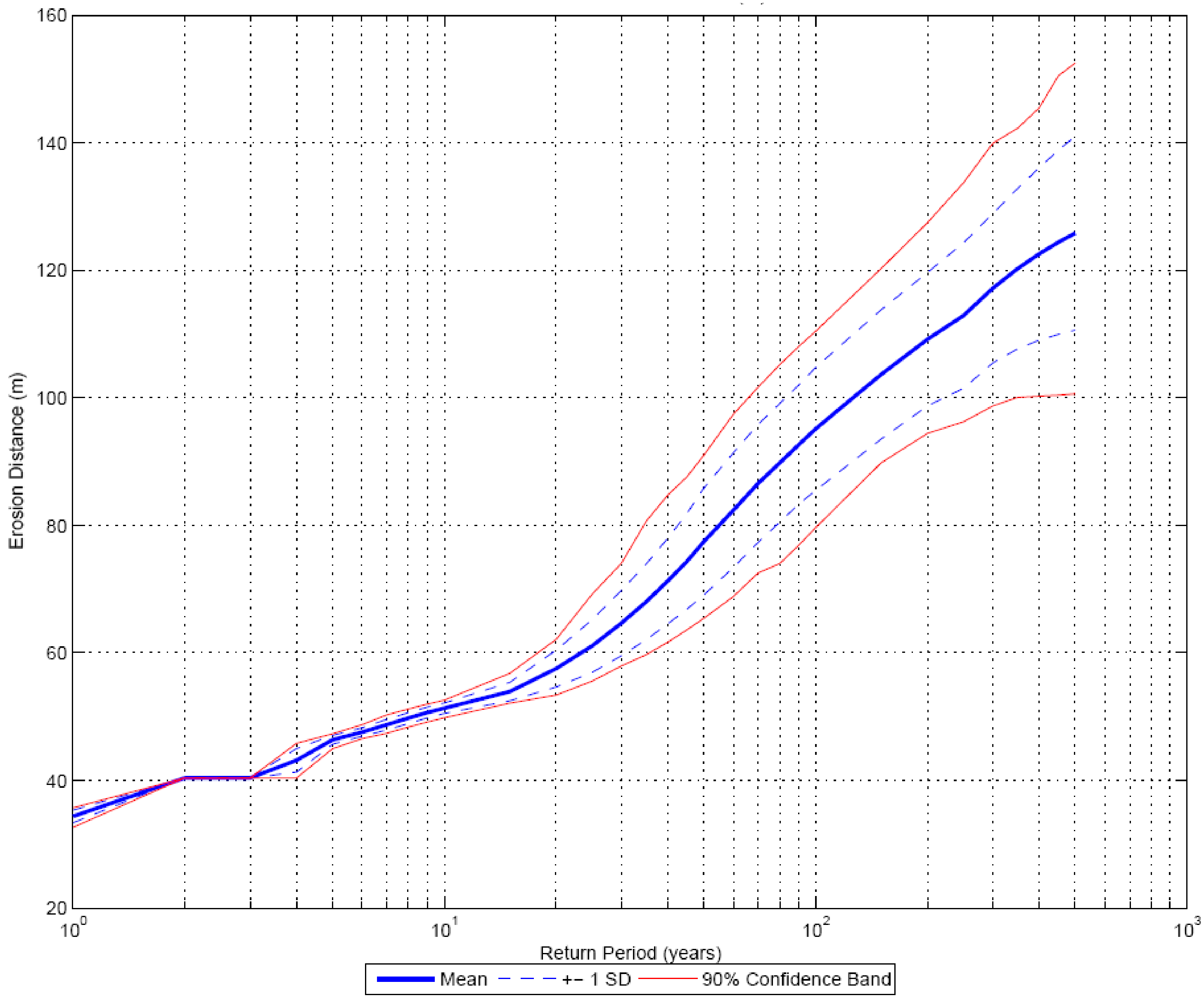


Figure 6-51. Erosion distance vs. frequency for Fire Island subreach F-R2 (1 m = 3.28 ft).

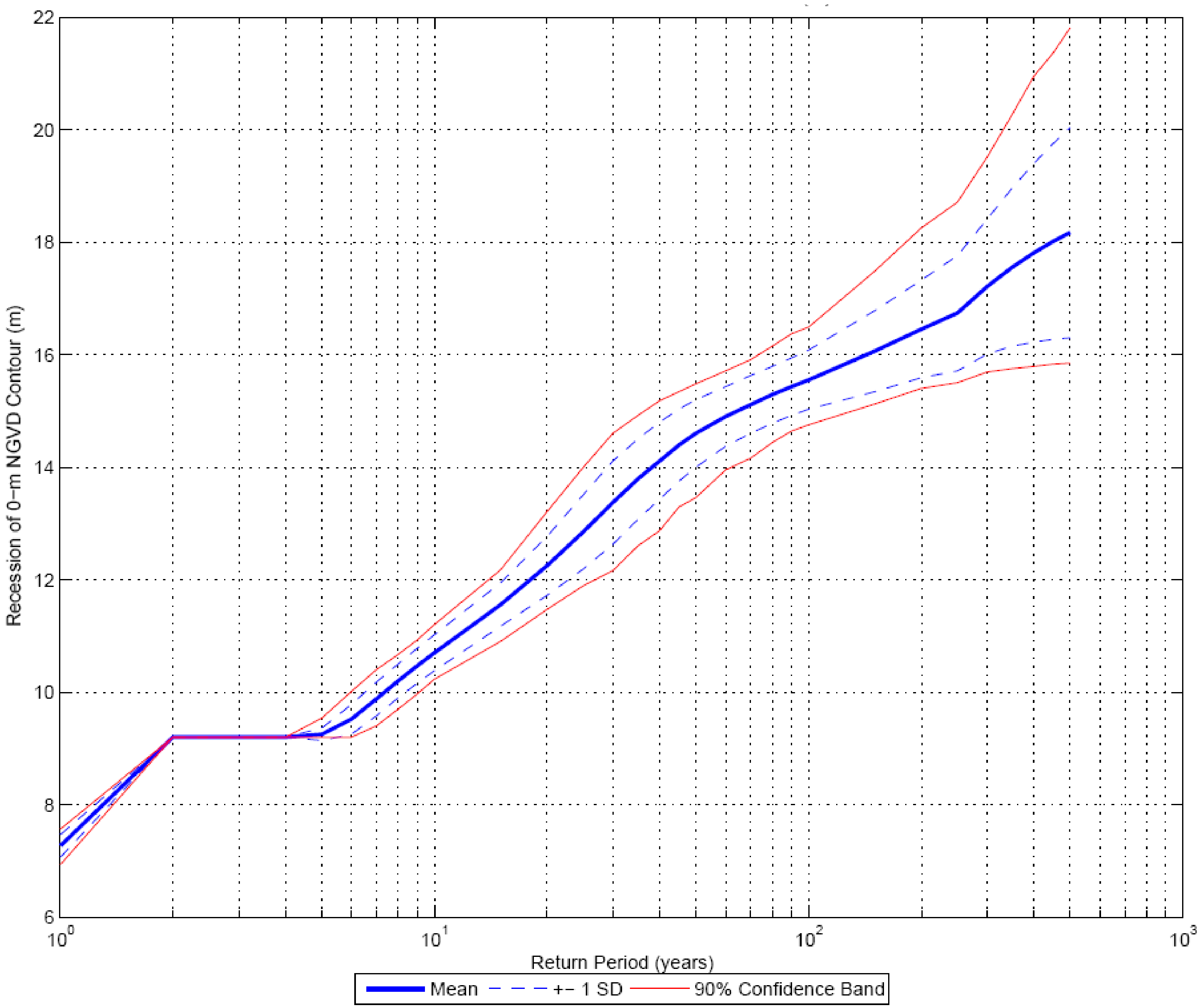


Figure 6-52. Recession of 0-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R2 (1 m = 3.28 ft).

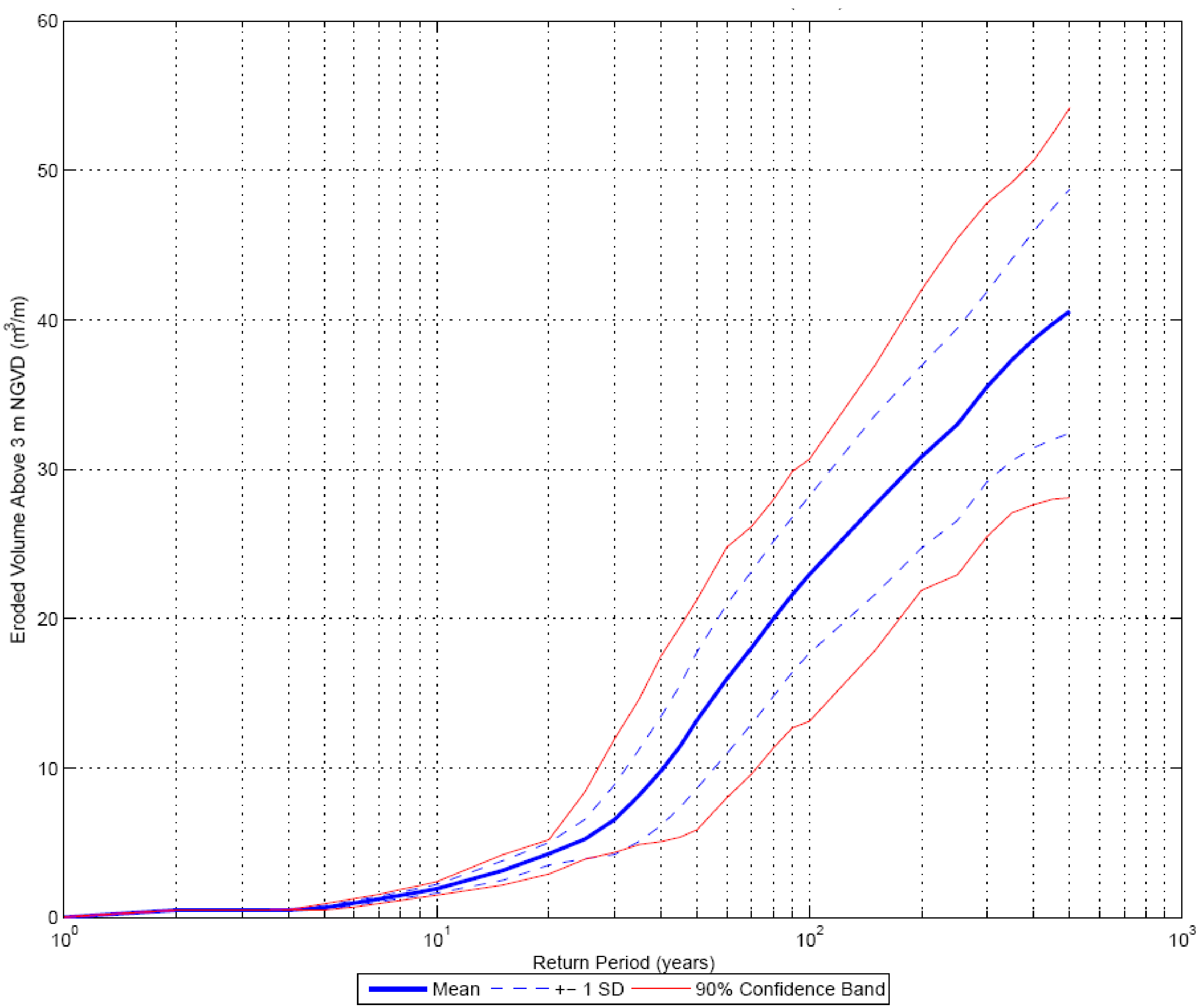


Figure 6-53. Eroded volume above 10-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R2 (1 m^3/m = 10.8 ft^3/ft).

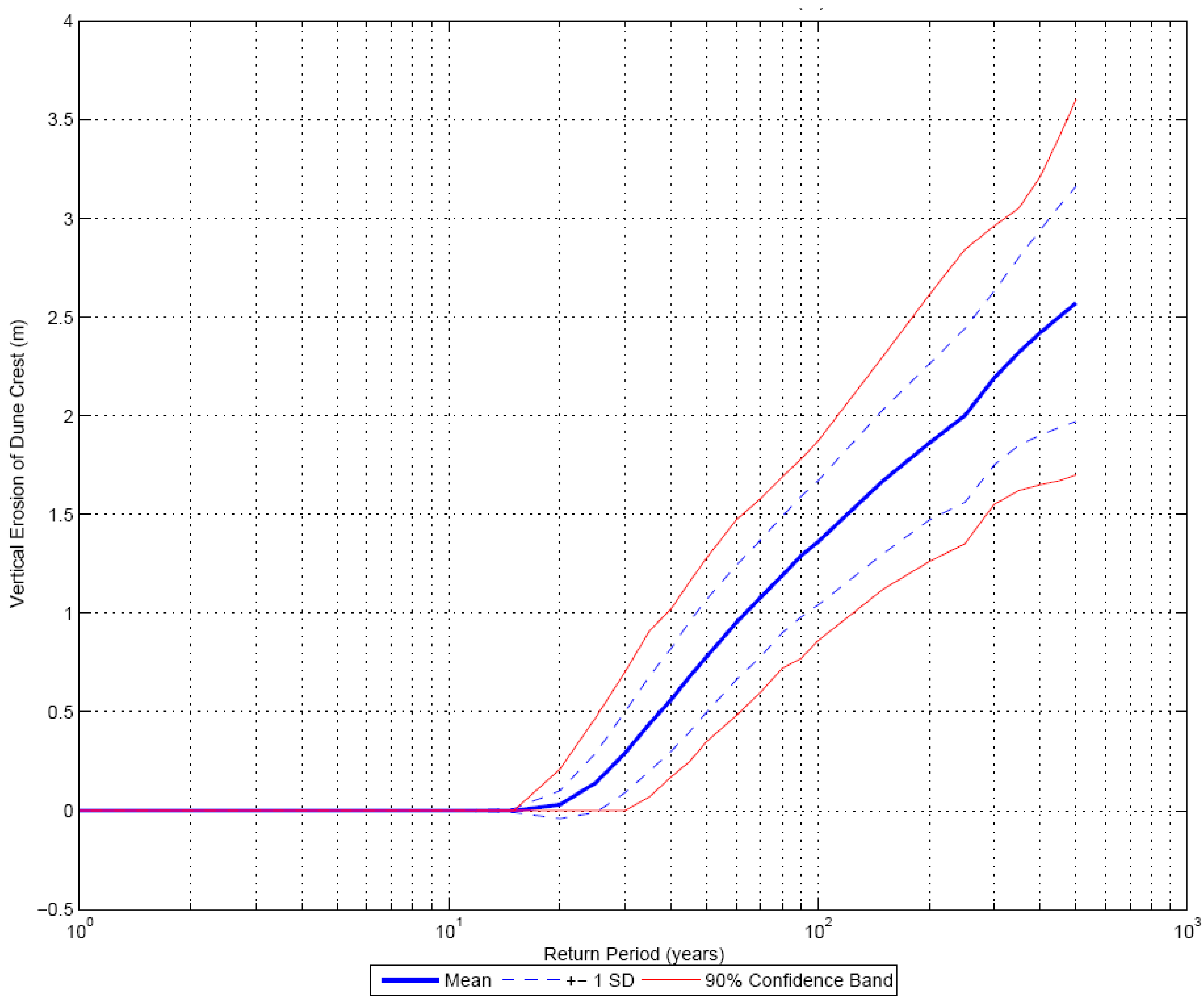


Figure 6-54. Vertical erosion of dune crest vs. frequency for Fire Island subreach F-R2 (1 m = 3.28 ft).

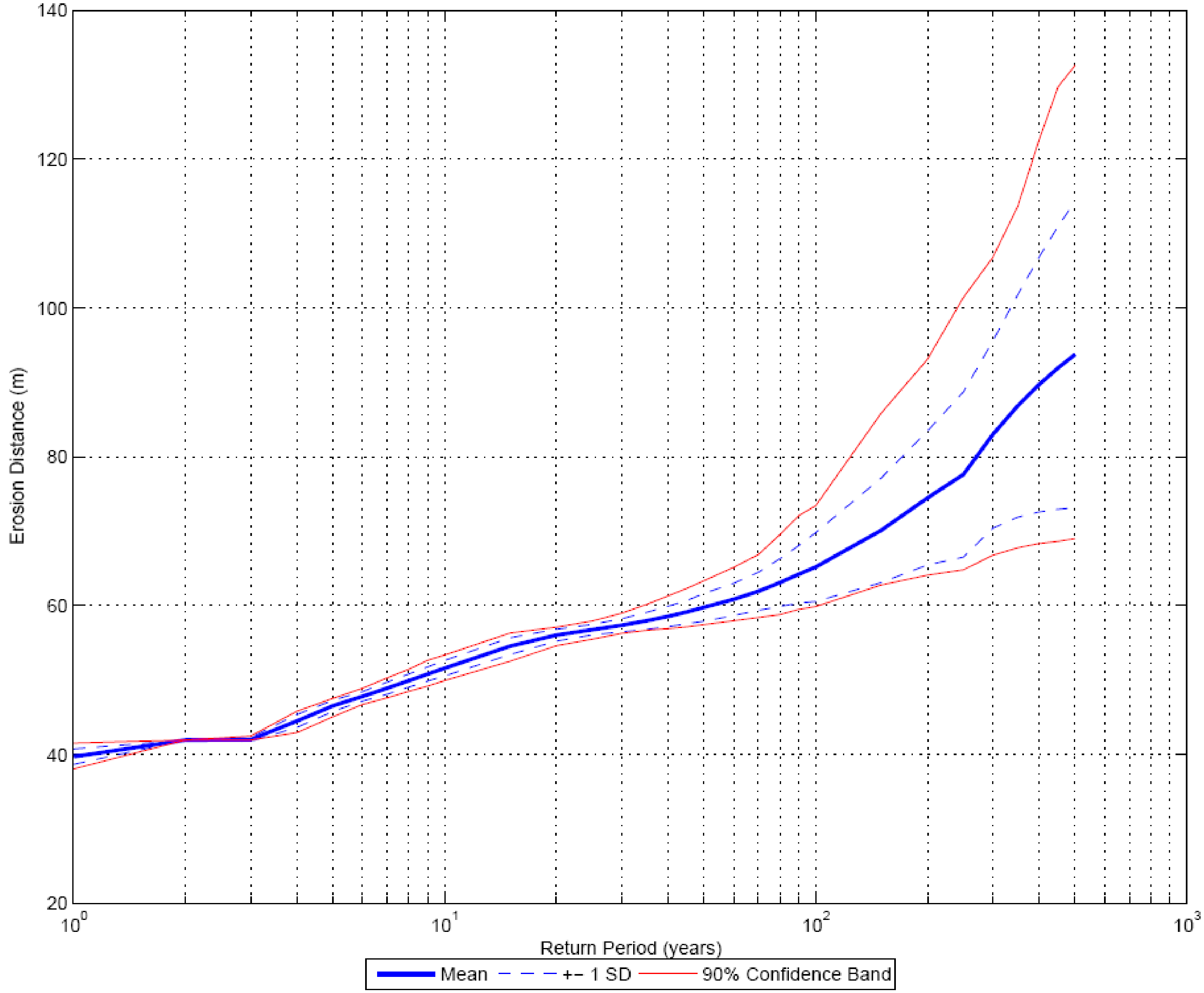


Figure 6-55. Erosion distance vs. frequency for Fire Island subreach F-R5 (1 m = 3.28 ft).

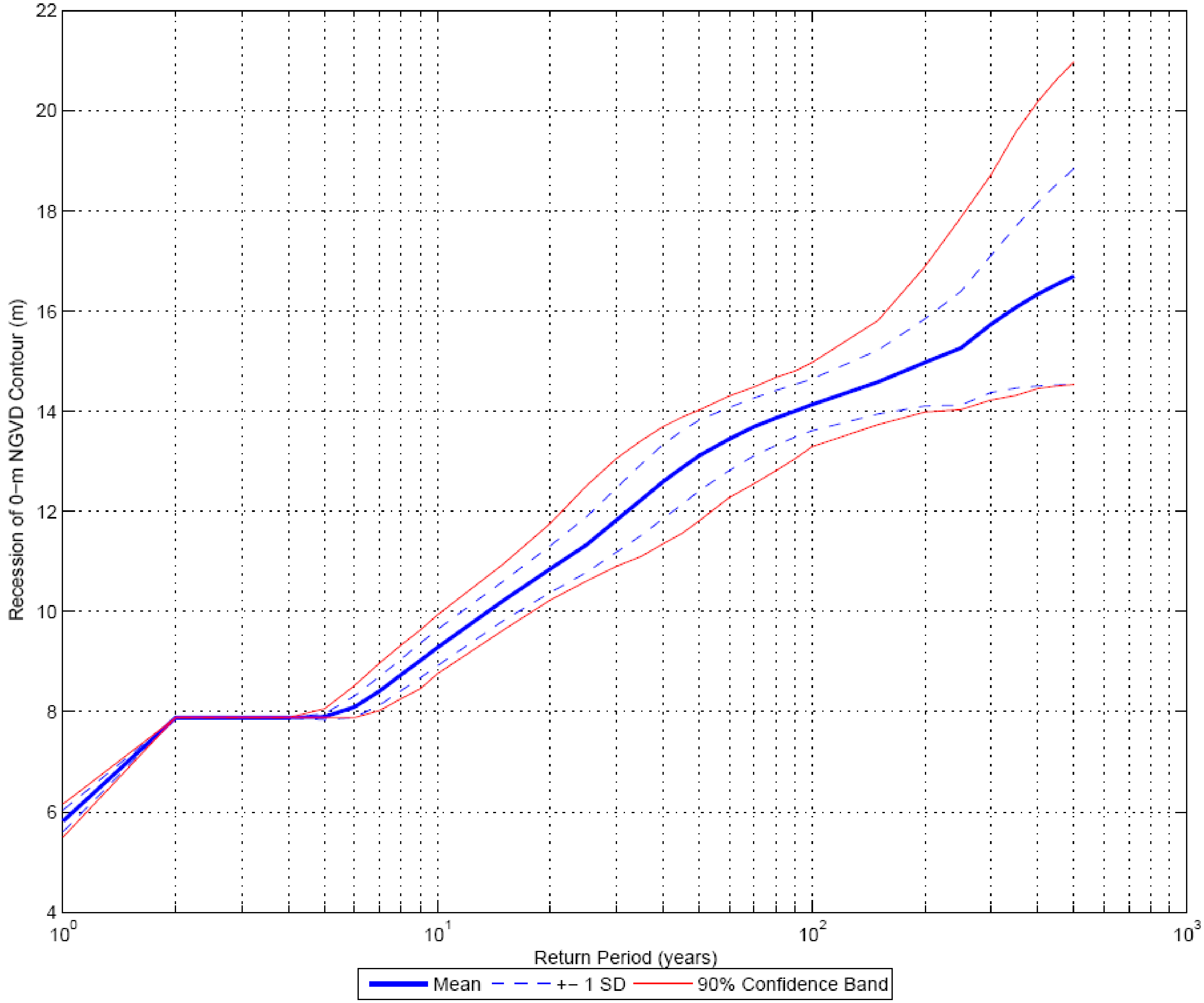


Figure 6-56. Recession of 0-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R5 (1 m = 3.28 ft).

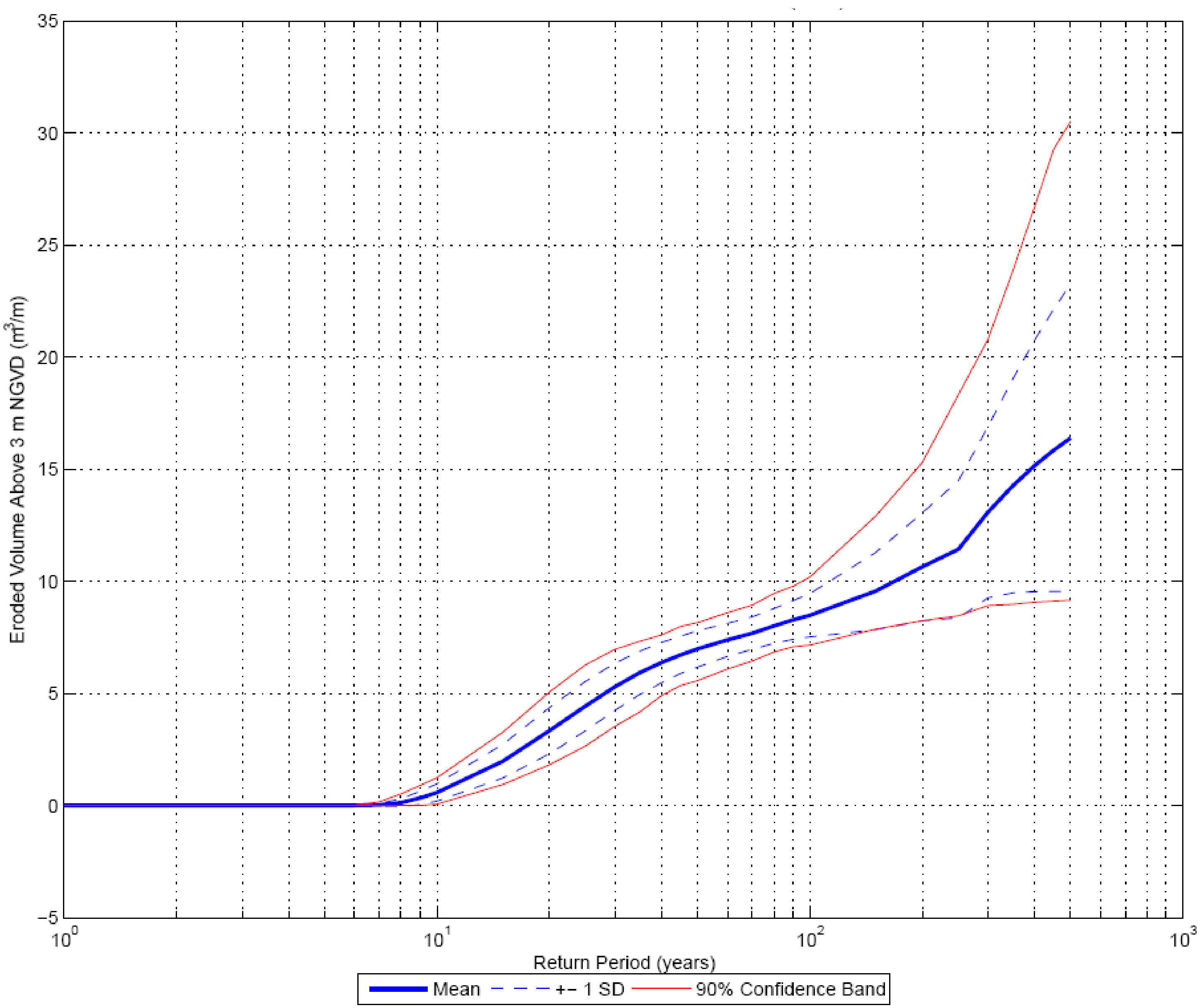


Figure 6-57. Eroded volume above 10-ft NGVD29 elevation vs. frequency for Fire Island subreach F-R5 (1 m^3/m = 10.8 ft^3/ft).

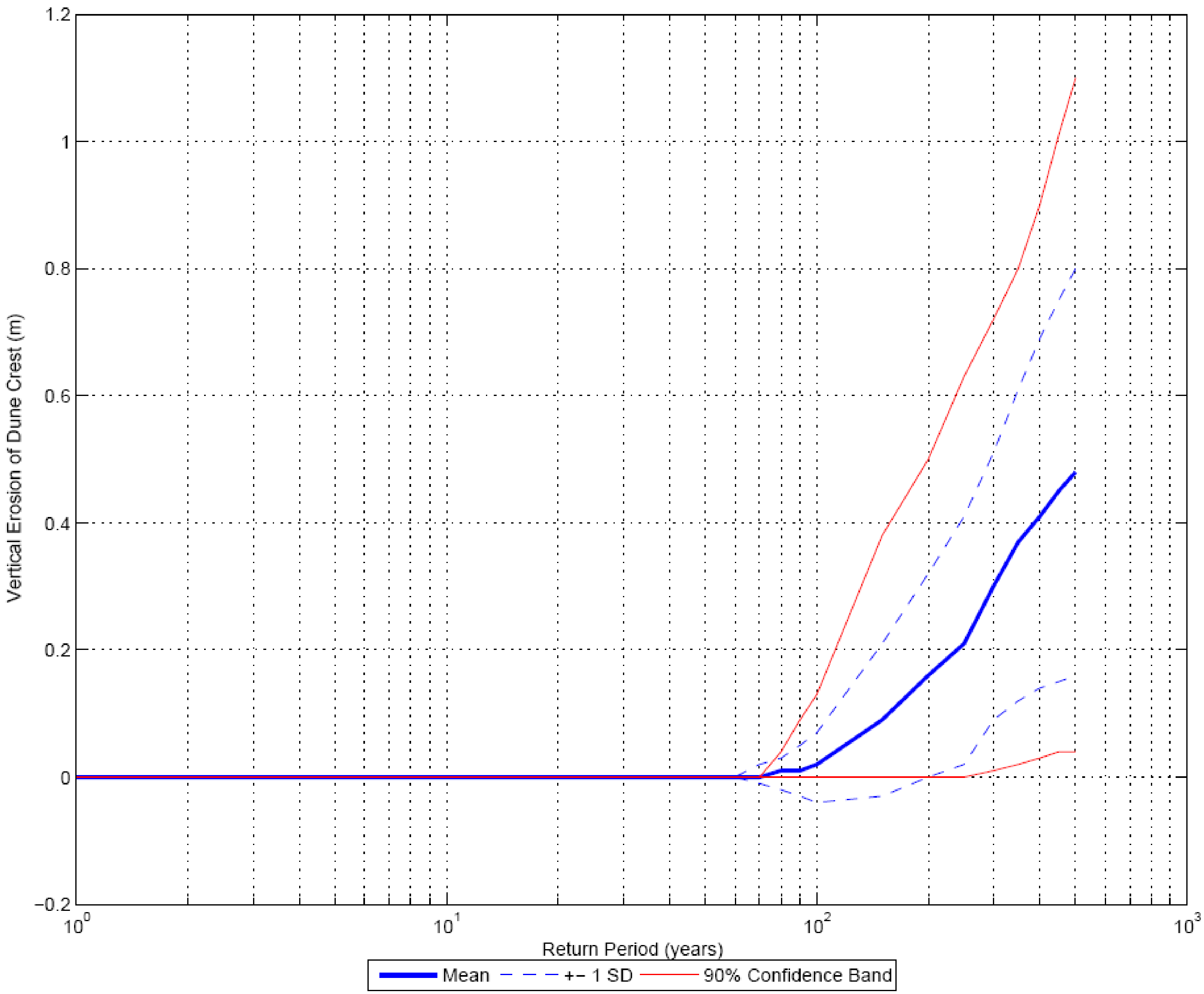


Figure 6-58. Vertical erosion of dune crest vs. frequency for Fire Island subreach F-R5 (1 m = 3.28 ft).

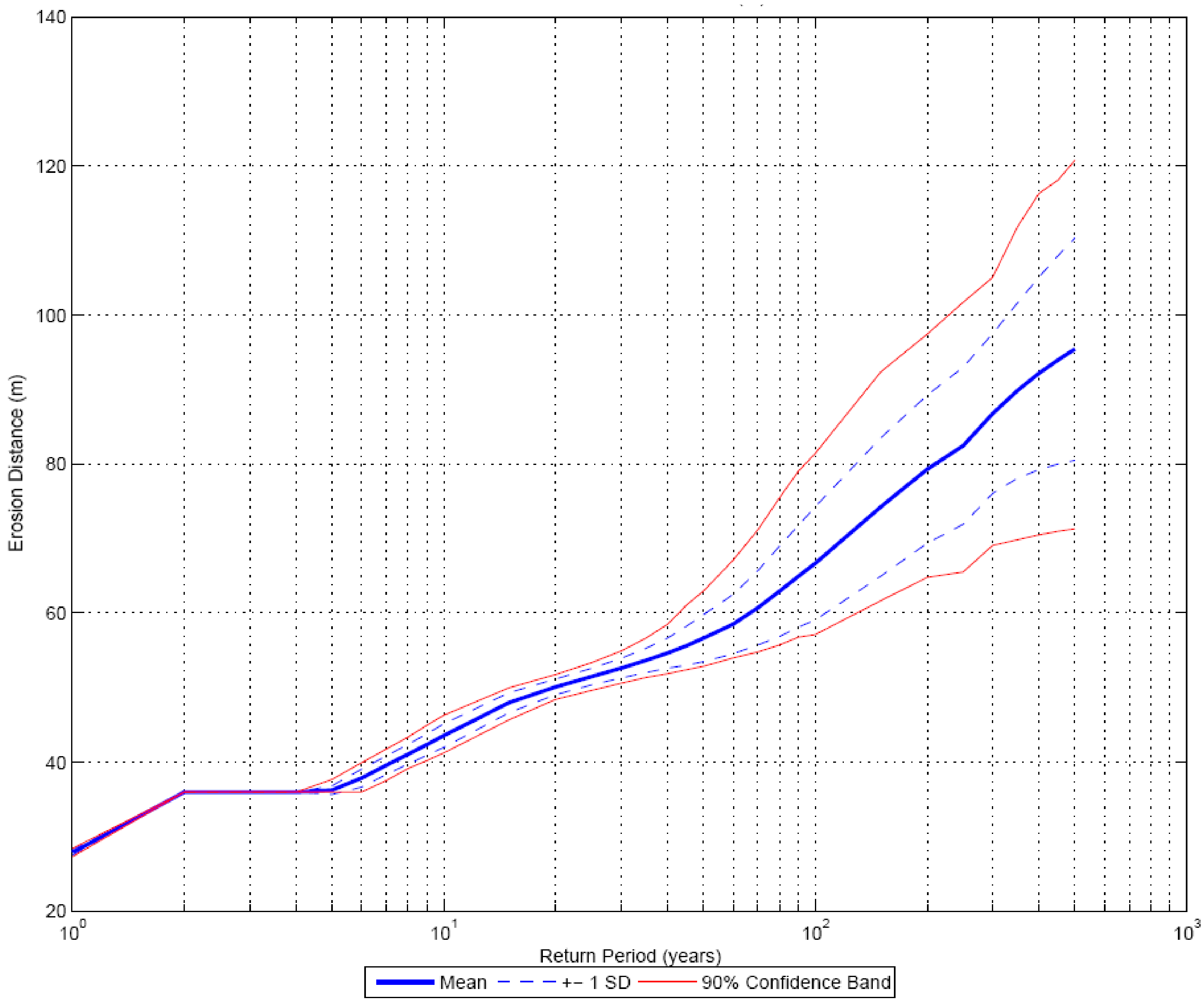


Figure 6-59. Erosion distance vs. frequency for Westhampton subreach W-R5 (1 m = 3.28 ft).

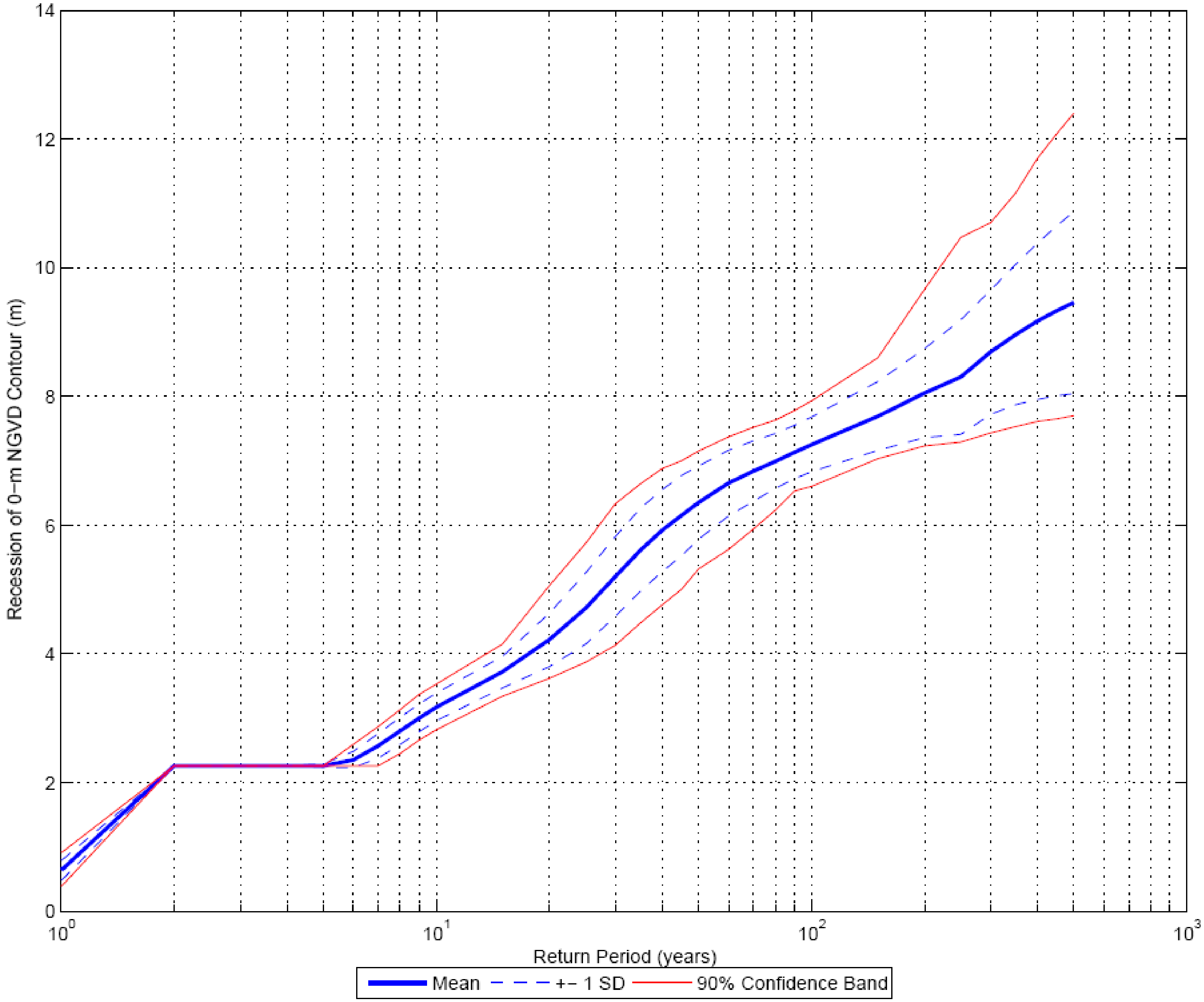


Figure 6-60. Recession of 0-ft NGVD29 elevation vs. frequency for Westhampton subreach W-R5 (1 m = 3.28 ft).

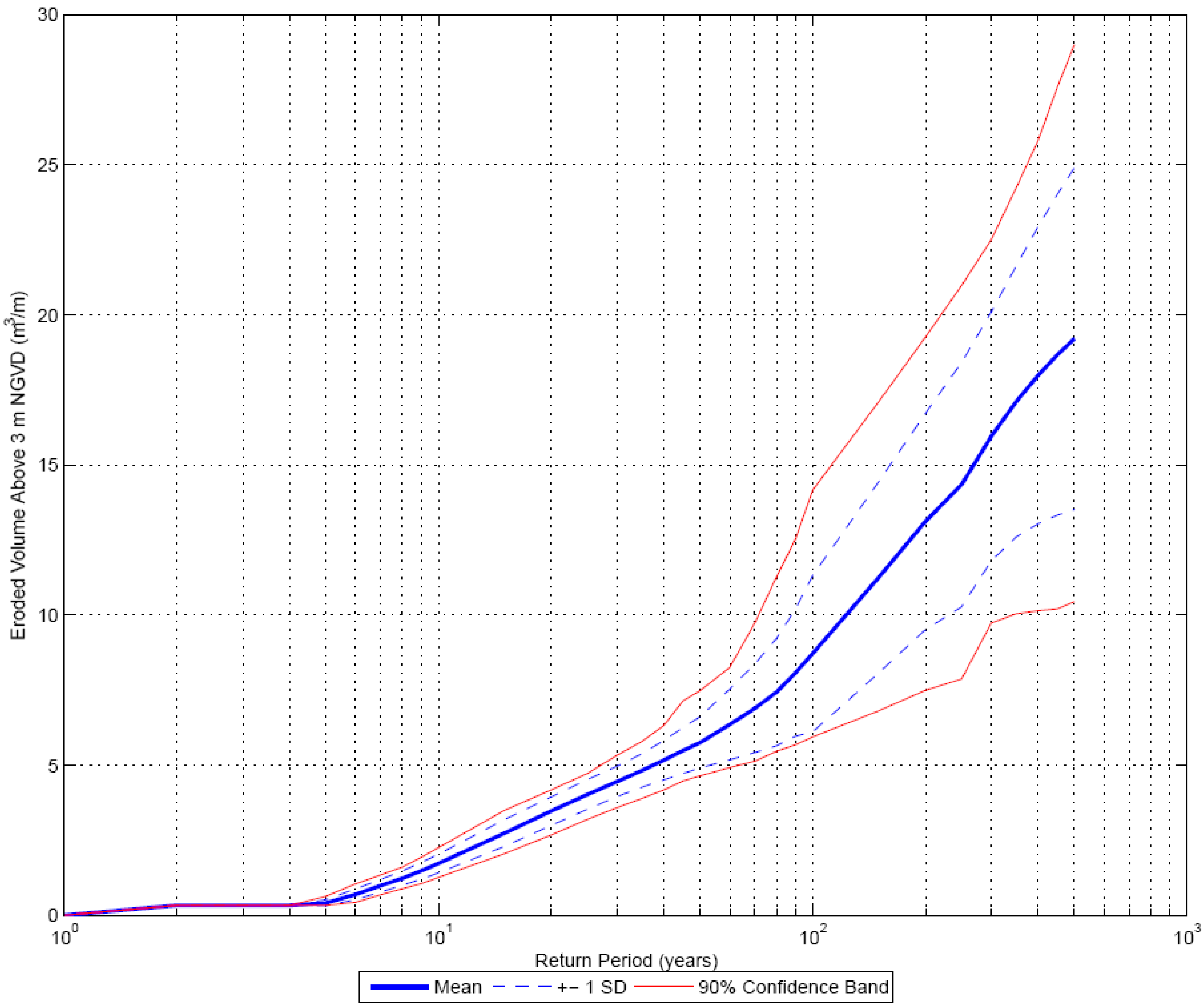


Figure 6-61. Eroded volume above 10-ft elevation vs. frequency for Westhampton subreach W-R5 (1 $m^3/m=10.8\ ft^3/ft$).

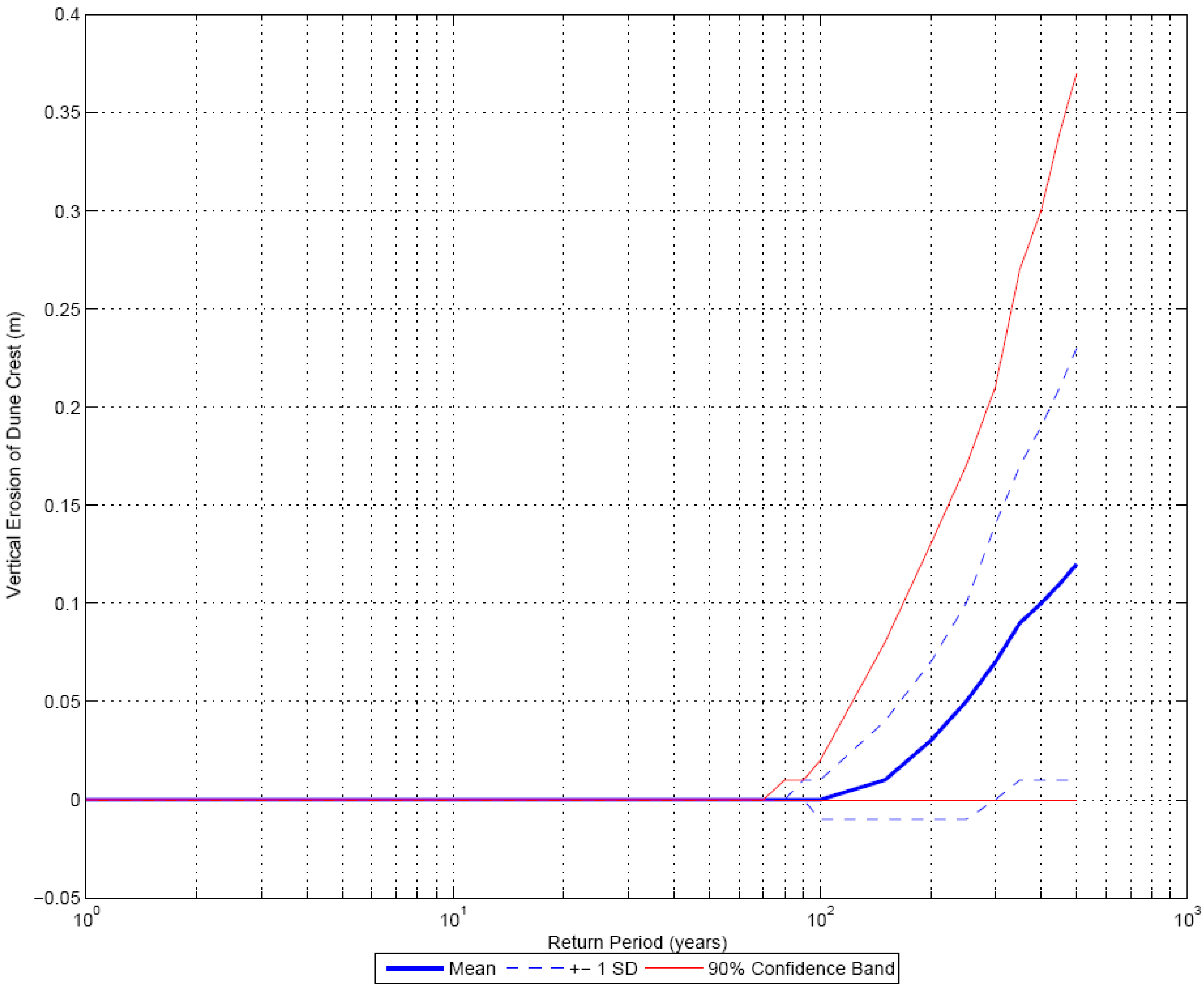


Figure 6-62. Vertical erosion of dune crest vs. frequency for Westhampton subreach W-R5 (1 m = 3.28 ft).

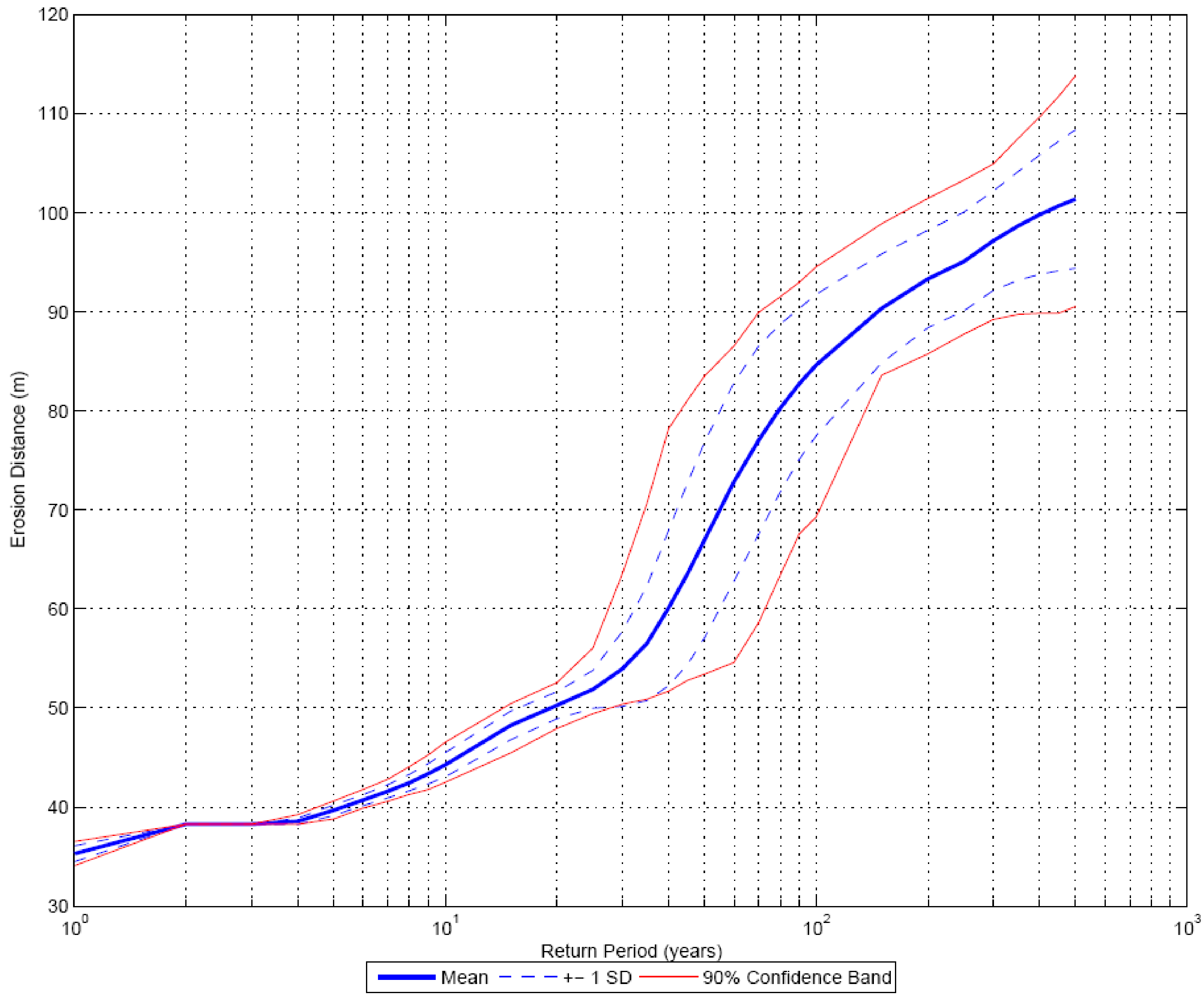


Figure 6-63. Erosion distance vs. frequency for Ponds subreach P-R5 (1 m = 3.28 ft).

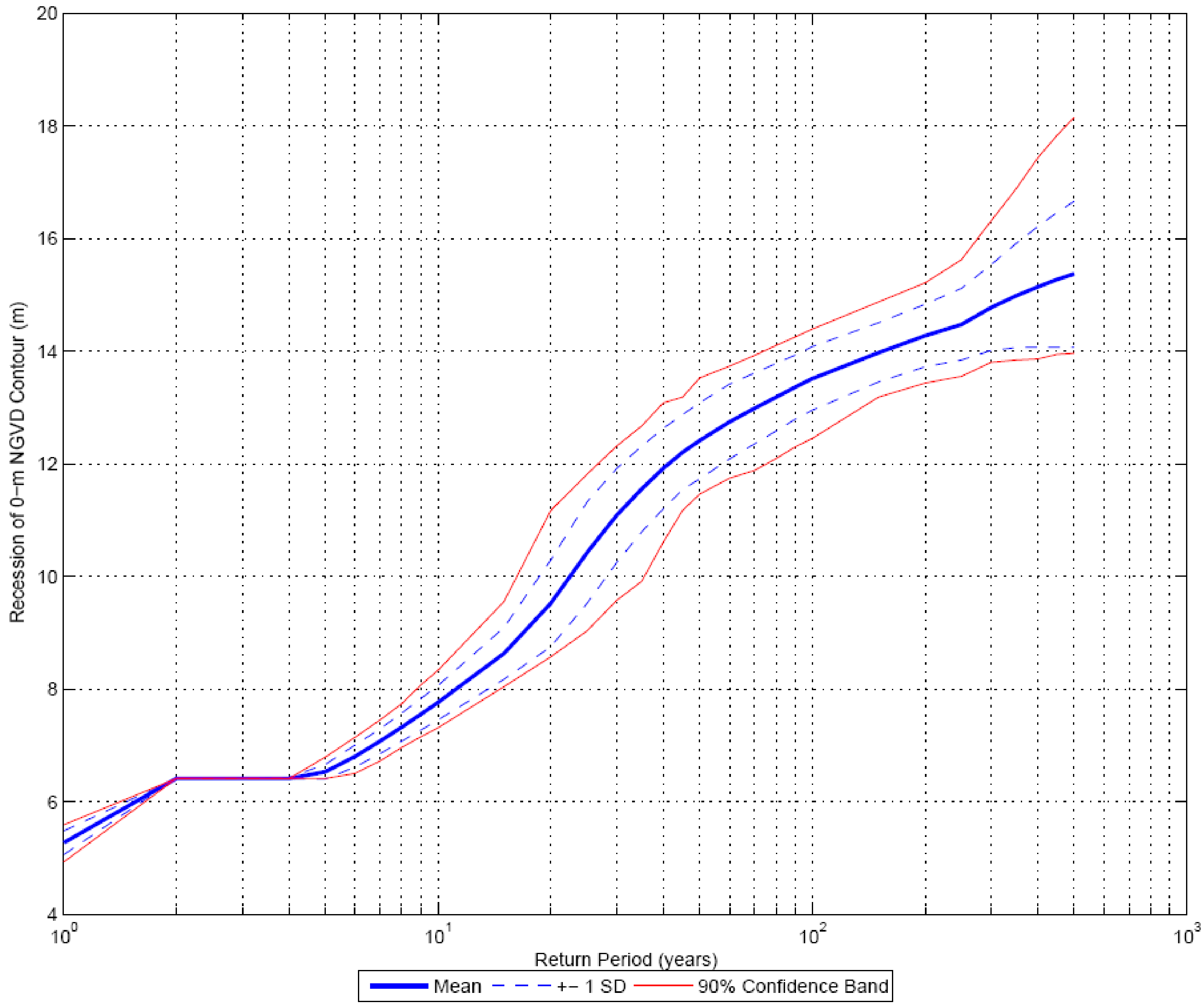


Figure 6-64. Recession of 0-ft NGVD29 elevation vs. frequency for Ponds subreach P-R5 (1 m = 3.28 ft).

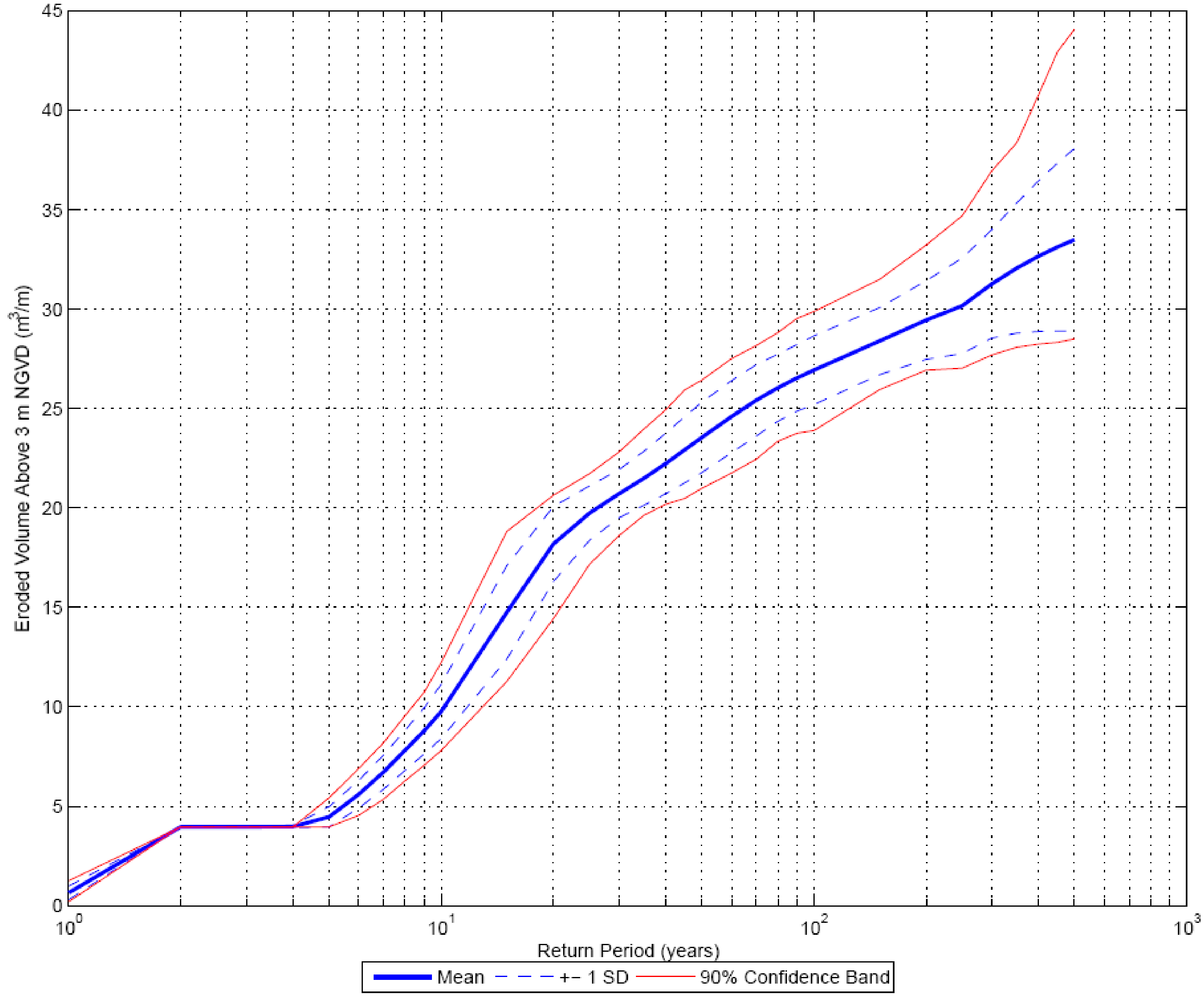


Figure 6-65. Eroded volume above 10-ft NGVD29 elevation vs. frequency for Ponds subreach P-R5 (1 m3/m = 10.8 ft3/ft).

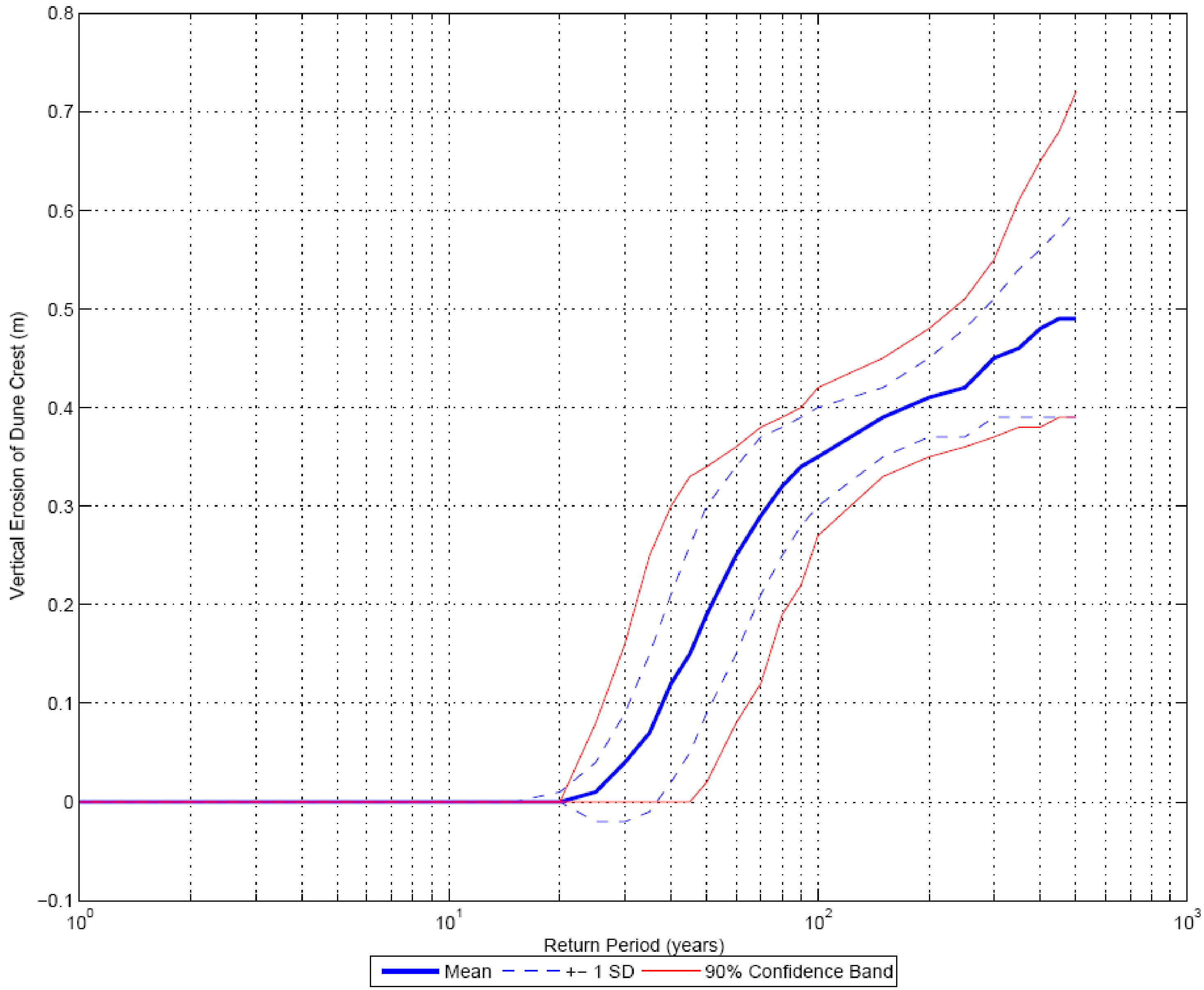


Figure 6-66. Vertical erosion of dune crest vs. frequency for Ponds subreach P-R5 (1 m = 3.28 ft).

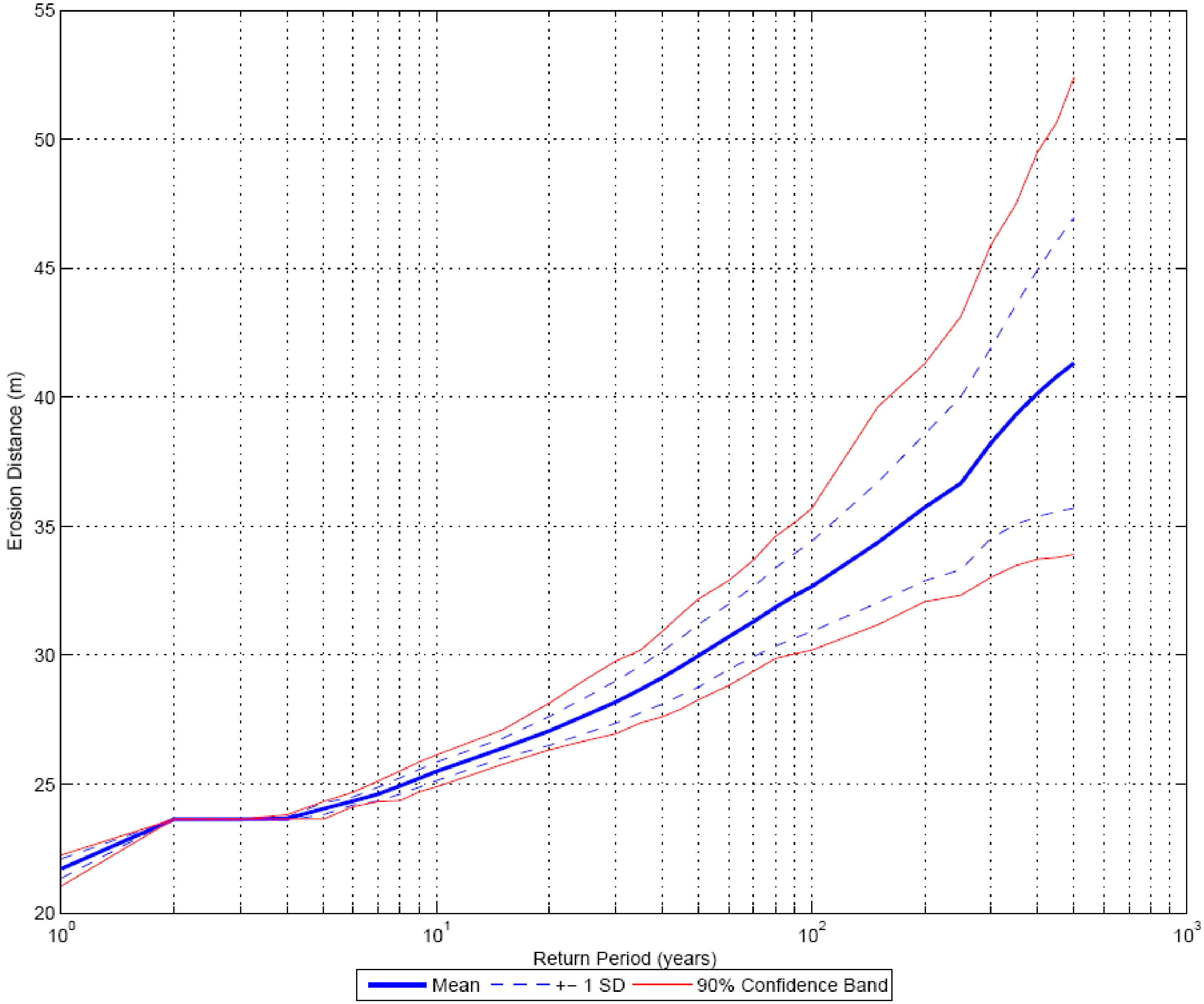


Figure 6-67. Erosion distance vs. frequency for Montauk Point subreach M-R3 (1 m = 3.28 ft).

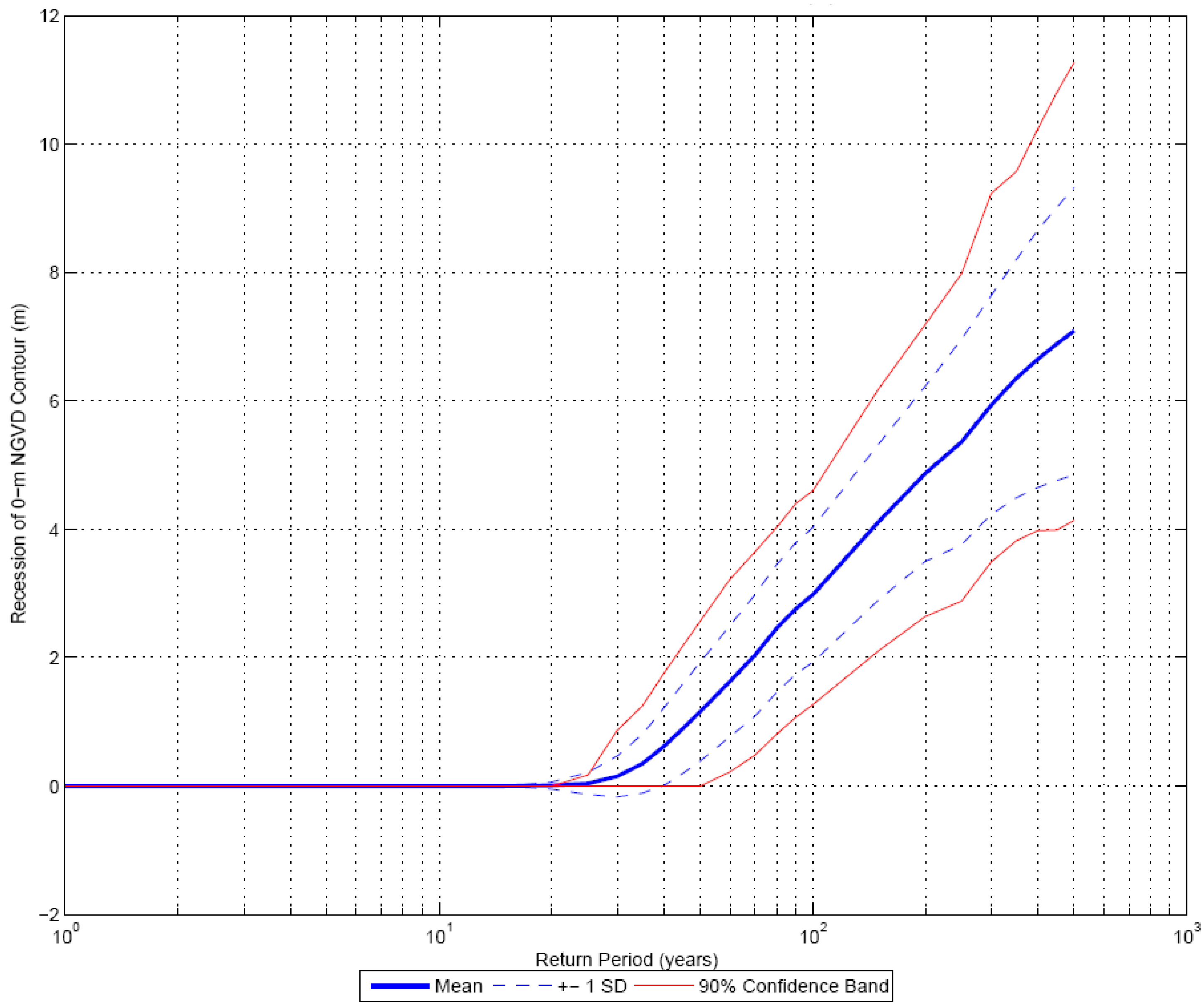


Figure 6-68. Recession of 0-ft NGVD29 elevation vs. frequency for Montauk Point subreach M-R3 (1 m = 3.28 ft).

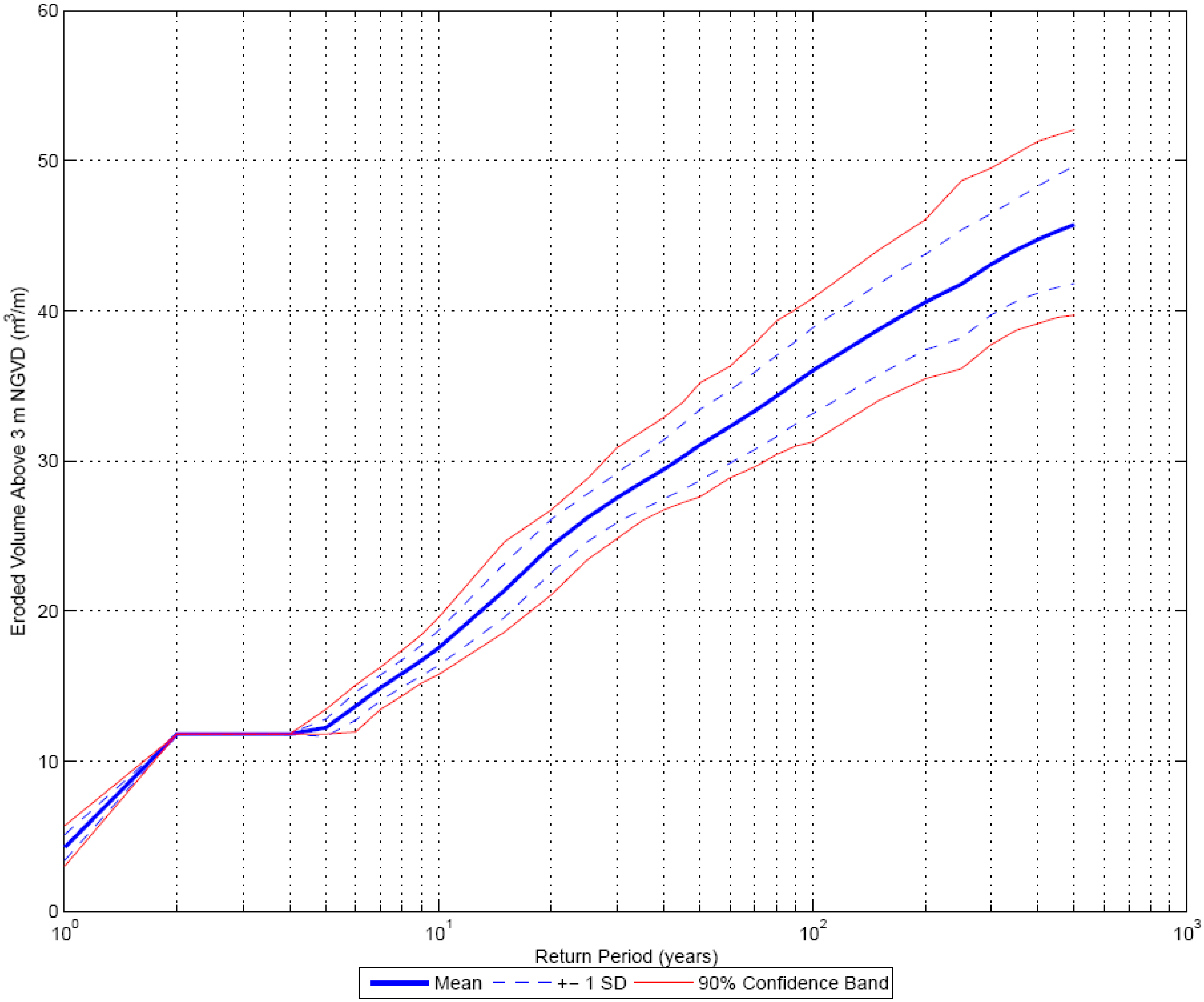


Figure 6-69. Eroded volume above 10-ft elevation vs. frequency for Montauk Point subreach F-R2 (1 m^3/m = 10.8 ft^3/ft).

6.3 Sediment Budget

In order to assist in the planning, design and formulation of coastal storm damage risk reduction measures for a large scale project such as FIMP, the issues must be understood from a regional standpoint. To aid with this, a Sediment Budget is typically constructed. Sediment movement patterns, sources and sinks between Fire Island Inlet and Montauk Point have been studied since the 1960's, which has led to various sediment budgets being developed over the years. The following sections summarize historic and more recent work.

Current (2016) Sediment Budget Considerations

Besides the breach in the Wilderness Area, there have been no other regional modifications within the last fifteen years that would modify the sediment budget for the project area. The Department of Interior, and Stony Brook University have been monitoring the breach area since November 2012, and have reported no changes to the regional sediment budget (in other words, to date the impact on the regional sediment budget is within only one to two miles east and west of the breach and the impact on littoral transport is localized.) The following web sites describe the evolution of the breach and the flood and ebb shoals: http://po.msrc.sunysb.edu/GSB/ ; https://www.nps.gov/fiis/learn/nature/monitoring-the-breach-at-old-inlet.htm.

The USGS has a body of research on the offshore ridges in the vicinity of western Fire Island. A field of shoreface-connected sand ridges that thin in the westward direction have been identified. It was hypothesized that these features may reflect onshore sediment transport west of Watch Hill from erosion of the Cretaceous strata traveling via sand waves. It was further hypothesized that removal of material from these ridges may interrupt the onshore migration of material from the ridges to the shore face. USACE acknowledges that the potential for this onshore movement is a plausible process. The Recent work in the following paragraph is considered work from 1995 to 2001, when the last extensive bathymetric data collection of the inlets, flood and ebb shoals was undertaken.

6.3.1 Previous Work

Gravens et al. (1999) developed a ***Historical*** sediment budget representative of coastal sediment transport pathways and magnitudes during the 1979 to 1995 period. In addition, the authors developed an ***Existing*** sediment budget reflecting littoral transport processes along the barrier island and inlets as of the time of their study (c. 1999). Both budgets were based on an analysis of the mainland and barrier island shorelines within the FIMP project area conducted by the Coastal Hydraulics laboratory (CHL), and an analysis of the three inlets contained in the FIMP project area conducted by Moffatt and Nichol (M&N) (see USACE-NAN, 1998). The authors applied shoreline position data available in 1979, 1983 and 1995 to derive estimates of volume change for each sediment budget cell by assuming the shoreline translated parallel to itself over the active profile depth. The latter is measured as the difference in elevation between the top of the seaward-most active berm and the depth of closure. Gravens et al. used profile data in 1979 and 1995 to compute an active profile depth of 10.5 (34.4 ft) as representative of the beach profiles within FIMP. The two budgets are referred to herein as the ***Historical (1979-95)*** and ***Existing (c. 1999)*** sediment budgets.

Gravens et al. divided the 133-km project shoreline extending from Fire Island Inlet to Montauk Point into three major morphological reaches (Figure 6-70): (1) Montauk Reach extending from Montauk Point in the east to Shinnecock Inlet in the west (58.1 km), (2) Westhampton Reach extending from Shinnecock Inlet to Moriches Inlet (24.8 km), (3) Fire Island Reach extending from Moriches Inlet to Fire Island Inlet (49.5 km). The Montauk Reach (M) is characterized by high bluffs rising more than 25 m above NGVD from Montauk Point to Montauk Beach (budget cell M5), which is located approximately 8 km to the west of Montauk Point. These bluffs, which are formed by a Pleistocene outcropping, are considered to be a source of material to the littoral sediment transport system. The shoreline to the west for about 6 km is characterized by a beach and dune system backed by mainland (budget cell M4). The next 30 km are characterized by a sandy beach backed by mainland and several ponds and small bays which are not typically connected to the ocean, unless during and immediately after storms, or after having been opened by locals to improve water quality (budget cells M3 and M2). The remaining 13 km of the Montauk Reach are characterized by a barrier island beach, which fronts the eastern half of Shinnecock Bay (budget cell M5 and the updrift beach at Shinnecock Inlet).

The westernmost 8.6 km in the Westhampton Reach (downdrift beach cell at Shinnecock Inlet and budget cell W4) include a stretch of barrier island fronting the western half of Shinnecock Bay and the narrow canal that connects Shinnecock Bay and Quantuck Bay. This cell includes the undeveloped area within Shinnecock Inlet Park and the developed communities of Tiana and Hampton Beach. The barrier continues west 2.1 km (budget cell W3) to the start of the Westhampton groin field, a 5.5 km stretch of barrier island (budget cell W2) stabilized between 1965 and 1970 with 15 groins (one additional, short, groin was recently added in 1998 as part of the Westhampton Interim Project). The remaining 5.2 km of barrier island in the Westhampton Reach (budget cell W1 and the updrift beach at Moriches Inlet) include Pikes Beach and Cupsogue County Park.

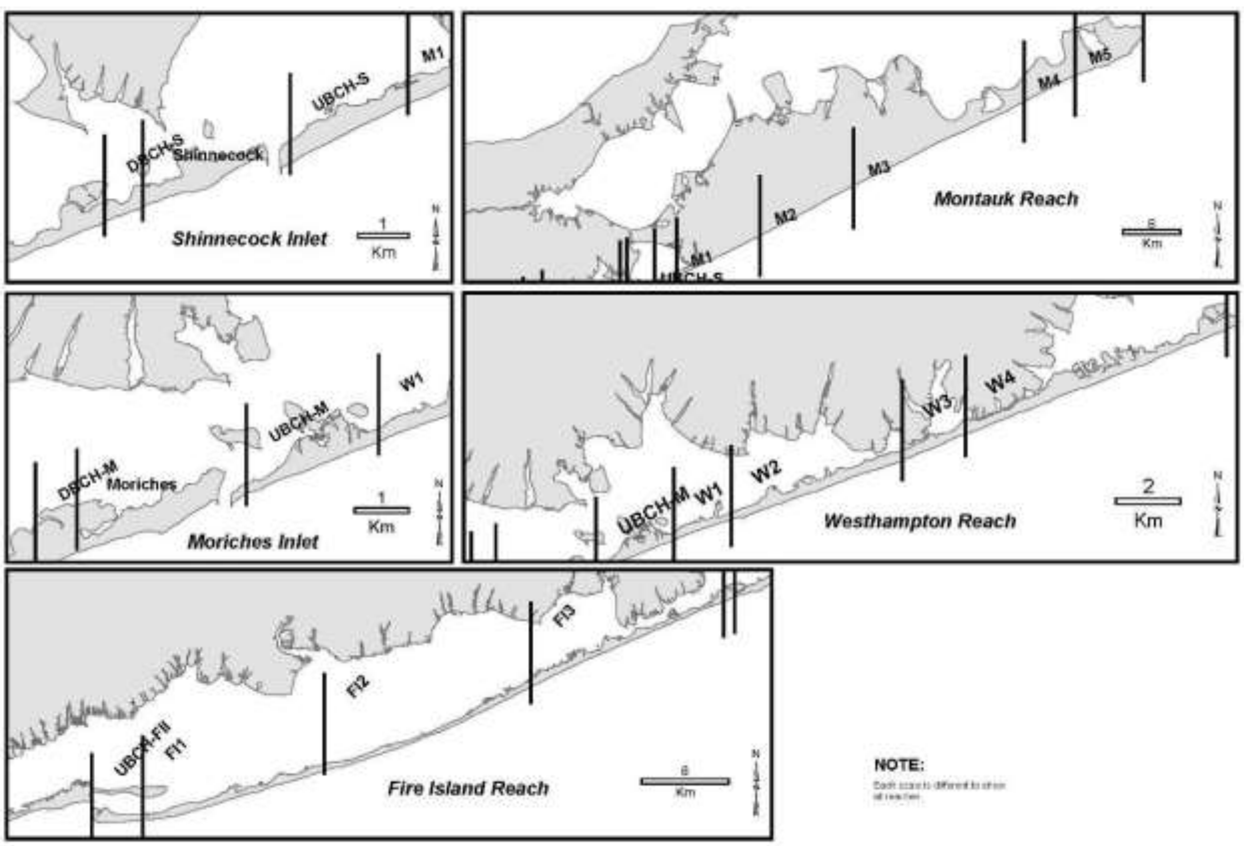


Figure 6-70. Sediment Budget Cells

The eastern portion of Fire Island (roughly 7.2 km including the downdrift beach at Moriches Inlet and budget cell FI3) is characterized by mostly undeveloped barrier island including Smith Point County Park and roughly the eastern two thirds of the Otis Pike Wilderness Area, both part of the Fire Island National Seashore. The next budget cell along central Fire Island (FI2) is roughly 15 km long and it includes the western one third of the Wilderness Area and alternating developed and undeveloped regions of Fire Island from the Watch Hill Visitor Center to Cherry Grove. The remaining 17 km of Fire Island (budget cell FI1 and the updrift beach at Fire Island Inlet) include a relatively continuous stretch of developed barrier island (roughly 8 km from Oakleyville to Kismet) flanked by two undeveloped regions: Sunken Forest to the east and the Fire Island Lighthouse tract and Robert Moses State Park to the west.

The Historical (1979-95) and Existing (c. 1999) regional sediment budgets are reproduced in Figure 6-71. Conclusions from their study are summarized in the following paragraphs. For a more detailed discussion see Gravens et al. (1999).

The Historical [1979-1995] and Existing [c. 1999] condition sediment budgets provide estimates of net longshore sand transport rates, including engineering activities (beach fill placement and dredging), and sources and sinks representative of the Fire Island to Montauk Point study area. These sediment budgets indicated net LST that fell within accepted ranges as derived by previous researchers and as calculated through independent analyses. Furthermore, differences from earlier sediment budgets (such as west of the Westhampton Groin Field) appeared reasonable given knowledge of the engineering activities and coastal processes occurring during the time periods represented in the Historical (1979 to 1995) and Existing (~1999) conditions.

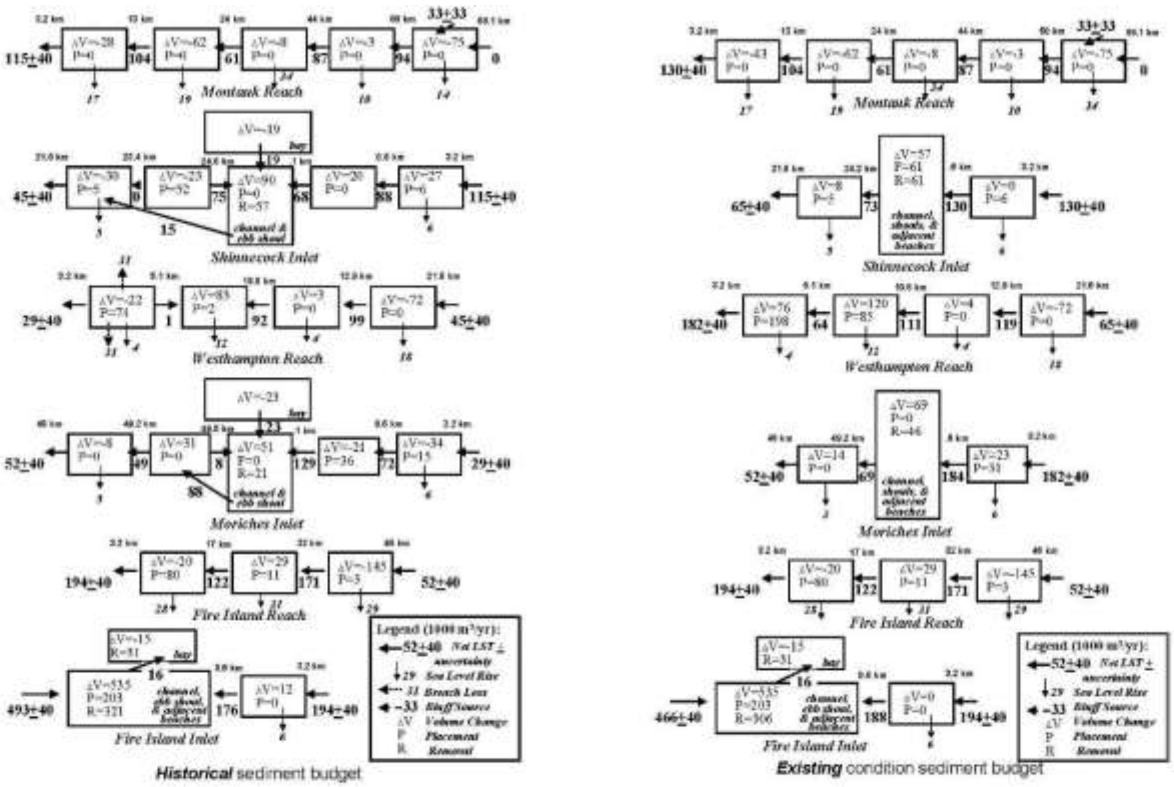


Figure 6-71. Previous Sediment Budgets

It was found that beach fill placement (and/or transfer of littoral material to adjacent beaches) is a significant process and constitutes an important mechanism in maintaining the study area beaches. The majority of the beachfill placement was assumed to be by mechanical means, through dredging of the inlets and bays, and placement on the adjacent beaches. It was found that from 1933 to 1979 and 1979 to 1995, the cumulative rate of beach fill placed from Montauk Point to Fire Island was 295,000 and 309,000 cu m/year, respectively. However, it was estimated that only 25 percent of fills placed to close breaches entered the alongshore movement, which reduced the 1979 to 1995 value to 208,000 cu m/year. Similar values for the 1979 to 1997 time period were determined to be 468,000 (total fill) and 357,000 cu m/year (adjusted for breach fill). These rates of beach fill placement were of the same order as estimates of the net longshore sand transport rate at Fire Island Inlet (Taney (1961a,b): 344,000 cu m/year; RPI (1985): 240,000 cu m/year; Kana (1995): 360,000 cu m/year; growth rate of Democrat Point prior to stabilization (this study): 159,000 to 238,000 cu m/year; impoundment rate at Fire Island East jetty (this study): 385,000 cu m/year (high; may include ebb shoal welding)). Thus, on a regional scale, it was determined that future projects must maintain these nourishment rates to preserve present-day beach conditions. It was also concluded that offshore sources of sediment may exist, but its contribution to the littoral zone was limited to 75,000 cu m/year.

6.3.2 Recent Work

The recent work modified the sediment budgets developed by Gravens et al. by considering more recent data, especially new conditions and management practices at the three inlets in the FIMP project area. First, a sediment budget for the period 1995 to 2001, herein referred to as the ***Recent (1995-2001)*** regional sediment budget, was developed for the project shoreline and the three inlets. This budget was based on the 1995 shoreline previously digitized by CHL, a recent (2001) shoreline digitized from orthorectified aerial photography by CENAN, short (i.e., wading depth) and long (i.e., to or beyond depth of closure) beach profile surveys collected in 1995 and 2001 by CENAN, and several inlet surveys collected between 1995 and 2002 by CENAN and others. This short-term sediment budget was prepared to assess any recent changes in the previously identified medium- to long-term trends. Note, however, that these short-term results cannot, in general, be used to predict long-term or even medium-term sediment transport trends. Thus, a new sediment budget incorporating the long-term trends identified by Gravens et al., recent changes, and existing shoreline and inlet management practices was also developed. This new "representative" budget is referred to herein as the ***Existing (c. 2001)*** sediment budget and should be considered an "update" of the of the *Existing (c. 1999)* conditions budget developed by Gravens et al.

The reach definitions for most of the cells of the regional budget remained similar to the ones in the Gravens et al. analysis to facilitate assimilation of the previous estimates and comparisons with the previous sediment budgets. The inlet cells (Fire Island Inlet, Moriches, and Shinnecock) encompass the sub-divisions specified in the inlet sediment budget presented in Section 5. Table 6-4 lists the beginning and ending stations (from east to west starting at Fire Island Inlet) for each of the regional sediment budget cells. For consistency and to make comparisons with previous work easier, the current sediment budget update is also presented in metric units.

Table 6-4. Regional Sediment Budget Cell Stations

Morphologic Zone	CHL Stationing (km east of each inlet)	Regional Stationing (km east of Fire Island Inlet)
Fire Island Inlet	0	0
	0.075	0.075
UBCH-FI	0.075	0.075
	3.8	3.8
FI1	3.8	3.8
	17	17
FI2	17	17
	32	32
FI3	32	32
	46	46
DBCH-M	46	46
	46.8	46.8
Moriches	46.8	46.8
	0.6	50
UBCH-M	0.6	50
	3.2	52.6
W1	3.2	52.6
	5.1	54.5
W2	5.1	54.5
	10.8	60.2
W3	10.8	60.2
	12.9	62.3
W4	12.9	62.3
	21.6	71
DBCH-S	21.6	71
	22.4	71.8
Shinnecock	22.4	71.8
	0.6	74.8

Table 6-4. Regional Sediment Budget Cell Stations		
Morphologic Zone	**CHL Stationing (km east of each inlet)**	**Regional Stationing (km east of Fire Island Inlet)**
UBCH-S	0.6	74.8
	3.2	77.4
M1	3.2	77.4
	13	87.2
M2	13	87.2
	24	98.2
M3	24	98.2
	44	118.2
M4	44	118.2
	50	124.2
M5	50	124.2
	58.1	132.3

6.3.3 Methodology and Data Sources

The basic sediment budget equation for a control volume, or cell, is expressed as (adapted from Rosati and Kraus, 1999):

$$\sum Q_{IN} - \sum Q_{OUT} - \sum \Delta V + P - R = residual \qquad \text{Equation 6-1}$$

where all terms are expressed as a volume or as a volumetric change rate. Q_{IN} are the sources (e.g., bluff erosion, incoming Longshore Sediment Transport, LST) to the control volume, conversely, Q_{OUT} are the sinks (e.g., outgoing LST) to the control volume. ΔV is the net volume change within the cell, P and R are the amounts of material placed in and removed from the cell, respectively, and *residual* represents the degree to which the sediment budget is balanced. For a balanced budget, the residual is zero.

6.3.3.1 Beach Profile Data

Beach profiles were collected by CENAN throughout the FIMP study area on several separate dates (March 1995, October 1995, March 1996, October 1996, March 1997, March 1998, October 1998, March 1999, October 1999, March 2000, and March 2001). These profile datasets were available as part of the Atlantic Coast of New York Erosion Monitoring Program (ACNYMP) and incorporated into the subject sediment budget.

6.3.3.2 Shoreline Data

The recent analysis leaned on Gravens et al. (1999), which compiled and analyzed a total of 13 historical shoreline position datasets as part of their study (1830, 1870, 1887, February-May 1933, October 1938, March 1962, December 1979, April 1983, March 1988, and March/April 1995) Details about the origin of the aerial photography are given in Gravens et al (1999). An additional set was also incorporated from April 2001.

Also, All-Terrain-Vehicle (ATV) data was used that was collected for the following dates: August 1993, September 1994, August 1995, November 1996, January 1997, May 1997, September 1997, January 1998, and September 2001. The same baseline established by Gravens et al. was used in this study and is shown in Table 6-5.

Table 6-5. Shoreline Analysis Baseline Information						
Shoreline Segment	Point of Origin		Point of Termination		Orientation (deg)	Length (m)
	Easting (m)	Northing (m)	Easting (m)	Northing (m)		
Gilgo	346350.00	49300.00	359587.26	53857.95	N 71 E	14000
Fire Island	358192.34	51878.58	405468.26	68156.99	N 71 E	50000
Westhampton	404995.00	67720.00	428334.51	76679.20	N 69 E	25000
Montauk	428275.00	76400.00	480220.68	102867.65	N 63 E	58000

6.3.3.3 Volume Changes

In order to develop the *Recent (1995-2001)* and *Existing (c. 2001)* sediment budgets, volume changes in each cell were computed using three data sources: (1) long profiles, (2) a combination of long and short (i.e. wading) profiles, and (3) digitized shorelines. Volume differences were divided by the time between surveys to obtain a volume change rate. Where short profiles were used to supplement the long profiles, volume changes across the subaerial portion of the profile were summed, a contour change rate was calculated at shoreline and multiplied by the approximate depth to closure, 7.0 m (Gravens et al., 1999), then added to the subaerial changes and divided by the time between surveys. Shoreline change rate was multiplied by the active profile depth, 10.5 m (Gravens et al., 1999) to obtain a volume change rate. In general, volume changes based on profile data were preferred over changes based on shoreline data if profile density was adequate (at least one profile per km of shoreline). However, this approach was modified in areas where additional shoreline data was available (Fire Island) or where changes based on profile data seemed unrealistic based on previous sediment budgets, net longshore sediment transport computed with GENESIS (see below) or basic understanding of coastal processes in the FIMP area.

6.3.3.4 Sea Level Rise

Cross-shore sediment losses due to sea level rise were incorporated as in Gravens et al. (1999) (after Bruun, 1962) which provides a generally-accepted, simple approach to an otherwise complex process. Specifically, a volumetric loss rate due to relative sea level rise of 2.3 m^3/m/yr based on relative SLR rate of 0.003 m/yr was applied to all ocean shoreline cells in the shoreline-based volume change analysis. Therefore, the total sediment sink along the shorelines due to sea level rise is estimated to be roughly 305,000 m^3/yr. 134,000 m^3/yr from Montauk Point to Shinnecock Inlet, 57,200 m^3/yr from Shinnecock to Moriches Inlet, and 114,000 m^3/yr along Fire Island (Gravens et al., 1999).

6.3.3.5 Contribution of Montauk Point Bluffs

Gravens et al. (1999) presents estimates of a sediment source from the Montauk Bluffs on the order of 30,000 m^3/yr, obtained using shoreline change and profile data as well as sediment grain size analysis. In this update, available profile data, which includes the face of the bluff, were used to quantify volume changes throughout the Montauk Bluff area. Therefore, these volume changes are used in the update directly without separate consideration of the exact bluff contribution.

6.3.3.6 Offshore Sediment Sources

A number of previous studies (e.g., Williams, 1986, Williams and Meisburger, 1987, Williams and Morgan, 1993, Schwab et al. 1999, Schwab et al. 2000) suggest the possibility of a contribution of sediment to the coastal sediment budget from offshore sources. The present study also recognized this possibility based on the estimated volume changes and computed potential longshore sediment transport rates. However, it was determined that this source was not required to meet the accepted range of longshore sand transport rates at Fire Island Inlet. Furthermore, the USGS and the USACE have engaged in discussions to cooperatively resolve the potential impacts of offshore sand mining to cross-shore sediment transport via a monitoring/adaptive management strategy.

6.3.3.7 Overwashing and Breaching Losses to the Bays

Significant storm events that produced overwashing and breaching were not present between 1995 and 2001. Therefore, the *Recent (1995-2001)* regional sediment budget did not include sediment losses caused by overwash or breaching. Earlier studies (Kana 1995) where overwash and breaching were part of the data set showed that the total annualized contribution was relatively small: 25,000 m^3/yr or 0.2 m^3/m/yr (RPI, 1985). Therefore, the contribution of breaches and overwashes was also neglected in the formulation of the Existing (c. 2001) sediment budget.

6.3.3.8 Wind-blown Sediment Transport

Similar to Gravens et al. (1999), it was assumed that the FIMP area is relatively well-established and vegetated. Therefore, the contribution of wind sediment transport to the littoral system was minor and was neglected.

6.3.3.9 Inlet Sediment Budgets

Sediment budget cells at each of the three inlets have been updated and are discussed in detail in Section 6.4. Beach profile and shoreline change data were used to assess volume change in shoreline cells adjacent to the inlets as discussed above. Bathymetric survey comparisons were conducted using a series of synthetic grids at each inlet.

6.3.3.10 Engineering Activities

Details of engineering activities and beach fill placement from 1998 to 2002 were obtained from CENAN and other state and local stakeholders. Activities from 1995 to 1998 were compiled by Gravens et al. (1999). Activities prior to 1995 are presented in Gravens et al. (1999).

6.3.3.11 Uncertainty

Volume changes and sediment transport quantities required for the formulation of a coastal sediment budget cannot be measured directly and therefore values of such quantities have to be obtained through indirect and/or incomplete measurements (e.g., shorelines or beach profiles), with predictive formulas, or through estimates based on experience and judgment. According to Kraus and Rosati (1998a), these values can be considered as consisting of two terms: (1) Best Estimate ± (2) Uncertainty. The values presented in the following sections are considered a “Best Estimate” and are based on various sources including incomplete measurements (beach profiles, inlet surveys), indirect measurements (shorelines), numerical estimates of longshore sediment transport, and numerous assumptions regarding coastal processes and sediment transport pathways within the FIMP project area, particularly at the three inlets.

Kraus and Rosati (1998a) provide various representative examples of uncertainty analysis and show that uncertainty in sediment budget can be large. In fact, the maximum uncertainty computed by the authors was greater than the estimates themselves and the “best” uncertainty was only about 50% smaller. This despite the fact that some of the assumed “input” uncertainty values are relatively small compared to other published estimates. For example, in their uncertainty analysis example, the “best” (rms) estimate of uncertainty regarding the active profile depth was 0.3 m for an assumed value of 8 m. However, Morang et al. (1999) estimated error associated with profile interpretation at 0.15 m, short-term temporal variability at more than 2 m, and spatial variability along the FIMP area at 3 m.

Given the myriad of data sources used in this study and the fact that most of the uncertainty is not easy to identify much less calculate (e.g., lack of overlapping coverage at the inlet surveys or differences in datum correction methods) an attempt to quantify the total uncertainty associated with the volume changes and longshore sediment transport rates presented below was not made. Instead, based on the estimates of uncertainty for the various components of the sediment budget, it was concluded that uncertainty represents a significant percentage of the estimates included in the proposed sediment budgets, perhaps as much as the estimates themselves in some cases. Nonetheless, it was judged that the proposed sediment budgets provide a realistic, albeit only semi-quantitative, description of the sediment transport processes that can be used to assist in the planning, design, and formulation of coastal storm damage risk reduction measures for the FIMP project area.

6.3.4 Recent (1995-2001) Regional Sediment Budget

6.3.4.1 Volume Change Rates

Volume change rates for the 1995-2001 period within the regional sediment budget cells were computed using the long profile data, long and short profile data, and shoreline data described above. Results of the regional volume change analysis are presented in Table 6-5. Volume change rates from each data source are plotted in Figure 6-72 through Figure 6-75.

Table 6-6 illustrates the magnitude of the uncertainty associated with volume change estimates. For example, the changes computed along Fire Island using USACE shorelines digitized from available aerial photos in the spring of 1995 and 2001 are remarkably different than the changes computed with field data collected by USGS using ATVs and GPS in late summer on 1995 and 2001. Some of these differences are probably due to methodology (scanning and digitizing the HWL on an aerial is very different than “driving” the HWL in the field) and some due to seasonal effects on the onshore/cross-shore distribution of sediment.

Also worth noting are the differences between volumes computed from shoreline data and profiles. Unfortunately it can only be speculated as to which of the two datasets is more accurate, because each has their own inherent accuracy issues. However, volume changes based on profile data were preferred over changes based on shoreline data if profile density was at least one profile per km of shoreline.

Table 6-6. Volume Change Rates by Reach and Data Source (1995 to 2001)

Morphologic Zone	Stationing (km east of each inlet)	Long Profile Density (Profiles/km)	Short & Long Profile Density (Profiles/km)	Volume Change Rate (Long Profiles) 1000 m3/yr	Volume Change Rate (Long & Short Profiles) 1000 m3/yr	Volume Change Rate (USACE Shoreline Change) 1000 m3/yr	Volume Change Rate (USGS Shoreline Change) 1000 m3/yr	Sea Level Rise (1000 m3/yr)
Fire Island Inlet	0 0.075							
UBCH-FII	0.075 3.8	0.81	0.81	26	-34	-4	204	8
FI1	3.8 17	0.53	1.48	-139	137	-152	248	28
FI2	17 32	0.57	1.77	131	89	-142	344	31
FI3	32 46	0.32	0.93	-402	-420	-294	111	29
DBCH-MI	46 46.8	--	1.25	--	57	-16	--	0
Moriches Inlet	46.8 0.6							
UBCH-MI	0.6 3.2	2.50	2.88	151	75	57	--	6
W1	3.2 5.1	2.11	2.89	123	343	237	--	4
W2	5.1 10.8	1.14	1.75	411	255	162	--	12
W3	10.8 12.9	0.71	1.19	-122	-25	-4	--	4
W4	12.9 21.6	0.57	1.15	-255	-57	-146	--	18
DBCH-SI	21.6 22.4	--	2.50	--	2	-12	--	2
Shinnecock Inlet	22.4 0.6							
UBCH-SI	0.6 3.2	0.96	1.15	59	108	27	--	6
M1	3.2 13	0.36	1.07	33	-59	-165	--	17
M2	13 24	0.55	1.50	21	-150	-254	--	19
M3	24 44	0.43	0.93	-265	-425	-105	--	34
M4	44 50	0.50	1.08	89	82	-32	--	10
M5	50 58.1	0.06	0.86	-73	-80	68	--	14

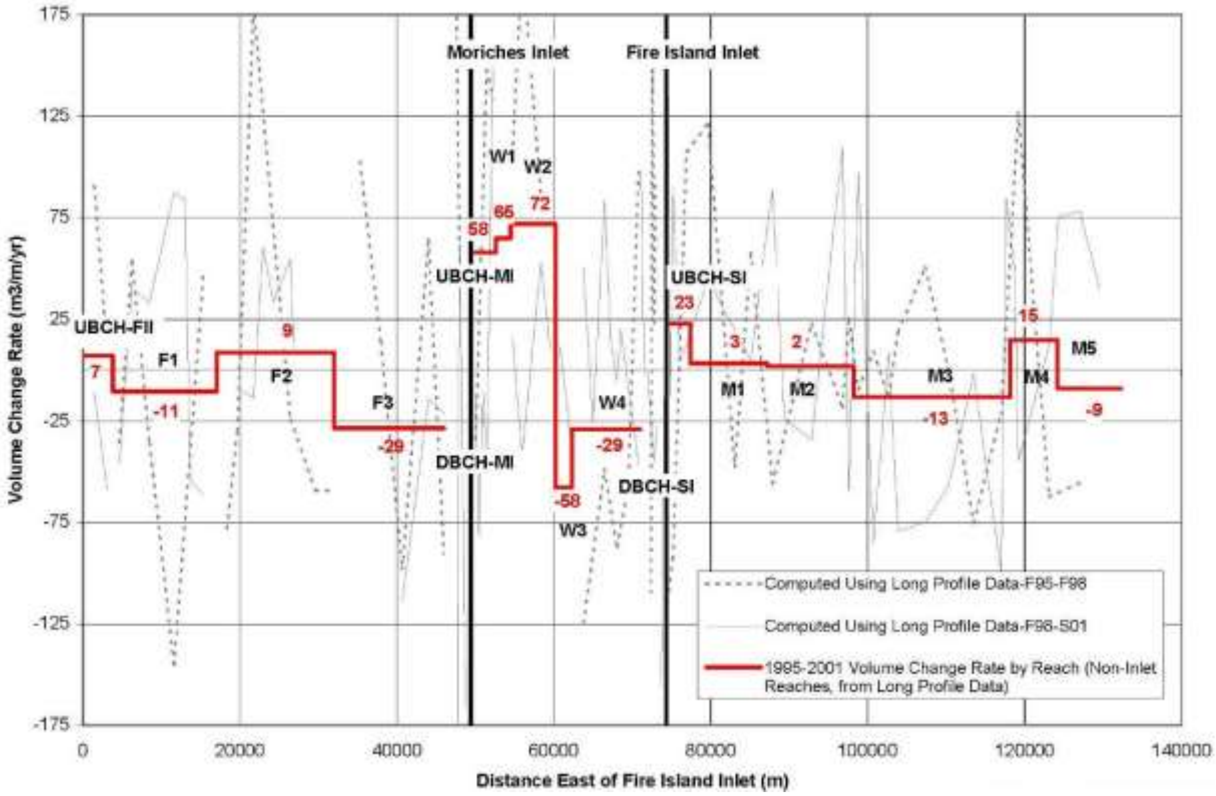


Figure 6-72. Volume Change Rates Computed from Long Profiles (1995-2001)

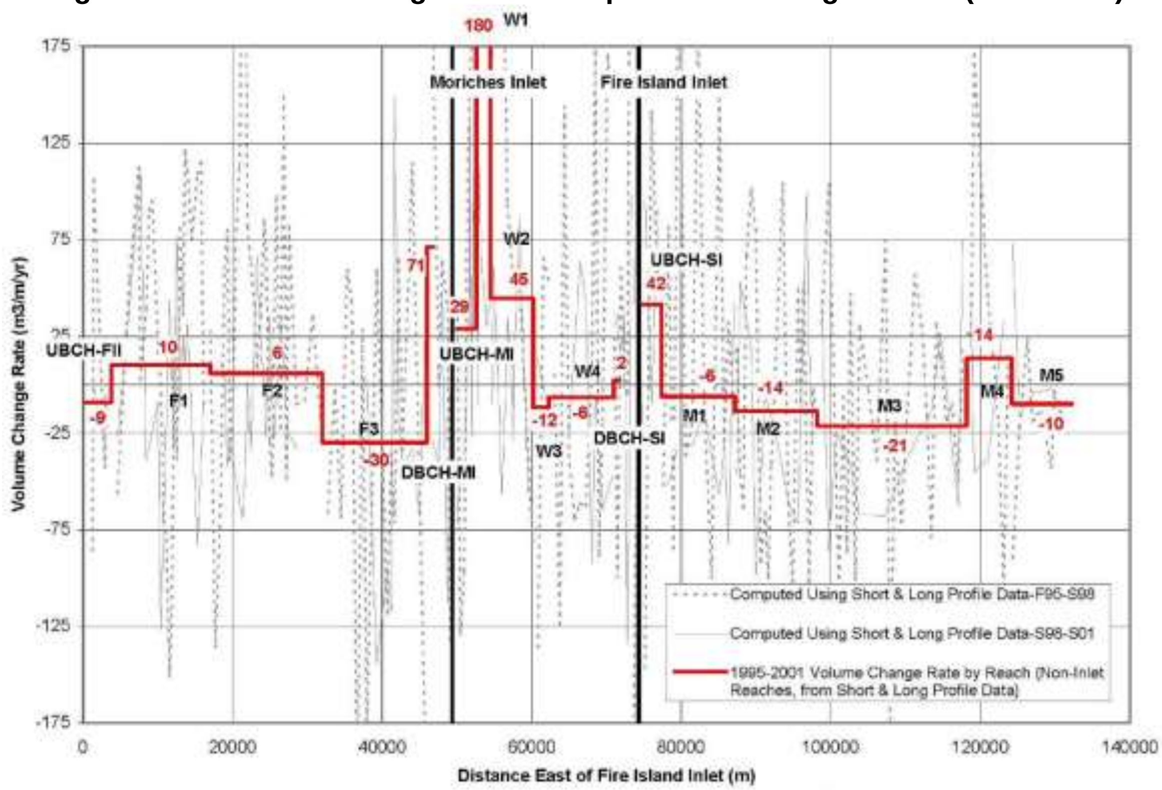


Figure 6-73. Volume Change Rates Computed from Short and Long Profiles (1995-2001)

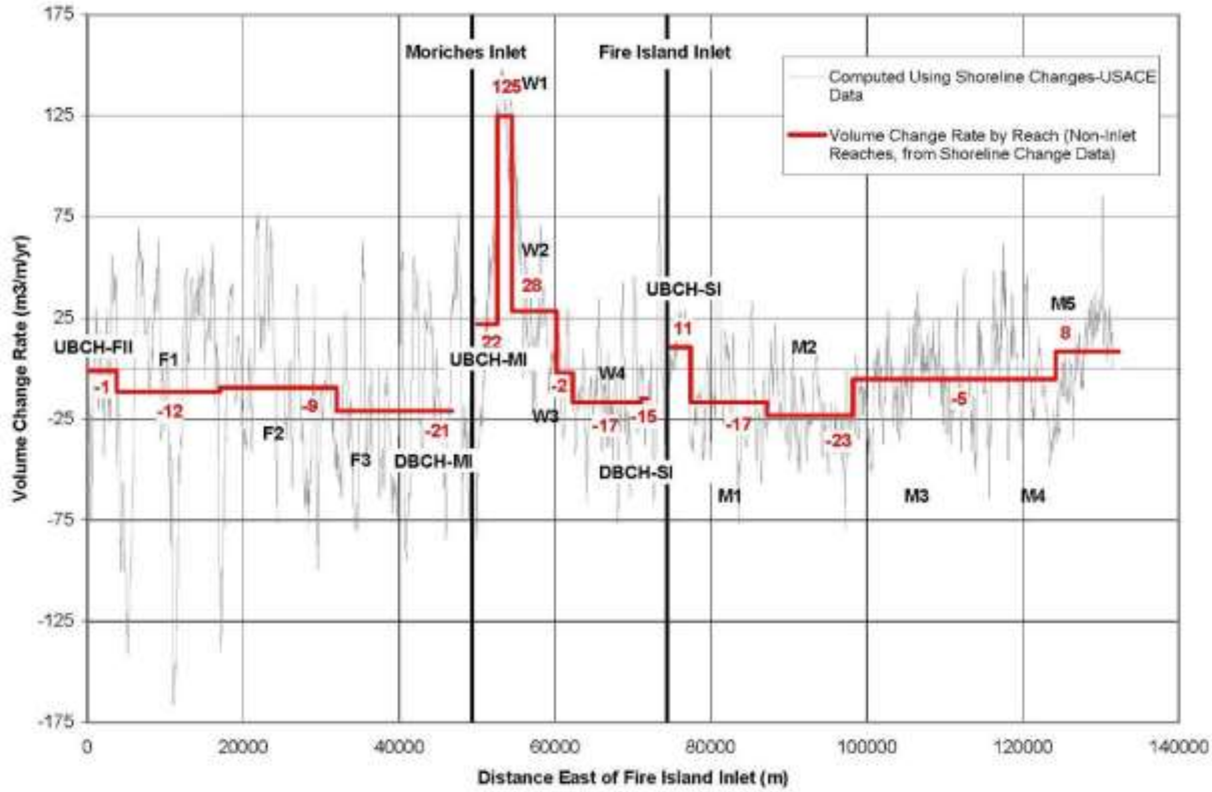


Figure 6-74. Volume Change Rates Computed from Shoreline Changes (USACE data, 1995-2001)

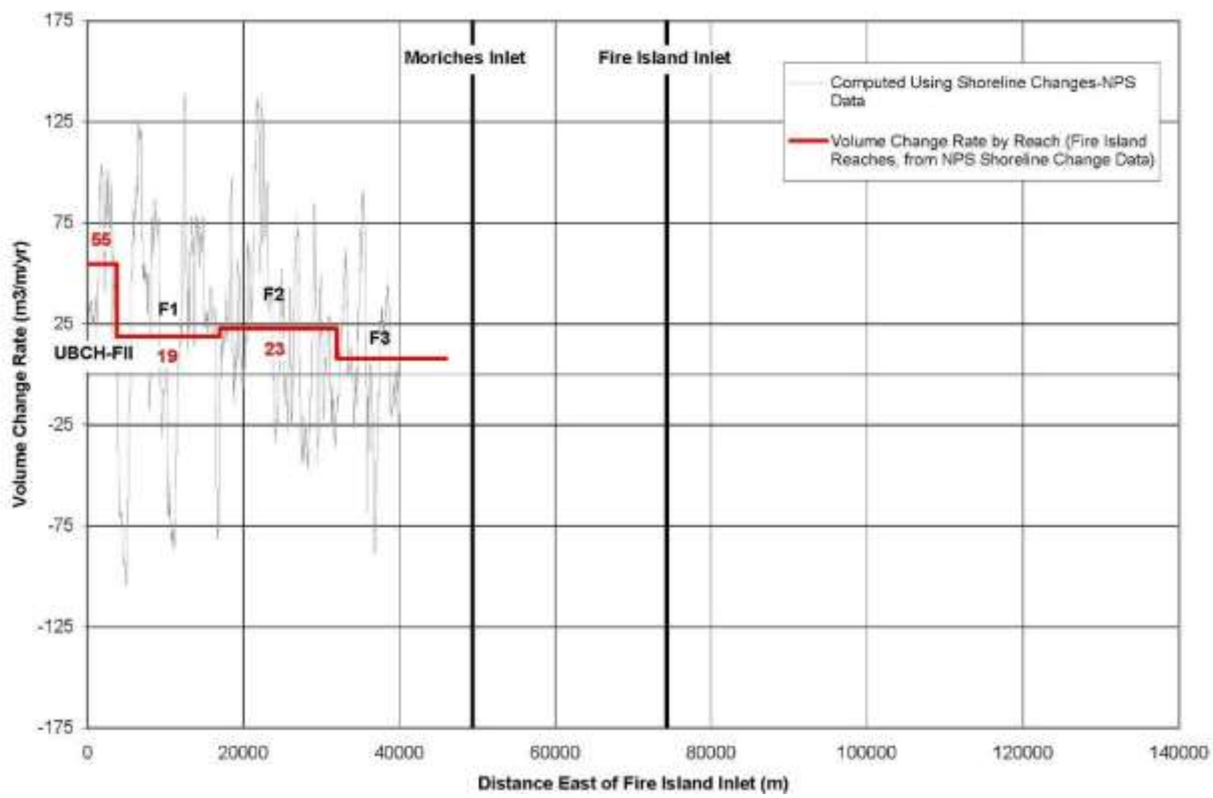


Figure 6-75. Change Rates Computed from Shoreline Changes (USGS data, 1995-2001)

6.3.4.2 Sediment Budget

The Recent (1995-2001) sediment budget was developed cell by cell from east to west. The volume changes presented in Table 6-5 were used with results of potential sediment transport calculations to build the regional budget. This process was based not only on the calculations themselves, but also on previous work and engineering judgment.

Montauk Point provides a convenient boundary condition for longshore sediment transport estimates and sediment budget formulation. Specifically, if zero longshore transport at the east end of the Montauk bluffs morphological reach (M5) is assumed, transport rates at the western end of that reach and at the boundaries between reaches farther west can be computed by solving the sediment budget equation for each reach. Therefore, the regional sediment budget was developed by starting at Montauk and progressing west until reaching Fire Island Inlet. A very similar approach was used in developing most of previous sediment budgets (e.g., Gravens et., 1999 and Kana, 1995). Computed transport rates at the updrift boundary of the inlet cells were also compared to previous estimates based on updrift jetty impoundment or updrift spit growth (Fire Island Inlet) and, in the case of Shinnecock Inlet, with a numerical estimate of potential longshore sediment transport. Table 6-7 shows shows the inputs used in the Sediment Budget Equation shown in Section 6.3.3.

Table 6-7. *Recent (1995-2001)* Sediment Budget Equation Inputs

Reach	ΣQ_{in} (m^3/yr)	ΣQ_{out} (m^3/yr)	$\Sigma \Delta V$ (m^3/yr)	*P* (m^3/yr)	*R* (m^3/yr)	*Residual* (m^3/yr)	*LST* (m^3/yr)
M5	0	81,000	-80,000	1,000	0	0	81,000
M4	81,000	-1,000	82,000	0	0	0	-1,000
M3	-1,000	70,000	-71,000	0	0	0	70,000
M2	70,000	220,000	-150,000	0	0	0	220,000
M1	220,000	279,000	-59,000	0	0	0	279,000
UBCH-S (east)	279,000	246,000	33,000	0	0	0	246,000
Shinnecock Inlet	246,000	253,000	-7,000	0	0	0	253,000
UBCH-S (west)	253,000	251,000	2,000	0	0	0	251,000
W4	251,000	308,000	-57,000	0	0	0	308,000
W3	308,000	331,000	0	23,000	0	0	331,000
W2	331,000	327,000	411,000	407,000	0	0	327,000
W1	327,000	437,000	123,000	233,000	0	0	437,000
UBCH-M (east)	437,000	368,000	69,000	0	0	0	368,000
Moriches Inlet	368,000	366,000	2000	0	0	0	366,000
UBCH-M (west)	366,000	345,000	21,000	0	0	0	345,000
FI3	345,000	421,000	-63,000	13,000	0	0	421,000
FI2	421,000	393,000	132,000	104,000	0	0	393,000
FI1	393,000	318,000	75,000	0	0	0	318,000
UBCH-FII (east)	318,000	394,000	-14,000	62,000	0	0	394,000
Fire Island Inlet	394,000	-140,000	159,000	0	375,000	0	-140,000
UBCH-FII (west)	-140,000	145,000	28,000	313,000	0	0	145,000

Qualitatively, this budget is similar to previous studies in that it shows increasing transport from east to west and it also shows that erosion along the beaches from Montauk Point to Southampton is the main source for a relatively large net westerly directed longshore sediment transport rate updrift of Shinnecock Inlet. The budget also shows erosion along the two barrier island reaches downdrift of Shinnecock and Moriches Inlet: W4 (Tiana Beach) and FI3 (Smith Point County Park and the eastern end of the Wilderness Area), respectively. In fact, erosion rates in reach W4 are very similar to those shown in Kana (1995) and in Gravens et al. (1999), which were approximately 50,000 to 60,000 m^3/yr. On the other hand, erosion rates in the FI3 cell during the 1995-2001 period were roughly half of those shown in those two studies (100,000 to 120,000 m^3/yr). As explained above, this new result seems reasonable considering that Moriches Inlet appears to have been bypassing sand fairly efficiently in recent years.

Perhaps the most significant difference between the *Recent (1995-2001)* budget and previous studies (particular Gravens et al., 1999 and USACE-NAN, 1999) is that Shinnecock and Moriches inlet, and to smaller extent the Westhampton groin field, do not appear to be intercepting as much of the westerly sand flow as they had in the past. This seems reasonable considering that these two inlets have now been open for more than 70 years and stabilized, with rock jetties for over 50 years. And although recent inlet modifications at Moriches Inlet (1986) and Shinnecock Inlet (1990) caused profound changes to the configuration of the channel and the ebb shoal, they do not appear to have caused a significant net increase in ebb shoal volume. However, this finding should be viewed somewhat skeptically until additional surveys are collected and analyzed over the next decade or so to confirm or refute it.

As in the previous studies, particularly in Kana (1995), central Fire Island shoreline (cell F2) appears to be fairly stable or even slightly accreting. The *Recent (1995-2001)* budget also shows net accretion in western Fire Island (75,000 m^3/yr in cell FI1), whereas Gravens et al. suggested very little net accumulation (8,000 m^3/yr) and Kana showed significant erosion (more than 150,000 m^3/yr) despite some fill (roughly 25,000 m^3/yr) being placed in this area during the analysis period for that budget (1955-1979). Kana also shows high erosion rates within Robert Moses State Park between 1955 and 1979 (42,000 m^3/yr) despite fill at rate of 14,000 m^3/yr.

Computed net westerly transport entering Fire Island Inlet between 1995 and 2001 (394,000 m^3/yr) compares favorably with the range of estimates (including Panuzio, 1969; RPI, 1985; Kana, 1995) prior to Gravens et al. (1999), which shows a significantly lower estimate of 194,000 m^3/yr. Increased sediment supply from updrift as a result of more efficient bypassing around Shinnecock and Moriches Inlet and, more importantly, the Westhampton groin field, combined with a large amount of fill placed at Westhampton may be at least partially responsible for increased westerly transport along Fire Island and at Fire Island Inlet between 1995 and 2001. In previous studies, these large westerly transport estimates were arrived at on the basis of historic spit growth analysis at Fire Island and updrift fillet accumulation after construction of the Democrat Point breakwater, however updrift volume changes from Fire Island to Montauk Point did not support that much transport at Fire Island and thus required other sources of sediment such as an offshore supply. Kana (1995) speculated that up until the early 1900s the source of this sediment was an abandoned delta off western Fire Island whereas between 1979 and 1995 this relict source had largely disappeared and the foreshore in western Fire Island was being "cannibalized" instead. Note that the more recent spit growth and impoundment analysis performed by Gravens et al. (1999) suggest slightly lower longshore sediment transport rates than Taney (1961a,b): 159,000 to 300,000 m^3/yr based on spit growth[4] and 385,000 m^3/yr based on impoundment at Democrat Point. The authors considered the latter estimate to be most likely "high" because it probably included "some contribution due to onshore welding of the eastern portion of the Fire Island ebb shoal" after construction of the east jetty.

It is important to note that the *Recent (1995-2001)* sediment budget does not require an offshore sediment source to yield a net westerly transport rate at Fire Island Inlet similar to other estimates that are based on spit growth prior to stabilization or impoundment at Democrat Point. However, this does not necessarily mean that there is no offshore source. In fact, accumulation within the inlet and dredging rates still yield a somewhat low westerly transport rate on Gilgo Beach, downdrift of Fire Island Inlet (145,000 m^3/yr). This rate would likely increase if an offshore source of sediment was added.

6.3.5 *Existing (c. 2001)* Regional Sediment Budget

As explained above, the *Recent* sediment budget is only representative of the 1995-2001 period and should not be used to predict medium- to long-term trends (10-20 year) in the FIMP area. A new *Existing* sediment budget was developed for that purpose. This *Existing (c. 2001)* regional sediment budget incorporates, to the extent possible, relevant long-term trends identified in Gravens et al. (1999) as well as recent changes shown in the 1995-2001 sediment budget. This includes relatively new inlet and shoreline management practices such as the deposition basin at Shinnecock Inlet and the Westhampton Interim Project.

To develop this new *Existing (c. 2001)* regional sediment budget, the *Recent (1995-2001)* regional budget was used in conjunction with the previous *Historic (1979-1995)* and *Existing (c. 1999)* regional sediment budget developed by Gravens et al. In most cases, estimates of volume change rates for the barrier island cells under *Existing (c. 2001)* conditions were computed as a prorated average of the *Recent (1995-2001)* and *Historic (1979-1995)* changes, which effectively results in an estimate of the long-term (1979 to 2001) changes in that cell. 1995-2001 estimates were used in cells where the recent trends are considered more representative of existing and future conditions (e.g., FI3). At the inlets, an attempt was made to account for recent management and morphological evolution changes without discounting previously identified long term trends and established theories such as the impact of inlets on longshore sediment transport and barrier island processes.

It was assumed that beach fill practices in Montauk Beach (cell M5), Westhampton, and Fire Island (mostly at Fire Island Pines, the westernmost Fire Island communities, and RMSP) would continue at a rate similar to the 1990s and early 2000s, with the exception of large storms or specific hot spots. These conditions may require placement of fill in areas that did not receive fill during that period (e.g., Ocean Beach) which would affect the sediment budget. Assumptions regarding the behavior of the fill placed at Westhampton Beach were made based on previous work by Gravens et al. (1999) and the changes observed since project construction in 1996-97.

---

[4] Gravens et al. (1999) developed two estimates based on different active beach depths. See Gravens et al. (1999) for details.

Computed longshore sediment transport rates were compared with results from previous studies and checked against estimates developed by Gravens et al. (1999) using the Wave Information Studies (WIS) 1976 to 1994 database and the shoreline evolution model, GENESIS. Gravens et al. calculated net and gross LST rates from Fire Island to approximately 6 km west of Montauk Point. Their model was calibrated such that the magnitude of the potential sediment transport rate at Fire Island Inlet agreed with accepted rates. Therefore, the long-term accuracy of these computed potential transport rates is limited by the accuracy of the accepted rates at Fire Island inlet and the degree to which the wave climate in the 1976 to 1994 is representative of average long-term conditions. Nonetheless, results of the *Existing (c. 2001)* conditions sediment budget were checked against the model results and assumptions and/or results were modified, if necessary.

The proposed *Existing (c. 2001)* conditions regional sediment budget is summarized in Table 6-8. This budget reflects coastal processes, inlet management activities, and beach fill placement rates assumed to be representative of the present time (c. 2001) and medium- to long-term conditions in the FIMP project area.

Table 6-8. *Existing (c. 2001)* Sediment Budget Equation Inputs

Reach	ΣQ_{in} (m^3/yr)	ΣQ_{out} (m^3/yr)	$\Sigma \Delta V$ (m^3/yr)	*P* (m^3/yr)	*R* (m^3/yr)	*Residual* (m^3/yr)	*LST* (m^3/yr)
M5	0	91,000	-90,000	1,000	0	0	91,000
M4	91,000	65,000	26,000	0	0	0	65,000
M3	65,000	64,000	1,000	0	0	0	64,000
M2	64,000	134,000	-70,000	0	0	0	134,000
M1	134,000	157,000	-23,000	0	0	0	157,000
UBCH-S (east)	157,000	151,000	6,000	0	0	0	151,000
Shinnecock Inlet	151,000	119,000	32,000	0	0	0	119,000
UBCH-S (west)	119,000	117,000	2,000	0	0	0	117,000
W4	117,000	172,000	-55,000	0	0	0	172,000
W3	172,000	167,000	5,000	0	0	0	167,000
W2	167,000	192,000	100,000	125,000	0	0	192,000
W1	192,000	267,000	50,000	125,000	0	0	267,000
UBCH-M (east)	267,000	238,000	29,000	0	0	0	238,000
Moriches Inlet	238,000	213,000	25,000	0	0	0	213,000
UBCH-M (west)	213,000	211,000	2,000	0	0	0	211,000
FI3	211,000	274,000	-63,000	0	0	0	274,000
FI2	274,000	296,000	78,000	100,000	0	0	296,000
FI1	296,000	351,000	25,000	0	0	0	351,000
UBCH-FII (east)	351,000	40,4000	9,000	69,000	0	0	40,4000
Fire Island Inlet	40,4000	-79,000	108,000	0	375,000	0	-79,000
UBCH-FII (west)	-79,000	206,000	28,000	313,000	0	0	206,000

Overall, the *Existing (c. 2001)* sediment budget shows longshore sediment transport rates that fall within the range of previously published estimates (e.g., 151,000 m^3/yr, 238,000 m^3/yr, and 404,000 m^3/yr entering Shinnecock, Moriches, and Fire Island Inlets, respectively). Transport appears to increase from east to west and the initial source of sediment feeding the net longshore sediment transport from east to west appears to be erosion along the beaches from Montauk Point to Southampton, specifically in cells M5, M2, and M1.

The budget suggests that the effects of the Westhampton groin field have been largely offset by the construction of the Westhampton Interim Project. Specifically, the estimate of sediment entering Moriches Inlet (238,000 m^3/yr) is higher than values presented in other recent studies (e.g., Kana, 1995) and very similar to the estimate by Taney (1961a,b) of 230,000 m^3/yr under conditions prior to the construction of the Westhampton groin field.

Also similarly to previous studies, the *Existing (c. 2001)* condition budget suggests erosion along the two barrier island reaches downdrift of Shinnecock and Moriches Inlet: W4 (Tiana Beach) and FI3 (Smith Point County Park and the eastern end of the Wilderness Area), respectively, albeit at somewhat smaller rates, particularly at cell FI3. This reduction may be a result of increased bypassing at Shinnecock and Moriches Inlet in recent years.

Nonetheless, the three inlets in the FIMP study area, particularly Fire Island Inlet, continue to be a sediment sink. Available surveys and assumptions regarding the effects of sea level rise on inlet morphology suggest that Shinnecock, Moriches, and Fire Island Inlet accumulate 32,000, 25,000, and 108,000 m^3/yr, respectively. Therefore, the total loss to the system is 165,000 m^3/yr, which represents a significant percentage of the average longshore sediment transport along the FIMP shoreline. However, it is important to note that approximately 431,000 m^3/yr of beach fill dredged from offshore sources is placed along the shoreline between Montauk Point to Fire Island Inlet, mostly as part of the Westhampton Interim Project (250,000 m^3/yr).

Offshore sediment sources are not explicitly included in the *Existing (c. 2001)* condition regional sediment budget because it was not required to balance the budget at Fire Island Inlet or to yield reasonable estimates of longshore transport entering and exiting the inlet. However, the possibility of its existence and contribution to the nearshore sediment transport system was recognized. Specifically, differences between potential net transport computed with GENESIS and transport computed based on volume changes in central Fire Island suggest an onshore sediment flux of approximately 200,000 m^3/yr to explain the well documented relative shoreline stability in this area. This value matches the estimate suggested by Schwab et al. (2000) based on the sediment budget by Kana (1995). However, Gravens et al. (1999) suggested a lower value, 75,000 m^3/yr, based on results from their sediment budget and Fire Island spit growth estimates

A relatively large number of data sources were used to develop this sediment budget, including shorelines digitized from aerial photography, shorelines surveyed using an ATV and a GPS system, beach profile surveys, boat-based bathymetric surveys, and LIDAR surveys. There are obvious benefits associated with a large dataset, such a spatial and temporal coverage. However, large differences in the results obtained from each dataset (e.g., volume changes based on shoreline vs. profile data) also underscore the significant level of uncertainty associated with this type of study. Although a detailed quantitative analysis was not possible because many of the individual uncertainty contributions cannot be determined (e.g., uncertainty due to lack of survey coverage at the inlets or due to differences in datum reduction methodologies), it is judged that the uncertainty in the estimates presented above is significant, perhaps as much as the estimates themselves in some cases. Even so, it is concluded that the proposed *Existing (c. 2021)* condition sediment budget provides a realistic, albeit semi-quantitative, description of the sediment transport processes that can be used to assist in the planning, design, and formulation of coastal storm damage risk reduction measures for the FIMP project area.

6.4 Inlet Processes

To assess the efficacy of any proposed inlet modification alternatives, the changes in seabed morphology induced by the alternatives must be estimated. The proposed modification must meet the stated goals of navigation and improved bypassing, without exacerbating existing problems or creating new ones. Typically, the most efficient method of calculating these effects is using numerical models.

Detailed modeling over large space and time scales may provide a reasonable estimate of the expected morphological changes near an inlet under different conditions. However, modeling of all hydrodynamic and wave events along with associated morphological changes requires excessively long simulation times. As a result, much of the research on morphological evolution of sandy and muddy coastlines has recently focused on how to make predictions with microscale (process based) models using input and process filtering (reduction) techniques (DeVriend et al., 1993; Whitehouse and Roberts, 1999; EMPHASYS Consortium, 2000). This approach reduces computational intensity by selecting a limited number of representative hydrodynamic and sediment transport conditions to use as input to a microscale process-based model. One example of input filtering is the use of a representative “morphological tide” where the sediment transport and bed evolution is driven by the average tide that would move the same amount of material per cycle as the full tidal time series. Hydrodynamics simulation is only required over one tide cycle rather than over a weeks- or months-long simulation.

A similar approach can be used for schematizing the influence of waves in morphological models. These techniques offer the advantage of reduced model run times provided their accuracy has been tested. However, it is important to note that even state-of-the-art, microscale, sediment transport models cannot explicitly incorporate all of the physical processes that drive morphological evolution. Input and process filtering only represent an additional simplification of simplifications already inherent in the formulation of the sediment transport equations.

Representative tidal variation and wave climate forcing may be applied to represent a seasonal wave climate until the morphological changes are so significant that the hydrodynamic conditions have to be recalculated. In this way, transport and bottom computations are repeated a number of times, until bottom changes are sufficiently large that a full hydrodynamic computation is required. This reduces the number of hydrodynamic runs, which is the most computationally demanding element of the morphological process.

Overall, morphological evolution is a very difficult process to model given the inherent uncertainties. Nevertheless, if model results are acceptable, it is a great tool to compare different alternatives and study the impacts.

The following sections present the modeling system capabilities and results for the three inlets comprising FIMP. Inlet modeling encompassing all physical processes was completed for the Existing Condition. Simplified inlet modeling that excluded morphological changes, sediment transport processes and contributions of overwash and/or breaching to bay water levels was performed for the Future Without Project Condiiton. The only components included were tide, storm surge, waves and winds. Modeling was not performed for the future improved conditions.

The background information including bathymetry, waves, tides and model calibration is extensive and has been documented in several earlier documents. Therefore, this information was not included in this appendix, but is available upon request to the New York District.

6.4.1 Modeling System

Morphological models of Shinnecock, Moriches and Fire Island inlets were developed using the morphological model of the general Delft3D modeling system. This model (Delft3D-MOR) fully integrates the effects of waves, currents and sediment transport on morphological evolution. Delft3D-MOR includes the following components:

- *Waves (Waves module*): The HISWA model (Holthuijsen et al, 1989) solves refraction and dissipation of directionally spread random waves. Several computations through a tidal cycle are carried out in one call. Model formulation is similar to STWAVE.
- *Hydrodynamics (Flow Module):* Delft3D Flow is a multidimensional (2D or 3D) hydrodynamic (and transport) simulation program which calculates non-steady flow and transport phenomena that result from tidal and meteorological forcing on a curvilinear, boundary-fitted grid. In 3D simulations, the vertical grid is defined following the sigma co-ordinate approach. The model solves the Navier-Stokes equations for incompressible fluid under the shallow water and the Boussinesq assumptions. In the vertical momentum equation, the vertical accelerations are neglected, resulting in the hydrostatic pressure equation.
- *Sediment transport (Sand or Silt Module):* This model applies the time dependent results obtained from the Waves and Flow modules to calculate the sediment transport in the curvilinear flow grid. In the case of non-cohesive sediment, the model can either calculate the total transport or account separately for bed-load and suspended sediment transport. A special version of this model may be used to calculate the sediment transport for cohesive material. The implemented sediment transport formulas are: Engelund-Hansen, Meyer-Peter-Muller, Swanby (Ackers-White), General Formula based on Meyer-Peter-Muller, Bijker with Waves, Van Rijn, and Ribberink – Van Rijn.
- *Bottom changes (Bottom Module):* Computes the bed level variation induced by the sediment transport module by solving the bed level continuity equation.

Each component of the model is developed and calibrated separately, then combined to simulate bed morphology. The model allows the simulation of time scales from days to years. The morphological process is built up from morphological time steps, which consist of a simulation of wave-current interaction over a period of time, followed by the computation of the average sediment transport over that period, and the bottom update.

6.4.2 Existing Condition

6.4.2.1 Shinnecock Inlet

The following paragraphs describe the observed patterns in the model results in regards to hydrodynamics, waves, sediment transport, and morphology for the existing conditions at Shinnecock Inlet. Results are based on the calibrated versions of the models described in Section 6.4.1.

Hydrodynamics

Figure 6-76 illustrates the modeled current speed and vectors during a typical peak flood tide at Shinnecock Inlet. Current speeds are relatively low over the ebb shoal (0.3-0.7 m/s) and do not increase significantly until the immediate vicinity of the jetties. In the throat of the inlet, currents are strong (over 2 m/s). Currents remain high over the flood shoal (1.0 m/s) and into the channel past the commercial fishing docks and Ponquogue Bridge (0.9 – 1.0 m/s). Because of the relatively shallow bay inside the inlet, the deeper channels attract more flow and consequently have higher currents. Velocities remain lower outside the inlet because flow is drawn from all directions over relatively constant depths. Flow accelerates through the constriction caused by the inlet and the fixed jetties.

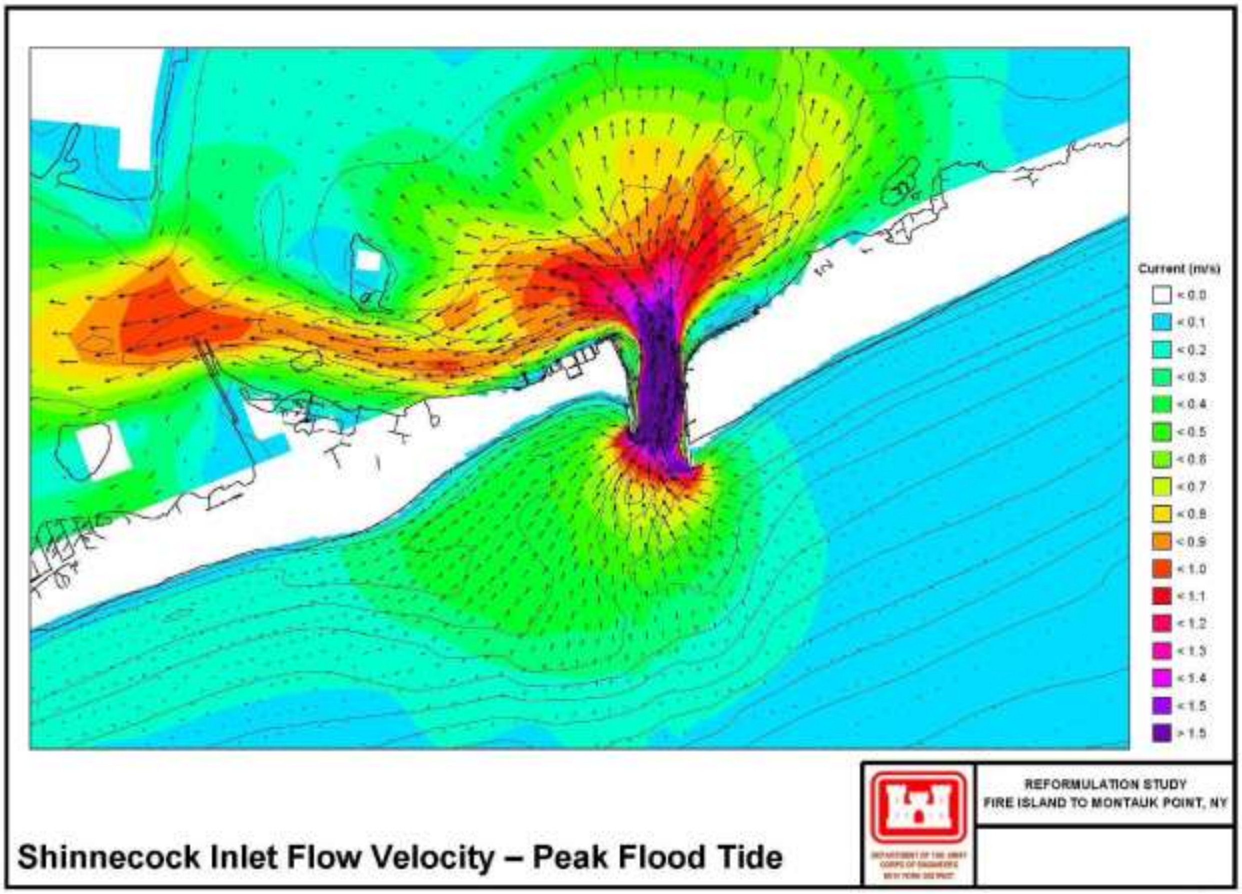


Figure 6-76. Shinnecock Inlet Peak Flood Tide

Figure 6-77 shows modeled flow patterns during a typical ebb tide. Flow is drawn from the interior of Shinnecock Bay and ejected through the inlet. Current speeds in the interior channels is of the same order as during the flood tide (0.9 – 1.0 m/s) in the opposite direction. Flow is constricted from the bay into the throat of the inlet. Velocities in the throat are again high (over 2 m/s), but now the flow velocity is maintained out over the ebb shoal as a jet. The values of the ebb velocities are smaller than those during flood. This corresponds to the definition of Shinnecock inlet as a flood dominated inlet (Militello and Kraus, 2001). The alignment of the jet principally follows the alignment of the deposition basin, skewed a bit to the west, probably due to the offset of the western jetty. Morphological modeling results show that the channel tends to align with the flow, relocating to the west in a more NE-SW alignment, between maintenance dredging projects.

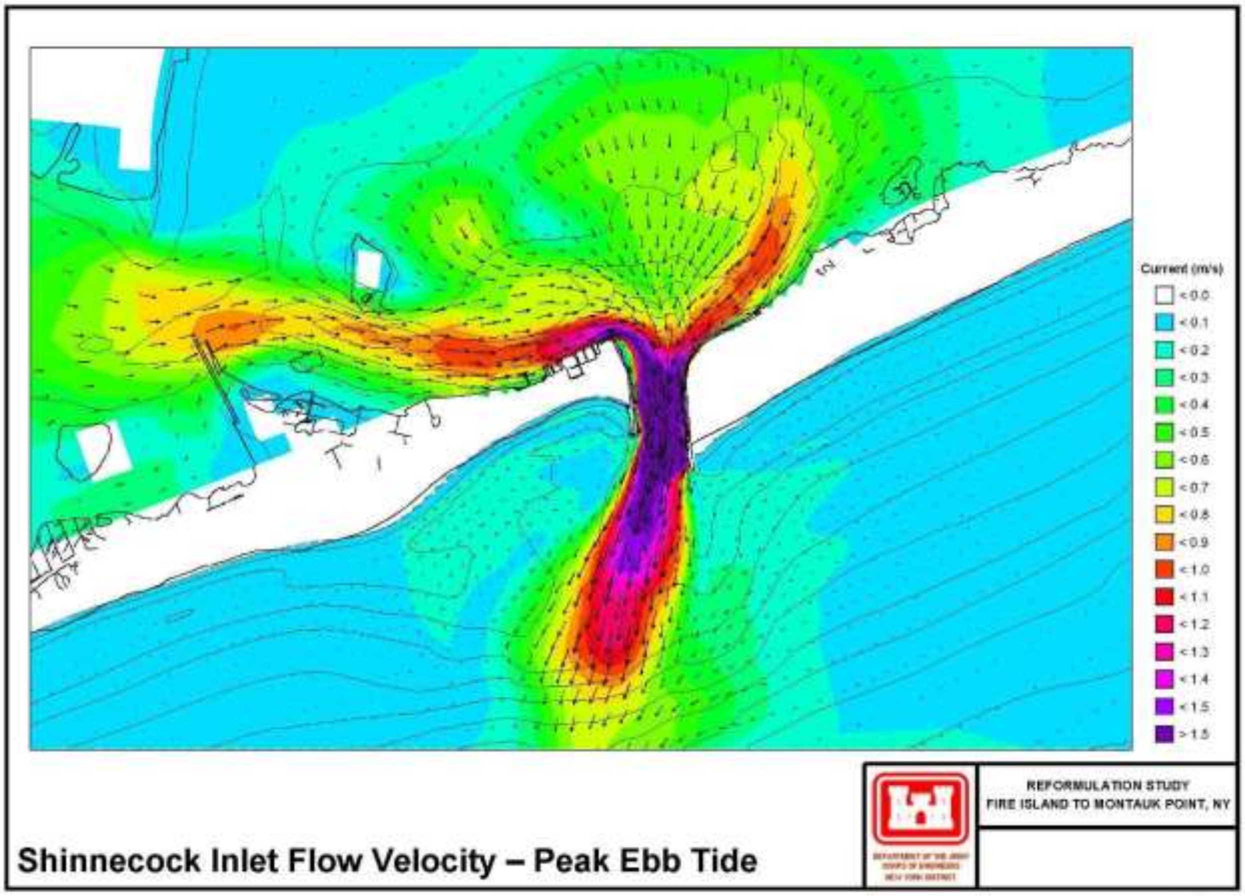


Figure 6-77. Shinnecock Inlet Peak Ebb Tide

Waves and Sediment Transport Potential

Figure 6-78 to Figure 6-81 display wave shoaling/refraction coefficients and initial sediment transport rates for the four wave sectors delineated for the morphological model input filtering. For Shinnecock Inlet, the four schematized wave directions are 105°, 115°, 145°, and 210° clockwise from north (Nautical convention) at a depth of 25 m. Each plot shows, in the lower frame, wave refraction/shoaling coefficients (*KrKs*) for a 1-meter, 9-second offshore wave (the wave used as the average morphological wave condition for each principal direction). The top frame of each plot shows the resulting tidally-averaged sediment transport rates. These rates were computed by combining the bottom stresses resulting from currents averaged over the representative tide, wave orbital velocity, and radiation stress-induced currents. The sediment transport rates represent the initial potential at the beginning of the morphological modeling. Note that in a conventional sediment transport modeling effort, these rates would be extrapolated over time to compute the bed change. In the morphological analysis, the rates are altered to account for the bed evolution and its effects on waves and currents.

Figure 6-78 plots the wave patterns and sediment transport patterns resulting from a wave with an offshore direction of 105 degrees. This wave condition occurs approximately 20% of the time. The waves are traveling obliquely to the shoreline. The wave direction vectors in the lower panel of the figure show nearshore waves oriented toward the northwest. The plot shows waves breaking along the shoreline east of the inlet and on the eastern jetty. Waves shoal up on the eastern side of the deposition basin, over the east lobe of the ebb shoal. Waves focus on and shoal over the west lobe of the ebb shoal and break on the shoreline west of the inlet. Wave heights are greatly reduced in the throat of the inlet due to sheltering of the eastern jetty and the fast currents in the throat.

The upper panel of Figure 6-78 shows the results of the sediment transport potential due to this wave condition. Longshore transport of sand is very strong on the eastern shoreline due to wave breaking. The oblique angle of incidence of the waves increases the strength of the westward flow. Transport at the eastern jetty is also strong, showing transport into the inlet entrance and the deposition basin. There is strong transport potential over the west lobe of the ebb shoal. Transport vectors are directed along the shoal toward where the ebb shoal welds to the shoreline. On the west side coastline, there is a moderate longshore transport toward the west, from the fillet on the western jetty toward the ebb shoal and from ebb shoal west toward Westhampton Beach. There is also significant transport potential in the throat of the inlet and in the entrance channel/deposition basin and around the west jetty. These potentials are mainly due to the tidal currents. Wave heights in these areas are not great and the water depths are typically large (>15 feet). The depth averaged tidal currents, however, are strong (Figure 6-76 and Figure 6-77). Tidal average transport potentials in the channel are directed outward south of the west jetty and inward north of the jetty. The strength of the potentials lessens away from the throat of the inlet. It is expected that some deposition may occur in these areas along the gradient of the potential.

Figure 6-79 shows modeled wave patterns and transport potentials for waves arriving from 115° (ESE). Wave patterns are similar to those from Figure 6-78. A notable difference is that waves from this angle strike the coastline west of the inlet more perpendicularly. The resulting longshore transport is weaker westward from the inlet. Transport potential between the deposition basin and the west lobe of the ebb shoal is stronger.

Figure 6-80 reports modeled wave patterns and transport potentials for a wave direction of 145° (SE). Waves arrive nearly perpendicular to the offshore contours. Wave coefficients are greater, since waves largely do not refract. Because of the normal approach, the longshore transport travels both east and west along the coastline. Nodal points develop on either side of the inlet, coinciding to the points where the ebb shoal meets the shoreline. Outward of the nodal points, longshore transport is away from the inlet. Inside the nodal points, transport is toward the inlet.

Figure 6-81 shows modeled wave patterns and sediment transport potentials for waves from 210° (SW). Waves point at an eastward angle to the coast in this orientation. Strong eastward longshore transport occurs on the shoreline on both sides of the inlet. Transport on the west lobe of the ebb shoal occurs closer to shore and is directed more toward the inlet. Transport along the bottom of the deposition basin is strong from the east jetty to the west side of the basin, before decreasing in intensity. Sediments transported away from the jetty are expected to deposit on east side of the deposition basin.

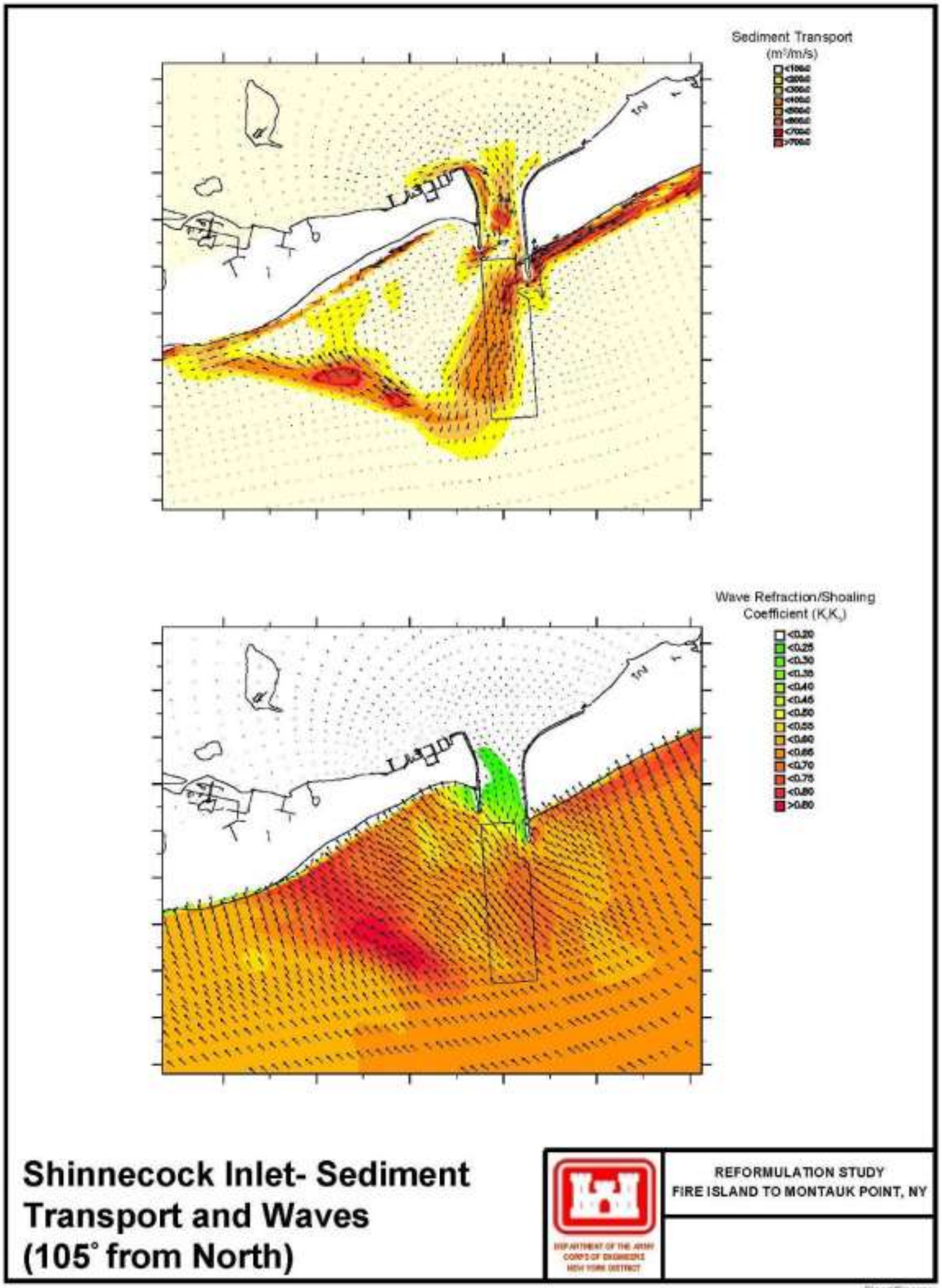


Figure 6-78. Shinnecock Inlet Wave and Sediment Transport Patterns (105 deg)

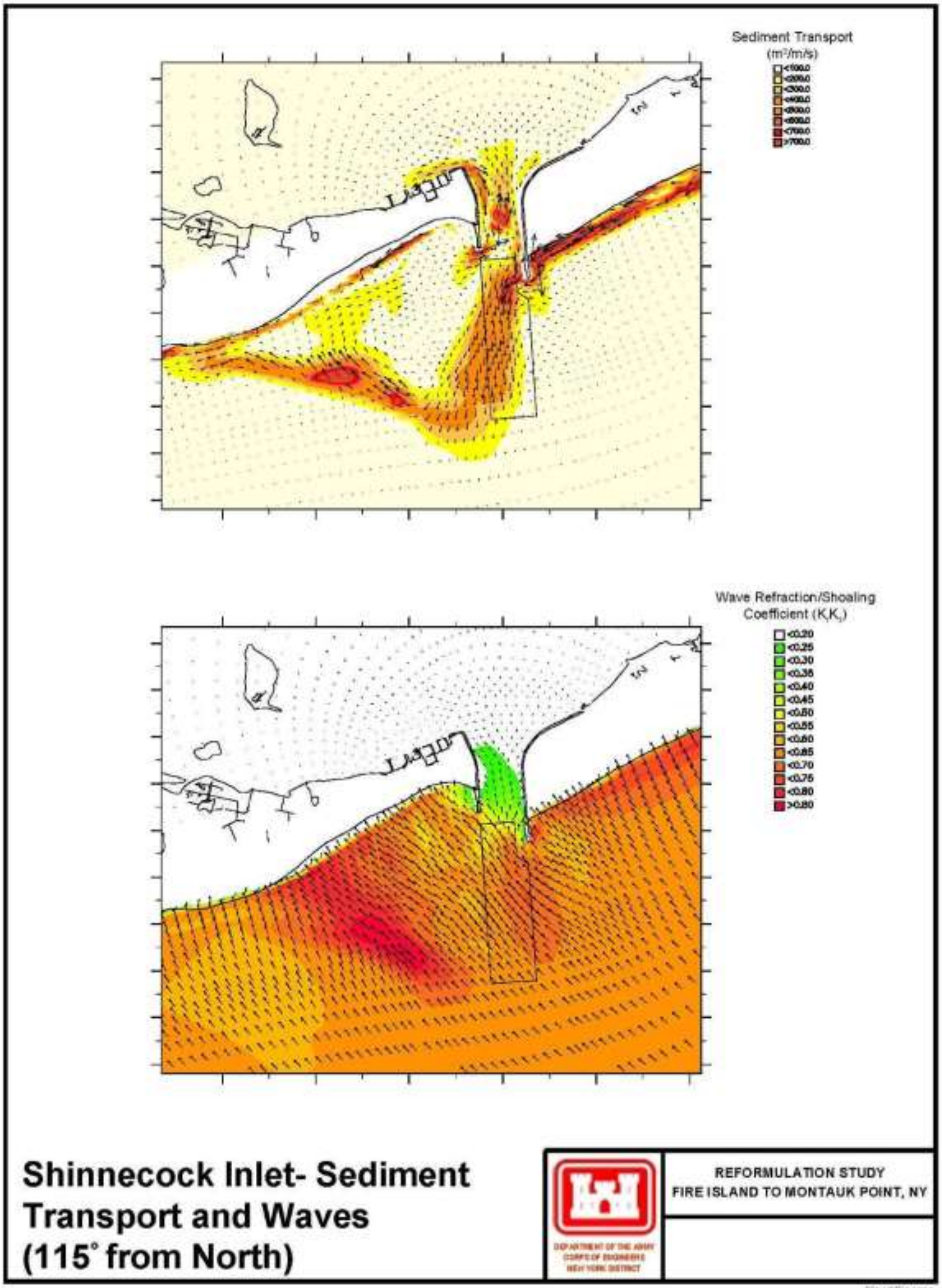


Figure 6-79. Shinnecock Inlet Wave and Sediment Transport Patterns (115 deg)

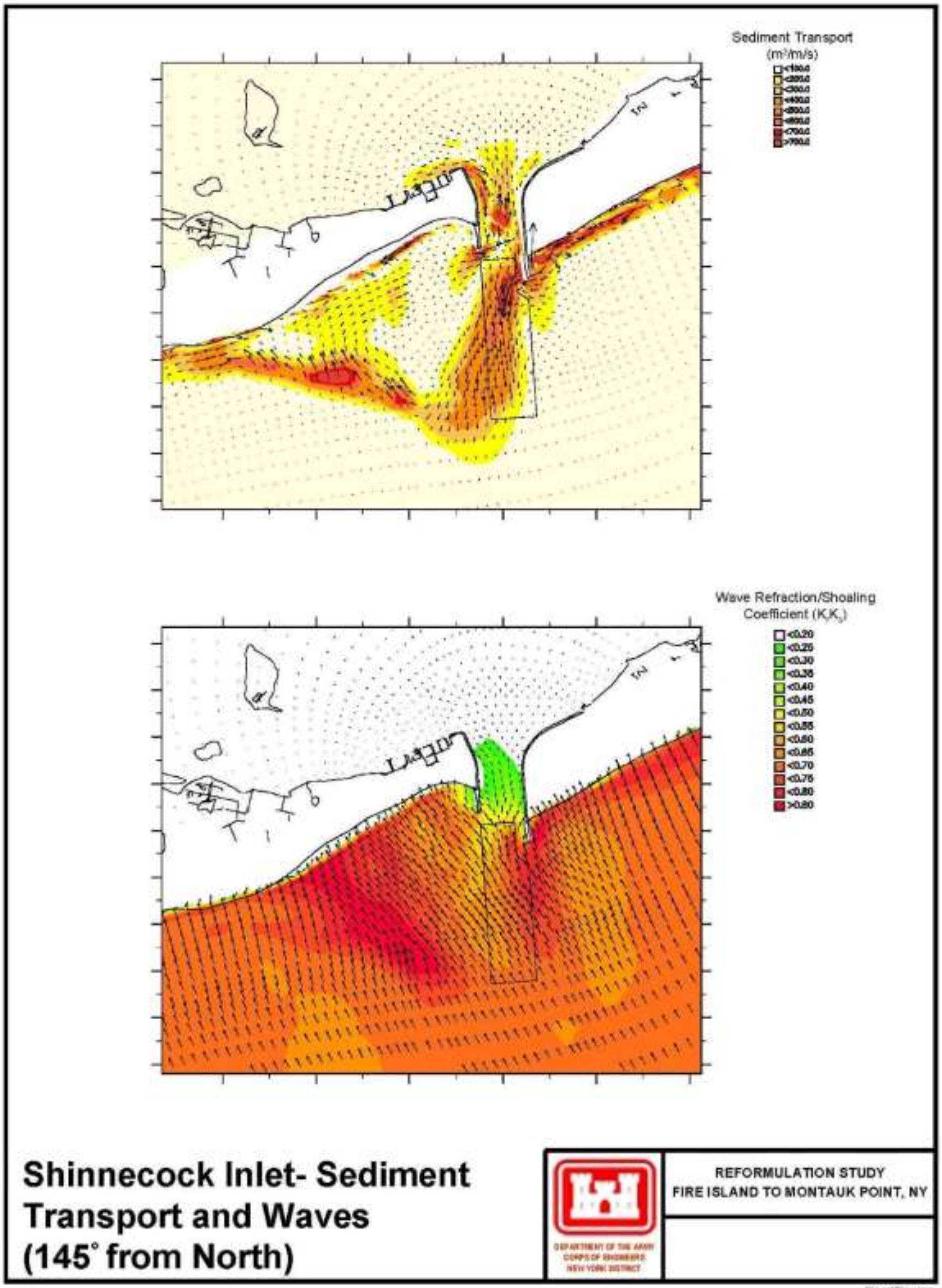


Figure 6-80. Shinnecock Inlet Wave and Sediment Transport Patterns (145 deg)

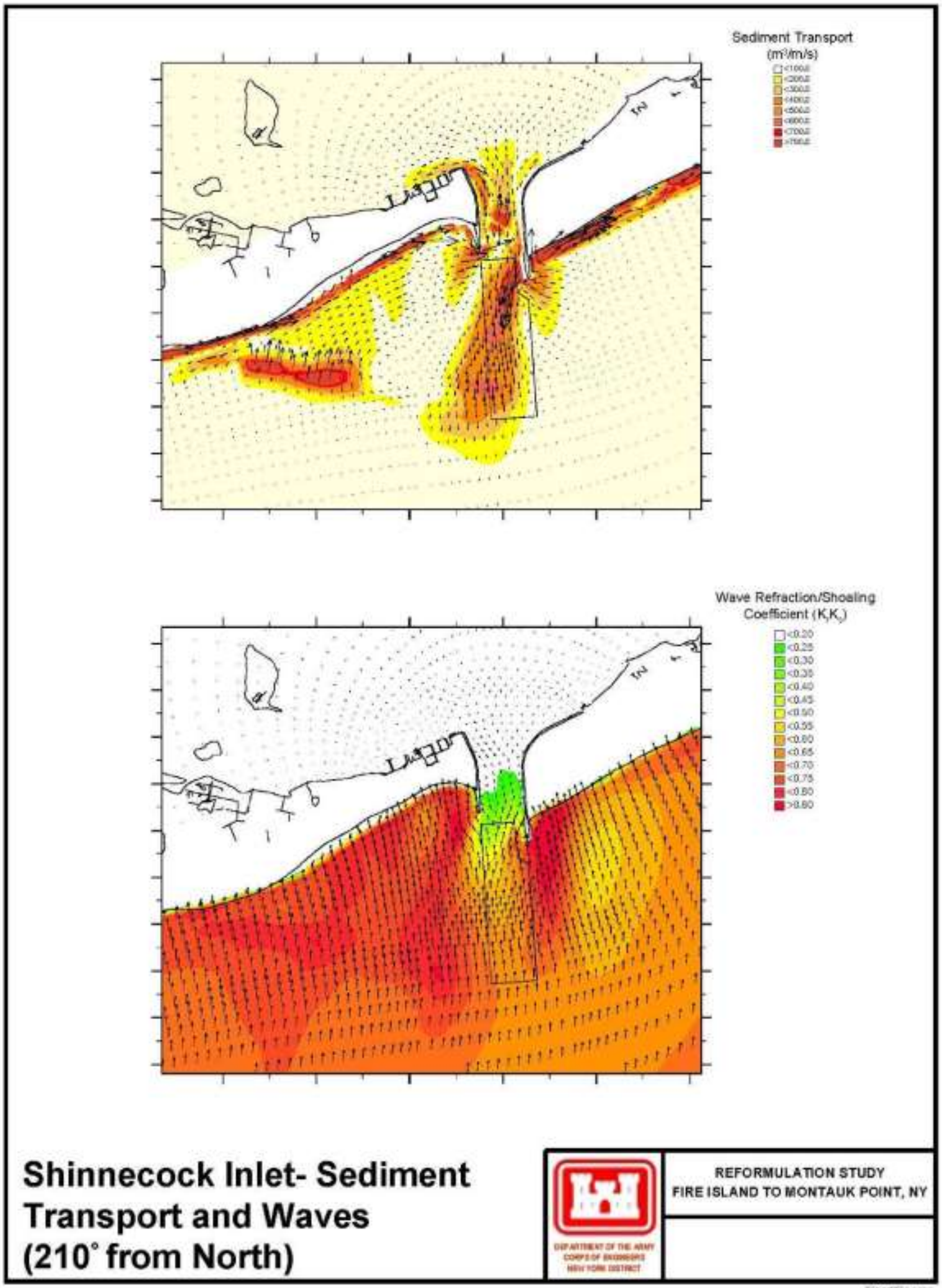


Figure 6-81. Shinnecock Inlet Wave and Sediment Transport Patterns (210 deg)

6.4.2.2 Moriches Inlet

The following paragraphs describe the observed patterns in the model results in regards to hydrodynamics, waves, sediment transport, and morphology for the existing conditions at Moriches Inlet. Results are based on the calibrated versions of the models described in Section 6.4.1.

Hydrodynamics

Figure 6-82 and Figure 6-83 present current patterns and velocities for the peak flood and peak ebb tide, respectively, during the representative morphological tide. Hydrodynamic patterns are similar to those of Shinnecock inlet. The inlet throat experiences high currents (1.0 – 2.0 m/s) on both flood and ebb tide. Similarly to Shinnecock inlet, maximum flood velocities are larger than maximum ebb. The velocities in the interior channels are higher during ebb tide, while during flood the incoming flow spreads out over the flood shoal at about 1.0 m/s. Currents over the ebb shoal on the flood tide are lower (0.5 m/s) than during the ebb tide jet (0.9 – 1.3 m/s).

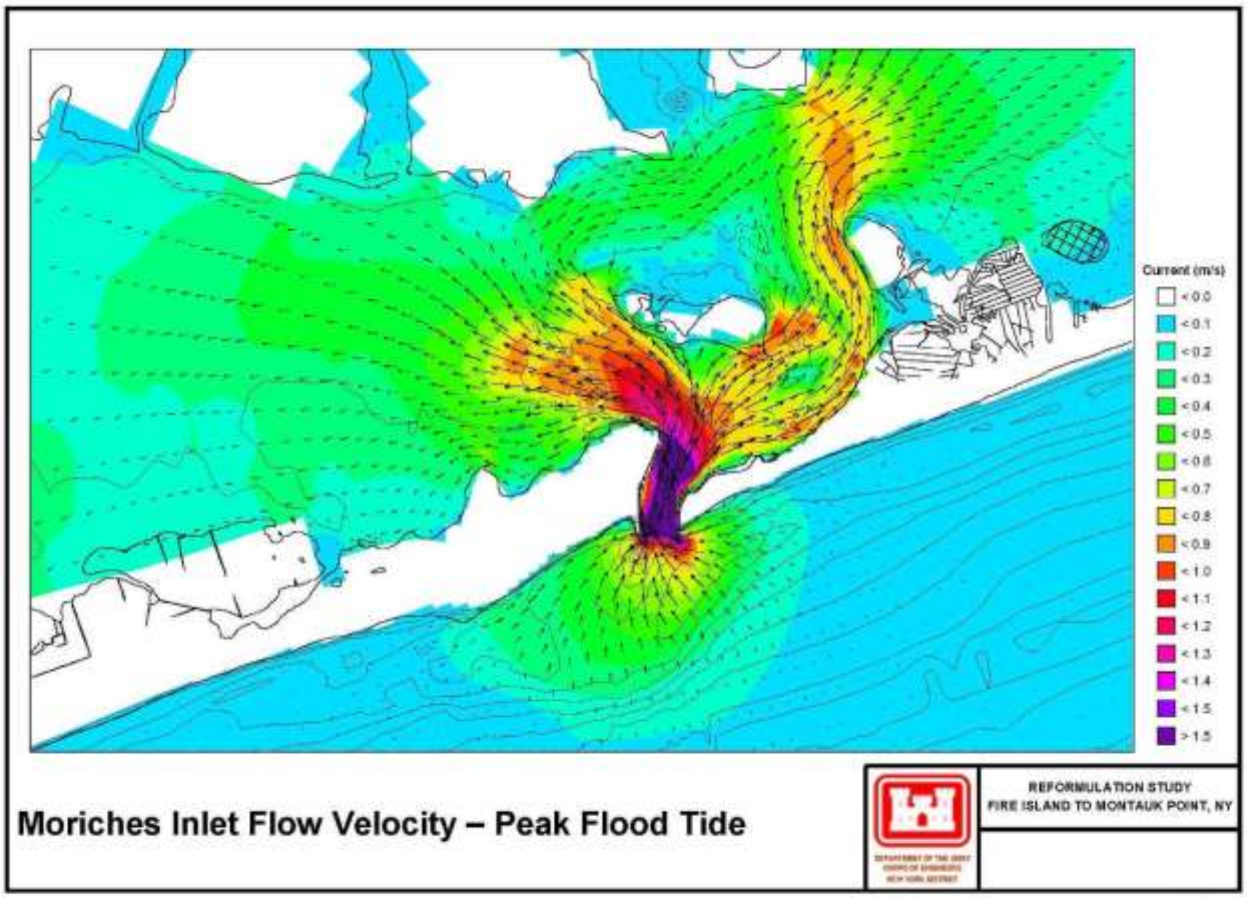


Figure 6-82. Moriches Inlet Peak Flood Tide

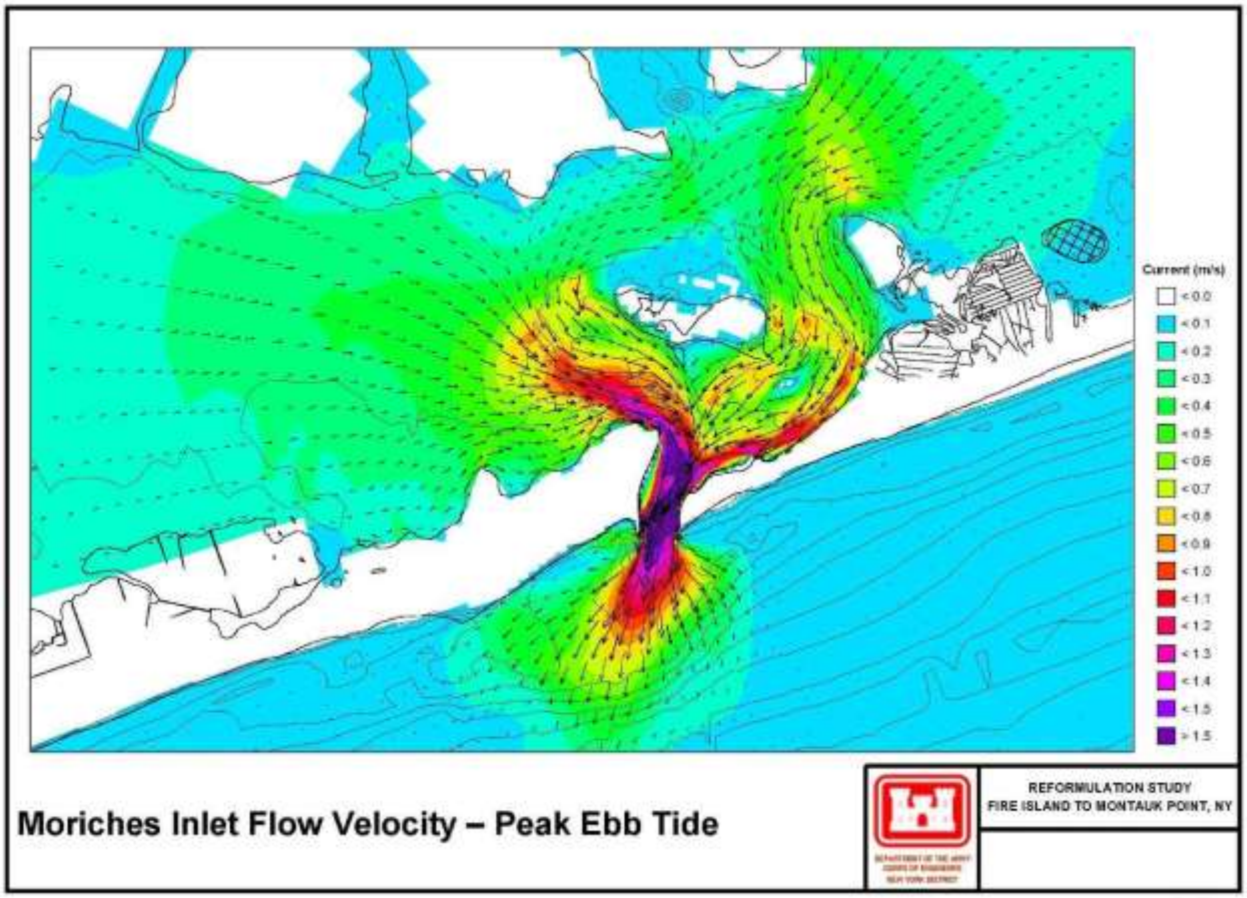


Figure 6-83. Moriches Inlet Peak Ebb Tide

Waves and Sediment Transport Potential

Figure 6-84 to Figure 6-87 show wave and sediment transport patterns for waves arriving from ESE to SW. Patterns are generally similar to those for Shinnecock Inlet, with strong westward longshore transport east and west of the inlet for waves arriving from 110° and 135°. Nodal points in longshore transport form for waves arriving from 165°, and longshore transport shifts to the east for waves arriving from the SW.

Waves break over the west lobe of the ebb shoal from all wave directions. Transport potentials over the shoal are active, with several areas of high potential and large gradients. Transport vectors along the edge of the deposition basin are generally southward for all wave directions, further westward vectors are in towards shore and along the crest of the shoal, and vectors are directed offshore and westward nearer to the shore.

An interesting feature of Moriches inlet is that the shoreline between west jetty and the point where the west lobe of the ebb shoal attaches to the shore is oriented normal to waves arriving from the SE. The longshore transport in this area is low for SE waves, and the shoreline is likely in equilibrium with the predominant wave direction.

For SW waves, the refracted wave vectors are oriented parallel with the deposition basin. Waves breaking over the east lobe of the ebb shoal direct sediment transport toward the east side of the inlet, reversing the direction of transport predominant from the SE and S waves.

Transport potential in the inlet is driven mainly by the average tidal currents. In contrast with Shinnecock where transport inside the jetties is directed inward and outside directed outward, transport at Moriches is directed inward on the western side of the throat and outward on the eastern side.

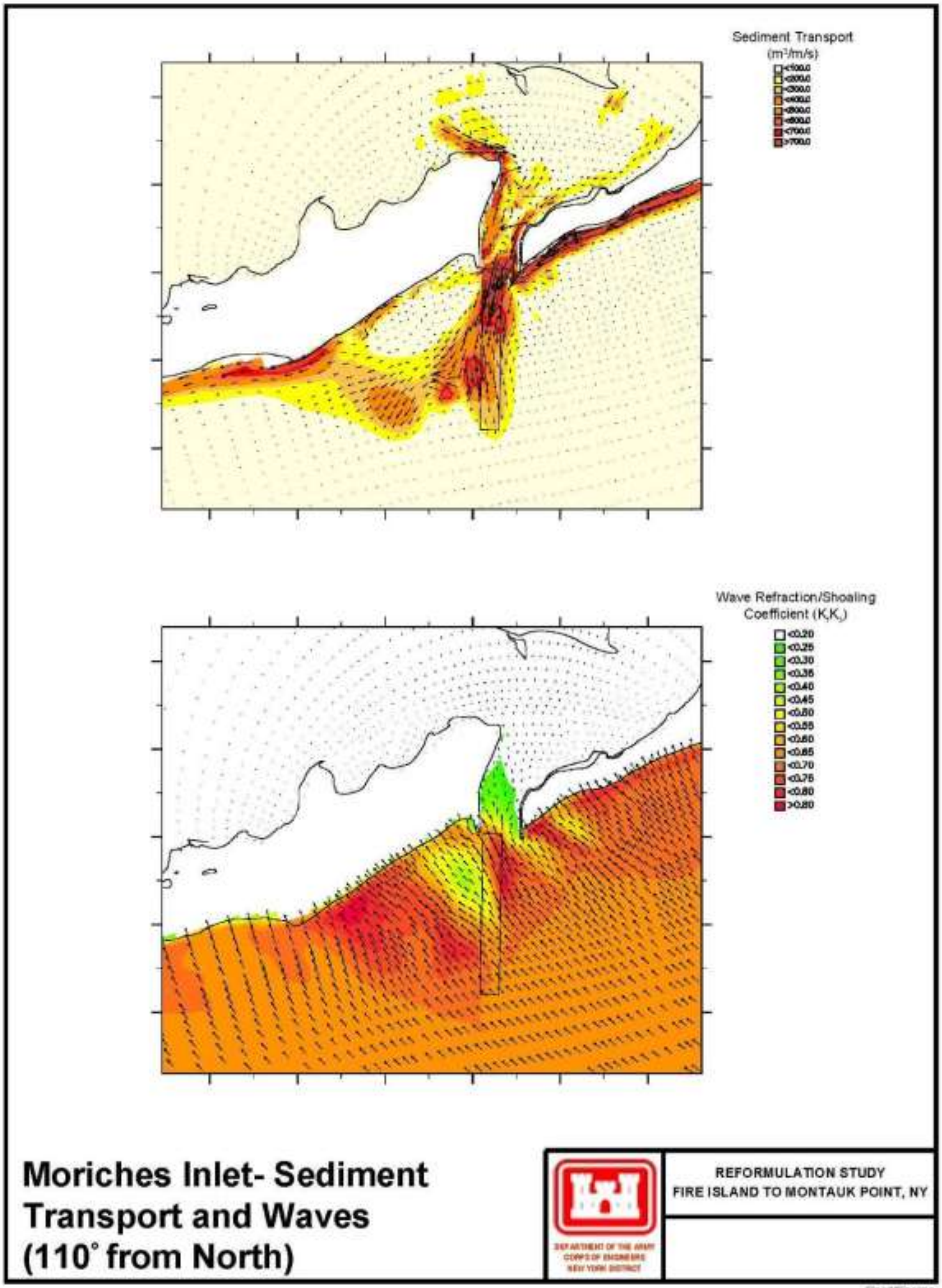


Figure 6-84. Moriches Inlet Wave and Sediment Transport Patterns (110 deg)

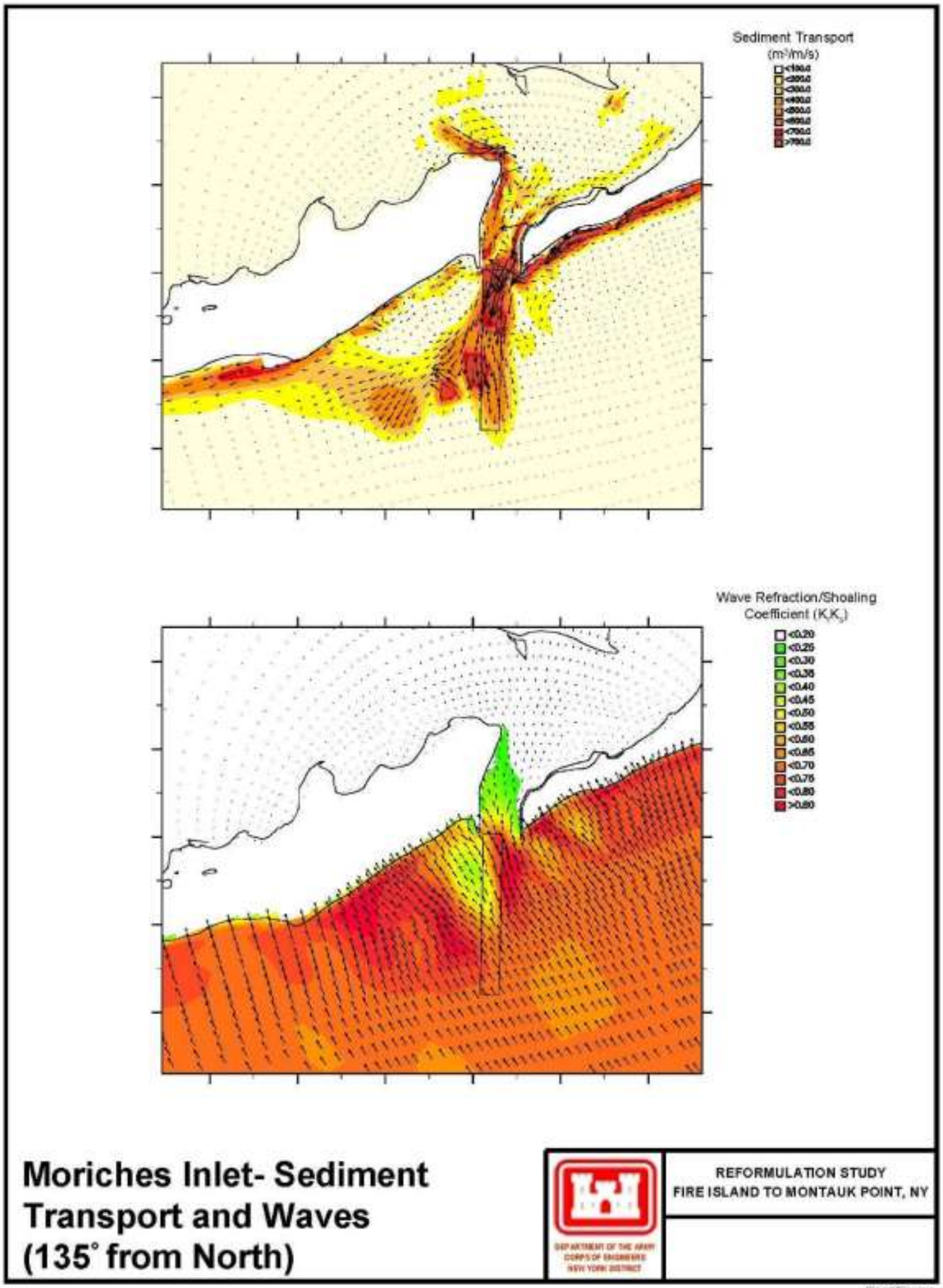


Figure 6-85. Moriches Inlet Wave and Sediment Transport Patterns (135 deg)

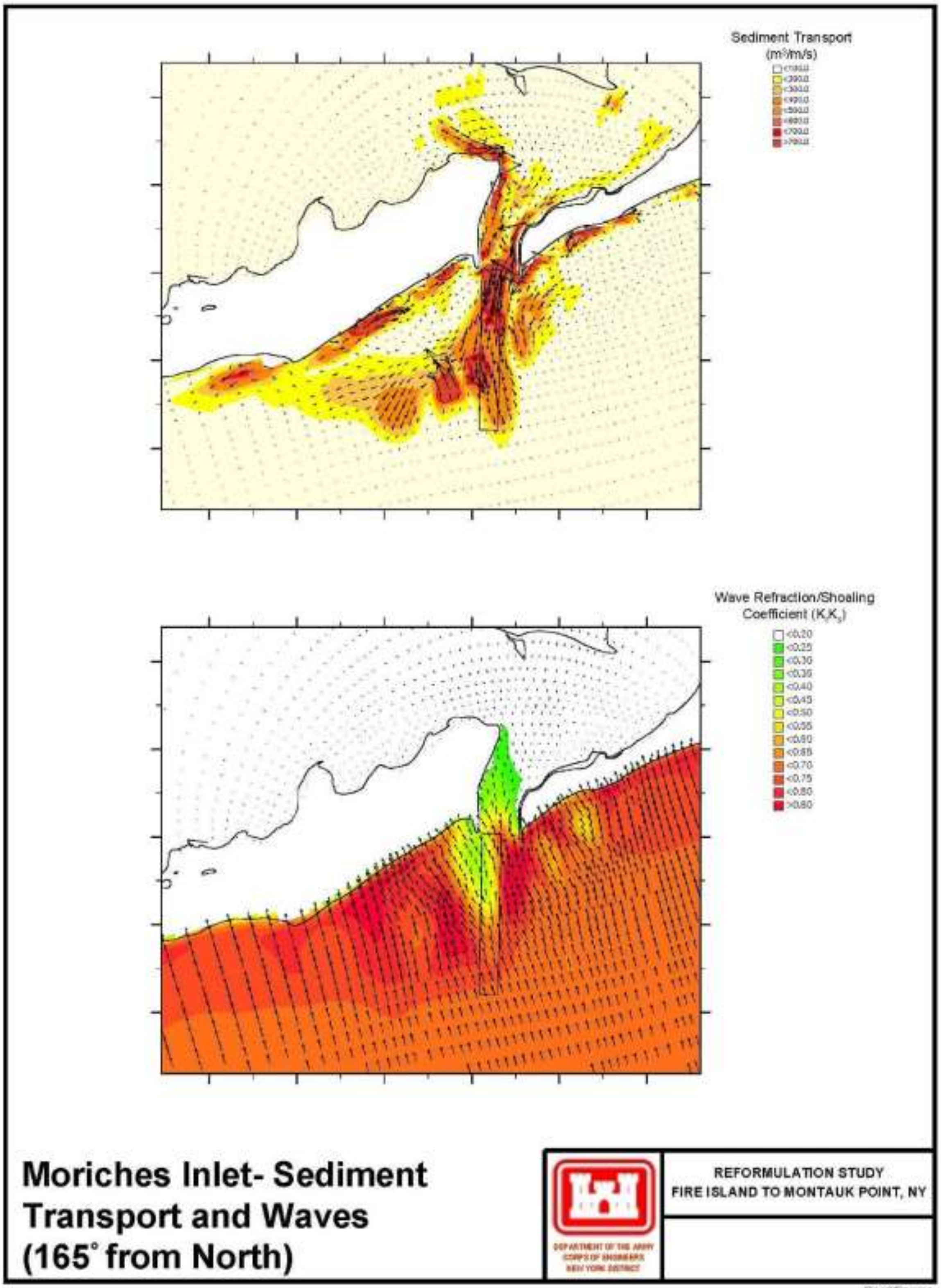


Figure 6-86. Moriches Inlet Wave and Sediment Transport Patterns (165 deg)

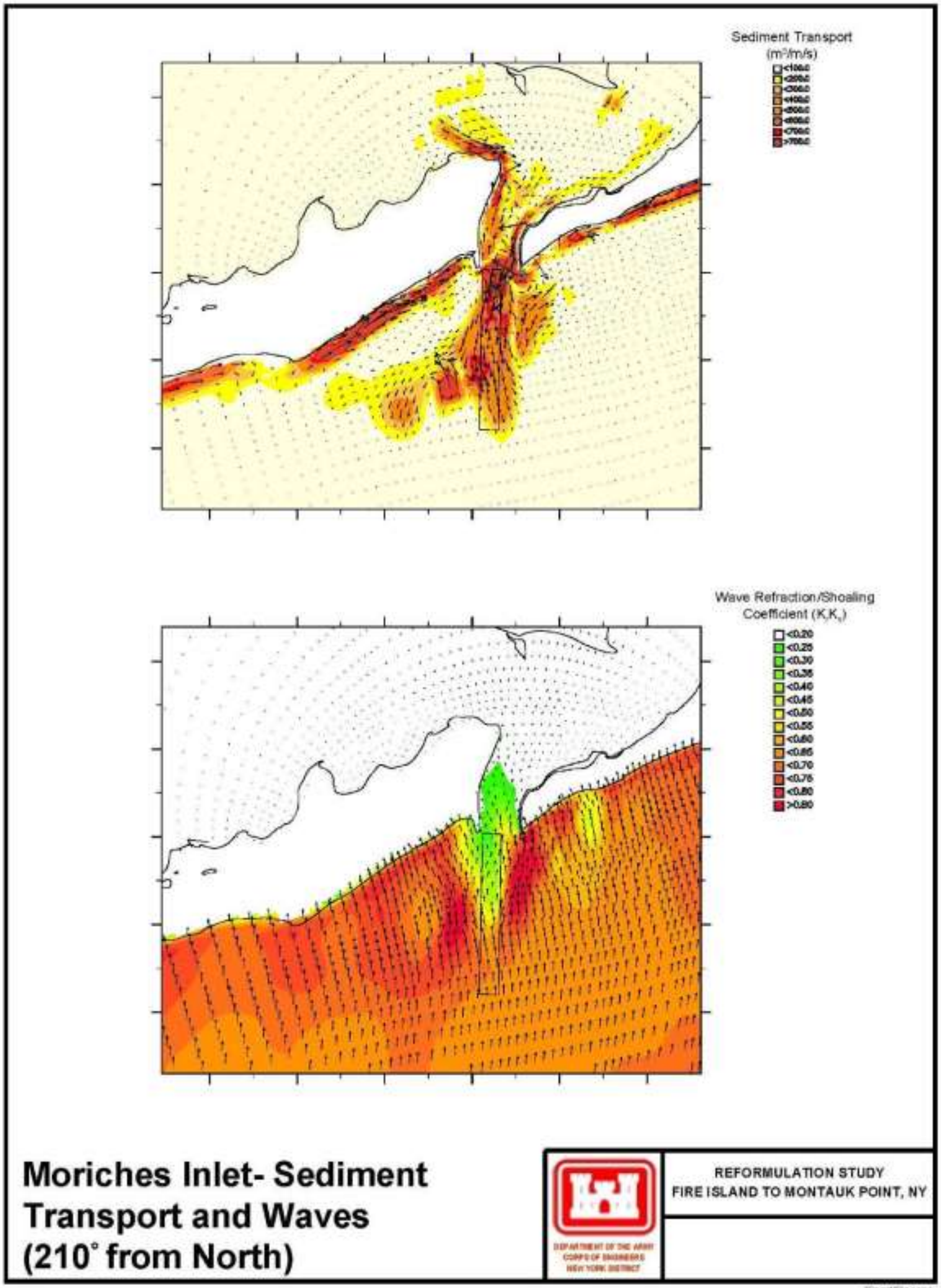


Figure 6-87. Moriches Inlet Wave and Sediment Transport Patterns (210 deg)

6.4.2.3 Fire Island Inlet

The following paragraphs describe the observed patterns in the model results in regards to hydrodynamics, waves, sediment transport, and morphology for the existing conditions at Fire Island Inlet. Results are based on the calibrated versions of the models described in Section 6.4.1.

Hydrodynamics

Figure 6-88 and Figure 6-89 present current patterns and velocities for the peak flood and peak ebb tide, respectively, during the representative morphological tide. The character of Fire Island Inlet is very different from the other two inlets. Fire Island is much older than Moriches or Shinnecock. The inlet is oriented east-west instead of north-south. Velocities are higher through the throat and interior channel during flood tide than during ebb tide. Because the throat of the inlet is wider than either Moriches of Shinnecock, peak velocities are lower (1.5 m/s). Velocities over the ebb shoal are higher during ebb than flood, but the velocity vectors fan out over the shoal more than in the other inlets because the deposition basin is not oriented with the ebb flow.

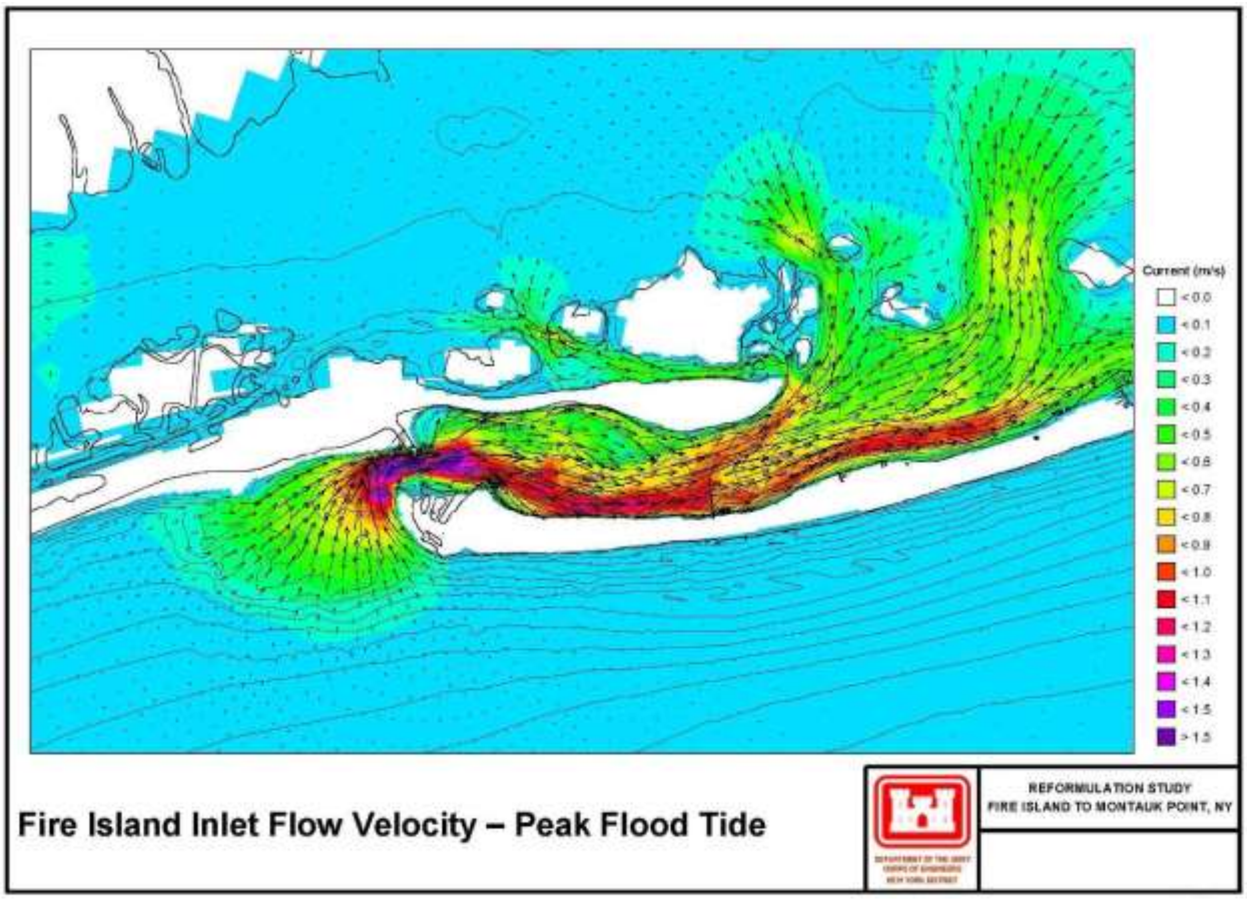


Figure 6-88. Fire Island Inlet Peak Flood Tide

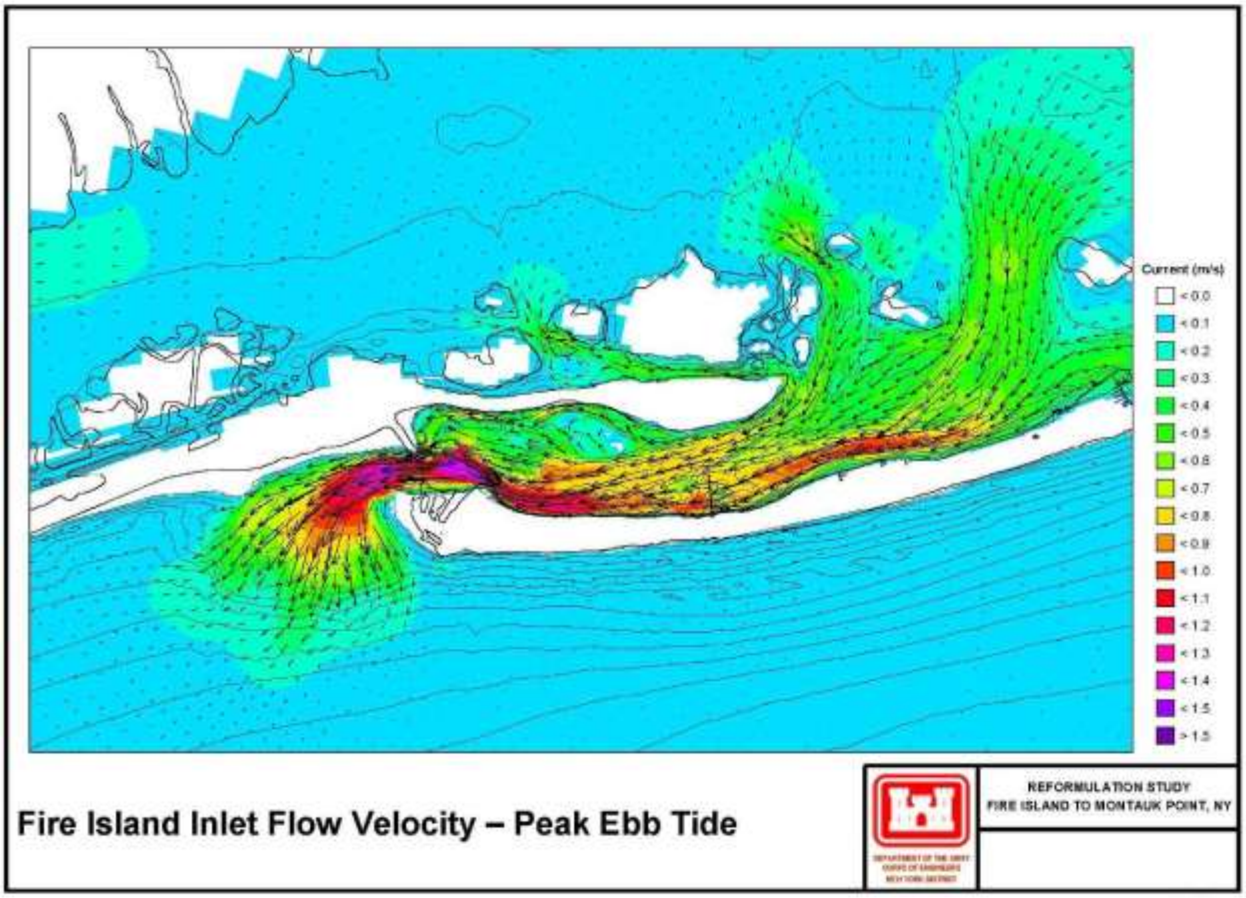


Figure 6-89. Fire Island Inlet Peak Ebb Tide

Waves and Sediment Transport Potential

Figure 6-90 to Figure 6-93 show wave and sediment transport patterns for waves arriving from ESE to SW. Waves and sediment transport along the shoreline east of the inlet behaves similarly to the other two inlets. For waves from ESE to S, longshore sediment transport is directed eastward. For SW waves longshore transport is eastward.

In the mouth of the inlet and west of the inlet, the transport patterns are different from the other inlets. From the tip of the federal jetty to the end of Democrat Point, longshore transport at the shoreline is always directed into the inlet. Waves from all directions breaking on this segment of shore direct transport inward, this is reinforced by the direction of the tidal current during flood tide. This segment of shore is protected from tidal currents during ebb tide. This may explain the rapid shoaling in the deposition basin and the growth of Democrat Point.

In the throat of the inlet, transport potentials are negligible. This is due to the lower average velocities in the throat. This indicates that there is likely little sediment exchange through the inlet.

West of the inlet, there is little longshore transport except during SW waves when there is a moderate transport potential toward the inlet. Between the point where the west side of the ebb shoal welds to shore and the northern jetty, there is a mild longshore return transport toward the jetty. These results indicate that the shoreline of Fire Island west of the inlet appears to be in equilibrium with the predominant wave direction.

The transport potentials over the ebb shoal are much milder than in the other two inlets and follow generally the orientation of the average tidal currents regardless of wave direction. This would seem to indicate a slow outward growth of the ebb shoal, but that the shoal is more or less in equilibrium with the wave climate.

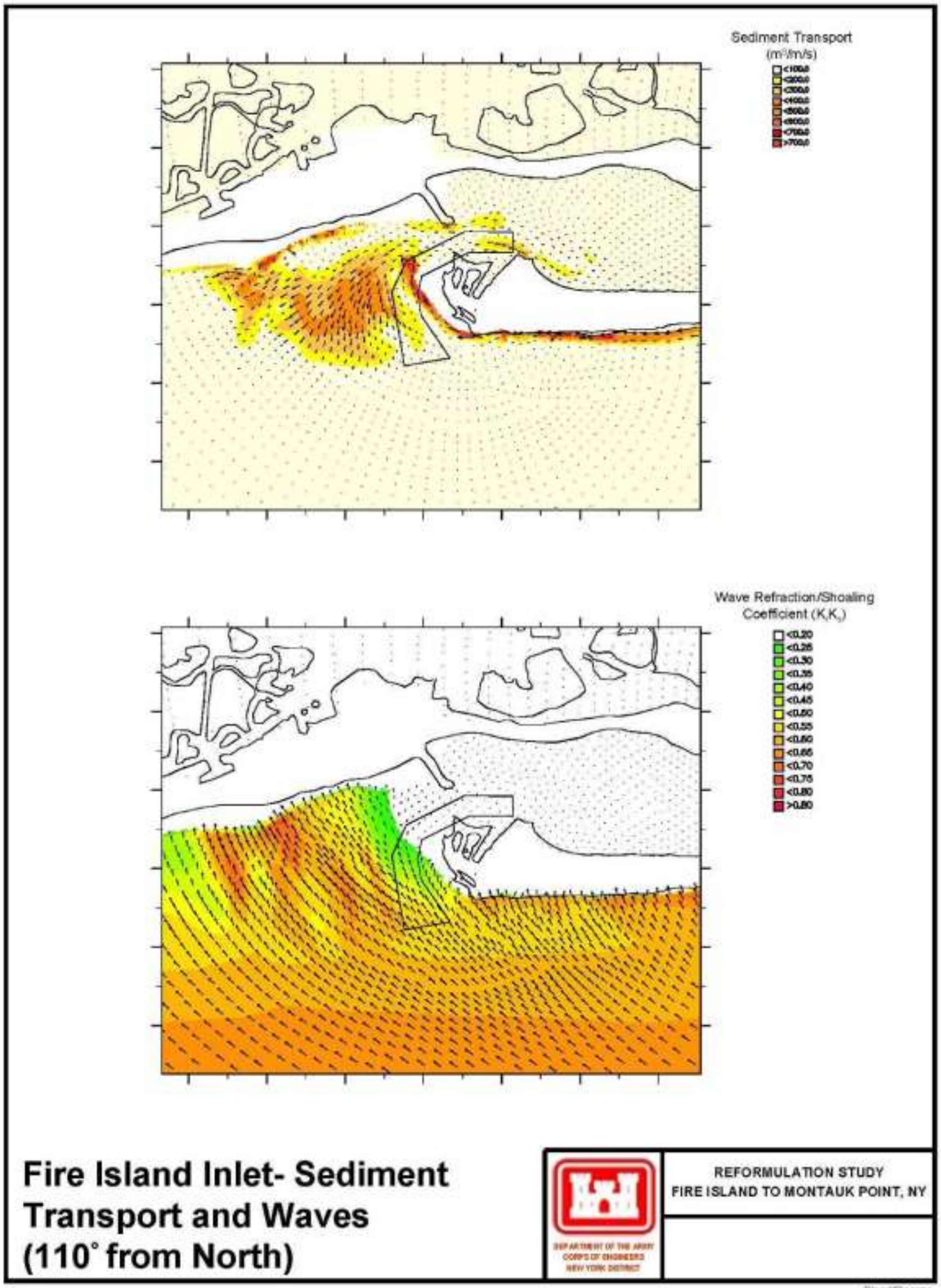


Figure 6-90. Fire Island Inlet Wave and Sediment Transport Patterns (110 deg)

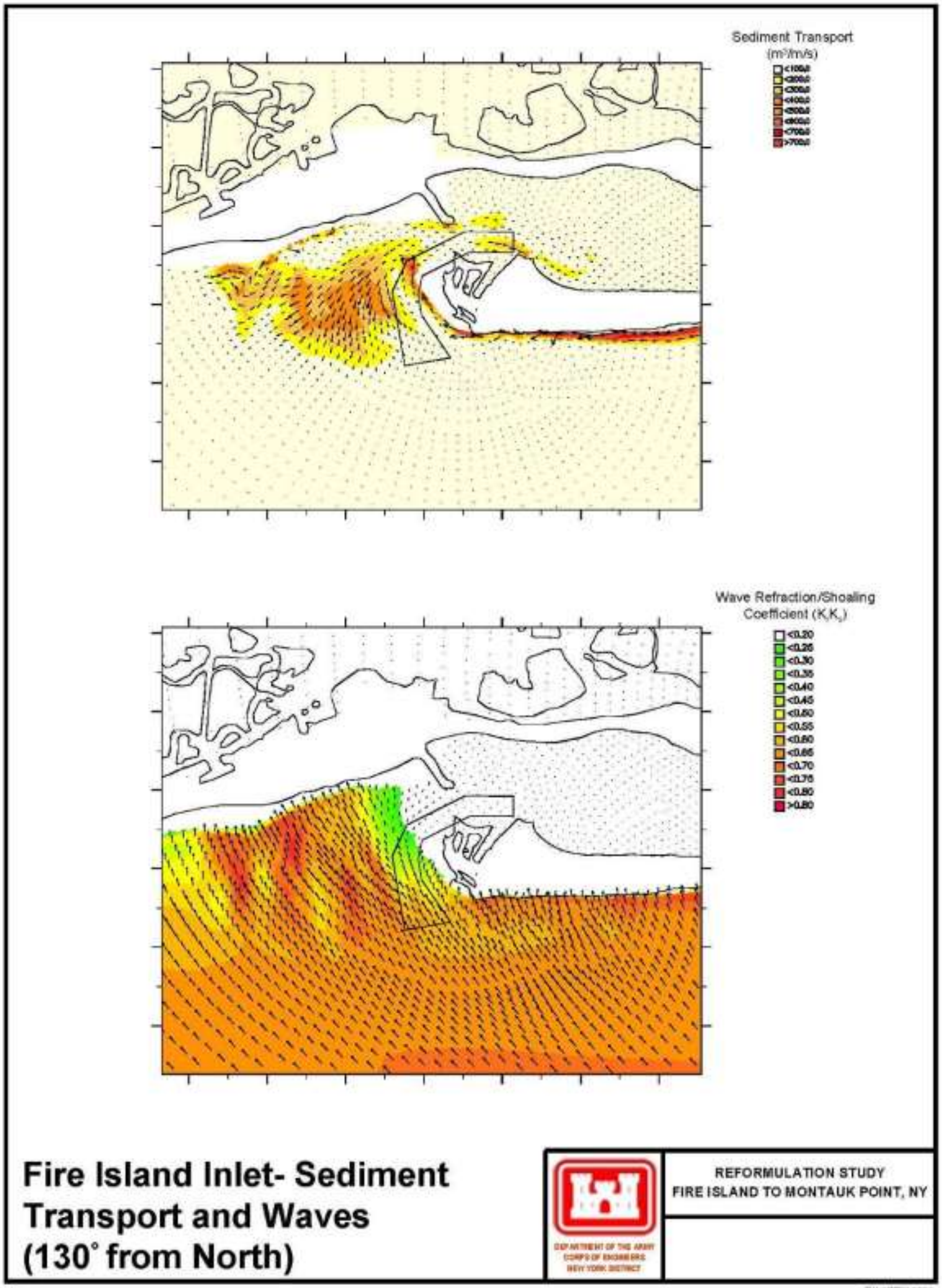


Figure 6-91. Fire Island Inlet Wave and Sediment Transport Patterns (130 deg)

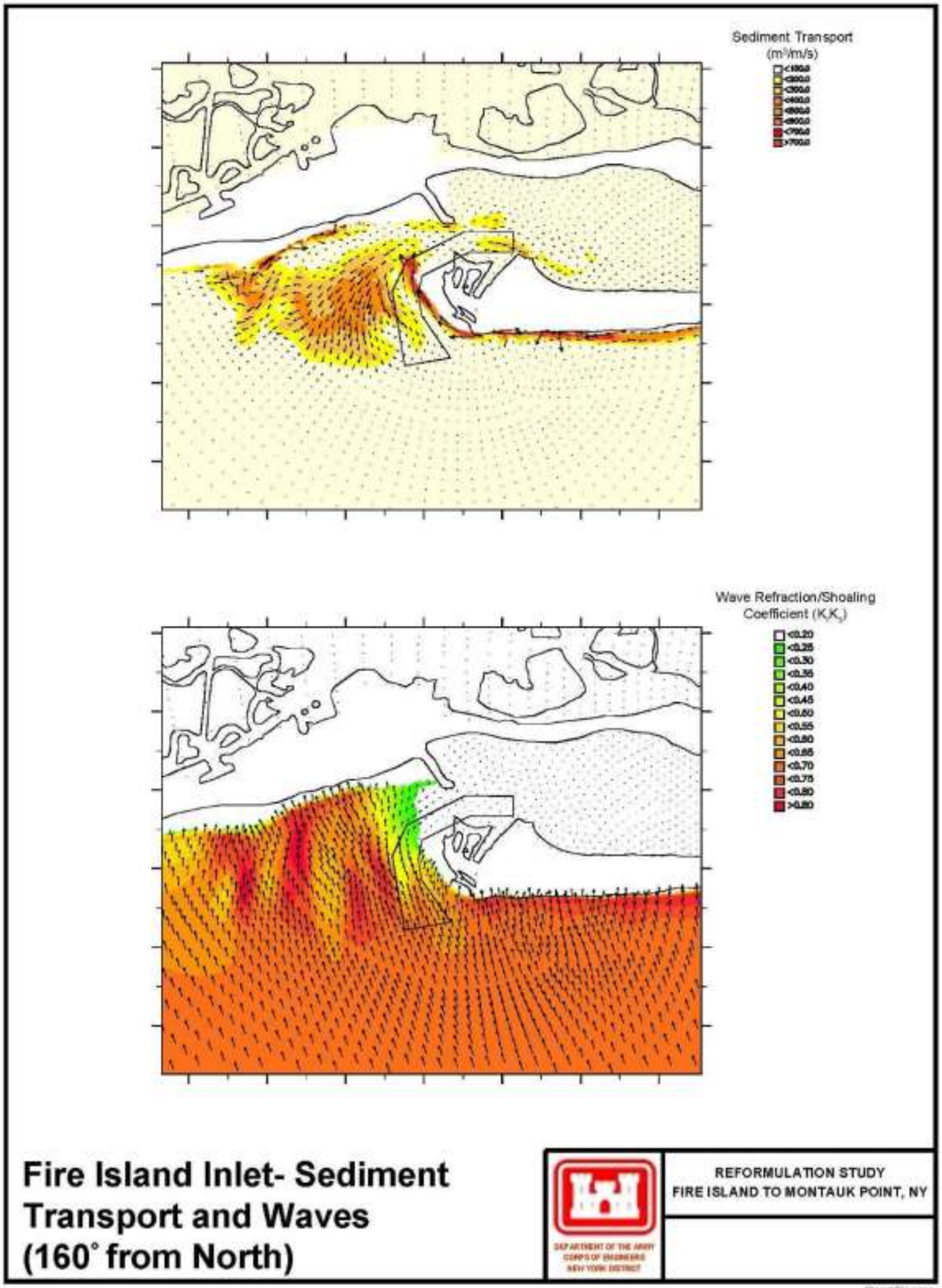


Figure 6-92. Fire Island Inlet Wave and Sediment Transport Patterns (160 deg)

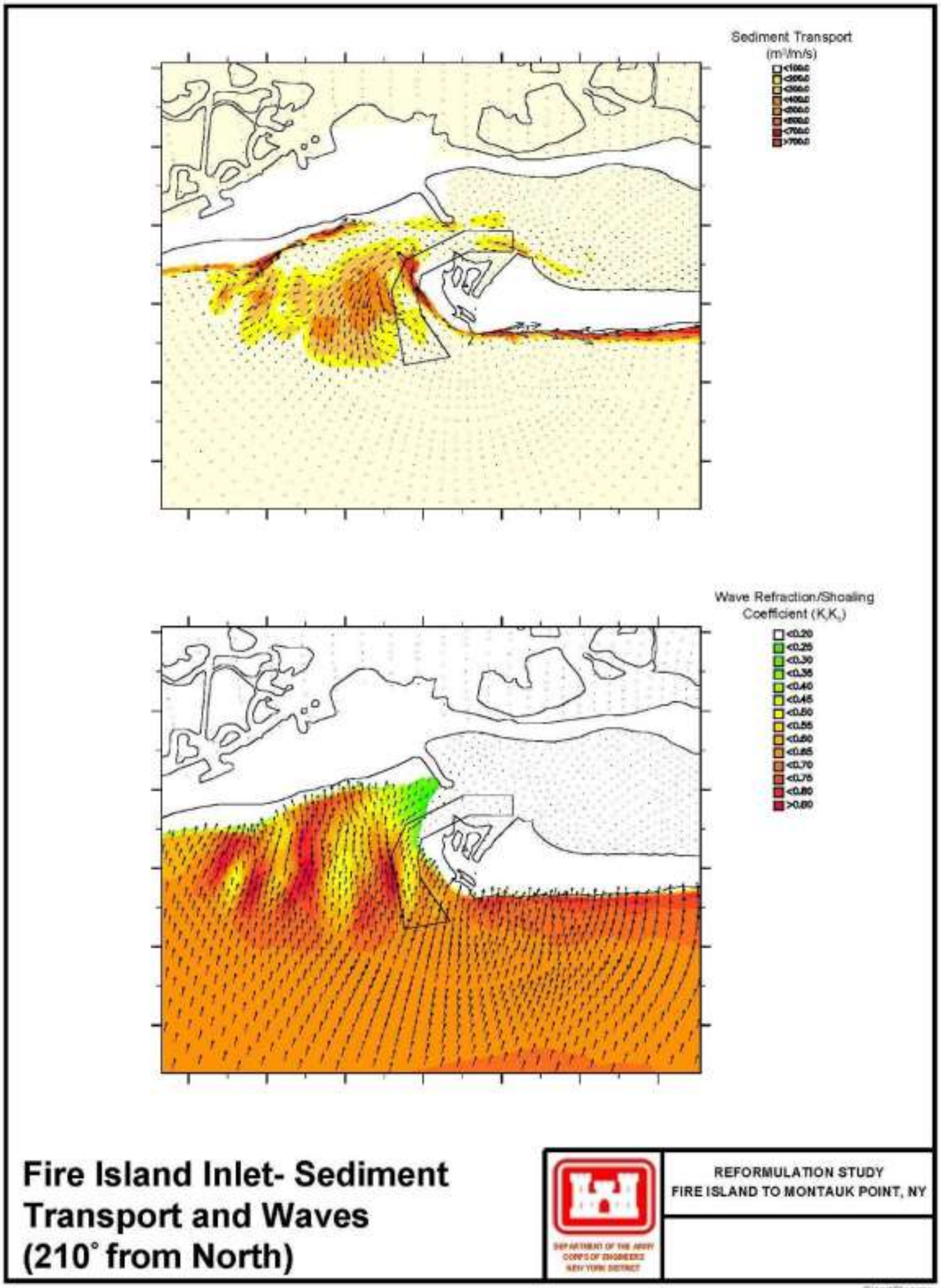


Figure 6-93. Fire Island Inlet Wave and Sediment Transport Patterns (210 deg)

6.4.3 Consideration of Alternative Project Modification Conditions

As previously stated, simulations were performed that only included tide, storm surge, waves and winds. The only geomorphological change included was reducing the existing inlet width from approximately 800 feet to 600 feet at Moriches and Shinnecock Inlets (considered as two possible inlet modification alternatives, see Sub Appendix A3, Inlet Modifications for a completed description of all inlet modification alternatives) . Normal tides and two of the most significant storms on record were simulated: the 1938 Hurricane and December 1992 Nor'easter.

A summary of the modeling results for normal and storm tide conditions under existing, narrower and narrower/deeper inlet cross-sections is shown in Table 6-9. As shown in the table, the potential changes range from 0% (normal tidal range for narrower/deeper Moriches Inlet) to 7.3% (1938 Hurricane water levels in Shinnecock Bay for a narrower inlet). It should be stated again that the results for the storm simulations do not include morphological evolution and the attendant inlet scour that typically occurs during high water levels. It seems logical to assume that increased velocities for narrower inlets would increase scour and offset the reductions in water level, which are relatively small to begin with.

Armoring the bottom of the inlet throat would be required to offset this effect. However, this would not be a simple solution, and it would likely require placing some kind of scour blanket over a fairly large area. In addition, a 200 ft narrowing increases peak velocity 1 to 2 ft/sec resulting in velocities at the inlets over 8 ft/sec, which may have significant impacts on navigation. Further inlet narrowing, unless accompanied by inlet deepening (which would offset any reductions in bay water level), would likely result in velocity increases that would make navigation through the inlet very dangerous.

Table 6-9. Alternative Project Modification Condition Results

	Tidal Range		1938 Hurricane		1992 Nor'easter	
	(ft)	% reduction from existing	Avg Max Water Level (ft)	% reduction from existing	Avg Max Water Level (ft)	% reduction from existing
Moriches Inlet						
Existing Section (800 ft)	2.29		4.79		4.71	
Narrower Section (600 ft)	2.18	4.6%	4.53	5.4%	4.39	6.8%
Narrower/deeper Section	2.29	0%	4.72	1.5%	4.59	2.5%
Shinnecock Inlet						
Existing Section (800 ft)	2.96		6.41		5.94	
Narrower Section (600 ft)	2.85	3.6%	5.94	7.3%	5.82	2.0%
Narrower/deeper Section	2.93	1%	6.19	3.4%	5.89	0.8%

6.5 Long-Term Breach Processes

This section summarizes the estimated long-term (up to 12 months) breach growth rates and associated back bay sediment transport volumes and areas. In particular, the analyses herein focus on the 10 areas most likely to breach (Table 6-10). The methodology, assumptions and results follow closely the work performed by USACE as part of the original Breach Contingency Plan (BCP, USACE-NAN, 1995) with additional analyses performed as part of the FIMP study.

Economic model lifecycle simulations include the possibility of future breach formation and growth. Changes in bay water levels and subsequent inundation damages caused by the breaches are captured in the economic model, as well as the cost of closing the breaches. The economic analysis is based on the predicted breach growth characteristics described below. Delft3D hydrodynamic model simulations were performed for a range of predicted breach sizes to evaluate the impact of breaches on bay water levels.

6.5.1 Long-Term Breach Growth Estimates

USACE-NAN examined historic breach data to determine long-term growth characteristics and sediment transport processes for the Breach Contingency Plan Report (1995). Breach growth characteristics for the BCP Report (1995) were based on three breaches that occurred during and after 1938 and remained open for several months or more: Shinnecock Inlet Breach (September 1938), Cupsogue Breach (January 1980), and Pikes Beach Breach (1992). Breach growth characteristics from USACE-NAN (1995) were revaluated in 2013 following Hurricane Sandy based on observations at the Old Inlet Breach within the Wilderness Area of the Fire Island National Seashore that opened during Hurricane Sandy.

Shinnecock Inlet opened on September 21, 1938 as was subsequently stabilized. The January 14, 1980 nor'easter opened a breach adjacent to Moriches Inlet at Cupsogue and was closed by a fill project completed on February 25, 1981. The nor'easter that struck Westhampton on December 10, 1992, created two breaches. One of the breaches was closed mechanically using dredged material from the Intracoastal Waterway (ICW) within a month of opening. The one that remained open expanded to an average width of 1,800 feet before being closed mechanically in October 1993.

Hurricane Sandy, October 2012, resulted in three barrier island breaches within the Study Area. One of the breaches within the Wilderness Area of the Fire Island National Seashore was not closed immediately following the storm. After the initial formation of the breach during Hurricane Sandy the breach grew rapidly for several months before breach growth slowed. DOI has been monitoring the Old Inlet Breach and is preparing an Environmental Impact Statement (Plan/EIS) to determine how best to manage the breach that was created in Fire Island's federally-designated wilderness area.

USACE-NAN (1995) presented a method for estimating breach along-shore cross-sectional area versus time according to the following exponential breach growth equation:

$$A(t) = A_0\left(1 - e^{-kt}\right)$$

Where t is the time in months from breach initiation, A_0 is the maximum breach cross sectional area, and k is the breach growth coefficient which varies from 0.15 to 0.40 month.

A_0 was stablished in the Breach Contingency Plan Report (1995). *A_0* represents the long term stable inlet cross sectional area and was estimated using the inlet stability analysis originally developed by Escoffier (1940), where the range of breach growth rate was estimated using the breach growth data from Cupsogue, adjacent to Moriches Inlet (1980) and at Pikes Beach (1992). These parameters vary depending on the bay where the breach occurs and were obtained as part of the breach inlet stability analysis (USACE-NAN, 1995). Breach growth would be attended by a reduction of tidal inlet area, although the trade-off between inlet and breaches areas may not be absolute. This behavior was observed during the breach at Moriches Inlet in 1980 when cross-sectional surveys of the breach and inlet indicated that the total area of both inlets was constant at approximately 23,000 square feet.

USACE-NAN (1995) applied the upper limit of the breach growth in Shinnecock Bay while the lowest range was used at Great South Bay. The breach growth equation and selected parameter values compare favorably with survey data for the 1980 and 1992 breaches at Cupsogue and Pikes Beach, respectively (USACE-NAN, 1995). The observed breach size at Old Inlet following Hurricane Sandy was smaller than originally predicted by USACE-NAN (1995). Therefore, the breach growth predictions for Great South Bay were modified to include the equal possibility of smaller breach with a maximum breach cross sectional area, A_0, of 6,500 ft^2. The smaller breach size (6,500 ft^2) combined with a k of 0.2 month-1 yields and area of 4,850 ft^2 at 7 months, which is consistent with observations at Old Inlet.

Estimated *A_0* and range of *k* values are summarized for Great South Bay, Moriches Bay, and Shinnecock Bay in Table 6-11. Estimated potential breach cross-sectional areas are shown in Table 6-12, assuming probable breach closure scenarios based on the experience at Westhampton Beach and recommendations of the Breach Contingency Plan (i.e., 1 to 12 months). The Minimum and Maximum rates shown in Table 6-11 reflect uncertainty in the breach growth rate (see range of values for parameter *k* in Table 6-10). Estimated breach widths based on an average breach depth of 7 ft below MSL (USACE-NAN, 1995) are shown in Table 6-13.

Table 6-10. More likely breach locations.

Design Subreach		Baseline Breaching Risk (RP in years)	FVC Breaching Risk (RP in years)
ID	Name		
GSB-1B	FI Lighthouse Tract	184[1]/500[2]	34/106
GSB-2B	Town Beach to Corneille Estates (at Robins Rest)	141/500	15/34
GSB-3D	Talisman to Water Island	213/500	12/31
GSB-3G	Davis Park	145/500	73/288
GSB-4B	Old Inlet (West)	45/82	7/22
GSB-4B	Old Inlet (East)	24/118	19/84
MB-1B	Smith Point CP - East	26/145	9/141
SB-1B	Sedge Island	251/500	48/291
SB-1C	Tiana Beach	72/336	74/326
SB-2B	WOSI	30/266	8/25
[1] Partial Breaching Risk [2] Full Breaching Risk			

Table 6-11. Breach growth parameters.

Project Reach	A_0 (sq. ft.)	k $(\text{month})^{-1}$
GSB - Small	6,500	0.15-0.3 (0.2 average)
GSB - Large	36,200	0.15-0.3 (0.2 average)
MB	16,000	0.15-0.4 (0.3 average)
SB	17,750	0.15-0.4 (0.3 average)

Table 6-12. Estimated long-term potential breach cross-sectional areas.

Project Reach	Range Value	Breach Areas (sq. feet)				
		1 Month	3 Months	6 Months	9 Months	12 Months
GSB - Small	Minimum	890	2,350	3,850	4,820	5,030
	Maximum	1,660	3,850	5,430	6,060	6,320
GSB-Large	Minimum	5,040	13,120	21,480	26,820	30,220
	Maximum	9,380	21,480	30,220	33,770	35,210
MB	Minimum	2,230	5,800	9,490	11,850	13,360
	Maximum	5,270	11,180	14,550	15,560	15,870
SB	Minimum	2,470	6,430	10,530	13,150	14,820
	Maximum	5,850	12,400	16,140	17,270	17,600

Table 6-13. Estimated long-term potential breach widths.

Project Reach	Range Value	Breach Areas (feet)				
		1 Month	3 Months	6 Months	9 Months	12 Months
GSB-Small	Minimum	130	340	550	690	780
	Maximum	240	550	780	870	900
GSB - Large	Minimum	720	1,870	3,070	3,830	4,320
	Maximum	1,340	3,070	4,320	4,820	5,030
MB	Minimum	320	830	1,360	1,690	1,910
	Maximum	750	1,600	2,080	2,220	2,270
SB	Minimum	350	920	1,500	1,880	2,120
	Maximum	840	1,770	2,300	2,470	2,510

6.5.2 Breach Sediment Transport Estimates

During a breaching event, the fate of sediments displaced from the barrier island depends largely on how the barrier island breached (i.e. oceanward or bayward). When a breach opens via ebb flows, the displaced sediments are moved offshore, as in the case of Shinnecock Inlet in 1938. When a breach opens due to overwash and storm flows from the ocean side, displaced sediments are moved into the adjoining back bay (e.g., Moriches Bay 1962, 1980 and 1992). Breaches that remain open will also influence sediment transport dynamics by redirecting/trapping longshore sediment transport during the period that the breach remains open.

The numerical model framework used for this study included the possibility of simulating the breaching of the barrier island from the bay to the ocean, since for each storm included the effect of wind, waves and increase water levels in the bays. However, none of the simulated storms, even for cases with a low barrier island conditions have generated a breach from the bay to the ocean. In all the cases when a breach occurred, it happened from the ocean to the bay.

6.5.2.1 Historic Breach Sediment Transport

Breach sediment transport volumes based on the volume of barrier island sediments removed during breach formation and growth are shown in Table 6-14 for several past breaches. These volumes are reasonable when compared to total bay deposition estimated from aerial photographs, hydrographic surveys and existing literature (Moffatt & Nichol, 2000). Pre-storm barrier island volumes above NGVD were estimated using topographic maps and records of breach widths. Barrier island volumes below NGVD were based on breach cross-sectional area and island width. The sum of these volumes was adopted as the volume of barrier island sediments displaced by the breach. Bay deposition volumes correspond to breach formation and the period of breach persistence, including storm-related bay sediment transport and long-term breach scouring.

6.5.2.2 Estimates of Potential Sediment Transport from Future Breaches

Breach sediment transport into the adjoining bays was separated into two phases: (1) sediment losses from the barrier island during the breaching storm and (2) long-term sediment losses from the barrier island and trapping of longshore sediment transport. For the purposes of the present evaluation, it was assumed that future breaches occur landward from the ocean to bay.

6.5.2.3 Initial Breach Formation

Total sediment volumes entering the bay during the formation of a breach was assumed to be the sum of the breach cross-sectional area multiplied by the barrier island width and the volume of material located above NGVD between the bay and ocean shorelines multiplied by the breach width. Initial breach cross-sectional area was assumed as the average of historic breach measurements with a width of 650 feet and depth of 4 feet below MSL.

Bay deposition volumes based on these assumptions are shown in Table 6-15, which represents bay deposition associated with potential breach sites listed in Table 6-10. The NGVD to MSL difference was accounted for and deducted from the barrier island volume to make sure we were not double counting in the volume calculation.

The bay deposition volumes shown in Table 6-15 appear to be consistent with estimates of “in-bay overwash” volumes estimated using morphological changes computed by Delft3D for Baseline Conditions (BLC). These estimates, which have been summarized in USACE-NAN memorandums (USACE-NAN, 2006a and 2006b), suggest that at Old Inlet, the in-bay sediment volumes (deposition during initial breach formation - sediment losses from the barrier island during the breaching storm) corresponding to return periods of 50 to 200 years would be on the order of 100,000 to 400,000 cubic yards, respectively. At SPCP, this range increases to 300,000-800,000 cubic yards. A significant part of the in-bay deposition at SPCP is due to overwash processes that do not necessarily lead to a breach. This may explain why the empirical estimate in Table 6-15, which would only account for transport due to the breach, appears to be lower than the values based on the model results. At Tiana Beach the range is roughly 200,000 to 500,000 cubic yards and 100,000 to 200,000 thousand cubic yards at WOSI.

Table 6-14. Historic breach sediment transport volumes.

Location	Date	Displaced Barrier Island Volume (cy)	Total Bay Deposition (cy)	Duration (months)	Bay Deposition Rate (cy/month)
Westhampton	1962	145,000	150,000	1	150,000
Moriches Inlet	1980	414,000	1,000,000	9	110,000
Westhampton	1992	467,000	600,000	10	60,000
Total		1,026,000	1,750,000	20	90,000

Table 6-15. Estimated bay deposition during initial breach formation.

Design Subreach		Breach Cross-Sectional Area (feet)	Barrier Width (feet)	Barrier Volume above NGVD (cy/ft)	Breach Width (feet)	Bay Deposition (cy)
ID	Name					
GSB-1B	FI Lighthouse Tract	2,600	1,500	220	650	270,000
GSB-2B	Town Beach to Corneille Estates (at Robins Rest)	2,600	1,200	180	650	220,000
GSB-3D	Talisman to Water Island	2,600	600	150	650	150,000
GSB-3G	Davis Park	2,600	1,200	250	650	260,000
MB-1B	Smith Point CP - East	2,600	800	230	650	220,000
SB-1B	Sedge Island	2,600	1,200	250	650	260,000
SB-1C	Tiana Beach	2,600	500	150	650	140,000
SB-2B	WOSI	2,600	600	100	650	120,000

6.5.2.4 Long-term Deposition Volumes

Long-term bay deposition following breach formation reflects the initial breaching event (and the estimated volumes shown in Table 6-15), and then expansion of the breaches following the empirical growth formula presented above.

Cross-sectional areas shown in Table 6-12 were multiplied by barrier island widths to calculate the volume of barrier island sediment (below NGVD) removed due to the breach. Unit barrier island volumes above NGVD were then multiplied by breach widths, which were calculated based on breach cross-sectional areas and depth. Total bay deposition values shown in Table 6-16 represent the combined sediment volumes above and below NGVD, including transport during breach formation.

It is important to note that the estimates presented in Table 6-16 are approximations of a highly complex process. Nonetheless, comparison with the historic bay deposition quantities presented in Table 6-16, which also reflect contributions from longshore sediment transport, suggests that barrier island scouring volumes are a reasonable indicator of bay deposition volumes. However, a portion of the barrier island sediments are undoubtedly moved offshore. This observation suggests that a portion of longshore sediment transport entering the breach is deposited bayward, but it approximately equals the volume of barrier island sediments moved offshore.

Table 6-16. Estimated bay deposition volumes during breach growth.

Design Subreach		Bay Deposition (x 1000 cy)				
ID	**Name**	**1 Month**	**3 Months**	**6 Months**	**9 Months**	**12 Months**
GSB-1B	FI Lighthouse Tract	320	800	1,240	1,480	1,610
GSB-2B	Town Beach to Corneille Estates (at Robins Rest)	260	650	1,000	1,190	1,300
GSB-3D	Talisman to Water Island	160	410	630	750	820
GSB-3G	Davis Park	300	740	1,150	1,370	1,490
MB-1B	Smith Point CP - East	250	570	810	900	940
SB-1B	Sedge Island	350	810	1,140	1,270	1,330
SB-1C	Tiana Beach	180	410	570	640	670
SB-2B	WOSI	160	370	520	580	600

6.5.2.5 Long-term Deposition Areas

Deposition areas (above and below MSL) associated with sediment transport volumes presented above were estimated based on recent estimates of "in-bay overwash" areas and volumes estimated using morphological changes computed by Delft3D for Baseline Conditions (USACE-NAN, 2006a and 2006b). Specifically, these estimates suggest that during initial breach formation (i.e., during the storm), the average thickness of the in-bay sediment layer deposited by overwash and breaching processes is on the order of 5 ft. On the other hand, previous literature suggests average "overwash depths" between 0.8 and 1.6 feet (Moffatt & Nichol, 2000). Unfortunately, long-term (i.e., 1 to 12 month) data on the thickness of the sediment layer created by a breach (including the area below MSL) is not readily available[5]. Therefore, for the purposes of this analysis, an average thickness of 3 ft (i.e., and roughly the mean of the model- and literature-based estimates) was assumed. However, it should be noted that there is a considerable amount of uncertainty regarding this estimate, which is in addition to the uncertainty related to the deposition volume estimates. Deposition area estimates based on this assumption are presented in Table 6-17.

[5] According to Moffatt & Nichol (2000), the 1992 breach at Westhampton, which remained open approximately 10 months, deposited roughly 600,000 cy in the bay and created roughly 30 acres of new "land". However, the area below MSL has not been reported and thus it is difficult to compute the average thickness of the deposition layer (both above and below MSL).

Table 6-17. Estimated total bay deposition areas during breach growth.

Design Subreach		Bay Deposition Area (acres)				
ID	**Name**	**1 Month**	**3 Months**	**6 Months**	**9 Months**	**12 Months**
GSB-1B	FI Lighthouse Tract	40	99	154	183	200
GSB-2B	Town Beach to Corneille Estates (at Robins Rest)	32	81	124	148	161
GSB-3D	Talisman to Water Island	20	51	78	93	102
GSB-3G	Davis Park	37	92	143	170	185
MB-1B	Smith Point CP - East	31	71	100	112	117
SB-1B	Sedge Island	43	100	141	157	165
SB-1C	Tiana Beach	22	51	71	79	83
SB-2B	WOSI	20	46	64	72	74

Finally, above and below MSL areas were estimated based on Delft3D morphological model results and the subsequent analysis performed by USACE-NAN (2006a and 2006b). Specifically, model results suggest that the area above MSL varies between 15 and 40% of the total deposition area. However, recent experience and observations from the Wilderness Area breach suggest that most of sediment transport does not result in the elevation of habitat above MSL. Therefore, the estimates presented in Table 6-18 and Table 6-19below reflect the assumption that the area above MSL is 15% of the total area for all breaches.

Table 6-18. Estimated bay deposition areas above msl during breach growth

Design Subreach		Bay Deposition Area (acres)				
ID	**Name**	**1 Month**	**3 Months**	**6 Months**	**9 Months**	**12 Months**
GSB-1B	FI Lighthouse Tract	6	15	23	28	30
GSB-2B	Town Beach to Corneille Estates (at Robins Rest)	5	12	19	22	24
GSB-3D	Talisman to Water Island	3	8	12	14	15
GSB-3G	Davis Park	6	14	21	25	28
MB-1B	Smith Point CP - East	5	11	15	17	17
SB-1B	Sedge Island	7	15	21	24	25
SB-1C	Tiana Beach	3	8	11	12	12
SB-2B	WOSI	3	7	10	11	11

Table 6-19. Estimated bay deposition areas below msl during breach growth

Design Subreach		Bay Deposition Area (acres)				
ID	**Name**	**1 Month**	**3 Months**	**6 Months**	**9 Months**	**12 Months**
GSB-1B	FI Lighthouse Tract	34	84	131	156	170
GSB-2B	Town Beach to Corneille Estates (at Robins Rest)	27	68	105	125	137
GSB-3D	Talisman to Water Island	17	43	66	79	86
GSB-3G	Davis Park	32	78	121	144	157
MB-1B	Smith Point CP - East	26	60	85	95	99
SB-1B	Sedge Island	37	85	120	134	140
SB-1C	Tiana Beach	19	43	60	67	71
SB-2B	WOSI	17	39	55	61	63

6.5.2.6 Expected Number of Breaches

The lifecycle simulation model and breaching risks summarized in Table 6-10 above were used to estimate the expected number of breaches that are likely to occur in the Future Without Project Condition (FWOPC), and in the Future With-project Condition (FWPC), over 50 years assuming the historic rate of relative sea level change (see Table 6-20). For reference, the range of uncertainty in these estimates (25th and 75th percentile) is also shown for the FWOPC and FWPC scenarios

Table 6-20. Estimated Number of Breaches over 50-year Project Life

Breach Area	Potential Breach Locations	WOPFC (mean)	25th percentile	75th percentile	WPFC (mean)	25th percentile	75th percentile
1	Fire Island Lighthouse	**1.7**	1.0	2.0	**1.1**	0.0	2.0
2	Kismet to Corneille						
3	Talisman to Blue Pt.	**2.1**	0.0	3.0	**1.7**	0.0	3.0
4	Davis Park						
5	Old Inlet West	**N/A**	N/A	N/A	**N/A**	N/A	N/A
6	Old Inlet East						
7	Smith Point County	**1.6**	1.0	2.0	**0.5**	0.0	1.0
8	Sedge Island	**1.1**	0.0	2.0	**0.4**	0.0	1.0
9	Tiana Beach						
10	West of Shinnecock	**1.7**	0.0	3.0	**1.3**	0.0	2.0
	Total	**8.2**	2.0	12.0	**5.0**	0.0	9.0

6.5.2.7 Lifecycle Estimates of Breach Sediment Transport

The above information, including breach sediment transport estimates and lifecycle breaching events, were used to estimate the amount of sediment that would be transported into the bay in the without project condition, where breaches would be open for a period of one year. The with-project analysis considered the change in breach frequency at a given location, based upon the recommended plan (see Section 9.0), and the estimated amount of time any breach would remain open, based upon the proposed breach response in each location. In each scenario, the estimated mean number of breaches over the 50-year project life (Table 6-20) and the estimated amount of sediment transport per breach (Table 6-16 and Table 6-18) were multiplied together to determine the total amount of cross-island sediment transport in terms of volume and area above MSL. Table 6-21 shows a summary of the results.

Table 6-21. Lifecycle Estimates of Cross-island Sediment Transport due to Breaching

<table>
<tr><th rowspan="2">Breach Area</th><th rowspan="2">Potential Breach Locations</th><th colspan="2">WOPFC</th><th colspan="2">WPFC</th><th colspan="2">Difference</th></tr>
<tr><th>Volume (CY)</th><th>Area above MSL (acres)</th><th>Volume (CY)</th><th>Area above MSL (acres)</th><th>Volume (CY)</th><th>Area Above MSL (acres)</th></tr>
<tr><td>1</td><td rowspan="2">FI Lighthouse Tract & Kismet to Corneille States (1)</td><td rowspan="2">2,470,000</td><td rowspan="2">46</td><td rowspan="2">800,000</td><td rowspan="2">15</td><td rowspan="2">1,670,000</td><td rowspan="2">31</td></tr>
<tr><td>2</td></tr>
<tr><td>3</td><td rowspan="2">Talisman to Blue Pt. Beach & Davis Park (2)</td><td rowspan="2">2,430,000</td><td rowspan="2">45</td><td rowspan="2">980,000</td><td rowspan="2">18</td><td rowspan="2">1,450,000</td><td rowspan="2">27</td></tr>
<tr><td>4</td></tr>
<tr><td>7</td><td>SPCP (1)</td><td>1,504,000</td><td>28</td><td>285,000</td><td>5</td><td>1,219,000</td><td>23</td></tr>
<tr><td>8</td><td rowspan="2">Sedge Island & Tiana Beach (1)</td><td rowspan="2">1,100,000</td><td rowspan="2">20</td><td rowspan="2">244,000</td><td rowspan="2">5</td><td rowspan="2">856,000</td><td rowspan="2">16</td></tr>
<tr><td>9</td></tr>
<tr><td>10</td><td>WOSI (1)</td><td>1,020,000</td><td>19</td><td>481,000</td><td>9</td><td>539,000</td><td>10</td></tr>
<tr><td colspan="2">Total</td><td>8,524,000</td><td>158</td><td>2,790,000</td><td>52</td><td>5,734,000</td><td>107</td></tr>
<tr><td colspan="6">Talisman to Blue Point Beach & Davis Park Sand Placement</td><td>-1,450,000</td><td>-27</td></tr>
<tr><td colspan="6">Overall Target (50-Year Project Life)</td><td>4,284,000</td><td>80</td></tr>
<tr><td colspan="6">Decadal Target (Over Five Decades)</td><td>856,800</td><td>16</td></tr>
<tr><td colspan="8">(1) Assumes 12-month closure in WOPFC and rapid (3-month) breach closure in WPFC</td></tr>
<tr><td colspan="8">(2) Assumes 12-month closure in WOPFC and conditional breach response with 6-month</td></tr>
</table>

Table 6-21 shows that the expected change between the with and without project condition in the number of breaches, and expected change in the duration of a breach remaining open, results in a difference of approximately 5.7 MCY of sand into the bay, and a difference of approximately 107 acres of habitat above MSL. Since no action is being taken at Talisman to reduce the likelihood of breaching, it is assumed that the difference in the amount of sediment transport into the bay (approximately 1.5 MCY and 27 acres of habitat above MSL) at this location could be offset, through a combination of 1) sand transported into the bay, while the breach is open, and 2) placement of sand in the bay as a plan feature in the closure process. This assumption has not been applied in other breach locations, because the plan in all other locations includes project features that reduce the potential for breaching.

With the assumption that locations of conditional breach response would be sediment neutral, the expected difference for sediment transport into the bay due to breaching is 4.3 MCY of sand, and 80 acres of habitat above MSL over the 50-year project life (equivalent to 1.6 acres/year). Similar to the proposals for reestablishing alongshore sediment transport, it is not expected that this entire quantity of sand or acreage of habitat would be constructed during initial construction. Instead, it is expected that there would be a component of initial construction that would meet a portion of this amount, and the project would include recurring costs over the project life (similar to reestablishment of alongshore transport) for meeting the lifecycle objectives of cross-island transport. These lifecycle efforts would be based upon monitoring and adaptive management of the coastal process features, and could include renourishment of the project features or additional, similar coastal process features in new locations, identified through the monitoring and adaptive management.

Moreover, early successional habitat established by breaching and/or overwash is temporary and time dependent. New bare sand areas will naturally vegetate at a rate dependent upon several conditions, including the potential for future breaching or overwash which would reset the state of succession. Recent monitoring of the post-Sandy overwash and restoration areas at Smith Point County Park suggest that these areas have significantly revegetated. Specifically, by 2016 vegetation growth had exceeded the 30% vegetation cover trigger specified in U.S. Fish and Wildlife Service's (USFWS) Biological Opinion. In fact, as of the 2016 survey, vegetation covered 50-75% of the management/restoration areas (other than Great Gun Restoration Area). For the purposes of this analysis an annual vegetation rate of 10% has been assumed (i.e., complete revegetation after 10 years). Assuming 10% annual vegetation, and based on 1.6 acres/year difference in breaching related sediment transport into the bays between with- and without-project conditions, the average bare sand acreage difference over the 50-year project life would be approximately 8 acres.

6.5.2.8 Uncertainty

As stated upfront, it is readily acknowledged that there is a tremendous amount of uncertainty in the projections presented in this analysis. Table 1 shows the range in the potential number of breaches that could occur over the project life, based upon the uncertainty analysis contained in the lifecycle modeling, and the unknowns regarding future storms. The 25-75% range estimates presented in Table 1 suggest that there is a 50% probability that the overall impact of the project on cross-island sediment transport related to breaching would be between 2.4 MCY / 45 acres and 5.2 MCY / 97 acres. Conversely, there is 50% probability that the impact will be smaller or greater than that range. In addition to the uncertainty regarding the expected number of breaches, there is also uncertainty in the breach characteristics (size of the breach, sediment transport associated with the breach, resulting natural bayside breach features, and bayside features that are indirectly created as a result of any closure operation), based both on the underlying uncertainty in breach processes, and in the approach used in developing these estimates.

Overriding all this analysis is also the projection of future sea level rise. This analysis is based upon the historic rate of RSLC. A projection of a greater increase in RSLC would result in a greater number of breaches in both the without-project condition, and the with-project condition. This analysis also focuses on the changes in breach potential as a result of the proposed action in the recommended plan, and does not consider the effect that prior actions within the project area may have had on cross-island sediment transport (acknowledging that there are past activities that have both increased cross-island transport and decreased cross-island transport at particular locations).

6.6 Overwash Processes

Overwash, the landward transport of beach/dune sediments, is also a potential contributor to cross-island sediment transport. Consequences of this process are highly dependent on site-specific conditions, including the volume and disposition of overwashed sediments, barrier island width, adjacent bay water depths and character of the backbarrier environment. Historically, overwashing has involved significant volumes of beach sediments.

The actual consequence of these occurrences is strongly dependent on the width of the overwashed barrier island, adjacent bay water depths and character of adjacent backbarrier habitat. At narrow barrier island locations backed by shallow bay waters, overwash may deposit in the bay providing substrate for future marsh development. On the other hand, wide barrier island segments are more resistant to overwashing causing materials to be deposited either on the barrier itself or on leeward marshes (where present). This situation can result in the establishment of a secondary dune system or marsh burial. Some overwashed sediments are deposited on adjacent roadways and other developed areas and then mechanically moved seaward as part of dune rebuilding.

6.6.1 Overwash Deposition

Overwash, the landward transport of beach/dune sediments, is also a potential contributor to cross-island sediment transport. Consequences of this process are highly dependent on site-specific conditions, including the volume and disposition of overwashed sediments, barrier island width, adjacent bay water depths and character of the backbarrier environment. Historically, overwashing has involved significant volumes of beach sediments.

The actual consequence of these occurrences is strongly dependent on the width of the overwashed barrier island, adjacent bay water depths and character of adjacent backbarrier habitat. At narrow barrier island locations backed by shallow bay waters, overwash may deposit in the bay providing substrate for future marsh development. On the other hand, wide barrier island segments are more resistant to overwashing causing materials to be deposited either on the barrier itself or on leeward marshes (where present). This situation can result in the establishment of a secondary dune system or marsh burial. Some overwashed sediments are deposited on adjacent roadways and other developed areas and then mechanically moved seaward as part of dune rebuilding.

As part the of the FIMP Reformulation Study, USACE developed a methodology to estimate significant overwash deposits (USACE, 2006a). Specifically, the goal of this analysis was to determine approximate dimensions and locations of new habitat area created by sand overwash deposits resulting from specific actual or possible storm events impacting the barrier islands between Fire Island Inlet and Montauk Point.

Estimates were made based on output from the Delft3D MOR morphological change model using the 1996 ocean and bay shorelines as a reference. Simulation results for storms listed in Table 6-22 below were examined, for the Baseline Conditions (BLC), representing 2000 topography, and Future Vulnerable Conditions (FVC) that can be expected to occur based on existing erosional trends.

Table 6-22. Storms Used for Significant Overwash Computation

Baseline Conditions	Future Vulnerable Conditions
Historical September 1938	Historical September 1938
September 1938 Alternate Tide	Historical September 1944
November 1950 Alternate Tide	Historical November 1950
September 1985 Alternate Tide	Historical March 1962
	Historical December 1992

Delft3D output graphics were used to determine the location of overwashes, partial breaches and full breaches. Areas of overwash were measured separately for on-land overwash and in-bay overwash, using the 1996 bay shoreline as the delimiter. Results for the for the most vulnerable FIMP locations and for relatively small (10-year return Period) and large (100-year Return Period) events are summarized in Table 6-23. Note that the analysis at the time included to two additional vulnerable locations at Old Inlet in the Fire Island Wilderness Area. However, those results are not shown since that is generally the area where the existing Sandy breach is currently located. The results summarized in Table 6-23 confirm the historical knowledge that in-bay overwash deposition areas are smaller than the on-land changes, particularly for small storm events.

Table 6-23. Overwash Deposition Estimates (in acres) for Small (10 yr) and Large (100 yr) Events

Design Subreach		Baseline Conditions (BLC)				Future Vulnerable Conditions (FVC)			
		on-land		in-bay (above MSL)		on-land		in-bay (above MSL)	
ID	Name	Small (10 yr)	Large (100 yr)	Small (10 yr)	Large (100 yr)	Small (10 yr)	Large (100 yr)	Small (10 yr)	Large (100 yr)
GSB-1B	FI Lighthouse Tract	10	16	0	0	25	50	0	5
GSB-2B	Town Beach to Corneille Estates (at Robins Rest)	0	20	0	3	110	25	5	20
GSB-3D	Talisman to Water Island	0	0	0	0	10	10	0	10
GSB-3G	Davis Park	0	2	0	0	0	0	0	0
MB-1B	Smith Point CP - East	5	50	5	42	25	5	10	80
SB-1B	Sedge Island	0	20	0	0	15	75	0	10
SB-1C	Tiana Beach	10	50	0	18	20	70	3	20
SB-2B	WOSI	0	22	0	3	5	10	2	4
TOTAL		**25**	**180**	**5**	**66**	**210**	**245**	**20**	**149**

6.6.2 Estimated Reduction in Annual Overwash Areas

The overwash area vs. frequency relationships summarized in Table 6-23 above were used to develop estimates of FWOPC overwash deposition areas by considering cumulative annual exceedance probabilities and the average of the BLC and FVC results as being representative of the average condition over the project lifetime. Unfortunately, model results are only available at the breach vulnerable reaches listed in Table 6-23. Therefore, estimates for other reaches had to be approximated based on the results from the closest vulnerable reach and scaled based on reach length. In general, all the other reaches are less vulnerable to overwash, so the BLC condition, as opposed to the more degraded BLC-FVC average, was assumed to estimate the annual overwash areas at these other locations.

FWOPC annual total and in-bay overwash results are summarized in Table 6-24. Only reaches where significant differences are expected in FWOPC and FWPC conditions and overwash response were considered. For example, any reaches where the proposed plan is only Reactive or Conditional/ Contingent Breach Response were not included in the analysis. This is because in these reaches the proposed breach response plan is only expected to impact cross-shore sediment volumes in the event of a breach and these impacts have already been captured in the breaching volumes/areas estimates presented in 6.5 above. Overwash processes should otherwise remain largely un-affected in the FWPC.

In addition, it was assumed that on-land overwash deposits (computed as part of the “total” overwash numbers) in developed areas (e.g., Kismet to Lonelyville) would not likely persist long-term as bare-sand habitat and therefore these on-land overwash areas were not considered in the analysis. On the other, in-bay overwash area differences were evaluated at every project reach.

Table 6-24. FWOPC Annual Overwash Estimates

Design Subreach	Reach Name	Reach Length (feet)	Total Annual Overwash (acres/year)	In-Bay Annual Overwash (acres/year)
Great South Bay (GSB)				
1A	Robert Moses State Park - East	19,000	1.32	0.00
1B	FI Lighthouse Tract	6,700	3.20	0.04
2A	Kismet to Lonelyville	8,900	0.17	0.17
2B	Town Beach to Corneille Estates	5,100	0.57	0.57
2C	Ocean Beach & Seaview	3,800	0.07	0.07
2D	OBP to Point O' Woods	7,400	0.14	0.14
3A	Cherry Grove	3,000	0.07	0.07
3C	Fire Island Pines	6,600	0.15	0.15
3G	Davis Park	4,100	0.09	0.09
	Subtotal GSB		**5.8**	**1.3**
Moriches Bay (MB)				
1A	Smith Point CP- West	6,300	0.82	0.82
1B	Smith Point CP - East	13,500	4.85	2.16
2A	Great Gun	7,600	2.55	1.20
2B	Moriches Inlet - West	6,200	1.74	0.81

2C	Cupsogue Co Park	7,500	2.10	0.98
2D	Pikes	9,700	1.26	1.26
2E	Westhampton	18,300	1.91	1.91
	Subtotal MB		**15.2**	**9.1**
Shinnecock Bay (SB)				
1A	Hampton Beach	16,800	0.82	0.82
1B	Sedge Island	10,200	4.85	2.16
1C	Tiana Beach	3,400	2.55	1.20
1D	Shinnecock Inlet Park West	6,300	1.74	0.81
2A	Ponquogue	5,300	2.10	0.98
2B	WOSI	3,900	1.26	1.26
2C	Shinnecock Inlet - East	9,800	1.91	1.91
3A	Southampton Beach	9,200	0.82	0.82
	Subtotal SB		**3.5**	**0.8**
TOTAL			**24.5**	**11.3**

FWPC estimates of overwash were developed by considering the changes in the probability of an overwash event due to the effect of the proposed plan. Specifically, along the developed reaches where beach and dune fill has proposed for the first 30 years, SBEACH simulations suggest that the selected 90 ft. width berm and +15 ft. dune will provide a 25-yr level of protection against overwash initiation[6]. Therefore, annual overwash estimates for these reaches were reduced accordingly by removing any overwash below that threshold. For reaches where Proactive Breach Response is the proposed solution, overwash below the 10-yr level was removed to reflect the impacts of the proposed +13 ft. dune in the Proactive Breach Response fill template. For years 31 through 50, there would also be Proactive Breach Response in the developed reaches, which would slightly increase estimated annual overwash areas relative to years 0 to 30. A summary of the results based on these assumptions is presented in Table 6-25 below.

Table 6-25. FWPC Annual Overwash Estimates

Design Subreach	Reach Name	Proposed Plan	Total Annual Overwash (acres/year)	In-Bay Annual Overwash (acres/year)
Great South Bay (GSB)			**Y0-30 / Y31-50**	**Y0-30 / Y31-50**
1A	Robert Moses State Park - East	Beach, no Dune, Renourishment	0.8 / 1.3	0.0 / 0.0
1B	FI Lighthouse Tract	Proactive Breach Response	1.9 /1.9	0.0 / 0.0
2A	Kismet to Lonelville	Beach, Dune and Renourishment	0.2 / 0.2	0.2 / 0.2
2B	Town Beach to Corneille Estates	Beach, Dune and Renourishment	0.2 /0.4	0.2 / 0.4
2C	Ocean Beach & Seaview	Beach, Dune, Renourish, Groin Modification	0.1 /0.1	0.1 /0.1

[6] Excess runup, the difference between the wave runup elevation and the profile crest height, was used as the indicator of potential overwash.

2D	OBP to Point O' Woods	Beach, Dune and Renourishment	0.1 /0.1	0.1 /0.1
3A	Cherry Grove	Beach, Dune and Renourishment	0.1 / 0.1	0.1 / 0.1
3C	Fire Island Pines	Beach, Dune and Renourishment	0.1 / 0.2	0.1 / 0.2
3G	Davis Park	Beach, Dune and Renourishment	0.1 /0.1	0.1 /0.1
	Subtotal GSB		**3.5 / 4.3**	**0.8/1.1**
Moriches Bay (MB)				
1A	Smith Point CP- West	Beach, Dune and Renourishment	0.6 / 0.8	0.6 / 0.8
1B	Smith Point CP - East	Proactive Breach Response, sand bypassing	3.2 / 3.2	1.6 / 1.6
2A	Great Gun	Proactive Breach Response, sand bypassing	2.1 / 2.1	1.0 / 1.0
2B	Moriches Inlet - West	Proactive Breach Response	1.4 / 1.7	0.6 / 0.8
2C	Cupsogue Co Park	Beach, Dune and Renourishment	1.6 / 2.1	0.7 / 1.0
2D	Pikes	Beach, Dune and Renourishment	0.9 / 1.3	0.9 / 1.3
2E	Westhampton	Beach, Dune and Renourishment	1.8 / 1.9	1.8 / 1.9
	Subtotal MB		**11.6 / 13.1**	**7.3 / 8.4**
Shinnecock Bay (SB)				
1A	Hampton Beach	Proactive Breach Response	0.0 / 0.0	0.0 / 0.0
1B	Sedge Island	Proactive Breach Response, sand bypassing	1.4 / 1.4	0.1 / 0.1
1C	Tiana Beach	Proactive Breach Response, sand bypassing	0.4 / 0.4	0.4 / 0.4
1D	Shinnecock Inlet Park West	Proactive Breach Response, sand bypassing	0.4 / 0.4	0.0 / 0.0
2A	Ponquogue	Proactive Breach Response	0.3 / 0.3	0.0 / 0.0
2B	WOSI	Proactive Breach Response, sand bypassing	0.1 / 0.1	0.1 / 0.1
2C	Shinnecock Inlet - East	Proactive Breach Response	0.0	0.0 / 0.0
3A	Southampton Beach	Proactive Breach Response	0.0	0.0 / 0.0
	Subtotal SB		**2.8 / 2.8**	**0.7 / 0.7**
TOTAL			**17.8 / 20.2**	**8.8 / 10.1**

Finally, expected differences between FWOPC and FWPC are presented in Table 6-26. This table shows that the proposed plan is expected to result in approximately 6.7 acres/yr less of total overwash and 2.5 acres/yr less of in-bay overwash above MSL (i.e., new land) in years 0 to 30. In years 31 to 50 there would be a reduction of 4.4 acres/yr of total overwash and 1.1 acres/yr of in-bay overwash above MSL (i.e., new land) in these reaches.

Table 6-26. Estimated Reduction in Annual Overwash Areas

Design Subreach	Reach Name	Years 0 to 30		Years 31 to 50	
		Total Annual Overwash	In-Bay Annual Overwash	Total Annual Overwash	In-Bay Annual Overwash

		Reduction (acres/year)	Reduction (acres/year)	Reduction (acres/year)	Reduction (acres/year)
Great South Bay (GSB)					
1A	Robert Moses State Park - East	0.54	0.00	0.00	0.00
1B	FI Lighthouse Tract	1.31	0.00	1.31	0.00
2A	Kismet to Lonelyville	0.02	0.02	0.00	0.00
2B	Town Beach to Corneille Estates	0.38	0.38	0.19	0.19
2C	Ocean Beach & Seaview	0.01	0.01	0.00	0.00
2D	OBP to Point O' Woods	0.02	0.02	0.00	0.00
3A	Cherry Grove	0.01	0.01	0.00	0.00
3C	Fire Island Pines	0.02	0.02	0.00	0.00
3G	Davis Park	0.02	0.02	0.00	0.00
	Subtotal GSB	**2.3**	**0.5**	**1.5**	**0.2**
Moriches Bay (MB)					
1A	Smith Point CP- West	0.21	0.21	0.00	0.00
1B	Smith Point CP - East	1.69	0.56	1.69	0.56
2A	Great Gun	0.42	0.21	0.42	0.21
2B	Moriches Inlet - West	0.33	0.16	0.00	0.00
2C	Cupsogue Co Park	0.52	0.25	0.00	0.00
2D	Pikes	0.33	0.33	0.00	0.00
2E	Westhampton	0.14	0.14	0.00	0.00
	Subtotal MB	**3.6**	**1.9**	**2.1**	**0.8**
Shinnecock Bay (SB)					
1A	Hampton Beach	0.21	0.21	0.00	0.00
1B	Sedge Island	1.69	0.56	1.69	0.56
1C	Tiana Beach	0.42	0.21	0.42	0.21
1D	Shinnecock Inlet Park West	0.33	0.16	0.00	0.00
2A	Ponquogue	0.52	0.25	0.00	0.00
2B	WOSI	0.33	0.33	0.00	0.00
2C	Shinnecock Inlet - East	0.14	0.14	0.00	0.00
3A	Southampton Beach	0.21	0.21	0.00	0.00
	Subtotal SB	**0.8**	**0.2**	**0.8**	**0.2**
TOTAL ANNUAL REDUCTION		**6.7**	**2.5**	**4.4**	**1.1**

6.6.3 Average Reduction in Overwash Bare-sand Habitat over Project Life

As with the breaching transport and habitat analysis presented above, an annual vegetation rate of 10% has been assumed (i.e., complete revegetation after 10 years) for early successional habitat established as a result of overwash. This assumption combined with the expected annual differences in overwash presented in the tables above results in an expected average reduction of the total and in-bay overwash bare sand habitat over the 50-year project life of 30 and 11 acres, respectively. These results are summarized in Table 6-27.

Table 6-27. Average Reduction in Overwash Bare-sand Habitat over 50-year Project Life

Design Subreach	Reach Name	Total Annual Bare-sand Overwash Reduction (acres)	In-Bay Annual Bare-sand Overwash Reduction (acres)
Great South Bay (GSB)			
1A	Robert Moses State Park - East	2.44	0.00
1B	FI Lighthouse Tract	5.92	0.00
2A	Kismet to Lonelyville	0.10	0.09
2B	Town Beach to Corneille Estates	1.69	1.56
2C	Ocean Beach & Seaview	0.04	0.04
2D	OBP to Point O' Woods	0.08	0.08
3A	Cherry Grove	0.04	0.04
3C	Fire Island Pines	0.09	0.08
3G	Davis Park	0.09	0.09
	Subtotal GSB	**10.5**	**2.0**
Moriches Bay (MB)			
1A	Smith Point CP- West	0.96	0.88
1B	Smith Point CP - East	7.62	2.33
2A	Great Gun	1.91	0.88
2B	Moriches Inlet - West	1.51	0.67
2C	Cupsogue Co Park	2.36	1.05
2D	Pikes	1.47	1.35
2E	Westhampton	0.62	0.57
	Subtotal MB	**16.4**	**7.7**
Shinnecock Bay (SB)			
1A	Hampton Beach	0.00	0.00
1B	Sedge Island	2.54	0.00
1C	Tiana Beach	0.51	0.47
1D	Shinnecock Inlet Park West	0.00	0.00
2A	Ponquogue	0.00	0.00
2B	WOSI	0.34	0.31
2C	Shinnecock Inlet - East	0.00	0.00
3A	Southampton Beach	0.00	0.00
	Subtotal SB	**3.4**	**0.8**
AVERAGE REDUCTION OVER PROJECT LIFE		**30**	**11**

6.6.4 Comparison to Historic Overwash

In general, historical data suggests that the principal impact of overwash is the increase of barrier island elevations as salt marsh habitats are converted to barrier island environments. The net result of overwash is that bay shorelines have either remained relatively stable or marsh acreage has been lost while subaerial barrier island habitat has increased. Leatherman and Allen (1985) found that overwash has contributed little to new land creation and barrier island migration. They estimated the total contribution of overwash to new marshland to be about 5.7 acres between 1938 and 1962, mostly from storms in 1938, 1954, 1960 and 1960. This total area is equivalent to only 0.24 acres/year.

More recently, overwash resulted in approximately 34 acres of new land from 1980 to 1995 (2.3 acres/year), comprised of 30, 2.5 and 1.5 acres at Swan Island, Smith Point and Pelican Island, respectively (USACE, 1999). This new land area represents approximately 20 percent of the total overwash area experienced during this 15-year period (approximately 170 acres or 11.3 acres/year).

A review of post-Sandy aerial imagery supports the finding that the majority of the overwash habitat resulted in the conversion of one upland type to another. Specifically, Hurricane Sandy resulted in approximately 13.5 acres of "new land" because of overwash (excluding breach areas), with 11 acres of this new land in Smith Point County Park:

- Approximately 0.7 acres of "new land" at the Reagan Property,
- Approximately 0.3 acres of "new land" in the wilderness area east of the breach
- Approximately 9 acres near Pattersquash, and 2 acres near the SPCP breach of "new land" in Smith Point County Park
- Approximately 0.5 acres of "new land" in Tiana Beach

Compared to the overall amount of overwash that was formed due to Hurricane Sandy, the creation of only 13.5 acres, supports the previous findings that the majority of overwash results in habitat conversion, rather than the creation of new land.

As summarized above, historic rates of overwash and new land creation vary significantly, from 0.24 acres/year between 1938 and 1962 to 2.3 acres/year in the 1980 to 1995 period. Between 1995 and 2017, the only significant in-bay overwash event was Hurricane Sandy in 2012, which resulted in 13.5 acres of new land due to overwash (equivalent to 0.6 acres/year). Therefore, the estimates of future with- and without-project overwash areas presented above appear to be conservative relative to historic observations. Specifically, the FWOPC estimates summarized in Table 6-24 suggest approximately 11.3 acres/year of in-bay overwash area above MSL for the reaches considered. Similarly, FWPC estimates summarized in Table 6-25 range from 8.8 to 10.1 acres/year, for years 0 to 30 and 31 to 50, respectively.

Given the uncertainty in the breaching and overwash area projections, it is recommended that an initial volume/area of sediment be targeted as the basis for coastal process reestablishment, and that these features be adaptively managed with continuing construction over the project life (akin to the sediment bypassing, and renourishment). Future construction could include the renourishment of these features, or alternately the construction of features in additional locations. The initial construction of these coastal process features is expected to occur in conjunction with the beachfill being undertaken along the adjacent ocean shoreline (a similar approach would also be expected during future construction). Since beachfill work is expected to occur over several years in multiple construction contracts, the construction of these coastal process features will be phased, to allow for lessons learned in the construction process.

7.0 ALTERNATIVES DEVELOPMENT

7.1 Introduction

The initial investigation of the full array of measures was undertaken to focus the alternative analysis on those measures which address the problems and opportunities in the Study Area. It leans upon the scientific and engineering material presented in previous sections and further develops the correlated economic impacts associated with them. The screening was completed in parts:

1. Initial screening of measures
2. Secondary screening of coastal storm damage risk reduction and restoration measures.
3. Detailed screening of coastal storm damage risk reduction and restoration measures.

This chapter summarizes the screening process, and identifies the coastal storm damage risk reduction and restoration measures recommended for further development in the second phase of evaluation, First Added Assessment of Alternative Measures. Additional details of the screening process are available in previous documents from the 2009 Reformulation Report (Alternative Screening Report, Non-Structural Supplemental Screening Memorandum, and the HEP Report). The information contained in this chapter was developed in the plan formulation process. Although dated, the information and decision making processes are still valid. Cost comparisons developed over the years are still relevenant and have not been updated for this portion of the plan formulation.

The purpose of the Screening of Alternative Measures is to identify potential solutions for the reduction of coastal storm damage risk to economic resources such as residences, commercial properties, and infrastructure; and restoration of coastal processes throughout the study area. This screening was preliminary and primarily intended to narrow the suite of possible solutions before proceeding to a more refined evaluation of selected measures. The detaled screening includes analyses of economic, environmental, and social and institutional issues, and consistency with the P & G's vision objectives to support plan selection.

<u>Coordination with Federal, State and Municipal Governments.</u> Throughout this process, involved Federal, State and municipal agencies were included in coordination meetings, and multiple meetings were held with the five Towns and incorporated villages within the study area to solicit their input on the array of alternatives under consideration. This included a workshop with all the project stakeholders to solicit input on the viability of non-structural measures. The results of the screening reflect the results of this coordination, and local preferences identified in this process.

7.2 Reach Delineation

The 83 mile study area shoreline was separated into segments to ease alternative development, evaluation, screening and design procedures. During the previous Reformulation Study efforts (circa 1998), the study area was separated into a series of reaches, namely: (1) project, (2) physical, and (3) economic reaches. For the new designation, the study area was reorganized into ten design reaches for preliminary design purposes. Design reaches are designated to correspond to project reach boundaries with further subdivisions within project reaches that represent segments where consistent design features are evaluated. The design reaches were further separated into design subreaches, which represent unique problem areas and/or design criteria. The following describe both the old and new delineated reaches.

Reaches for the Fire Island Inlet to Montauk Point (FIMP) study area were delineated based on site-specific project, physical and economic criteria. Reaches were defined to establish the basis for the independent evaluation of alternative storm-damage reduction measures.

7.2.1 Project Reaches

The principal factor considered in project reach delineation was the requirement to provide coastal storm damage risk reduction benefits for a contiguous area (e.g., all of Shinnecock Bay). Project reaches may be characterized by varying physical characteristics that influence the development of coastal storm damage risk reduction plans. Therefore, project reaches were subdivided into physical reaches to reflect changes in physical conditions important to design. The study shoreline was originally designated into five project reaches as follows:

Project Reach 1 – Montauk
Project Reach 2 – Ponds
Project Reach 3 – Shinnecock
Project Reach 4 – Moriches
Project Reach 5 – Fire Island

7.2.2 Physical Reaches and Design Subreaches

Physical reaches were defined as continuous shore segments having similar geomorphic features and environmental constraints. As stated above, physical reaches are subreaches of project reaches. Project features would be consistent within a physical reach, but may vary between neighboring physical reaches. Consequently, alternatives for a given project reach include the design features of each applicable physical reach. Design subreaches correspond to those areas where coastal storm damage problems and economic development may provide economic justification for coastal storm damage risk reduction plans, but were primarily selected based upon identified storm damage problems.

7.2.3 New (Year 2000) Reach Designations

New reach designations are established for the purpose of conceptual screening and design. New reach designations separate the project area into project reaches, design reaches, and design subreaches. Design reaches represent combined physical reaches delineated during previous efforts, and reflect areas where consistent coastal storm damage risk reduction features may be evaluated. Each of the designated reaches and subreaches start with a letter abbreviation representing the project reach location so that reach locations may be readily identified. Of primary importance to the Year 2000 reach designations is that it was used for the preliminary structural alternatives development shown in the "Basis of Design Report" prepared by Moffatt & Nichol Engineers in June, 2000. This report is referred to as "BDR" for FIMP study.

Table 7-1. Reach designations

New (Yr2000) Reach Designations			Old (circa 1998) Reach Designations					
Project Reach	Design Reach	Design Subreaches	Project Reach	Name	Physical Reach	Physical Name	Design Subreaches	Name
GSB	GSB-D1	GSB-D1A	5	Fire Island	5E	Robert Moses	5E-2	Fire Island Inlet -East
		GSB-D1B					5E-1	Robert Moses - West
		GSB-D1C			5D	USCGS	5D-2	Robert Moses - East
		GSB-D1D					5D-1	Coast Guard Station
	GSB-D2	GSB-D2A			5C	Atlantique	5C-3	Saltaire
		GSB-D2B					5C-2	Atlantique
		GSB-D2C					5C-1	Ocean Beach
		GSB-D2D			5B	Cherry Grove	5B-5	Ocean Bay Park
		GSB-D2E					5B-4	Sailors Haven
		GSB-D2F					5B-3	Fire Island Pines
		GSB-D2G					5B-2	Water Island
		GSB-D2H					5B-1	Davis Park
	GSB-D3	GSB-D3A			5A	Wilderness Area	5A-3	Wilderness Area - West
		GSB-D3B					5A-2	Old Inlet
		GSB-D3C					5A-1	Wilderness Area - East
MB	MB-D1	MB-D1A	4	Moriches	4D	Smith Point CP	4D-2	Smith Point - West
		MB-D1B					4D-1	Smith Point - East
	MB-D2	MB-D2A			4C	Moriches Inlet	4C-3	Great Gun
		MB-D2B					4C-2	Moriches Inlet - West
		MB-D2C					4C-1	Moriches Inlet - East
		MB-D2D			4B	Pikes	4B-1	Pikes
		MB-D2E			4A	Westhampton	4A-1	Westhampton
SB	SB-D1	SB-D1A	3	Shinnecock	3C	Tiana	3C-3	Hampton Beach
		SB-D1B					3C-2	Sedge Island
		SB-D1C					3C-1	Tiana Beach
	SB-D2	SB-D2A			3B	Shinnecock Inlet	3B-3	Ponquogue
		SB-D2B					3B-2	Shinnecock Inlet - West
		SB-D2C					3B-1	Shinnecock Inlet - East
	SB-D3	SB-D3A			3A	Southampton	3A-3	Southampton Beach
		SB-D3B					3A-2	Southampton
		SB-D3C					3A-1	Agawam
P	P-D1	P-D1A	2	Ponds	2C	Mecox	2C-4	Wickapogue
		P-D1B					2C-3	Watermill
		P-D1C					2C-2	Mecox Bay
		P-D1D					2C-1	Dune Road
		P-D1E			2B	Sagaponack	2B-3	Surfside Drive
		P-D1F					2B-2	Sapaponack Lake
		P-D1G					2B-1	Peters Lane
		P-D1H			2A	Georgica	2A-3	Wainscott
		P-D1I					2A-2	Georgica Pond
		P-D1J					2A-1	Apaquogue
M	M-D1	M-D1A	1	Montauk	1C	Amagansett	1C-1	Beach Hampton
		M-D1B			1B	Napeague	1B-2	East Hampton Beach
		M-D1C					1B-1	Hither Hills
		M-D1D			1A	Montauk Point	1A-2	Montauk Beach
		M-D1E					1A-1	Ditch Plains

7.3 Measures Considered

Measures were sought which reduce the risk of coastal storm damages and restore coastal processes in the study area; and when possible. avoid unnecessary adverse impacts to economic, social and environmental resources. The following list of measures was examined to determine its applicability within the study area, and to select those appropriate for further consideration in the development of alternatives during future study phases.

- No Action
- Non-Structural Measures
- Coastal Process Restoration Measures
- Sediment Management (including Inlet Modifications)
- Breach Response Measures
- Removal/Modification of Groins
- Beach Restoration
- Offshore Breakwaters (including Artificial Headlands or T-Groins)
- Seawalls (Rubble-mound)
- Groins
- Beach Restoration With Structures
- Levees and Floodwalls
- Storm Closure Gates

7.4 Initial Screening

An initial screening of measures was undertaken to identify the effectiveness of these measures in accomplishing the desired objectives. Based upon this initial screening, these measures were either recommended for further screening, or dropped from consideration. The following sections provide an overview of the measure and a summary of the results of the initial screening.

7.4.1 No Action

Simply stated, this plan means that no additional measures would be taken to provide for coastal storm damage risk reduction in the study area and assumes continuation of the future without-project condition. This plan is based on the description of the Without-Project Future Condition, which assumes continuation of the Westhampton Interim Project for thirty years, breach closure activities within a period of one year, continuation of inlet maintenance activities, and continuation of locally implemented measures, as described earlier in the report. This plan fails to meet any of the objectives or needs of the project. <u>While this plan was not considered for further development, it does provide the basis for measuring with-project benefits, and was recommended for further analysis.</u> Additionally, this plan would be implemented if there is no plan found to be in the Federal interest.

Non-Structural Measures

There are three main categories of non-structural plans: 1) building retrofits, 2) acquisition of threatened properties, and 3) land use management options. Building retrofits include raising the structure above the design flood, providing an impermeable barrier around the structure, wet floodproofing, or relocating the structure out of the flood plain. Wet floodproofing techniques allow floodwaters to enter the crawlspace or unfinished levels of the structure but relocates utilities and reduces the chance of utility infrastructure damage. Unlike floodproofing, acquisition of structures in the flood plain will prevent all damage to structures and may provide land for public use and conservation. However, buyouts may decrease the local tax base by removing land from private ownership. Land use management options include zoning regulations and other measures that restrict further development in areas where continued development is expected. Land use management is an effective way of controlling flood plain development and thereby minimizing future increases in the potential damage associated with flooding. Although land use regulation may be recommended, implementation of these measures is the responsibility of state or local governments, and would likely be an element of a Floodplain Management Plan. Non-structural techniques can also supplement the coastal storm damage risk reduction provided by other structural features, and can be evaluated as combined or stand-alone measures. Non-structural measures were recommended for further evaluation.

7.4.2 Coastal Process Restoration

As part of this study, a restoration framework was established which identified the objective of restoring coastal processes. The key difference between the restoration of coastal processes and restoration of a specific landform, is that restoration of coastal processes emphasizes realigning the processes with the natural functioning rather than achieving a specific habitat.

The restoration framework identified 5 key physical processes to be targeted for restoration, including 1) alongshore transport, 2) cross-island transport, 3) dune growth and evolution, 4) bay shoreline processes, and 5) estuarine circulation and water quality. There are a number of measures that can be applied to achieve these restoration objectives, which are presented further in the screening of restoration measures.

The restoration measures can generally fall in the types of effort to include: 1) restoring the process by removing or modifying the source of the disturbance, 2) restoring the process by mimicking what would occur naturally, with sustainable features, or 3) restoring the process by mimicking what could occur naturally , with features that require continued management to achieve the objectives. Coastal process restoration alternatives were recommended for further study.

7.4.3 Sediment Management (Inlet Sand Modification)

Sediment Management includes a range of measures designed to improve the littoral transport of material. These measures include those associated with improving the littoral transport at inlets, and also include the establishment of feeder beaches, designed to improve the effectiveness of sediment transport to downdrift shorelines.

Tidal inlets, either stabilized or unstabilized, represent littoral drift disruptions. Areas updrift (east in the study area) may be subject to accretion as longshore sediment transport is trapped. A portion of longshore sediment transport entering the inlet will also be transported cross-shore and be distributed into flood or ebb shoals adjacent to the inlet. The remaining portion of longshore sediment transport will bypass the inlet and nourish the downdrift beaches. Trapping of longshore sediment transport, either updrift or within the inlet and shoals, may create sediment transport deficits downdrift that may result in shoreline erosion. The erosion experienced downdrift of inlets may be marked and can more significant than experienced outside of the inlet vicinity. As this erosion can be partly assigned to sediment trapping caused by the inlet, measures to enhance/restore littoral drift across the inlets in the study have been investigated. These measures include dredging of inlet shoals and channels and/or excavating updrift deposits with placement downdrift, and other inlet design modifications (e.g., modification of inlet cross-sections to reduce shoaling) to aid natural bypassing. The sediment management measures were recommended for further evaluation, including consideration for improving longshore transport. At the inlets, measures are recommended for further consideration to balance the objectives of: 1) reliable navigation, 2) offsetting localized sediment disruption, and 3) uninterrupted regional sediment transport.

In addition to altering sediment transport pathways, inlets also serve as a conduit for floodwaters to enter the bays during storm events. Therefore, modifications of current inlet design and dredging practices that may provide measures to limit storm surge propagation through the inlets that leads to bay flooding have also been explored.

7.4.4 Breach Response Measures

Breaching refers to the condition where severe overwashing forms a new inlet which permits the exchange of ocean and bay waters under normal tidal conditions. The breach may be temporary or permanent depending on a number of factors; however, the breach must have a scoured depth below mean lower low water in order for water to exchange between the ocean and bay over a complete tidal cycle (to meet the definition of a breach). Factors which lead to the formation of a breach include narrow barrier island width, relatively low dune elevation, and relatively small island cross-section volume above some critical elevation. Once a breach has formed, the likelihood of it remaining open to form a permanent inlet depends on a number of factors including, size of the initial opening, adjacent bay side bathymetry, presence of other inlets, longshore drift rate, and ocean-bay tidal phase differences.

Breaches left unchecked, as evidenced by breach closure efforts in 1980 and 1993 just east of Moriches Inlet, will result in significant damages that could be avoided if pre-breach measures were planned to allow for rapid closure procedures. Previous studies (BCP, 1995) have also shown that delayed closure will also result in increased overall closure costs. Therefore, breach response measures, including plans for rapid closure and proactive measures, were recommended for further consideration.

7.4.5 Beach Restoration

Beach restoration generally involves the placement of compatible sand from an offshore source (borrow area) on an eroding shoreline to restore its form and to provide an adequate geometry to provide coastal storm damage risk reduction. Beach restoration may include the following options: (1) beach and dune fill, (2) dune fill only, (3) beachfill only or (4) beachfill placement in response to extreme events to close breaches (e.g., BCP). Selection of the desired configuration depends on site conditions, and must consider whether fill placement is intended to combat shore erosion, flood inundation, or both. A beachfill typically includes a berm backed by a dune and both elements combine to prevent inundation damages to leeward areas. Periodic renourishment is normally required to offset long-term and storm-induced erosion. At locations where long-term and storm-induced erosion are severe, renourishment and rehabilitation may prove costly. Beach restoration represents a quasi-natural method for reducing the risk of flooding and erosion damages, and is an important element for constructed coastal storm damage risk reduction measures that must combat severe erosion. Beach restoration is commonly used in concert with other structural features (e.g. offshore breakwaters, groins, buried seawalls etc.).

Quantities of offshore sand can sometimes be minimized by utilizing material otherwise available in the active littoral system, such as at stabilized inlets and nearby navigation channels. Common examples of alternative sand sources include the beneficial use of dredged inlet materials, inlet sand bypassing that acts to mechanically move beach sands across gaps (inlets) in the littoral system, stockpiles, feeder beaches and beach scraping.

Beach restoration measures were recommended for further consideration, to identify locations within the study area where the infrastructure at risk would support this type of solution.

7.4.6 Offshore Breakwaters

Offshore breakwaters are typically rubble-mound structures built seaward of the shoreline, and act to reduce wave energy reaching the shoreline. Offshore breakwaters may be built as a long continuous structure or as a series of shorter, segmented structures. The advantages of segmented breakwaters include cost-effectiveness and design flexibility. The effect of breakwaters is to cause gradients in wave energy in the lee of the structures that promote sediment deposition behind the breakwaters. When properly designed, these depositional features should not interrupt longshore sediment transport in a way that negatively impacts adjacent shorelines. As with other coastal structures, offshore breakwaters are often combined with beach restoration. For example, beach restoration may serve to reduce storm-induced damages, while the offshore breakwater system serves to reduce long-term erosion. The need for structural features combined with beach nourishment is particularly acute near inlets, where both long-term and storm-induced erosion may be severe. Beachfill and offshore breakwater combinations reduce the risk of coastal storm damage to the shoreline, and, when properly designed, will permit sand bypassing of the inlet. If located too far offshore, for instance, offshore breakwaters located near inlets may interfere with inlet behavior. Consequently, it is often advisable to locate the structures closer to shore where they would act as artificial headlands or combined with tradition groins to form T-groins. Breakwater placement closer to shore reduces construction costs and enhances fill stabilization relative to breakwaters located further offshore.

Based upon the initial screening, offshore breakwaters, as stand-alone features are not universally recommended for further consideration. Offshore breakwaters are not recommended for further consideration as structures combined with beachfill. Based upon the initial screening, breakwaters tend to be comparable to other coastal structures in stabilizing beachfill, but the costs associated with breakwater construction are much higher than other available methods. <u>Offshore breakwaters were considered further in conjunction with inlet modification alternatives, including the integration of breakwaters and groins in T-groin configurations. However, they were not considered as a stand alone alternative.</u>

7.4.7 Seawalls

Seawalls are generally used to reduce the risk of damage to upland structures from wave impact and erosion. Seawalls are typically rather massive structures as they are intended to resist the full force of storm waves. Seawalls normally require extensive toe protection to reduce the risk and magnitude of scour. Vertical seawalls are generally high and are often judged to be socially and aesthetically unacceptable. Moreover, vertical seawalls are vulnerable to catastrophic failures that may be attended by accelerated upland erosion. A rubble-mound seawall consisting of relatively large armor units and armored backslope provides a high level of stability when subjected to direct wave forces. An exposed rock structure in the absence of beach restoration does not abate shoreline erosion, because it does not provide the sand necessary to offset erosion processes. Seawalls are typically located landward of the active littoral zone, therefore, shoreline erosion is not affected. An alternative to a conventional rubble-mound or vertical seawall is a buried rubble-mound seawall placed landward of the shoreline; the rubble-mound seawall is often coupled with beach restoration. Example applications of a buried seawall are described in Headland (1992) and Basco (1998). The buried seawall has the appearance of a sand dune and is only exposed during severe events. When used in concert with beachfill, the seawall provides the last-line-of-defense to reduce the risk of coastal storm damage, while the beach restoration combats long-term shoreline erosion.

<u>Based upon the initial screening, seawalls as stand-alone measures are not recommended for further consideration. Seawalls, in the form of a reinforced dune, were considered further in the secondary screening to determine their applicability when considered in combination with beachfill.</u>

7.4.8 Groins

Groins are coastal structures, normally constructed perpendicular to the shoreline, which act to interrupt longshore sediment transport. Groins generally extend from the dune/beach interface to MSL water depths on the order of 10 to 12 feet and are designed to impound sand. At a single groin, the updrift impoundment of sand is generally offset by an equivalent amount of erosion downdrift of the structure. Groins are often constructed in series or fields to provide coastal storm damage risk reduction for continuous shoreline segments. In this arrangement, erosion is displaced to the most downdrift groin, rendering the downdrift area susceptible to accelerated erosion. Erosion downdrift of a groin field can be mitigated through the use of low, tapered groin transitions and/or beach nourishment. Groin fields can also be designed to transition to areas of lower erosion losses or to terminal structures, such as jetties. Furthermore, groin compartments should be filled initially in order to promote sand bypassing throughout the groin field. Groins fields may be particularly effective at areas characterized by significant longshore sediment transport or high erosion rates. Groins are, however, vulnerable to storm-induced or offshore erosion losses. These losses may be reduced by the use of T-groins that may be an effective solution in areas of severe erosion, such as in the vicinity of tidal inlets. T-groins combine the features of traditional groins and breakwaters by reducing both alongshore and cross-shore beach erosion losses.

Based upon the initial screening, groins as stand-alone features were not recommended for further consideration. Groins were considered further as measures which could be implemented in combination with beach nourishment. Groins and T-groins were also considered further in the context of inlet modification alternatives.

7.4.9 Beach Restoration and Structures

Life-cycle costs may be much higher for beach restoration in areas of severe erosion. Therefore, in these areas it is advisable to consider beach restoration in concert with structural options that augment coastal storm damage risk reduction against severe storms (i.e. seawalls) or stabilize the beachfill against long-term erosion (i.e. breakwaters and groins). These structures act to reduce long-term maintenance requirements and/or residual damages arising from severe storm effects. Beach restoration performance may also be improved by including structures at locations requiring only isolated (short) lines of coastal storm damage risk reduction. The principal consideration in these cases is the poor performance typically characteristic of small beachfill projects.

As presented above, the initial screening recommended consideration of beach nourishment in conjunction with structures. The secondary screening identified, for the locations where beachfill may be viable, the relative effectiveness of integrated coastal structures. Also as presented, the combination of beachfill and structures were also explicitly considered in the context of the inlet modification alternatives.

7.4.10 Removal/Modification of Groins

Groins serve to reduce the risk of storm damage to the shoreline fronted by these structures, but may adversely impact downdrift shorelines. Adverse impacts of groin fields may be mitigated through beachfill placement and/or groin transitions or it may be best to remove or modify existing groins. The functioning of the existing groin fields within the study area must be evaluated to determine whether groin removal or modification is advisable. Based upon the initial screening, the existing groins within the study area were evaluated further to consider the effectiveness of groin removal or modification, including shortening or notching.

7.4.11 Levees and Floodwalls

Levees and floodwalls are generally considered the most direct method to reduce the risk of damage to the backbay/mainland areas from tidal inundation. Levees and floodwalls are not suited to reduce the risk of coastal storm damage from wave action, and are not considered for oceanfront applications. They provide coastal storm damage risk reduction to developed areas by providing a continuous barrier around a group of structures and are often described as local storm damage risk reduction measures. The structures may be made of earthen materials, concrete, rock, metal sheetpiling or a combination of materials. Along the mainland shorefront, such features would tie into high ground at each end of a project segment. In general, levees (dike or embankment, comprised of rock or earthen materials designed to reduce the risk of flooding to low land areas) are less expensive than floodwalls (comprised of concrete and/or sheetpiling) but require more land. If a large area is to be included behind such structures, the numerous rivers or canals draining into the bays will either require closure gates and drainage facilities such as pump stations or will require the levees and floodwalls to surround the water course on both sides, frequently extending inland to high ground. This often requires significant roadway and bridge relocation as the existing structures are usually too low to cross over the levee or floodwall. The levee/floodwall must be accompanied by an extensive interior drainage system to impound and/or pump stormwater runoff.

The initial screening of alternatives considered levees and floodwalls. These measures were eliminated from general application, in that they were not economically viable, due to the mainland site constraints, and generally not supported by sponsors and stakeholders. <u>Levees and floodwalls were recommended for further consideration in the limited context of road raising alternatives, which can be considered as smaller scale measures that would accomplish objectives similar to the mainland non-structural building retrofits.</u>

7.4.12 Storm Closure Gates

Flood control closure gates are designed to prevent storm surges from entering tidal inlets and/or canals. As mentioned previously, closure gates are also included in levee and floodwall features for canal and creek closures. In the present context, closure gates could be considered at Fire Island, Moriches and Shinnecock Inlets, as well as Narrow Bay and Quogue and Quantuck Canals. Storm closure gates constructed at these locations could reduce inundation damages by limiting storm tidal flows into study area estuaries. While several types of closure gates exist, they can be primarily classified as either mobile or fixed systems. Mobile systems can be raised, lowered or otherwise removed when there is no threat of coastal flooding. Fixed systems restrict flow during storms by inducing hydraulic losses and/or limiting flow area.

The initial screening considered the relative cost and effectiveness of closure gates at the locations described above. The initial screening concluded that the cost for these structures exceeds the maximum benefits that could be derived, and that there were concerns regarding the environmental impact of these alternatives. <u>As a result, these storm closure gate measures were not recommended for further consideration.</u> As presented above, the inlet modification alternatives will consider if modifications to the inlet management practices could reduce tidal flow.

At the coastal ponds, consideration was given for water control structures, that similar to inlet closure gates, would provide a mechanisms to control the inflow and outflow of water from the ponds. These measures were developed as an alternative to the present practice, which is both the regularly scheduled and storm-induced opening and closing of the ponds. These inlet closure structures would be a necessary component of any plan that would include beachfill fronting the ponds. <u>These water control structures at the ponds were eliminated from consideration, since they were not locally supported because of the impact these structures would have on the ability of the Town Trustees to manage the ponds as they historically have.</u>

7.4.13 Results of Initial Screening

In conducting the initial screening of measures, the above alternative plans were looked at for their applicability for accomplishing the study objectives in the study area. As is presented in the summary of each measure, the following were recommended for further consideration in the secondary screening of alternatives.

- No Action
- Non-structural Measures
- Coastal Process Restoration
- Breach Response Measures
- Beach Restoration
- Sediment Management (including Inlet Modifications)
- Removal/Modification of Groins
- Beach Restoration with Structures
- Mainland Road Raising

The following section provides a summary of the secondary screening undertaken for the Storm Damage Reduction Measures. Following this section is a summary of the screening undertaken for the Coastal Process Restoration Measures.

7.5 Secondary Screening of Storm Damage Reduction Measures

The eight measures recommended for further consideration following the initial screening were developed to a conceptual level of detail to provide a basis for comparison and screening of different coastal storm damage risk reduction measures to establish their applicability throughout the study area in the secondary screening. The scope and complexity of each of the potential measures varies; as such, the extent of the screening varies, as well. For example, sediment management measures associated with the inlet are complex and wide ranging. As a result, the level of screening that has gone into this analysis was of a greater level of detail than other measures.

The following factors were considered for each measure to determine their applicability as part of potential plan alternatives.

- Performance – What is the role of the feature in the reduction of storm damages? Where is the feature located?
- Design – What are the specific feature requirements for the study area?
- Costs – What are the costs for measure construction and maintenance?
- Limitations – Does the measure fully address the problem? Can the measure be implemented?
- Impacts – What is the effect of the measures on the environment? Is the measure socially/aesthetically acceptable?

These screening factors helped to select cost-effective solutions for the reduction of storm damages, and minimize adverse social and environmental impacts.

Non-Structural Measures

The secondary screening of Non-structural measures followed the recommendations from the initial screening of alternatives, which recommended consideration of non-structural building retrofit alternatives, including land management strategies, and acquisition alternatives. In order to undertake this effort, the Corps conducted a supplemental screening of non-structural alternatives, as the basis for identifying and coordinating these available alternatives with the local sponsor and municipalities.

Each non-structural alternative was evaluated to determine whether it could perform the following functions:

- Reduce flooding damage to existing development;
- Reduce erosion and wave damage to existing development;
- Reduce flooding damage to future development and redevelopment;
- Reduce erosion and wave damage to future development and redevelopment;
- Avoid or minimize adverse environmental project effects;
- Preserve or enhance existing ecological resources;
- Preserve or enhance recreational access;
- Preserve community character.

Reduction in flooding, erosion, and wave damage would be achieved by modifying structures to a specified design level to lessen risks from these sources of damage. The term "existing development" includes regular maintenance and upkeep activities, but does not include substantial improvement or expansion of existing structures. The term "future development and redevelopment" includes new construction and modifications to existing structures requiring permit approval from local, county, and or state authorities. The preservation of community character would be met by preserving an area's existing visual character, cultural resources, population characteristics, transportation infrastructure, public recreational facilities, neighborhoods, and scale. The techniques evaluated are listed below, and are grouped into four main categories:

1. Land Use/Regulatory. Zoning/Land Use Controls, New Infrastructure Controls, Landform/Habitat Regulations, Construction Standards and Practices, Insurance Program Modifications, and Tax Incentives;
2. Building Retrofit. Relocation, Elevation, Free-Standing Structures, Dry Floodproofing, and Utilities Protection;
3. Land Acquisition. Purchase of Property, Exchange of Property, Transfer of Development Rights, Easements and Deed Restrictions;
4. Other. Wetland Preservation and Restoration, and Vegetative Stabilization.

The evaluation of alternatives was conducted on a project reach basis (Great South Bay, Moriches Bay, Shinnecock Bay, Ponds, and Montauk), with Great South Bay split into a barrier island and a mainland sub-section, to account for differing conditions in the two areas.

Non-structural Supplemental Screening Results

For the mainland reaches, the evaluation determined that all of the non-structural alternatives were found to meet or potentially meet the project objectives. No measures were eliminated from further consideration for these reaches during this phase. Because of the special circumstances of the barrier islands, three alternatives were eliminated from further consideration. New Land Use Controls were eliminated because the FIIS General Management Plan has effectively designated community districts to restrict installation of new infrastructure as a means of controlling development. Thus, this technique is already fully implemented on Fire Island. Free Standing Structures, such as ringwalls to reduce the risk of damage to individual buildings, and Dry Floodproofing were also eliminated for use on Fire Island. Free-standing barriers are prohibited in dune areas and the Coastal Erosion Hazard Area (CEHA); in addition, there is limited lot space on many of the interior parcels. In addition, the water diverted from flooding a structure using this method would only be transferred to adjoining properties. Dry Floodproofing is unsuited for use on the barrier islands, particularly given the depth of flooding that can occur in the shorefront areas. Dry floodproofing techniques typically requires a structurally sound slab foundation to prevent water from entering the structure from below, and the vast majority of buildings on Fire Island are constructed on pile foundations. Wet floodproofing techniques are also unsuitable for barrier island buildings for the same reason.

As part of the supplemental screening, the non-federal study sponsor, the New York State Department of Environmental Conservation (NYSDEC), stated that it did not support non-structural measures for buildings on the barrier island. The vast majority of these buildings are not primary residences and are only seasonally occupied; there are logistical issues associated with building retrofits, and the concern that retrofits would still leave structures vulnerable to ocean hazards, and increase the investment potentially at risk in environmentally sensitive areas are some of the reasons for this direction. NYSDEC chose instead to support the evaluation of non-structural measures for permanently occupied buildings on the backbay mainland of the project area. Nonstructural retrofits on the barrier islands were eliminated from further screening and will not be considered further.

The remainder of the techniques identified in the initial screening successfully passed this second round and were further evaluated, as detailed below. An important outcome of this supplemental screening was the identification of the techniques that should be evaluated for possible inclusion for Federal implementation in the recommended plan, and which techniques would be recommended for inclusion in a non-federally implemented Flood Plain Management Plan (FPMP) as a component of the overall collaborative plan. A number of the alternatives can be included in both. The USACE does not possess authority to modify or implement local land use regulations; this power rests at the municipal and state levels, and thus certain alternatives are assigned only to the FPMP. Table 7-2 below shows where (in terms of authority to implement) each alternative can be evaluated.

Based upon the findings of this screening, the recommendation was to further develop the non-structural alternatives in two main categories, 1) building retrofit alternatives along the mainland, and 2) land and development management alternatives that could be implemented to reduce development pressures, and the existing development in high hazard areas, where retrofits are not applicable.

Table 7-2 Summary Of Non-Structural Technique Evaluation

NON-STRUCTURAL TECHNIQUE	RECOMMENDED FOR FURTHER EVALUATION UNDER:		
	FIMP Reformulation Plan	Non-Federal Flood Plain Management Plan	
	USACE*	State	Local
Land Use and Regulatory Measures			
Zoning/Land Use Controls		+	+
New Infrastructure Controls		+	+
Landform and Habitat Regulations		+	+
Construction Standards and Practices		+	+
Tax Incentives		+	+
Building Retrofit Measures			
Relocation	+	+	+
Elevation	+	+	+
Free-Standing Barriers (mainland only)	+		
Dry Floodproofing (mainland only)	+	+	+
Utilities Protection	+	+	+
Land Acquisition			
Purchase of Property	+	+	+
Exchange of Property		+	+
Transfer of Development Rights		+	+
Easements and Deed Restrictions	+	+	+
Other			
Wetlands Protection & Restoration	+	+	+
Vegetative Stabilization	+	+	+
Post-Storm Response Planning	+	+	+
* It is acknowledged that there are other Federal agencies (including the NPS, within the jurisdictional boundaries of FINS; FEMA; and USFWS) that have a Federal Role in these activities			

7.5.1 Coastal Process Restoration

The FIMP Vision Statement establishes that measures to protect and restore coastal landforms and natural habitats on a system-wide basis be one of the FIMP Reformulation Study's objectives.

In order to establish specific objectives a Restoration Framework was developed. This framework called for the restoration of five coastal processes which are critical to the development and sustainability of the various coastal features (such as beaches, dunes, barrier islands and bluffs) that, together, form the natural system. The five Coastal Processes identified by the Restoration Framework as vital to maintain the natural coastal features are: Longshore Sediment Transport; Cross-Island Sediment Transport; Dune Development and Evolution; Estuarine Circulation; and Bayside Shoreline Processes.

The following is a brief description of the types of specific restoration that can be undertaken to achieve these restoration objectives

<u>Longshore Sediment Transport.</u>

Restoration of the longshore process can help to maintain a more natural shoreline condition, and a more natural beach profile. Restoring these processes can reduce the need for future activities to address erosion in these areas. Restoration of longshore transport can be undertaken through a number of options. The most effective way to accomplish this is in the removal of the barrier that is disrupting the transport. If removal of the barrier is not possible, modification of the structure (such as shortening or notching) could be considered. If neither of these options is viable, it may be possible to replicate the processes that would have naturally occurred (i.e. bypassing sand at the inlets).

Cross-Island Transport

Opportunities for restoration of this habitat are similar to those identified for longshore transport. The preferred approach would be to allow these processes to continue unimpeded, or promote the occurrence of these processes in areas where they have been negatively impacted. If these processes can't be restored through this process, it may be possible to replicate the processes as they would have naturally occurred (i.e. the construction or restoration of overwash habitats).

Dune Development and Evolution.

In much of the study area, the long-term trend is erosional. In these areas, under a natural condition, the dunes would tend to evolve and migrate over time. To varying degrees, the existing dunes are unable to do this due to development and the past efforts undertaken to maintain a beach and dune to protect existing development. Prior decisions have impacted the natural growth and evolution of the dunes. Significant amounts of dune habitat have been degraded due to the presence of buildings on the dunes. One opportunity for restoration of the dune process include removing structures to allow for improved dune functioning, and removal of buildings to provide the necessary space to allow for dune evolution. If this is not viable, the next available opportunity could be construction of a dune, or enhancement of an existing dune that is allowed to move over time through phased acquisition.

Bayside shoreline Processes.

The possible solutions for restoring these bayside processes include removal of the actions that have caused or are causing the disruption. There may be some areas where removal of bayside bulkheading or filling of channels could be a viable option. In areas where this is not feasible, the next set of scenarios could consider reducing the impact of these structures through modification of the structure. Lastly, it may be possible to consider taking actions to replicate the processes, through the infusion of material to offset the impact of the disturbance.

Estuarine Circulation

The magnitude of human changes within the estuary, and the complexity of the interaction between the physical processes and the environment make it difficult to identify a clear objective for the restoration of estuarine circulation processes, although the topographic and bathymetric changes within the estuaries can provide clear opportunities for habitat restoration

In the consideration of restoration alternatives, two main categories of process restoration present themselves:

1. Restoration of processes with the primary objective of storm damage reduction. These are restoration alternatives that were designed for the purpose of using habitat features for

coastal storm damage risk reduction purposes. These include measures such as sand bypassing, and some bayside habitat restoration in breach vulnerable areas.

2. Restoration of processes with the primary objective of habitat restoration. These are measures developed by an interagency team to identify optimal locations for restoration to primarily achieve ecological objectives, with a secondary objective of reducing the risk of coastal storm damages.

In order to achieve these objectives, the habitat restoration measures generally can be accomplished with the following measures as described below:

- Along the Atlantic Ocean shorefront, measures are developed to restore beach and dune habitat, including:

    - establishing optimal beach and dune conditions, accounting for footprint, slopes, and vegetative cover.
    - Restoring the beach and dune through removal of buildings in the dune
    - Restoring the beach and dune through removal of buildings and infrastructure to allow for dune migration
    - Removal or modification of coastal structures to allow for more natural beach and dune conditions.
    -

- In the interior of the island, measures are considered for restoring secondary dunes, and removing areas of disturbance to provide habitat connectivity from Ocean to Bay.
- Along the bayside shoreline, measures are developed to restore bayside habitats (inclusive of the bay islands),

    - Restoring bay beaches, wetlands, and subaquatic vegetation
    - Restoring these bayside habitats through removal or modification of bayside structures
    - Restoring these bayside habitats with the use of bayside structures to stabilize the restoration.

Identification and screening of potential restoration sites

The identification of potential restoration measures was undertaken as a site evaluation in conjunction with the development of the HEP model, which identifies habitat values at potential sites to quantify their potential for improvement. The identification of sites was undertaken collaboratively with the study team who provided input on desired locations for restoration and restoration objectives which could be accomplished. This screening resulted in the identification of a number of sites, which were ultimately screened down to 18 sites. This screening was based upon the site's ability to contribute to an identified restoration objective and advance the restoration of coastal processes, as well as their potential to contribute to storm damage reduction. These sites and the development of the restoration measures at these sites are described in detail in the Environmental Appendix.

7.5.2 Breach Response Measures

The secondary screening of breach-response measures focused on identifying barrier island areas with a higher breaching risk and investigating the costs associated with various breach response timeframes.

Although breach closure may be required at any location along the barrier islands fronting Great South Bay, Moriches Bay, and Shinnecock Bay, a few specific areas where breaching risk is significantly higher were identified to serve as the basis for the screening of breach response measures. These selected areas are those where a breach or partial breach was observed in the storm surge modeling simulations (USACE, 2005). Table 7-3 lists the specific locations where a breach, and therefore a breach closure, would be more likely. The full extent of the breaching potential at each of these locations is described in Section 4.0.

Breach stability analysis indicated a tendency for new breaches in the project area to remain open and possibly cause increased shoaling of existing inlets. To evaluate damages and closure construction costs attendant with a given breach, it was necessary to estimate the cross-sectional area of the breach with time. Survey data for the 1980 and 1992 breaches at Cupsogue and Pikes Beach, respectively, were used to estimate breach growth characteristics. An exponential equation that assumed breach cross-sectional area is asymptotic in time to a long-term stable value was fit to the data. The exponential breach growth is consistent with the physical nature of barrier island breaches. Breach cross-sectional area typically stabilizes as the scouring potential associated with tidal flow velocities balances forces attempting to close the breach. As tidal flow velocities decrease with increasing breach area, the rate of breach growth is initially rapid and slowly approaches an equilibrium condition.

Table 7-3. Likely Breach Locations

Location	Design Reach		Federal Tract
FI Lighthouse Tract	GSB-1B	FI Lighthouse Tract	Yes-Major
Robins Rest	GSB-2B	Town Beach to Corneille Estates	Yes-Small & adjacent to developed areas
Barrett Beach	GSB-3D	Talisman to Water Island	Yes-Major
Davis Park	GSB-3G	Davis Park	No
Old Inlet West	GSB-4B	Old Inlet	Yes-Major
Old Inlet East	GSB-4B	Old Inlet	Yes-Major
Smith Point CP	MB-1B	Smith Point CP – East	No
Sedge Island	SB-1B	Sedge Island	No
Tiana Beach	SB-1C	Tiana Beach	No
West of Shinnecock	SB-2B	WOSI	No

Note: based on Baseline (circa 2000) conditions

For this screening analysis, costs associated with closure delays of up to one year were considered (45 days, 3, 6, 9, and 12 months). This screening analysis shows that at all potential breach locations, it is more cost effective to close a breach immediately than to delay closure for 9 or 12 months. Immediate closure was recommended for further evaluated under the Phase 2 Alternative Assessment. As part of this analysis, consideration was also given for variations in the design cross-section, and the implementation criteria, such as a trigger point where action is taken.

7.5.3 Beach Restoration

The initial screening of measures recommended consideration of beachfill across the entire project area. In order to determine the appropriate spatial extent for consideration, the beachfill alternatives were developed further to be able to identify the relative degree to which infrastructure is at risk, as compared with a typical beachfill cost.

The secondary screening of beach restoration measures focused on identifying specific project reaches where beach fill could be economically justified. For areas along the barrier islands, there was no straight-forward assessment tool to evaluate damages, since along the barrier island there are also benefits that are derived from maintaining a stable barrier island conditions, which have to be considered when determining the viability of these areas.

Conceptual beach fill cost estimates were developed for each project reach using a typical beach fill template (90 ft wide berm and 15 ft NGVD dune). Costs are presented in terms of dollars per foot of beach restored in Table 7-4. Expected annual damages by reach were compared to these typical beachfill costs. This analysis was used to eliminate areas where the expected damages clearly would not support a beachfill alternative. The results of this analysis demonstrate that the beachfill alternatives in the majority of areas east of Shinnecock Inlet are not economically viable. Areas east of Shinnecock Inlet where beachfill is still considered include the areas with the greatest potential for damage per linear foot of project reach, which includes the areas of Downtown Montauk and in the vicinity of Georgica Pond. In the remainder of the areas, fill is not considered, but non-structural alternatives will be advanced.

It was recommended that beachfill be considered along the barrier island reaches, and evaluated further in the areas of Georgica Pond and Downtown Montauk.

Table 7-4. Approximate Beachfill Cost by Project Reach

Project Reach	Name	Annualized Cost per ft
GSB	Great South Bay	$260/ft
MB	Moriches Bay	$165/ft
SB	Shinnecock Bay	$520/ft
P	Ponds	$655/ft
M	Montauk	$510/ft

7.5.4 Sediment Management (including Inlet Modifications)

The secondary screening of sediment management considered a number of inlet modification alternatives, including dredging of inlet shoals and channels, excavating updrift deposits with placement downdrift, and other structural modifications to aid natural bypassing and reduce downdrift erosion (spur jetties, T-groins, etc.) The goal of the inlet modification alternatives was to develop alternatives that provide reliable navigation through the Federal navigation channels and maximize sand bypassing in order to restore, to the extent possible, natural sediment pathways and reduce adjacent shoreline erosion. Inlets are a complex, and dynamic system. <u>History has shown that modifications at inlets can result in unintended, negative secondary effects. For this reason, when conducting this alternative analysis, preference was given to alternatives that can achieve the objectives with a minimal amount of change, have a low risk, and are readily reversible or adaptable</u>.

This alternative analysis was conducted in an interagency setting, with input from members of a Coastal Technical Management Group (CTMG) which included representatives of NYS-DEC, NYS-DOS, and DOI (National Park Service). This group first brainstormed an initial concept list of inlet modifications alternatives and screening criteria. This initial list recorded all measures, regardless of consistency with USACE policies/authorization or the policies of any of the other agencies/sponsors represented in the meeting. More importantly, some of the alternatives discussed at this meeting do not qualify as complete inlet modification plans to the extent that they do not necessarily address all of the project needs as listed above.

At a subsequent CTMG meeting, the preliminary alternative screening analysis were presented and revised based upon agency input to arrive at a concept list of alternative inlet modification plans, the screening criteria, and screening methodology. Alternatives that were clearly inadvisable or included negative effects that could not be offset by any degree of benefits from other factors were eliminated. The following were considered to be fatal flaws:

- Not meeting all of the stated needs
- Exacerbating shoreline erosion
- Increasing barrier island breaching potential
- Significant uncertainty at a high cost
- Jeopardizing endangered species
- Significant inconsistency with applicable laws and regulations
- A similar, more effective option, is available

The following tables present the alternatives that were recommended for consideration in the detailed screening analysis.

Table 7-5. Preliminary List of Modification Alternatives Shinnecock Inlet	
1.	Authorized Project[7] plus Offshore Dredging
2.	Authorized Project plus Dredging the Flood Shoal
3.	Channel & Deposition Basin Realignment along the “Natural” Channel Thalweg
4.	Relocation of the Deposition Basin (Not Channel)
5.	Reduced Dimensions of Deposition Basin
6.	Authorized Project plus Dredging the Ebb Shoal (outside limits of Deposition Basin) with a Floating Plan
7.	Semi-fixed Bypass System
8.	Truck/Trailer Mounted System
9.	Authorized Project plus Spur Jetty (West)
10.	Authorized Project plus Shortening the East Jetty
11.	Change Distance between Inlet Jetties
12.	Authorized Project plus Nearshore Structures along West Beach
13.	Sand Trapping and Bypassing System Updrift
14.	Authorized Project plus Dredging the Ponquogue Ebb Shoal Attachment
15.	Authorized Project plus Relocation of the Maritime Center within Shinnecock Bay.

[7] The design capacity of the existing deposition basin is approximately 350,000 cubic meters, and the anticipated dredging interval was 1.5 years (USACE-NAN, 1988). Since 1990, however, the deposition basin has been dredged approximately every 4 years. This larger than anticipated interval is at least partly due smaller than expected sediment accumulation in the deposition basin.

Table 7-6. Preliminary List of Modification Alternatives Moriches Inlet	
1.	Authorized Project[8]
2.	Authorized Project plus Dredging the Flood Shoal.
3.	Channel & Deposition Basin Realignment.
4.	Relocation of the Deposition Basin (Not Channel)
5.	Increased Dimensions of Deposition Basin
6.	Dredging the Ebb Shoal (outside limits of Deposition Basin) with a Floating Plant
7.	Semi-fixed Bypass System
8.	Truck/Trailer Mounted System
9.	Authorized Project plus Extension the West Jetty
10.	Sand Trapping and Bypassing System Updrift
11.	Reduce Authorized Channel Depth

Table 7-7. Preliminary List of Modification Alternatives Fire Island Inlet	
1.	Existing Practice[9] (Dredging of Deposition Basin & Channel)
2.	Existing Practice plus Discharge farther West
3.	Optimize Existing Channel and Deposition Basin Configurations
4.	Eastern Realignment of Channel and Deposition Basin
5.	Existing Practice plus Dredging the Flood Shoal
6.	Dredging the Ebb Shoal (outside limits of Deposition Basin) with a Floating Plant
7.	Semi-fixed Bypass System
8.	Existing Practice plus Extension of the East Jetty
9.	Reconfiguration of the Sore Thumb (and Channel Realignment)
10.	Sand Trapping and Bypassing System Updrift
11.	Groins East of the Inlet
12.	Move the Inlet back to the Lighthouse Location

Screening of alternatives for each of the three inlets requires the careful balancing of multiple and, sometimes conflicting, criteria. For this study, an alternative selection decision matrix based on Multiple Criteria Decision Analysis (MCDA) principles was used as a screening tool. The matrix evaluates each of the alternatives based on their performance with regard to several

[8] The authorized project (USACE-NAN, 1982) calls for a "seasonal" channel and deposition basin maintenance schedule with an equivalent rate of 75,000 m^3/yr (98,000 cy/yr). The GDM suggests that dredging take place in the spring, so that depths of less than -10 feet MLW would only occur during the winter months when traffic through the inlet is minimal. Observed bottom changes after the 1996 and 1998 dredging events seem to support the expected design shoaling rates. However, actual dredging has only been performed every 4 years or more since project authorization.

[9] Channel and deposition basin are currently dredged approximately every two years resulting in approximately 279,000 m^3/yr (365,000 cy/yr), 80% of which are placed at downdrift at Gilgo Beach and 20% (depending on the need) are placed updrift within Robert Moses State Park.

criteria. In addition, the method weights the resulting overall values according to how well each alternative performs with regard to the stated project needs. Briefly, an overall value or score for each alternative was computed based on the following two basic scores:

Performance Score: How well the alternative meets the stated needs (accounting for risk & uncertainty inherent to each alterative and their expected performance), and
Total Criteria Score: How beneficial (or adverse) is each alternative with regard to a specific set of criteria.

Five general *Criteria Categories* with equal weight were defined: Environmental, Economical, Recreational, Engineering, and Cultural/Social, each including specific individual criteria. A single weighted average score for each Criteria Category was computed for each alternative based on the raw scores for each specific criteria (e.g., cost). The scoring process was based on a "qualitative value scale" method which assesses the performance of alternatives by reference to descriptive pointers (i.e., word descriptions) to which appropriate values are assigned. The *Performance Score* is computed based on how well each alternative meets the stated needs and how much risk & uncertainty is associated with the alternative with regards to those needs (measured in terms of percentage).

Specific screening criteria were reduced to a reasonable number that would adequately describe the pros and cons of each alternative by reflecting its impacts on the most relevant environmental, economic, recreational, engineering, social, and cultural conditions in the study area. At this level of the screening process, a concise but representative list of criteria allows for a more objective grading of the different alternatives because it does not unfairly weight very specific issues that happen to be included in the analysis, while neglecting other issues, which may be as important, but were forgotten or intentionally left out. It also minimizes the possibility of "double counting" the effects on certain issues that might otherwise be included under several different criteria.

Another important consideration in developing screening criteria was to ensure that screening process would account for relevant New York State Coastal Management Program (CMP) Policies (NYSDOS, 2002). The final list of criteria is shown in the following table:

Screening results are shown in the following tables. Note that the rankings reflect the recent findings with regard to coastal processes at the inlets (e.g., ebb shoal growth) and the sediment budgets. More importantly, although the resulting ranking depends on a relatively subjective assessment (as is always the case in this type of analysis), developing criteria and assigning scores does bring to focus each alterative and the associated pros and cons. More importantly, <u>the results of this screening were applied to identify alternatives that should be eliminated from further consideration, and also to identify the top alternatives that should be carried forward for more detailed investigations. This screening was not used to select only the top ranked alternative at each inlet.</u>

Table 7-8 Screening Criteria – Inlet Modifications
Environmental Criteria
1. Fish and Wildlife
2. Rare and Endangered Species
3. Water Quality
4. Tidal and Freshwater Wetlands
5. Sediment Pathways
6. Non-Structural Components
Economic Criteria
7. Lifecycle Costs
8. Flooding Risk
9. Commercial Fisheries
10. Waterfront Development and Commercial Fishing
11. Land Use and Ownership
Recreational Criteria
12. Recreational Fish and Wildlife Resources
13. Water and Foreshore Related Recreation
Engineering Criteria
14. Capacity
15. Source Flexibility
16. Placement Flexibility
17. Continuity
18. Performance
19. Reversibility
Cultural and Social Criteria
20. Historic, Cultural, and Scenic Resources
21. Local Concerns and Public Relations

Table 7-9 Screening Matrix Results – Shinnecock Inlet			
	Alternative Plan Description	**TOTAL SCORE (Max 1,000)**	**RANKING (out of 17)**
6	Authorized Project plus Dredging the Ebb Shoal	512	1
12	Authorized Project plus Nearshore Structures along West Beach	440	2
1	Authorized Project plus Offshore Dredging	429	3
7	Semi-fixed Bypass System (plus “reduced” Authorized Project)	385	4
5	Reduced Dimensions of Deposition Basin	378	5
4	Relocation of the Deposition Basin (Not Channel)	358	6
14	Authorized Project plus Dredging the Ponquogue Attachment	346	7
2	Authorized Project plus Dredging the Flood Shoal	342	8
10	Authorized Project plus Shortening the East Jetty	333	9
13	C. Offshore Breakwater	332	10
8	Truck/Trailer Mounted System (plus “reduced” Authorized Project)	328	11
15	Authorized Project plus Relocation of the Maritime Center	323	12
9	Authorized Project plus Spur Jetty (West)	306	13
13	B. Weir Jetty and Sediment Trap	301	14
3	Channel & Deposition Basin Realignment	290	15
13	A. Jetty Opening and Nearshore Breakwater	253	16
11	Change Distance between Inlet Jetties	189	17

Table 7-10 Screening Matrix Results – Moriches Inlet			
	Alternative Plan Description	**TOTAL SCORE (Max 1,000)**	**RANKING (out of 13)**
6	Authorized Project plus Dredging the Ebb Shoal	532	1
7	Semi-fixed Bypass System (plus Authorized Project)	449	2
5	Increased Dimensions of Deposition Basin	408	3
4	Relocation of the Deposition Basin (Not Channel)	408	4
3	Channel & Deposition Basin Realignment	404	5
2	Authorized Project plus Dredging the Flood Shoal	401	6
11	Reduced Authorized Channel Depth	399	7
10	C. Offshore Breakwater	387	8
8	Truck/Trailer Mounted System (plus Authorized Practice)	384	9
1	Authorized Project	384	10
10	B. Weir Jetty and Sediment Trap	338	11
10	A. Jetty Opening and Nearshore Breakwater	285	12
9	Authorized Project plus Extension of the West Jetty	274	13

Table 7-11 Screening Matrix Results – Fire Island Inlet			
	Alternative Plan Description	**TOTAL SCORE (Max 1,000)**	**RANKING (out of 13)**
6	Existing Practice plus Dredging the Ebb Shoal	483	1
4	Eastern Realignment of Channel and Deposition Basin	429	2
3	Optimize Existing Channel & Deposition Basin Configurations	419	3
1	Existing Practice	413	4
2	Existing Practice plus Discharge Farther West	397	5
7	Semi-fixed Bypass System (plus “reduced” Existing Practice)	378	6
5	Existing Practice plus Dredging the Flood Shoal	347	7
10	C. Offshore Breakwater	328	8
8	Existing Practice plus Extension of the East Jetty	314	9
11	Groins East of the Inlet (plus Existing Practice)	301	10
10	B. Weir Jetty and Sediment Trap	276	11
12	Move the Inlet Back to the Lighthouse	245	12
10	A. Jetty Opening and Nearshore Breakwater	233	13
9	Reconfiguration of the Sore Thumb (and Channel Realignment)	208	14

The secondary screening results presented in the tables above were considered in combination with additional input from New York State suggesting that more emphasis be placed on alternatives that may provide more continuous bypassing (e.g., using semi-fixed bypassing plant or shortening the east jetty at Shinnecock Inlet). The following alternative inlet management measures were selected for further development in the Phase 2, First Added Assessment of Alternative Measures.

Shinnecock Inlet

Alt. 1: Authorized Project (AP) + Dredging the Ebb Shoal
Alt. 2: AP + Nearshore Structures
Alt. 3: AP + Offshore Dredging
Alt. 4: AP + Semi-fixed Bypass System
Alt. 5: AP with Reduced Dimensions of Deposition Basin
Alt. 6: AP + Dredging the Flood Shoal
Alt. 7: AP + Shortening the East Jetty
Alt. 8: AP + West Jetty Spur

Moriches Inlet

Alt. 1: Authorized Project (AP) + Dredging the Ebb Shoal
Alt. 2: AP + Dredging the Ebb Shoal
Alt. 3: AP + Semi-fixed Bypass System
Alt. 4: AP + Dredging the Flood Shoal

Fire Island Inlet

Alt. 1: Existing Practice/ Authorized Project (AP)
Alt. 2: AP + Dredging the Ebb Shoal
Alt. 3: AP + Optimized Deposition Basin
Alt. 4: AP + Dredging the Flood Shoal

The further development of these alternative measures is presented in the detailed screening.

7.5.5 Removal/Modification of Groins

Initial screening recommended further evaluation of the existing groins within the study area to consider the effectiveness of groin removal or modification, including shortening, tapering, or notching. The purpose is to reduce or eliminate interruptions in longshore sediment transport and restore natural sediment movement. The total number of structures that could be classified as groins in the project area is 26, not including jetties and drainage outfalls. Existing groins are located in the Towns of Easthampton and Southampton (8), at Westhampton Beach (16) and along Fire Island (2).

To evaluate the effect of groin removal or modification, this screening applied a conceptual level analysis on the costs and benefits of groin removal compared to beach nourishment. For this conceptual screening, only the complete removal of the groins was examined.

A complete investigation into the feasibility or impacts of groin removal would require (1) historical shoreline and volumetric changes east and west of the structures before and after construction, (2) the contribution of the groins toward any irregularities in the existing beach layout, and (3) the groin impacts determined by the implementation of a shoreline change model. It is also important to determine if existing coastal storm damage risk reduction would be adversely affected in areas where groin removal would occur.

Evaluation of groin removal, in comparison with beachfill, shows that groin removal results in increased annualized costs with no readily identifiable benefit in terms of beachfill performance.

Total groin removal was not recommended for further consideration as an alternative, but modification of the existing groins was recommended for further consideration.

7.5.6 Beach Restoration with Structures

The secondary screening of beachfill alternatives identified locations where beachfill would be considered further, based upon the infrastructure at risk. Using these results, a secondary screening of structural measures was undertaken to identify if there are locations where structural measures would be warranted.

It is recognized that in areas where the rate of erosion is high, structural measures may be preferable as a means to reduce the long-term requirement for sand placement, and also as a means to provide more reliable storm damage reduction. As summarized in the initial screening, the structural measures considered include groins and breakwaters, both of which function to reduce storm losses in an area, and can reduce the need for long-term renourishment. Another structural measure recommended for further consideration was the "reinforced dune" that includes a stone revetment buried by a dune. This alternative can reduce the berm width required to provide a given level of shore protection, as compared to a beachfill alternative, thus reducing the amount of fill required.

The secondary screening of structural features was undertaken to look at the costs of the beachfill and structural alternatives, the erosion rates in the area, and the associated reliability of the storm damage reduction features. Based upon this information, alternatives were further screened to identify locations where structural measures would be beneficial to reduce long-term costs, and increase reliability. As explained above, structural alternatives work by either reducing erosion (groins and breakwaters) or increasing the shore protection (buried seawall). In the case of buried seawalls beachfill volume requirements were adjusted to account for the volume of the seawall itself and for the reduced beach berm. After comparing the costs of beach fill alone and beachfill plus seawalls it was evident that the seawall was not competitive for any of the design reaches.

However, in the case of the groins and offshore breakwaters, if the erosion rate is sufficiently high the increased first cost associated with construction of the structures may be offset by future savings in erosion reduction and increased reliability. A detailed analysis was conducted to determine the minimum erosion rate under which any of the structural alternatives would be cost effective. Costs included initial construction costs, renourishment costs, and emergency rehabilitation costs. A summary of results from this analysis are shown in Table 7-12**.** These results show that unless erosion rates are higher than 14 ft per year, groins are not cost-effective. For offshore breakwaters the required erosion rate is even greater. Only one design reach in the FIMP area, West of Shinnecock Inlet, has an average erosion rate of more than 10 ft/yr, roughly 25 ft/yr.

Table 7-12. Minimum Shoreline Erosion Rates for Structures to be Cost Effective (ft/yr)

	Design Level		
Structural Feature	**Small Design**	**Medium Design**	**Large Design**
Groins	14	16	18
Breakwaters	77	88	110

Based on these results it was concluded that the only location where structural measures appeared promising to reduce the long-term requirement for beachfill, and to provide more reliable shore protection is in the area immediately west of Shinnecock Inlet. The consideration of these measures were developed further as Shinnecock Inlet modification alternatives.

7.5.7 Mainland Road Raising

As described in the initial screening of alternatives, levee/floodwall measures were not recommended for further, comprehensive evaluation. Consideration was given to areas where road raising could serve as a localized coastal storm damage risk reduction measure.

For this secondary screening, road raising in selected mainland back bay residential areas was analyzed to explore if opportunities exist to reduce flooding risk to homes. Road raising is considered as a means to achieve storm damage reduction for a greater number of buildings at a reduced cost compared to individual-building nonstructural plans for a given area. In addition to reducing damage to structures, road raising can reduce outside physical costs such as the flooding of cars, and non-physical costs such as clean up and evacuation. Raised roads can also offer enhancements to local evacuation plans and public safety by reducing the risk of inundation of local roads within the area, and providing safer evacuation routes out of the area. Road raising may also be more acceptable to residents in some communities since it reduces the need for structural alterations to individual buildings that may disrupt the owners' lives and affect perceptions of property value.

Based on a review of topography, density of vulnerable structures, layout of residential streets, and environmental considerations such as the need to avoid wetland impacts, 24 potential road raising locations were identified. This list of locations was further refined to minimize the average length of road raising required per structure. Five areas have been selected for detailed analysis: Areas 4a, 8c, 8d/8e, 9b, and 52a. In these locations, it is likely that road raising would result in substantial cost savings compared to retrofit treatments.

Based upon this screening, road raising was recommended for consideration in discrete locations, in conjunction with the non-structural alternatives.

7.6 Conclusions, Alternative Measures Selection

In general, the following measures were recommended for further development in the Detailed Evaluation of Individual Storm Damage Reduction Measures. The specific recommendations include:

a) Breach Response Measures along the barrier island
b) Sediment Management, including Inlet Management Modifications
c) Non-Structural Retrofit Measures
d) Non-Structural Land and development management
e) Road Raising along the mainland
f) Beach Nourishment
g) Groin Modifications
h) Coastal Process Restoration Measures at locations throughout the Study Area

Breach Response Measures. Along the barrier island, there are locations which have been identified as vulnerable to breaching. At these locations, and at locations that may become vulnerable in the future, breach response plans were developed for further consideration. The development of these plans took into consideration the lessons learned from prior breach responses, and the analysis undertaken for the Breach Contingency Plan (USACE, 1995). The further design and development of breach response plans considered the design profile, implementing procedures (trigger for the action), and the need for lifecycle management of breach closures.

Sediment Management, including Inlet Modifications. As presented above, specific inlet modification alternatives were recommended for further examination at Shinnecock, Moriches and Fire Island Inlet to determine whether enhanced sand bypassing or modified inlet designs could potentially limit future storm damages and/or enhance the performance of plan alternatives. Opportunities for sediment management measures have also been considered further, in conjunction with the beachfill evaluation.

Non-Structural Retrofit Measures. Building Retrofit Measures will be considered at locations along the mainland back-bay area, and will consider the benefits and costs for various scales of coastal storm damage risk reduction.

Non-Structural, Land and Development Management. These measures were developed further to identify alternatives that could be implemented to address the existing land management challenges, and any additional challenges or opportunities that may increase in conjunction with the plan alternatives.

Road Raising along the mainland. Levees and floodwalls, because of their applicability to localized flooding problems, were recommended for further evaluation in the mainland areas of Project Reaches 1 to 3 as localized road raising measures, at four discrete locations identified above.

Beach Nourishment. As presented above, beachfill was considered further in locations where the without project damages indicate that a beachfill project could potentially be supported, based upon the level of damages. This includes the entire shoreline along Great South Bay, Moriches Bay and Shinnecock Bay. East of this area, evaluation of beachfill alternatives was limited to the areas of Georgica Pond and Downtown Montauk. Further evaluation of the beachfill plans considered variations in scale and alignment.

Groin Modifications. Groin modification alternatives were considered further at Ocean Beach, Westhampton, and Georgica. Complete groin removal was not be considered further.

Coastal Process Restoration Measures. These restoration features were developed further at locations throughout the Study Area to identify features that accomplish the NER objectives and can be integrated with the NED plan. These measures are discussed in further detail in the Environmental Appendix.

All other features (i.e., storm closure gates, coastal structures only) were eliminated from further consideration due to their failure to meet the objectives of the Reformulation Study.

7.7 Detailed Evaluation of Individual Storm Damage Reduction (SDR) Measures

7.7.1 Introduction

The evaluation of SDR alternatives was undertaken to develop each of the measures advanced from the Secondary Screening into a greater level of detail, and to provide for variations in the scale, and location of the project, to develop alternatives based upon specific design criteria. Each of these alternatives has been developed to include alternative descriptions, alternative plan layouts, alternative project costs, and alternative project benefits. This evaluation is focused on alternatives to accomplish the objective of storm damage reduction within the overall project evaluation criteria. In addition to addressing the effectiveness of the alternative in reducing storm damages, each alternative is also evaluated relative to how effective it is in meeting the objectives of the Vision, through the application of evaluation criteria.

The outcome of the evaluation of the individual SDR measures is the identification of alternatives that contribute to the overall project objectives and an assessment of whether these measures meet Corps implementation criteria. For storm damage reduction alternatives, these are alternatives that meet the requirements for providing net excess benefits.

In parallel with the evaluation of storm damage reduction alternatives were the development and evaluation of alternative measures to restore coastal processes. This is discussed in the Environmental Appendix.

Based on the Screening of Measures for the full array of storm damage reduction measures, the following types of storm damage reduction alternatives have been considered as appropriate for consideration for further development.

a) Breach Closure including Responsive and Proactive Breach Alternatives
b) Sediment management and Inlet Modifications
c) Non-Structural / Building Retrofits
d) Beachfill and Beachfill with Dunes
e) Groin Modifications
f) Land and Development Management

Each of these alternatives is described further in the following sections. These general measures have been developed further to provide alternatives of varying scales and of varying effectiveness in storm damage risk reduction. Cost estimates have been prepared for these alternatives, and each alternative has been evaluated relative to effectiveness in reducing storm damages and meeting the evaluation criteria.

The land and development management measures are described last in this chapter. This is done intentionally. Throughout the chapter, each alternative presents the land and development management challenges that may be created or increased, or opportunities that may arise for improved land and development management with the implementation of the alternative. These challenges and opportunities are used as a basis for introducing the land management and development management measures that may be available to address these challenges and opportunities.

7.7.2 Non-Structural Measures

General.

Non-Structural Measures, by definition are measures which seek to move the buildings being damaged, rather than redirecting the movement of water. As presented in the prior Chapter, a supplemental screening of non-structural alternatives was undertaken, which identified plans to be considered further, and whether they could be implemented as a part of a cost-shared project, or as an element of a locally implemented FPMP. This analysis looked at three types of non-structural alternatives: 1 – Land Management, 2 – Acquisition, and 3 – Building Retrofit.

This section focuses on Building Retrofits. (Land Management and Development Management are addressed later in this section). The screening of alternatives identified that opportunities exist for Federal participation in retrofit of structures, with a focus on the mainland, backbay shores.

Design

In order to evaluate these alternatives, an algorithm was applied to evaluate six non-structural approaches for individual buildings in the back bay mainland areas. The measures considered were wet flood proofing, dry flood proofing, elevation, acquisition, flood walls for individual buildings, and rebuilding. Five separate alternatives were considered to provide coastal storm damage risk reduction from flooding with a 1% annual chance of exceedance (plus freeboard) corresponding to the baseline-condition landward limits of the 2-, 6-, 10-, 25- and 100-year floodplains. After evaluating the measures for each building, the least-cost measure deemed technically feasible was selected. The four smaller alternatives were found to be cost-effective, while the 100-year floodplain alternative was determined to be cost-prohibitive and was screened out from further consideration.

Retrofitting

This evaluation focused on retrofitting techniques for buildings on the mainland, and not for barrier island structures. On the barrier island, elevation was determined unsuitable because of the difficulties in logistical and site access; transporting materials to the site is made more difficult by the lack of roads, and the limited lot space of many buildings prevents the use of standard cribbing and jacking techniques to elevate the building.

The following, six non-structural flood proofing alternatives were considered during the evaluation process.

Dry Flood Proofing. Dry Flood Proofing measures allow flood waters to reach the structure but diminish the flood threat by preventing the water from getting inside the structure walls. Dry Flood Proofing measures considered in this screening make the portion of a building that is below the flood level watertight through attaching watertight closures to the structure in doorway and window openings. Detached levees and floodwalls were not considered due to the density of structures in the floodplains.

Wet Flood Proofing: allowing flood water to enter lower, non-living space areas of the structure via vents and openings to reduce hydrostatic pressure and in turn reduce flood-related damages to the structure's foundation. This technique can be used along with the protection of utilities and other critical equipment, which can include permanently raising machinery, critical equipment, heating and cooling units, electrical outlets, switches, and panels and merchandise/stock above the estimated flood water height. It can also involve construction of interior or exterior floodwalls, utility rooms, or additional living space to compensate for space subject to flooding, and the use of flood resistant materials.

Elevation: raising the lowest finished floor of a building to a height above the design flood level. This option was considered both as a stand-alone measure and in conjunction with additional construction. In some cases, the structure is lifted in place and foundation walls are extended up to the new level of the lowest floor. In other cases, the structure is elevated on piers, posts, or piles;

Acquisition: removal of the structure from the floodplain through demolition. Lands are then preserved for open space uses;

Relocation: moving the structure out of the floodplain, either within the existing property boundary (if sufficient space is available) or to another property;

Rebuild: demolishing a flood-prone structure and replacing it with a new structure built to comply with local regulations regarding new construction and substantial improvements in a floodplain, and therefore is at a lower risk. The rebuild option was considered only where the costs were found to be less than those associated with an otherwise recommended treatment.

7.7.2.1 Cost Criteria

After evaluating a series of alternatives for each representative building, the least cost alternative was selected wherever possible. Wet flood proofing tended to be the least costly option, followed by dry flood proofing. In general, acquisition and relocation were the costliest alternatives, followed by elevation.

7.7.2.2 Assumptions Inherent to the Screening of Alternatives

Because this was an alternative comparison, there were a limited number of unit costs developed, and certain assumptions were made to expedite the analysis. Table 7-13 summarizes the assumptions that were made during the screening of non-structural alternatives for representative buildings.

7.7.2.3 Application to the Overall Floodplain, Generalized Design Criteria – The Flood Proofing Screening Algorithm

A flood proofing screening algorithm was used to screen alternatives for representative buildings. Alternatives were considered based on two conditions: one with flood levels above the main floor, and one with levels below the main floor. The screening process was conducted using the previously identified representative buildings, assumptions, and criteria. Using this process, the following non-structural alternatives were identified for the development of detailed unit costs (and inclusion into the flood proofing computer model). The relationships in the algorithm are illustrated in Table 7-14.

Separate from the five non-structural plans, relocation on the existing lot was considered for the back bay areas but was found to be infeasible because back bay land plots tend to be too small and flat to meet the criteria for relocation outside of the floodplain within the existing property boundaries.

Acquisition

Acquisition was also considered as an option for backbay structures, but was found to be generally cost-prohibitive due to high property values in the study area. However, Suffolk County has expressed an interest in pursuing structure acquisition as an option. USACE regulations require that for the purpose of estimating benefits and costs, acquisition costs be estimated under a flood-free condition, which requires extensive appraisals. Thus, for planning purposes only, acquisition costs have been computed as the sum of the depreciated structure replacement value plus a land cost of $100,000; an administrative cost of $30,000; and a demolition cost of $15,000. On completion of the algorithm, the recommended treatment cost was compared to the acquisition cost and acquisition was identified as the preferred treatment if it was found to be the lowest cost alternative. Under these conditions, land costs were found to preclude most potential acquisition candidates from being recommended for this treatment.

A reevaluation of the acquisition option could be applied in a combined NED/NER approach, whereby acquired land could be considered for environmental restoration. Building acquisition instead of elevation is also an option in the few mainland areas designated as “V” or “high velocity” zones on the FEMA Flood Insurance Rate Maps. There are approximately 290 V-zone buildings currently proposed for elevation under the 100-year protection plan. To acquire these structures would increase the plan cost by approximately $72 million dollars, and thus is not likely to be cost-effective over elevation.

Results

Table 7-15 presents the first cost of construction for alternatives Nonstructural 1 through 4 (also called NS-1, NS-2, NS-3, and NS-4). Costs for the baseline 100-year plan (which was determined to be cost-prohibitive) are included for comparison purposes only.

Table 7-13. Assumptions inherent to the screening of back bay alternatives for representative buildings.

General Assumptions	• Flood velocity is negligible. • Debris impacts will not be considered. • There are limited areas designated as "V-Zone" by FEMA, subject to 3-foot breaking waves. The majority of back bay areas are considered non-V-Zone and thus not subject to wave and erosion impacts. • All buildings selected for treatment will be protected to the 100-year level, plus one 1 foot of freeboard. • Buildings elevated in non-coastal areas will be raised (finished floor elevation) to the 100-year water surface plus 1 foot of freeboard. • Flooding is gradual (no flash flooding).
Foundation Walls	• All basement foundation types are assumed to be unreinforced, 8" concrete masonry units (CMUs).
Raised Structures (Crawlspace)	• No utilities are located in the crawlspace. • Wet flood proofing of raised structures includes the elevation of utilities only, and where necessary, the installation of vents or louvers to allow adequate venting.
Slab-On-Grade Structures	• Wet flood proofing is possible if the expected flood elevation is below the main floor (shallow flooding). This alternative includes the elevation of utilities only. • Consistent with Corps' flood proofing guidance, structures will not be dry flood proofed for flooding depths greater than 2 feet plus one foot of freeboard for a maximum 3 feet of dry flood proofing (See Attachment 1 for supporting calculations).
Structures With Basements	• All basements are unfinished and contain major utilities.
Bi-Levels	• The lower portion of the first floor walls are masonry construction. • The foundation is slab-on-grade. • The main floor can be raised separately from the lower level by lifting off the sill of the masonry wall.
Raised Ranches	• The first floor (lower) walls are masonry. • The foundation is slab-on-grade. • The main floor can be raised separately from the lower level (similar to a structure with a basement).
Split-Levels	• The lower level is slab-on-grade. • The lower portion of the lower level walls are masonry construction. • The main floor level is raised over a crawl space. • The main floor and upper level can be separated from the lower level by raising at the sill.

Table 7-14. Flood-proofing alternatives identified for back bay unit cost estimating.

Typical Structure Type	Flood Level	Protection* Level Condition 1	Protection* Level Condition 2	Flood Proofing Alternative
Slab-On-Grade	>= Main Floor	Protection Level – Ground < 3	n/a	Sealant & Closures
		Protection Level – Ground >= 3	n/a	Elevate Building
	< Main Floor	< Main Floor	n/a	Raise AC
		>= Main Floor	Protection Level – Ground < 3	Sealant & Closures
			Protection Level – Ground >= 3	Elevate Building
Basement-Subgrade	>= Main Floor	n/a	n/a	Elevate Building
	< Main Floor	< Main Floor		Fill Basement + Utility Room
		>= Main Floor	n/a	Elevate Building
Raised (Crawlspace)	>= Main Floor	n/a	n/a	Elevate Building
	< Main Floor	< Main Floor	n/a	Raise AC + Louvers
		>= Main Floor	n/a	Elevate Building
Basement-Walkout	>= Main Floor	n/a	n/a	Elevate Building
	< Main Floor	< Main Floor	Protection Level – Ground < 3	Interior Floodwall
			Protection Level – Ground >= 3	Raise Lower Floor + Space
		>= Main Floor	n/a	Elevate Building
Bi-Level/Raised Ranch	>= Main Floor	n/a	n/a	Elevate Building
	< Main Floor	< Main Floor	Protection Level – Ground <= 3	Sealant & Closures
			Protection Level – Ground >3	Raise Lower Floor + Space
		>= Main Floor	n/a	Elevate Building
Split Level	>= Main Floor	n/a	n/a	Elevate Building
	< Main Floor	< Main Floor	Protection Level – Ground < 3	Sealant & Closures
			Protection Level – Ground >=3	Elevate Building
		>= Main Floor	n/a	Elevate Building

* For purposes of Non-Structural Measures, the term “protection” refers to storm damage risk reduction, not absolute protection from damage.

Table 7-15. Comparison of Alternative Non-Structural First Costs

Project Reach	Econ. Reach	Number of Buildings, Reach Total	Design Water Elevation*	2yr Water Elevation	Number of Buildings, 2yr Plan	First Cost, 2yr Plan	6yr Water Elevation	Number of Buildings, 6yr Plan	First Cost, 6yr Plan	10yr Water Elevation	Number of Buildings, 10yr Plan	First Cost, 10yr Plan	25yr Water Elevation	Number of Buildings, 25yr Plan	First Cost, 25yr Plan	100yr Water Elevation
(Quogue to	8b	119	10.26	4.71	0	$0	5.15	0	$0	5.62	0	$0	6.65	3	549,500	9.26
	10.1	39	9.19	4.76	2	$170,500	5.24	8	$2,153,000	5.71	8	$2,153,000	6.55	18	$3,518,000	8.19
	10.2	6	9.46	4.91	2	$200,000	5.43	2	$220,000	5.92	2	$220,000	6.73	2	$200,000	8.46
	10.3	204	10.06	4.89	18	$2,998,000	5.31	29	$4,833,500	5.84	29	$4,833,500	6.87	51	$7,091,500	9.06
	10.4	260	10.26	4.71	12	$1,530,000	5.15	31	$4,360,500	5.62	31	$4,360,500	6.65	55	$6,723,000	9.26
	11.1	281	9.91	4.87	8	$923,000	5.54	28	$3,833,500	6.03	71	$9,389,500	6.95	71	$9,049,500	8.91
	11.2	626	9.70	4.78	3	$358,000	5.45	27	$3,741,500	5.93	27	$3,741,500	6.85	85	$13,553,000	8.70
	12	786	9.39	4.95	4	$541,500	5.53	19	$2,876,500	6.16	73	$13,529,000	7.19	140	$22,181,500	8.39
	13.1	297	9.67	5.02	48	$7,500,000	5.89	48	$8,417,000	6.64	94	$15,874,000	7.67	118	$16,927,500	8.67
	13.2	588	9.67	5.02	47	$7,069,000	5.89	47	$7,874,000	6.64	109	$18,097,500	7.67	138	$19,606,000	8.67
	Subtotal	*3,206*			*144*	*$21,290,000*		*239*	*$38,309,500*		*444*	*$72,198,500*		*681*	*$99,399,500*	
Bay (Smith	16.1	137	8.21	4.22	3	$367,500	4.85	3	$404,500	5.24	6	$906,000	5.87	6	$795,000	7.21
	16.2	318	8.27	4.13	62	$10,859,000	4.68	64	$10,943,000	5.07	85	$14,861,500	5.70	85	$15,044,500	7.27
	16.3	432	8.44	4.09	46	$8,461,500	4.65	46	$8,346,500	5.06	65	$11,040,000	5.75	65	$11,021,000	7.44
	16.4	611	8.44	4.09	66	$12,106,000	4.65	66	$11,985,000	5.06	116	$21,484,000	5.75	116	$21,842,000	7.44
	17.1	226	7.76	4.26	31	$8,540,000	4.96	31	$9,129,000	5.35	46	$10,644,000	6.01	77	$17,294,000	6.76
	17.2	94	8.21	4.22	0	$0	4.85	0	$0	5.24	1	$113,500	5.87	1	$113,000	7.21
	18.1	3,070	7.94	3.91	140	$18,116,000	4.70	356	$46,507,500	5.30	543	$66,688,500	6.10	924	$82,689,000	6.94
	18.2	208	8.47	4.22	16	$1,722,500	5.07	25	$3,252,000	5.85	25	$3,252,000	6.66	41	$4,438,500	7.47
	18.3	1,343	8.49	4.24	124	$16,865,500	5.11	194	$29,781,000	5.75	194	$29,781,000	6.57	329	$62,346,000	7.49
	Subtotal	*6,439*			*488*	*$77,038,000*		*785*	*$120,348,500*		*1,081*	*$158,770,500*		*1,644*	*$215,583,000*	
South Bay	20	571	6.71	3.15	0	$0	4.02	30	$2,607,500	4.44	30	$2,607,500	5.01	80	$5,474,500	5.71
	21.1	517	6.29	3.10	4	$463,000	4.23	48	$5,492,000	4.51	74	$8,438,000	4.88	81	$9,136,500	5.29
	21.2	1,641	6.29	3.10	24	$4,803,500	4.23	168	$30,232,000	4.51	203	$34,391,500	4.88	223	$36,508,500	5.29
	21.3	755	6.29	3.10	0	$0	4.23	9	$1,960,000	4.51	19	$4,438,500	4.88	21	$6,930,500	5.29
	21.4	747	6.37	3.20	9	$1,970,500	4.02	78	$9,267,500	4.36	79	$9,376,000	4.83	79	$8,471,000	5.37
	21.5	225	6.37	3.20	1	$130,000	4.02	5	$664,000	4.36	6	$754,500	4.83	13	$1,263,000	5.37
	21.6	428	6.65	3.22	13	$1,457,500	3.89	13	$1,611,500	4.18	50	$6,566,000	4.82	50	$5,879,000	5.65
	22.1	1,961	6.30	3.21	156	$18,626,000	4.34	474	$58,724,000	4.61	491	$60,712,500	4.93	495	$54,373,500	5.30
	22.2	2,095	6.20	3.19	38	$4,545,000	4.31	163	$22,450,500	4.54	196	$26,750,500	4.85	214	$27,815,500	5.20
	23.1	364	5.48	3.09	1	$95,500	3.74	1	$118,500	3.97	1	$118,500	4.22	12	$684,500	4.48
	23.2	1,746	5.48	3.09	59	$6,312,000	3.74	101	$12,231,000	3.97	122	$15,471,000	4.22	311	$27,682,500	4.48
	23.3	2,985	5.46	3.14	21	$1,871,000	3.64	30	$3,094,000	3.89	31	$3,241,000	4.18	166	$8,687,500	4.46
	24	3,175	6.07	3.28	16	$2,056,500	3.80	22	$2,649,500	4.02	158	$19,113,000	4.48	189	$20,839,000	5.07
	25.1	1,960	6.56	3.37	6	$802,000	4.45	135	$11,242,500	4.71	138	$11,484,000	5.07	262	$17,718,000	5.56
	25.2	2,413	6.07	3.28	40	$8,141,500	3.80	42	$7,761,500	4.02	494	$48,298,000	4.48	507	$45,380,000	5.07
	26.1	1,715	7.69	3.95	23	$2,860,000	5.00	370	$42,486,500	5.36	371	$42,504,000	5.96	405	$41,352,000	6.69
	26.2	4,703	6.56	3.37	17	$1,963,500	4.45	282	$22,306,000	4.71	313	$23,473,500	5.07	704	$40,586,000	5.56
	26.3	2,323	6.56	3.37	17	$2,246,000	4.45	416	$41,886,500	4.71	416	$41,886,500	5.07	779	$63,293,500	5.56
	Subtotal	*30,324*			*445*	*$58,343,500*		*2,387*	*$276,785,000*		*3,192*	*$359,624,500*		*4,591*	*$422,075,000*	
Reaches		**39,969**			**1,077**	**$156,671,500**		**3,411**	**$435,443,000**		**4,717**	**$590,593,500**		**6,916**	**$737,057,500**	

1) *Note: Design Water Elevation is 100-yr water elevation + 1 Foot freeboard
(For structures in V Zones, Design Water Elevation is listed elevation + 4 feet)

2) 100-year plan (Baseline condition) was determined to be cost-ineffective and is included for comparison purposes only. These costs have not been updated to October 2007 price level.

Evaluation of Storm Damage Reduction Effectiveness.

The reduction in storm damages arising from retrofit treatments or other actions applied directly to individual structures was modeled using the Lifecycle Damage Analysis Model, with the stage-damage relationships in each reach modified to reflect the application of the nonstructural methodology described in earlier sections. The four nonstructural alternatives analyzed were based on applying nonstructural measures to back bay mainland structures in the baseline 2-year, 6-year, 10-year, and 25-year floodplains. This protection corresponds to nonstructural plans NS-1, NS-2, NS-3, and NS-4 respectively. Table 7-16 presents a summary of the number of buildings affected by each plan, by Reach.

Table 7-16 –Structures Where Nonstructural Alternatives Reduce Risk of Damages

Planning Unit	Nonstructural 1	Nonstructural 2	Nonstructural 3	Nonstructural 4
Great South Bay	445	2,387	3,192	44,591
Moriches Bay	488	785	1,081	1,644
Shinnecock Bay	144	239	444	681
Project Total	*1,077*	*3,411*	*4,717*	*6,916*

These non-structural alternatives are implemented on a volunteer basis. For evaluation purposes, the benefits and costs are shown for all structures which fall within the footprint of the non-structural plan. This represents the maximum reduction in damages associated with this project alternative. The ability to achieve this reduction however, depends upon the extent of participation in the program.

Table 7-17 presents the modeled annual damages resulting from the implementation of the four nonstructural alternatives.

These damages have been compared with those associated with the without-project condition to generate the nonstructural project benefits, which are presented in Table 7-18. As shown in the table, these plans reduce the storm damages to flood-prone structures in the mainland back bay areas, but do not reduce damages on the barrier islands or in mainland shorefront areas. Although they appear not to address damages arising due to barrier island breaching, mainland inundation damages caused by breaching would be reduced somewhat by nonstructural plans.

Table 7-17. Annual Damages: Nonstructural Alternatives

Damage Category	Nonstructural 1	Nonstructural 2	Nonstructural 3	Nonstructural 4
Total Project				
Tidal Inundation				
Mainland	$52,392,700	$36,102,000	$29,230,500	$22,880,500
Barrier	$12,998,600	$12,998,600	$12,998,600	$12,998,600
Total Inundation	$65,391,300	$49,100,900	$42,229,100	$35,879,100
Breach				
Inundation	$9,242,500	$9,242,500	$9,242,500	$9,242,500
Structure Failure	$395,700	$395,700	$395,700	$395,700
Total Breach	*$9,638,200*	*$9,638,200*	*$9,638,200*	*$9,638,200*
Shorefront	$7,388,900	$7,388,900	$7,388,900	$7,388,900
Public Emergency				
Other				
Total Storm Damage	$82,418,400	$66,128,000	$59,256,200	$52,906,200

Interest Rate 5.125%, Project Life 50 years

Table 7-18. Annual Benefits: Nonstructural Alternatives

Benefit Category	Nonstructural 1	Nonstructural 2	Nonstructural 3	Nonstructural 4
Total Project				
Tidal Inundation				
Mainland	$21,842,800	$38,133,300	$45,005,000	$51,355,000
Barrier	0	0	0	0
Total Inundation	*$21,842,800*	*$38,133,300*	$45,005,000	*$51,355,000*
Breach				
Inundation	$0	$0	$0	$0
Structure Failure	$0	$0	$0	$0
Total Breach				
Shorefront Damage	$0	$0	$0	$0
Public Emergency				
Other				
Total Storm Damage Reduction				
Costs Avoided	$0	$0	$0	$0
Breach Closure	$0	$0	$0	$0
Beach Maintenance				
Other				
Land Loss				
Total Benefits	***$21,842,800***	***$38,133,300***	**$45,005,000**	***$51,355,000***

Interest Rate 5.125%, Project Life 50 years

The costs associated with the application of nonstructural treatments and actions are presented in Table 7-18. The total investment costs include contingencies, and allowances for Engineering and Design, Supervision and Administration, and temporary accommodation for the occupants of structures undergoing significant nonstructural treatments. The total investment costs also reflect opportunity costs associated with interest during construction.

Table 7-19. Annual Costs: Nonstructural Alternatives

Cost Category	Nonstructural 1	Nonstructural 2	Nonstructural 3	Nonstructural 4
Total Project				
Total First Cost	$156,671,500	$435,443,000	$590,593,500	$737,058,000
Total IDC	$3,142,368	$13,817,329	$18,734,435	$15,208,000
Total Investment Cost	*$159,813,900*	*$449,260,329*	*$609,327,935*	*$752,266,000*
Interest and Amortization	$8,923,700	$25,085,829	$34,023,695	$42,005,100
Operation & Maintenance	$0	$0	$0	$0
BCP Maintenance	$0	$0	$0	$0
Monitoring	$0	$0	$0	$0
Renourishment	$0	$0	$0	$0
Total Budgeted Cost	$8,923,700	$25,085,829	$34,023,695	$42,005,100
Annual Breach Closure Cost	$1,372,884	$1,372,884	$1,372,884	$1,372,884
Major Rehabilitation				
Total Additional Cost	$1,372,884	$1,372,884	$1,372,884	$1,372,884
Total Annual Cost	***$10,296,600***	***$26,458,713***	***$35,396,579***	***$43,378,000***

Interest Rate 5.125%, Project Life 50 years

Table 7-20, which presents the net benefits and benefit-cost ratios of the four nonstructural alternatives shows that all the alternatives analyzed are cost-effective in reducing storm damage. Nonstructural Alternative 2 appears to provide the greatest storm damage reduction benefits in excess of cost. A closer inspection of the results shows that the differences in net excess benefits between nonstructural 2 and 3 is very small, and alternative 3 provides significantly greater coastal storm damage risk reduction to a larger number of structures. The difference in the design criteria for these 2 alternatives is also very small, generally less than 0.5 ft difference in the storm surge height). This small difference is difficult to resolve with the accuracy of the existing data. Given this small difference in design criteria, and the relatively small difference in net excess benefits between these alternatives, both Nonstructural Alternative 2 and 3 have been identified as the plans that maximize net excess benefits, and are recommended for consideration in combination with other alternatives.

Table 7-20. Net Benefits and BCRs: Nonstructural Alternatives

	Nonstructural 1	Nonstructural 2	Nonstructural 3	Nonstructural 4
Total Project				
Total Annual Cost	$9,106,258	$26,458,713	$35,396,579	$37,814,205
Total Benefits	$21,842,762	$38,133,250	$45,005,002	$51,354,953
Net Benefits	$12,736,503	$11,674,536	$9,608,423	$13,540,748
Benefit-Cost Ratio	**2.40**	**1.44**	**1.27**	**1.36**
Great South Bay				
Total Annual Cost	$3,763,342	$16,824,750	$21,597,012	$21,770,091
Total Benefits	$7,779,888	$21,015,677	$24,846,235	$28,375,917
Net Benefits	$4,016,545	$4,190,927	$3,249,222	$6,605,827
Benefit-Cost Ratio	**2.07**	**1.25**	**1.15**	**1.30**
Moriches Bay				
Total Annual Cost	$4,086,723	$11,304,862	$9,333,069	$10,862,206
Total Benefits	$8,983,402	$10,989,258	$12,434,091	$14,327,878
Net Benefits	$4,896,679	-$315,605	$3,101,022	$3,465,672
Benefit-Cost Ratio	**2.20**	**0.97**	**1.33**	**1.32**
Shinnecock Bay				
Total Annual Cost	$1,213,068	$2,344,561	$4,267,127	$5,035,052
Total Benefits	$5,079,472	$6,128,315	$7,724,677	$8,651,157
Net Benefits	$3,866,405	$3,783,754	$3,457,549	$3,616,105
Benefit-Cost Ratio	**4.19**	**2.61**	**1.81**	**1.72**

Interest Rate 5.125%, Project Life 50 years

<u>Nonstructural/Raised Road Alternatives</u>

Road raising in selected mainland back bay residential areas was analyzed to explore whether it could achieve storm damage reduction for a greater number of buildings at a reduced cost compared to individual-building nonstructural coastal storm damage risk reduction plans for a given area. In addition to reducing damage to structures, road raising can reduce outside physical costs such as the flooding of cars, and non-physical costs such as clean up and evacuation. Raised roads can also offer enhancements to local evacuation plans and public safety by reducing the risk of inundation of local roads within an area, and providing safer evacuation routes out of the area. Road raising may also be more acceptable to residents in some communities since it reduces the need for structural alterations to individual buildings that may disrupt the owners' lives and affect perceptions of property value.

Based on a review of topography, the density of vulnerable structures, the layout of residential streets, and environmental considerations such as the need to avoid wetland impacts, 24 potential road raising locations were identified. This list of locations was further refined based on minimizing the average length of road raising required to reduce the risk of inundation. Five areas were consequently selected for detailed analysis: Areas 4a, 8c, 8d/8e, 9b, and 52a. An earlier stage of this study demonstrated that road raising in these areas would result in substantial cost savings compared to retrofit treatments. A more detailed process to optimize the crest elevations in these areas has since been completed, incorporating revised back bay stage-frequency relationships.

The optimization process examined crest elevations ranging from +5.25' to +7.5' (NGVD 29) for the various areas, and determined that road-raising is not cost effective for area 9b. The process identified +7' as the optimum road crest elevation for four remaining areas. This elevation would reduce the risk of damages due to still water flooding from storms with greater than a 1% annual chance exceedance in the future condition. In each of the four areas, crest elevations lower than +7' would also result in positive net benefits and could be implemented as components of a federal project. Theoretically, there are additional benefits to be gained from a slightly higher crest elevation in some areas; however, +7' has been judged to be the highest acceptable elevation for all four sites, since higher elevations would cause problems with the roadway side slopes encroaching further onto adjacent properties, and would necessitate excessive gradients on many adjoining residential driveways.

The four areas feasible for road-raising are shown in Table 7-21, which summarizes the road raising alternatives and compares the number of buildings protected by each alternative to the number of buildings protected by the nonstructural alternatives for the same area.

Table 7-21. Road Raising Areas

Area #	Town	Community	Approx. Length of Raised Road (Ft)	Structures Protected [1]	Nonstructural Treatments In Same Area[2]	Total First Cost[3]
4a	Babylon	Amityville	6,600	97	24	$2,541,000
8c	Babylon	Lindenhurst	5,300	240	42	$3,038,000
8d8e	Babylon	Lindenhurst	9,000	362	16	$4,829,000
52a	Brookhaven	Mastic Beach	10,500	355	234	$3,950,000

1. Structures enclosed by raised road and high ground with ground elevations below the raised road crest.
2. Nonstructural Plan 3.
3. Includes contingency, Engineering & Design, Supervision & Administration

Evaluation of SDR Effectiveness

The reduction in storm damages resulting from alternatives featuring a combination of nonstructural treatments and road raising in selected areas were analyzed using the Lifecycle Damage Analysis Model, with the stage-damage relationships in each reach modified to reflect the application of the nonstructural algorithm. Two combined nonstructural/road raising alternatives were analyzed, which represent the optimized raised road elevation nonstructural plans 2 and 3.

Table 7-22 presents the modeled annual damages resulting from the implementation of the two combined alternatives.

Table 7-22. Annual Damages: Nonstructural/Road Raising Alternatives

Damage Category	Nonstructural 2R	Nonstructural 3R
Total Project		
Tidal Inundation		
Mainland	$33,604,600	$27,110,300
Barrier	$12,998,600	$12,998,600
Total Inundation	*$46,603,200*	*$40,108,900*
Breach		
Inundation	$9,242,500	$9,242,500
Structure Failure	$395,700	$395,700
Total Breach	*$9,638,200*	*$9,638,200*
Shorefront	$7,388,900	$7,388,900
Public Emergency		
Other		
Total Storm Damage	***$63,630,300***	***$57,136,000***

Interest Rate 5.125%, Project Life 50 years

These damages have been compared with those associated with the without-project condition to generate the project benefits, which are presented in Table 7-23.

Table 7-23. Annual Benefits: Nonstructural/Road Raising Alternatives

Benefit Category	Nonstructural 2R	Nonstructural 3R
Total Project		
Tidal Inundation		
Mainland	$38,133,300	$45,005,000
Barrier	$0	$0
Total Inundation	$38,133,300	$45,005,000
Breach		
Inundation	$0	$0
Structure Failure	$0	$0
Total Breach	$0	$0
Shorefront Damage	$0	$0
Public Emergency		
Other		
Total Storm Damage Reduction	$38,133,300	$45,005,000
Costs Avoided		
Breach Closure	*0*	*0*
Beach Maintenance	*0*	*0*
Other		
Land Loss		
Total Benefits	$38,133,300	$45,005,000

Interest Rate 5.125%, Project Life 50 years

The costs associated with the plans combining the application of nonstructural treatments and actions and raised roads in selected areas are presented in TTable 7-24able 7-23. The total investment costs include contingencies, and allowances for Engineering and Design, Supervision and Administration, and temporary accommodation for the occupants of structures undergoing significant nonstructural treatments. The total investment costs also reflect opportunity costs associated with interest during construction.

Table 7-24. Annual Costs: Nonstructural/Road Raising Alternatives

Cost Category	Nonstructural 2R	Nonstructural 3R
Total Project		
Total First Cost	$422,029,000	$570,923,000
Total IDC	$13,291,800	$17,997,000
Total Investment Cost	*$435,320,800*	*$588,920,000*
Interest and Amortization	$24,307,500	$32,884,200
Operation & Maintenance		
BCP Maintenance		
Monitoring		
Renourishment		
Total Budgeted Cost	*$24,307,500*	*$32,884,200*
Annual Breach Closure Cost	$1,372,900	$1,372,900
Major Rehabilitation		
Total Additional Cost	*$1,358,040*	*$1,372,900*
Total Annual Cost	***$25,680,400***	***$34,257,100***

Interest Rate 5.125%, Project Life 50 years

Analysis of the two nonstructural/raised road alternatives shows that both the alternatives analyzed are cost-effective in reducing storm damage. Similar to the nonstructural evaluation, Nonstructural Alternatives 2R and 3R provide benefits in excess of cost. Although these plans did not consider road raising in combination with NS-1 and NS-4, it would be expected that road raising would be viable in combination with those measures.

TaTable 7-25ble 7-24, which presents the net benefits and benefit-cost ratios of the two nonstructural/raised road alternatives shows that both the alternatives analyzed are cost-effective in reducing storm damage. Similar to the nonstructural evaluation, Nonstructural Alternative 2R provides the greatest storm damage reduction benefits in excess of cost. However, it is important to note that because Nonstructural Alternative 3R is so close in design criteria and net benefits, it is effectively equal to Nonstructural Alternative 2R.

Table 7-25. Net Benefits and BCRs: Nonstructural/Road Raising Alternatives

	Nonstructural 2R	Nonstructural 3R
Total Project		
Total Annual Cost	$25,680,356	$34,257,036
Total Benefits	$39,742,523	$46,236,821
Net Benefits	$14,062,167	$11,979,785
Benefit-Cost Ratio	**1.55**	**1.35**
Great South Bay		
Total Annual Cost	$16,773,108	$21,784,819
Total Benefits	$22,099,368	$25,940,603
Net Benefits	$5,326,259	$4,155,783
Benefit-Cost Ratio	**1.32**	**1.19**
Moriches Bay		
Total Annual Cost	$6,438,522	$8,027,349
Total Benefits	$11,514,841	$12,571,542
Net Benefits	$5,076,319	$4,544,192
Benefit-Cost Ratio	**1.79**	**1.57**
Shinnecock Bay		
Total Annual Cost	$2,344,561	$4,267,127
Total Benefits	$6,128,315	$7,724,677
Net Benefits	$3,783,754	$3,457,549
Benefit-Cost Ratio	**2.61**	**1.81**

Interest Rate 5.125%, Project Life 50 years

Compatibility of Restoration Measures

There are several types of restoration measures that are compatible with the non-structural, retrofit alternatives. Given that these alternatives have been developed for the mainland floodplain area, there is limited geographic overlap with the restoration measures that focus on barrier island habitats. Non-structural measures, however, offer the opportunity for habitat restoration in instances where there are opportunities to restore the land in conjunction with an acquisition or relocation plan. As discussed above, the cost of acquisition is significantly higher than the cost of retrofit. These additional costs would have to be borne by the restoration.

Evaluation of Non-Structural Measures

NED Criteria. The analysis above shows that non-structural alternatives, and non-structural in combination with road raising are cost-effective storm damage reduction alternatives that contribute to reducing the damages primarily associated with flooding along the mainland backbay areas, independent of a barrier island breaching.

P&G Criteria. This is the evaluation of the alternatives relative to being complete, effective, efficient, and implementable. Mainland retrofit plans alone do not represent a complete solution, as they only address the damages that arise due to the relatively frequent flooding of the mainland. Relative to the purpose they are accomplishing, these alternatives are effective and efficient. These alternatives are also implementable, and generally supported by all parties.

Vision Criteria. Non-Structural measures were evaluated in relationship to the planning criteria developed to reflect the Project objectives and the project approach delineated in the "Vision Statement for the Reformulation Study". This systematic assessment ensures that the Vision Statement approach is fully integrated into the development and selection of the FIMP Plan. Table 7-26 provides a summary of the evaluation of these measures relative to the established criteria.

Land and Development Management Challenges and Opportunities.
The non-structural plans do not introduce land use and development management challenges, but instead introduce additional land use and development management opportunities that could be considered in conjunction with these alternatives. If there is a local desire for land acquisition rather than retrofit alternatives, these alternatives could consider if the additional costs for acquisition would be warranted to provide restoration of habitat to the underlying area.

Table 7-26. Non-Structural Retrofit Alternatives		
Evaluation Criteria	**Assessment**	**Rating**
The plan or measure provides identifiable reductions in risk from future storm damage.	Reductions in storm damage to the specific structures and contents are quantifiable.	Full
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science considered a lower priority.	Retrofits are a standard method for flood mitigation. Some individual structures may present design challenges, requiring a comparatively large cost contingency.	Full
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	The measures reduce physical impacts of flooding from the various sources for a limited number of structures. They do not address general floodplain impacts such as traffic delays, damage to cars and other physical property outside of the living areas, or non-physical costs such as flood evacuation or cleanup.	Full
The plan or measures incorporate appropriate non-structural features provide both coastal storm damage risk reduction and to restore coastal processes and ecosystem integrity	The non-structural features are specific to storm damage reduction.	Partial
The plan or measure help protect and restore coastal landforms and natural habitat.	The measures have no direct impact. Indirectly they may reduce the need for structural features.	No
The plan avoids or minimizes adverse environmental impacts	The plan minimizes environmental impacts.	Full
The plan addresses long-term demands for public resources.	There is no long term public involvement beyond monitoring to ensure that the use of the structure is consistent with any restrictions.	Full
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	NA	No
The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures	NA	No
The plan or measure incorporates appropriate alterations of inlet stabilization measures and dredging practices	NA	No
The plan or measure is efficient and represents a cost effective use of resources	Measures are cost efficient when targeted to frequently flooded structures.	Full
The plan or measure reduces risks to public safety.	Measures reduce damage only. It is important to maintain evacuation plans so that residents do not remain in homes that are inaccessible during a flood event.	No

Summary of Non-Structural Alternatives.

The analysis above shows that non-structural alternatives are cost-effective storm damage reduction alternatives that contribute to reducing the damages primarily associated with flooding along the mainland backbay areas, independent of a barrier island breach.

Non-Structural alternatives are recommended for further evaluation with alternatives NS-2, and NS-3, in conjunction with the road raising alternatives, which maximize net benefits.

The mainland non-structural alternative partially fulfill the vision objectives.

.

7.7.3 Breach Response Alternatives

7.7.3.1 General

Breach Response Alternatives are plans to be implemented either in response to the occurrence of a breach (reactive breach response), to close a breach quickly, or in response to conditions where a breach is imminent (proactive breach response). The variables accounted for in the design and evaluation of alternatives include: 1) the design cross-section, 2) the implementing method (reactive or proactive), and 3) the lifecycle maintenance of the alternative.

7.7.3.2 Design

Although the breach closure alternatives can be implemented at any location along the barrier islands fronting Great South Bay, Moriches Bay, and Shinnecock Bay, a few specific areas, where breaching risk is higher based on the baseline conditions, erosion rates and the future vulnerable condition estimates, were selected to serve as the basis for development of the Breach Response Alternatives. These selected areas are those where a breach or partial breach was observed in the baseline and future vulnerable conditions storm surge modeling simulations. Ten (10) vulnerable breach locations (based on 2000 LIDAR survey) were identified as shown in Figure 7-1.

Three breach closure cross-sections have been considered for these locations. The smallest breach closure template is a berm with height of +9.5 ft NGVD. The elevation of the berm was determined by the analysis of the relationship between overwash frequency and a range of potential breach closure section elevations. The analysis showed that a breach closure section of +9.0 ft NGVD will overwash several times per year, while a closure section of +10 ft NGVD would overwash once a year. Since the intent of the closure is to fill a breach, a specific berm width has not been established. Instead the intent is to generally match the berm width conditions prior to the breach and within adjacent areas. The design foreshore slope (from the seaward edge of the berm to MHW) is 1 on 12 which is also the same slope defined for the beach fill design templates. The design profile below MHW would match the representative morphological profile corresponding to each specific location. Bayside slopes would generally match the preexisting adjacent shorelines (this is a design element that can be altered as a restoration feature). Based on the existing topography the bayside design slope was selected as 1 on 20 from the bayside crest of the berm to an elevation of +6 ft NGVD. Two larger breach closure templates have been developed, to reduce the potential for rebreaching. These plans are similar to the first, but with an additional volume of sand in the shape of a trapezoidal dune at elevations +11' NGVD and +13'NGVD, respectively.

The typical cross-sections are illustrated in Figure 7-2 for Old Inlet West and West of Shinnecock Inlet. The typical breach closure plan layouts at Old inlet West and West of Shinnecock are shown in Figure 7-3 and Figure 7-4.

The total cross-shore cross sectional area for each template at each breach closure location is summarized in Table 7-27. A breach at Davis Park would have the largest cross sectional fill requirement, while a breach at WOSI would have the smallest. It should be noted, however, that the total volume requirement is based upon the combination of breach width (which varies over time) and design template area. A large area at Davis Park does not necessarily require the largest breach closure volume, since it is dependent upon growth rate, and time to closure.

Table 7-27. Breach Closure Plan Design Template Cross Sectional Area (sq. ft.)

	No Dune	+11 ft NGVD Dune	+13 ft NGVD Dune
FI Lighthouse Tract	9,811	9,860	9,960
Town Beach to Corneille Estates	12,918	12,967	13,067
Talisman to Water Island	15,367	15,416	15,516
Davis Park	15,389	15,438	15,839
Old Inlet West	14,727	14,776	14,876
Old Inlet East	12,327	12,376	12,476
Smith Point County Park	13,927	13,976	14,076
Sedge Island	14,127	14,176	14,276
Tiana Beach	13,327	13,376	13,476
WOSI	7,324	7,373	7,473

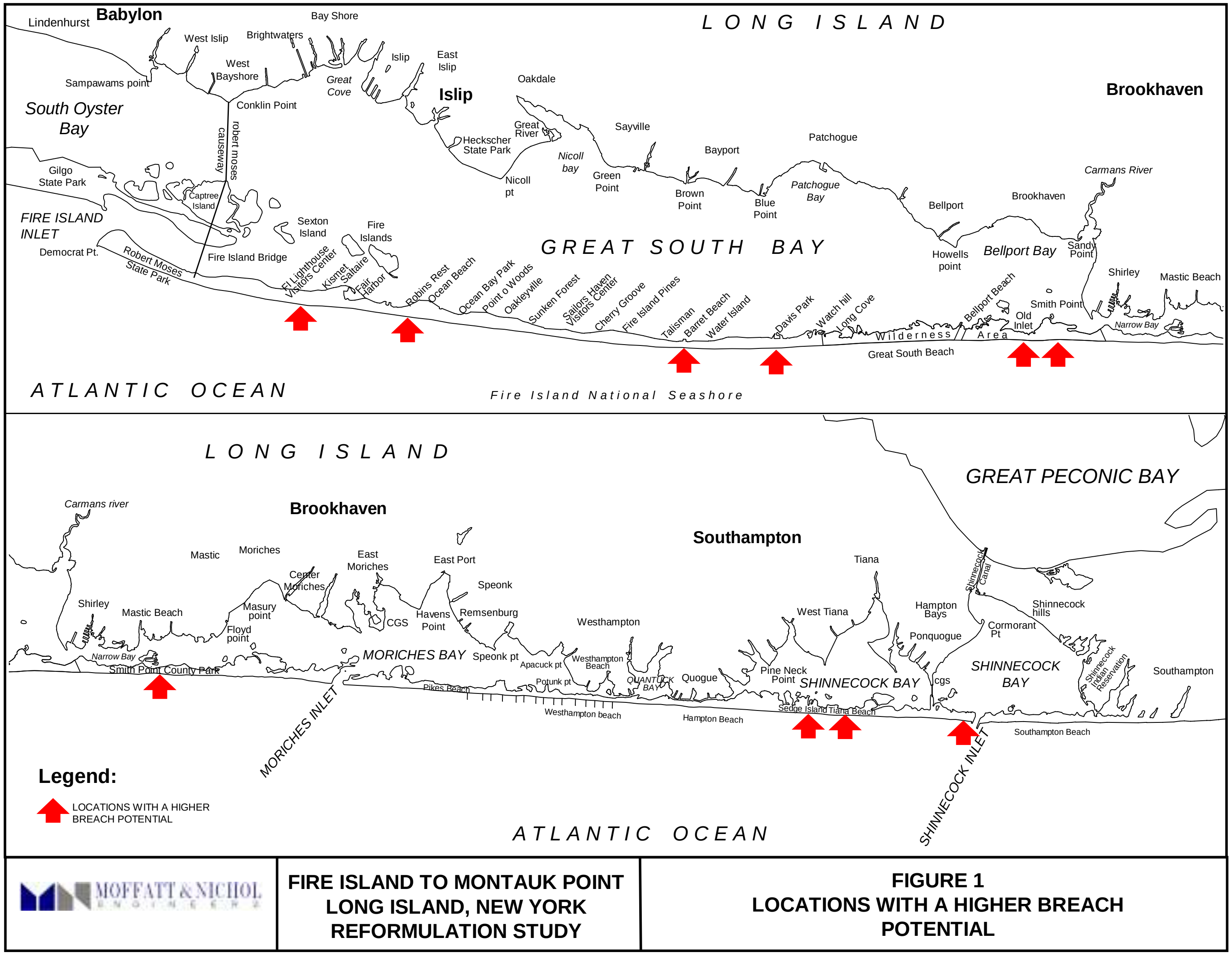


Figure 7-1. Vulnerable Breach Locations

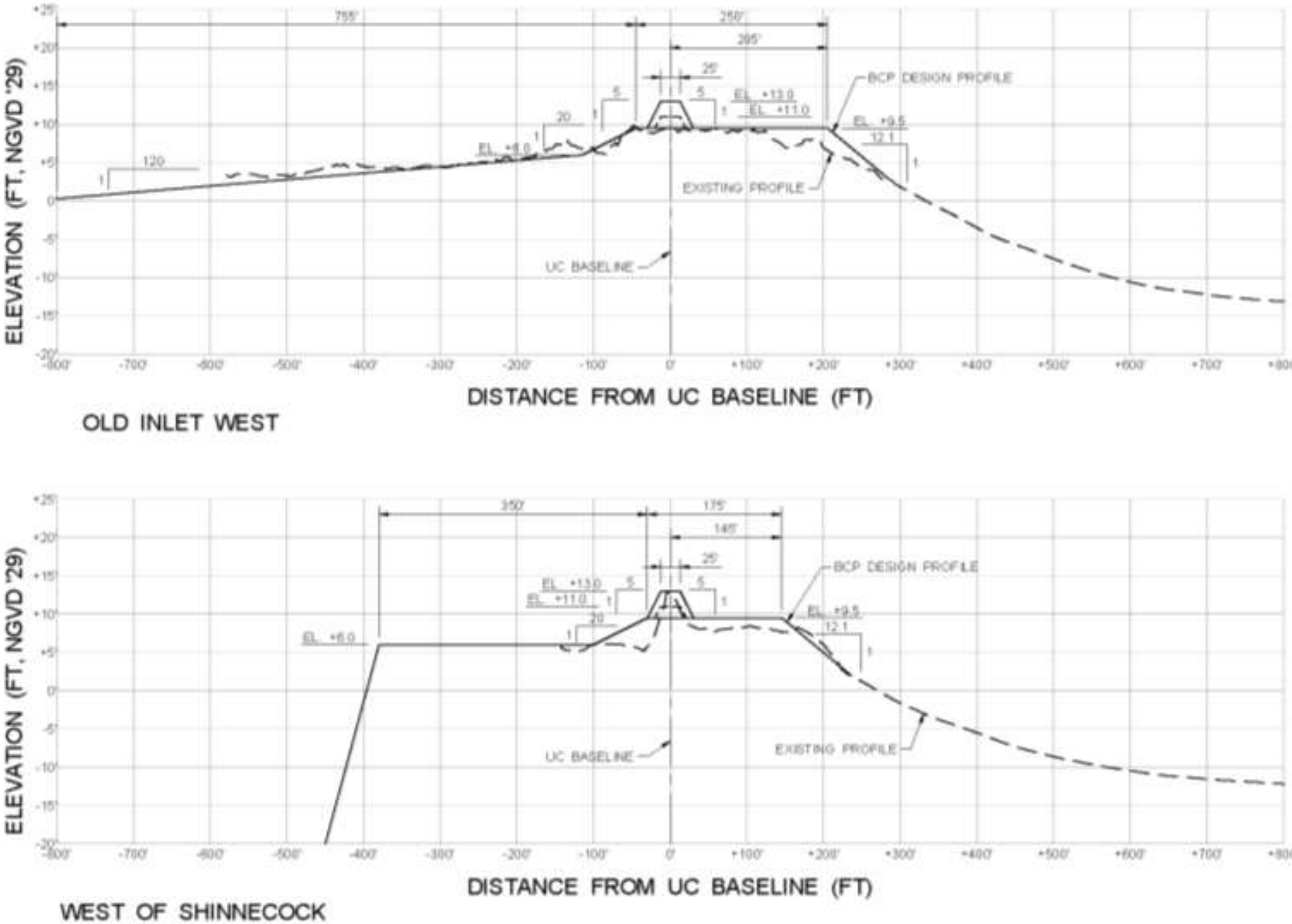


Figure 7-2. Typical Breach Closure Sections

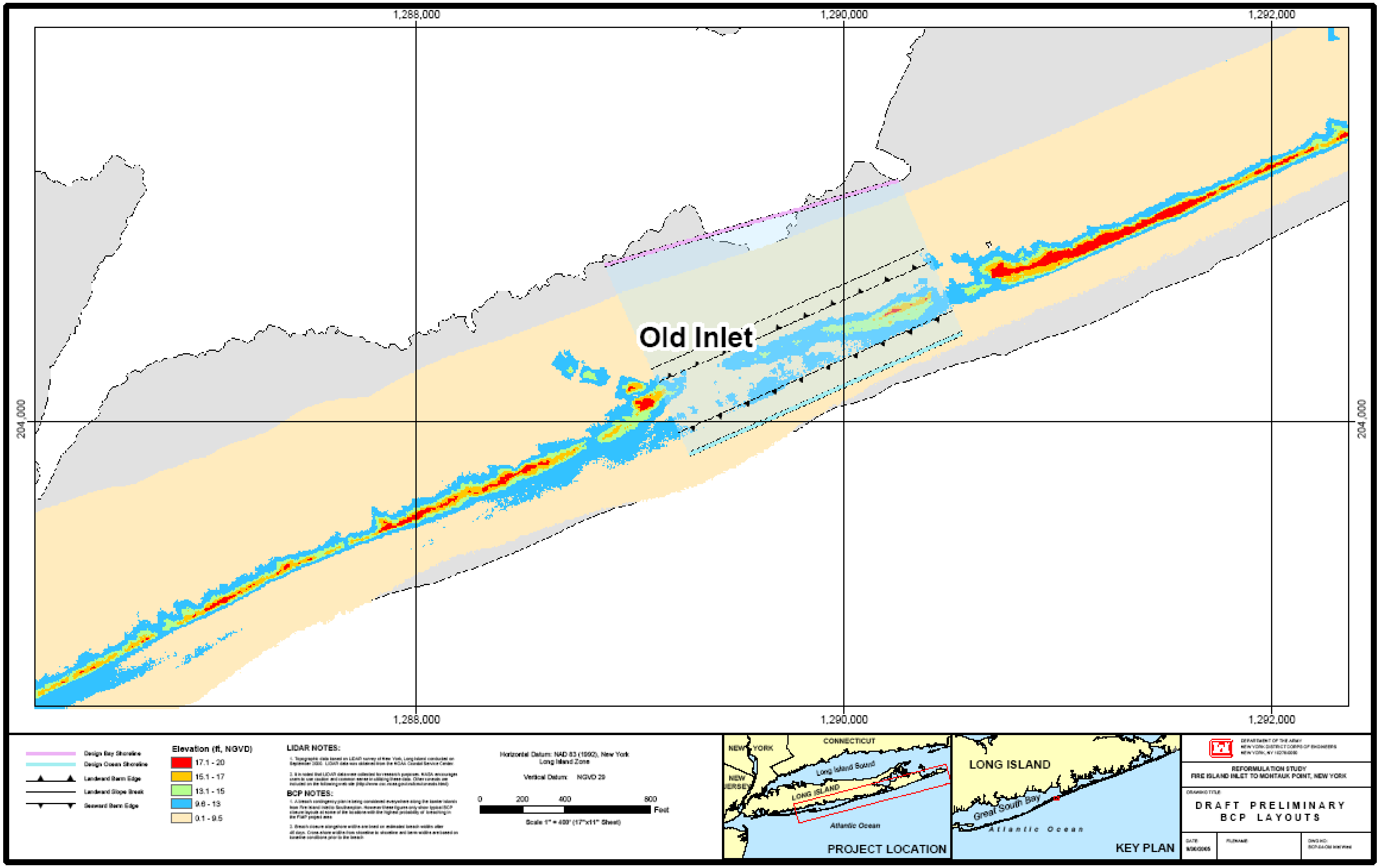


Figure 7-3. Typical Breach Closure Plan Layout at Old Inlet West

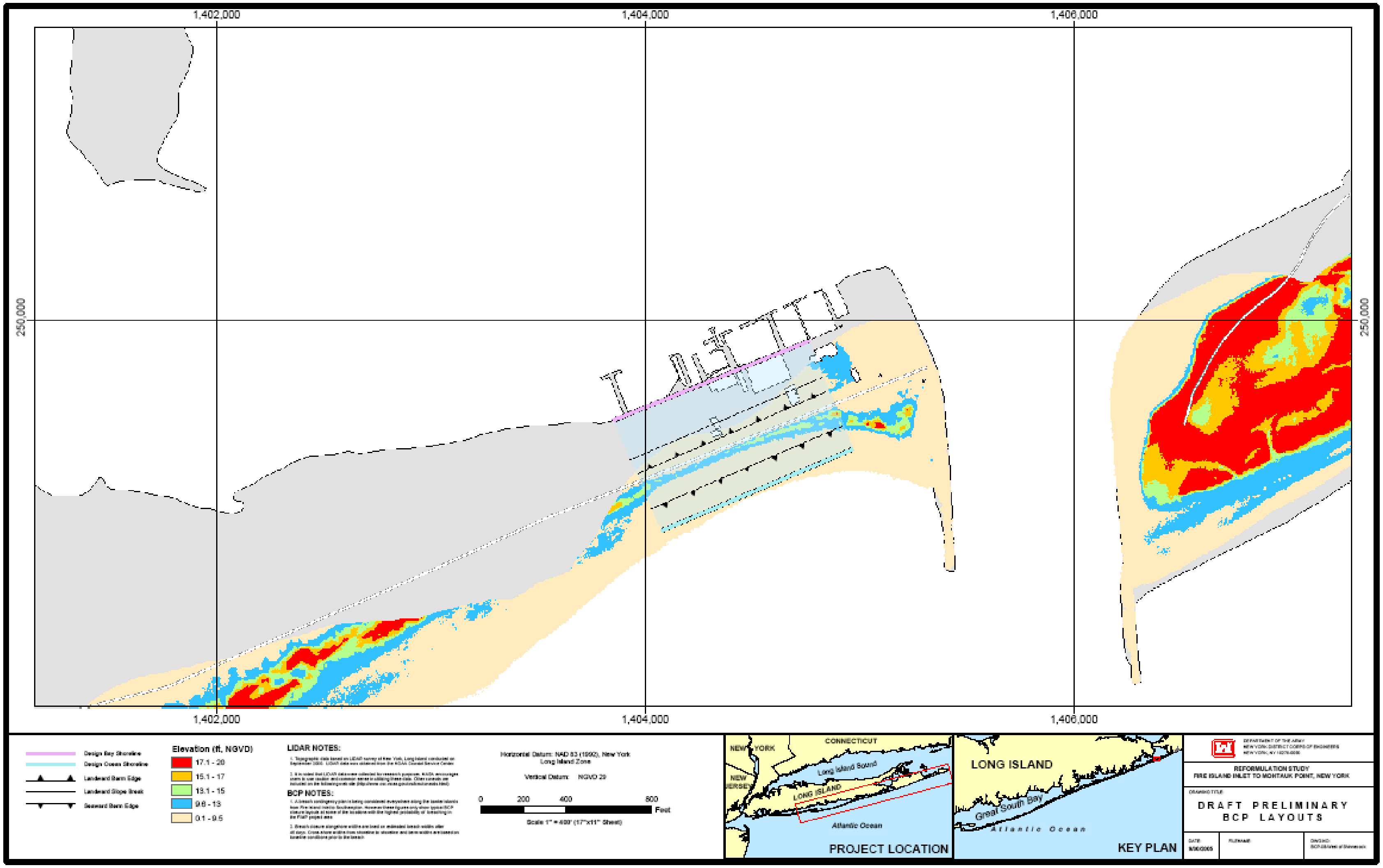


Figure 7-4. Typical Breach Closure Plan Layout at WOSI

7.7.3.3 Costs

In identifying the closure costs, a number of scenarios were evaluated, considering time to closure, volume of material required, mobilization costs, and the cost per CY for material placement. For some of the locations, the construction alternative that resulted in the smallest total breach width or lowest fill volume did not result in the least expensive closure cost. Table 7-28 presents the lowest cost breach closure construction alternative and the cost for each design template. It should also be noted that stockpile trucking is never part of a cost effective alternative; dredging-only closure options were the most cost effective.

Table 7-28. Breach Closure Cost by BCP Location and Design Template

	Construction Alternative Resulting in Lowest Total Cost	Total Project Cost		
		No Dune	+11 Dune	+13 Dune
FI Lighthouse Tract	Hopper Dredge	$11,157,000	$11,187,000	$11,249,000
Town Beach to Corneille Estates	Cutterhead Dredge	$ 9,591,000	$ 9,614,000	$ 9,663,000
Talisman to Water Island	Cutterhead Dredge	$ 6,676,000	$ 6,690,000	$ 6,717,000
Davis Park	Cutterhead Dredge	$ 6,682,000	$ 6,696,000	$ 6,723,000
Old Inlet West	Cutterhead Dredge	$ 6,826,000	$ 6,843,000	$ 6,876,000
Old Inlet East	Cutterhead Dredge	$ 7,629,000	$ 7,645,000	$ 7,679,000
Smith Point County Park	Hopper Dredge	$ 7,546,000	$7,561,000	$ 7,592,000
Sedge Island	Cutterhead Dredge	$ 6,645,000	$ 6,654,000	$ 6,672,000
Tiana Beach	Cutterhead Dredge	$ 6,495,000	$ 6,504,000	$ 6,523,000
WOSI	Hopper Dredge	$ 6,192,000	$ 6,209,000	$ 6,245,000

7.7.3.4 Lifecycle Maintenance

In the development of breach response alternatives, continued maintenance of the breach closure template was included, subsequent to a breach closure to maintain the protection afforded by the closure section, without waiting for another breach.

Since maintenance of the post-closure profile was assumed to be a component of each Breach Response Alternative, the lifecycle simulation models also evaluated the annualized costs of actions to restore the profile to the design section. The analyses allowed the post-closure profile at each location to degrade over time, and then implement restoration activities when certain conditions have been reached.

The Decision tree for the Breach Response Maintenance is shown in Figure 7-5 Decision tree for Breach Response Maintenance. As shown in the figure, the primary conditions that trigger restoration of the design profile were partial breaches and significant overwash events. For overwash events, restoration is also dependent on the required fill volume meeting minimum threshold volumes required to justify the mobilization of the appropriate equipment used for restoration. The equipment used was assumed to be dependent on the accessibility of each location: fill material was transported to site by trucks at those locations accessible by road, and other locations required the mobilization of a dredge. In the Otis Pike Wilderness Area, triggers for post-closure maintenance actions were restricted to the occurrence of partial breaches, in order to align Breach Closure Plans more closely with the current management polices in this area. In order to evaluate maintenance costs, the volume and cost data presented in Table 7-29 was input to the models.

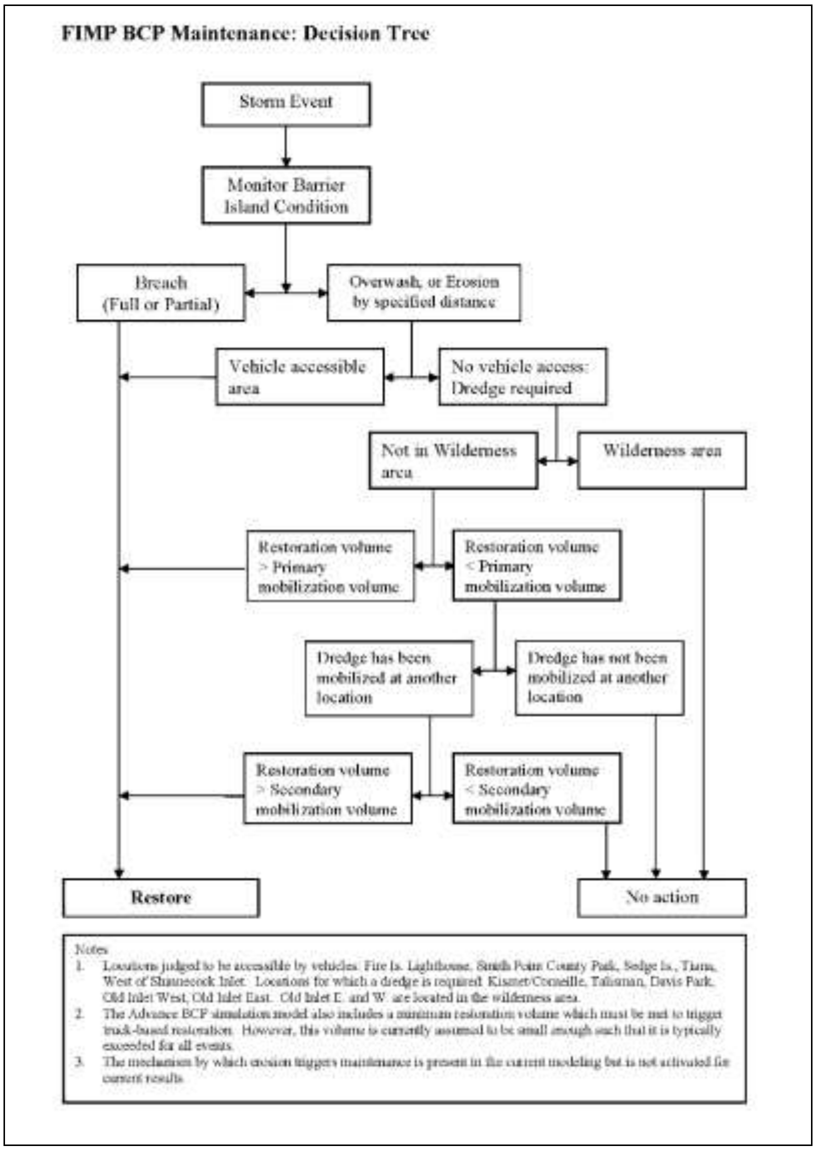


Figure 7-5. Decision tree for Maintenance of Breach Closure Template

Table 7-29. Primary Input Data for Evaluating Breach Closure Template Maintenance Costs

Input Quantity/Cost	Restoration Utilizing Trucking	Restoration Utilizing Dredge
Minimum Mobilization Volume	2,000 Cubic Yards	300,000 Cubic Yards
Initial Mobilization Cost	N/A	$1,000,000
Restoration Unit Price	$15 / Cubic Yard	$7 / Cubic Yard

7.7.3.5 Evaluation of Storm Damage Reduction Effectiveness

The reduction in storm damages arising from the implementation of these breach closure alternatives was modeled to quantify back bay inundation damages resulting from open breaches in the barrier islands, and structure failure damage, which results from the loss of buildings on the barrier islands when the land on which they stand is eroded by an expanding breach. This model is also used to quantify the costs associated with closing barrier island breaches, and with maintaining the design section in the post-closure time period.

The three alternative breach closure templates described above were evaluated and the resulting damages compared to those associated with the appropriate without-project condition. All breach closure alternatives were compared to the without-project scenario, which includes a delay of nine months prior to the start of construction. The results of the analyses are presented in Table 7-30 to TTable 7-33able 7-32. Table 7-30 presents the modeled with-project annual damages resulting from the implementation of the three breach closure alternatives considered.

Table 7-30. With Project Annual Damages: Breach Closure Alternatives

Damage Category	9.5'	11'	13'
Total Project			
Tidal Inundation			
Mainland	$66,638,700	$66,638,700	$66,638,700
Barrier	$11,668,400	$11,668,400	$11,668,400
Total Inundation	*$78,307,100*	*$78,307,100*	*$78,307,100*
Breach			
Inundation	$420,600	$314,800	$266,200
Structure Failure	$158,300	$172,800	$174,000
Total Breach	*$578,900*	*$487,600*	*$440,200*
Shorefront	$7,388,900	$7,388,900	$7,388,900
Public Emergency			
Other			
Total Storm Damage	***$86,274,900***	***$86,183,600***	***$86,136,200***

Interest Rate 5.125%, Project Life 50 years

These damages have been compared to those associated with the without-project condition to generate the project benefits, which are presented in Table 7-30. As described above, the Breach Closure alternatives only function to prevent breaches from remaining open. As such, the benefits are limited to reducing flooding due to breaches remaining open, and damages to structures on the barrier island, which represents a small portion of the overall damages in the study area.

Table 7-31. Annual Benefits: Breach Closure Alternatives

Benefit Category	9.5'	11'	13'
Total Project			
Tidal Inundation	$0	$0	$0
Mainland	$0	$0	$0
Barrier	$0	$0	$0
Total Inundation	*$0*	*$0*	*$0*
Breach			
Inundation	$8,821,900	$8,927,600	$8,976,300
Structure Failure	$237,400	$223,000	$221,700
Total Breach	*$9,059,300*	*$9,150,600*	*$9,198,000*
Shorefront Damage	$0	$0	$0
Public Emergency	$0	$0	$0
Other	$0	$0	$0
Total Storm Damage Reduction	*$9,059,300*	*$9,150,600*	*$9,198,000*
Costs Avoided			
Breach Closure	$2,159,600	$2,159,580	$2,159,580
Beach Maintenance	$0	$0	$0
Other			
Land Loss	$0	$0	$0
Total Benefits	***$11,218,900***	***$11,310,200***	***$11,357,600***

Interest Rate 5.125%, Project Life 50 years

7.7.3.6 Average Annual Breach Closure Plan Costs

To evaluate fully the relative costs of the various Breach Closure Plan alternatives, the annualized costs over the project life for each plan must be compared. The annualized costs for each alternative take into account the likely frequency with which each closure plan is implemented as well as the timing of implementation, and have been evaluated using the breach only lifecycle model. As presented previously, models have been developed to simulate the occurrence of breaches and hence breach closures during the project life, and the subsequent behavior of the profile at each location. This enabled the present worth of each closure cost to be calculated, totaled, and converted to an annualized value for comparison, by means of a Capital Recovery Factor.

The breach closure costs and profile maintenance costs associated with each alternative are presented in Table 7-32 using the input data and conditions presented above. The total investment costs include contingencies and allowances for Engineering and Design, and Supervision and Administration. Note that these breach closure plans have no first cost associated with them, and that the costs have been broken out into annual amounts that could be budgeted for (maintenance), and annual "breach closure funding" that is an expected annual emergency cost that would be necessary to implement this alternative.

Table 7-32. Annual Costs: Breach Closure Alternatives

Cost Category	9.5'	11'	13'
Total Project	$0	$0	$0
Total First Cost	$0	$0	$0
Total IDC	$0	$0	$0
Total Investment Cost	$0	$0	$0
Interest and Amortization	$0	$0	$0
Operation & Maintenance	$0	$0	$0
BCP Maintenance	$519,965	$367,761	$278,804
Monitoring	$0	$0	$0
Renourishment	$0	$0	$0
Total Budgeted Cost	*$519,965*	*$367,761*	*$278,804*
Annual Breach Closure Cost	$1,275,840	$1,042,386	$628,457
Major Rehabilitation	$0	$0	$0
Total Additional Cost	$0	$0	$0
Total Annual Cost	$1,795,805	$1,410,147	$1,159,867

Interest Rate 5.125%, Project Life 50 years

Table 7-33, which presents the net benefits and benefit-cost ratios of the three breach closure alternatives shows that all the alternatives analyzed are cost-effective in reducing storm damage, and that the +13' dune alternative would provide the greatest storm damage reduction benefits in excess of cost. Table 7-33 shows that as the scale of the Breach Closure Alternatives increase, the benefits increase, and the total annual costs decrease. The smaller alternatives breach more frequently, requiring more frequent breach closure, and also requiring a higher level of maintenance. For this reason, the +13 ft dune alternative is recommended to be considered further. However, knowing that there are environmentally sensitive areas where it may be desirable to promote some level of cross-shore transport, the 9.5 ft NGVD alternative is carried forward for consideration in these areas.

Table 7-33. Net Benefits and BCRs: Breach Closure Alternatives

	9.5’	11’	13’
Total Project			
Total Annual Cost	$1,795,805	$1,410,147	$1,159,867
Total Benefits	$11,219,152	$11,310,513	$11,357,843
Net Benefits	$9,423,347	$9,900,366	$10,197,976
Benefit-Cost Ratio	**6.25**	**8.02**	**9.79**
Great South Bay			
Total Annual Cost	$1,295,085	$699,934	$587,972
Total Benefits	$8,823,151	$8,904,345	$8,935,627
Net Benefits	$7,528,066	$8,204,411	$8,347,655
Benefit-Cost Ratio	**6.81**	**12.72**	**15.20**
Moriches Bay			
Total Annual Cost	$520,418	$419,740	$389,983
Total Benefits	$2,038,789	$2,055,494	$2,061,940
Net Benefits	$1,518,371	$1,635,754	$1,671,957
Benefit-Cost Ratio	**3.92**	**4.90**	**5.29**
Shinnecock Bay			
Total Annual Cost	$262,611	$261,079	$177,934
Total Benefits	$357,213	$350,673	$360,276
Net Benefits	$94,602	$89,594	$182,342
Benefit-Cost Ratio	**1.36**	**1.34**	**2.02**

Interest Rate 5.125%, Project Life 50 years

7.7.3.7 Proactive Breach Response

The Proactive Breach Response Plan is an alternative that includes measures to take action to prevent breaches from occurring at locations vulnerable to breaching, when a breach is imminent. This alternative provides a beach cross-section area that is comparable to the Breach Closure Alternatives, and smaller than a beachfill alternative.

These plans (as are the breach response plans) are not specifically designed with the intent of protecting ocean shorefront development from overwash, wave attack or storm induced erosion losses, and allow for a greater level of overwash and dune lowering during a storm, so long as the overwash extent is below the threshold that would result in breaching.

Based upon the results of the Breach Closure Alternatives analysis, the Proactive Breach Response Plan considered only the template with the +13 ft dune section. The cross-section is comparable to the Breach Closure Plan (BCP) with the + 9.5 ft NGVD berm and a +13 ft NGVD dune. The berm widths are generally described as 90 ft widths, but with the intent of matching the existing, adjacent shoreline. The fill alignment is generally consistent with the unconstrained dune alignment (or as far landward as possible accounting for real estate requirements).

Presently, this alternative has been developed assuming triggers for taking action, and triggers for renourishment of the profile in response to a partial breach or significant overwash. The proactive plans have been developed considering that a greater alongshore length of fill would be necessary, in comparison with the responsive plan, since the exact location of a breach is unknown.

Threshold for Proactive Breach Response

Available historic breaching information, including topography, waves, and water levels, and modeling results suggest that are there is more than one factor controlling the barrier island breaching risk. Modeling results show that depending on location and based on conditions prior to Hurricane Sandy, the partial breaching risk ranged from less than a 25 year return period at Old Inlet East to over 250 year return period at Sedge Island (Table 7-34). At Smith Point County Park (SPCP) the risk for breaching was approximately at the 26 year return period. Hurricane Sandy opened a 1,500-foot-wide breach just east of Moriches Inlet in Cupsogue County Park, a 500-foot-wide breach to the west of Moriches Inlet at Smith Point County Park, and a third breach at Old Inlet within the National Park Service's Fire Island Wilderness Area. Therefore, both the SPCP and Old Inlet breach locations coincided with areas previously identified as vulnerable to breaching by the 25 year storm, approximately.

Table 7-34. Baseline Overwash and Breaching Risk (Return Period) Potential

Location	Overwash	Partial Breaching	Full Breaching
FI Lighthouse Tract	14	184	> 500
Robins Rest.	9	141	> 500
Barrett Beach.	20	213	> 500
Davis Park	22	145	> 500
Old Inlet West	10	45	82
Old Inlet East	5	24	118
Smith Point CP	8	26	145
Sedge Island	25	251	> 500
Tiana Beach	7	72	336
West of Shinnecock	18	74	326
Note: based on Baseline (circa 2000) conditions			

Many other areas along the barrier island were subject to significant overwash, but did not breach. This is also consistent with pre-Sandy modeling results which as shown in Table 7-34 suggest overwash for a storm between a 5 and 25 year return period storm, depending on location.

Overall, the barrier island response appears to be most sensitive to elevation, width from ocean to bay, back bay bathymetry, and proximity to an existing inlet. Maximum barrier elevation, typically the dune crest, is important because it controls the overwash response. A high berm or dune, if combined with sufficient beach width, will prevent significant overwash, which is the precursor to beaching. However, elevation alone is not necessarily sufficient. Modeling results and data suggest that a high but relatively narrow dune with a narrow beach may be quickly eroded by surge and waves. Ultimately, whether the resulting overwash results in a breach will depend on the barrier island width at that location and the hydraulic gradient from ocean to bay. Locations farther away from an inlet (e.g., eastern Great South Bay prior to the Wilderness Area breach) have a tidal range significantly smaller than the adjacent ocean areas. The bayside storm surge hydrograph may also be significantly muted, particularly if the wind is pushing water away from the area. Therefore, if there is a significant overwash, and the ocean surge extends from ocean to bay, there is the potential for a difference in water levels that will be sufficient to drive the formation of a partial or even a full breach.

Barrier island sensitivity to breaching and the development of a response trigger is further complicated by the fact that the problem is not one-dimensional. A short narrow section of beach fronting an otherwise healthy dune may be result in localized overwash, but if the overwash is narrow relative to the barrier island width, it will not necessarily result in a breach. The same overwash over a longer section of the beach/barrier may allow for enough water discharge and scour to generate a breach. In the end, the likelihood of sufficient scour and a breach forming is a function of the hydraulic forcing (surge and waves) against the opposing “friction” generated by the cross-shore barrier island profile (height and width) and alongshore width of the flow path (a narrow “channel” will generate more resistance to flow than a wider one).

New proposed barrier island condition thresholds, or triggers, based on dimensions than can be easily measured and monitored as part of the FIMP project are proposed in the following sections. These triggers are also reach specific and consider historic breaching/overwash data, modeling results, and overall understanding of the hydraulic “conductivity” at each location. The triggers build on all of the engineering and modeling work that has been done in support of the FIMP Reformulation Study to date, including beach profile modeling (SBEACH), two-dimensional waves, storm surge, sediment transport and morphological modeling (ADCIRC, SWAM, and Delft3D), shoreline erosion modeling (GENESIS) and an engineering assessment of the potential future changes in the barrier island in response to continued erosion and storm impacts which was performed to define future barrier island conditions.

As explained above, breaching response is a multidimensional problem, so there is not one single measurement than can be monitored and used as threshold for action. Therefore, it is proposed that the following relevant dimensions be measured and considered instead:

1. Barrier island width: distance between bay and ocean MHW contours
2. Elevation: generally characterized by volume/area above +10 ft NGVD29
3. Beach width: distance between baseline (generally the natural dune alignment) and the MHW contour

Specific PBRP thresholds by reach are summarized in Table 7-35.. When one or more of these proposed thresholds is exceeded, historic data and modeling results suggest that the risk of a partial breach is at the 25 year return period level and proactive action should be taken to rebuild the PBRP template and reduce the risk of breaching. Note that if one of these thresholds is met over a very small area but the barrier island is generally in good condition otherwise, the risk of breaching is significantly less than if the threshold is met over a large area. Therefore, recommend triggers are based on both widespread but not necessarily contiguous weakness within a reach and smaller, localized, but potentially weaker spots. Sub Appendix A5, Proactive Breach Response Triggers includes a more detailed description of how these triggers were established.

In summary, the proactive breach closure plan would only be implemented when the barrier island cross-section falls below the threshold condition; the proactive breach closure plan has no advanced fill volume at construction, and the proactive breach closure plan is a plan with less rigorously structured renourishment requirements

7.7.3.8 Evaluation of Storm Damage Reduction Effectiveness

The reduction in storm damages arising from the implementation of a proactive breach response alternative was modeled using the Breach Only Lifecycle Model to analyze the resulting effect on breach-related damages, and also the Lifecycle Damage Analysis Model to quantify back bay inundation damages, since it was assumed that a proactive breach closure alternative would impact on back bay water levels. One proactive breach response alternative was analyzed, featuring a design section taken from the +13' dune (reactive) breach closure alternative, and the results are presented in Table 7-36.

As part of the lifecycle modeling work, breach response triggers were defined based on the concept of "effective width". The effective width is an abstract measurement of the vulnerability to breaching and indirectly accounts for the beach and dune width. In other words, although as explained above breaching response is a multidimensional problem, limitations in the lifecycle model approach do not allow for detailed simulation in time of each barrier island metric (i.e., beach width, dune height and width, barrier island width, etc.) an its effects on barrier island breaching potential. Effective width is therefore an overall proxy that tries to incorporate the influence of all these parameters and can change over the lifecycle simulation in response to erosion. This effective width has generally been approximated as the width of the island above the berm elevation (9.5 ft NGVD), but this is a generalization that may not hold true at every breach vulnerable location.

These damages have been compared to those associated with the without-project condition to generate the project benefits, which are presented in Table 7-37. This table illustrates that benefits achieved from this plan are similar to those provided by the +13 ft Breach Response Plan.

Table 7-35. Summary of Proposed Proactive Breach Response Triggers

Reach			Barrier Island Width		Area Above +10 ft NGVD					Beach Width			
			Contiguous		Total		Contiguous			Total		Contiguous	
ID	Name	Length (ft)	Length	Width	Length	Width	Length	Width	Beach Width	Length	Width	Length	Width
GSB-1B	Fire Island Lighthouse (FILT)	6,700	200	1,000	2,000	50	100	50	100	3,000	100	1,000	100
MB-1B	Smith Point County Park (SPCP) East	13,500	200	400	2,000	100	100	100	150	6,000	150	500	100
MB-2A	Great Gun	7,600	200	400	2,000	100	100	100	150	4,000	150	500	100
MB-2B	Moriches Inlet - West	6,200	200	1,200	2,000	50	100	50	100	3,000	100	1,000	100
SB-1A	Hampton Beach	16,800	200	600	2,000	50	100	50	100	8,000	100	1,000	100
SB-1B	Sedge Island	12,200	200	500	2,000	100	100	100	150	6,000	100	500	100
SB-1C	Tiana Beach	3,400	200	400	2,000	100	100	100	150	2,000	100	500	100
SB-1D	Shinnecock Park West (SPW)	6,300	200	600	2,000	50	100	50	100	3,000	100	500	100
SB-2A	Ponquogue	5,300	200	600	2,000	50	100	50	100	3,000	100	1,000	100
SB-2B	West of Shinnecock (WOSI)	3,900	100	350	2,000	100	100	100	150	2,000	100	300	100
SB-2C	Shinnecock Inlet - East	9,800	200	800	2,000	50	100	50	100	5,000	100	1,000	100
SB-3A	Southampton Beach	9,200	200	600	2,000	50	100	50	100	5,000	100	1,000	100

Table 7-36. Annual Damages: Proactive Breach Response Alternative

Damage Category	Proactive Breach Response: +13' Dune
Total Project	
Tidal Inundation	
Mainland	$73,994,969
Barrier	$12,998,638
Total Inundation	*$86,993,607*
Breach	
Inundation	$342,200
Structure Failure	$116,900
Total Breach	*$459,100*
Shorefront	$7,388,900
Public Emergency	
Other	
Total Storm Damage	***$94,841,607***

Interest Rate 5.125%, Project Life 50 years

Table 7-37. Annual Benefits: Proactive Breach Response Alternative

Benefit Category	Proactive Breach Response: +13' Dune
Total Project	
Tidal Inundation	
Mainland	$240,500
Barrier	$0
Total Inundation	*$240,500*
Breach	
Inundation	$8,900,300
Structure Failure	$278,900
Total Breach	*$9,179,200*
Shorefront Damage	$0
Public Emergency	
Other	
Total Storm Damage Reduction	
Costs Avoided	
Breach Closure	$2,159,900
Beach Maintenance	
Other	
Land Loss	
Total Benefits	***$11,579,600***

Interest Rate 5.125%, Project Life 50 years

The costs associated with this alternative, which include implementation, profile maintenance and breach closure, are presented in Table 7-38. These costs are slightly higher than for the responsive alternative. When comparing costs with the benefits presented in Table 7-37, this shows that the proactive plan is cost-effective, and provides positive net benefits that are slightly less than the responsive plan.

Table 7-38. Proactive Breach Response Costs

Proactive BCP	
First Cost	*$0*
IDC	$0
Total Investment Cost	*$0*
Interest & Amortization	$0
O&M	$0
BCP Maintenance	$1,400,400
Monitoring	$0
Renourishment	$0
Total Budgeted Cost	*$1,400,400*
Annual Breach Closure Cost	$759,000
Major Rehabilitation	$0

Since the costs and benefits for these plans are so similar, the proactive breach response plan has not been carried forward as a separate alternative from the breach response plans. The differences between the proactive closure and the responsive closure will be accounted for in the implementing criteria for each site. One benefit of the proactive breach closure plan is that a greater amount of the project costs fall within a budgetable category, and are not required as emergency response costs.

Following the identification of a preferred plan there is an opportunity to refine the process for breach response, for locations where breach response is proposed. If a breach response plan is an element of a preferred plan it may be warranted to consider if a more structured response is warranted, which could take advantage of cost-savings associated with the combined plan (i.e. shared mobilization expenses), and allow for the breach closure plans to be more of a budgeted program, rather than depending upon emergency funding. These refinements can also consider, at sight specific locations what trigger point is acceptable for action to occur. Presently the trigger point for taking action has consistently been applied to consider a threshold with a relatively high vulnerability to breaching. This threshold can be revisited to accommodate the level of risk that is acceptable.

7.7.3.9 Compatibility of Breach Response Alternatives with Restoration Alternatives.

FIMP seeks to develop a plan which advances storm damage reduction in a manner that also balances environmental considerations. In order to develop alternatives that advance both initiatives, consideration is given for each alternative to identify restoration alternatives that would be compatible with the individual storm damage reduction features. As described previously, the criteria used in considering the complimentary nature of the restoration is: 1) does the restoration increase the SDR effectiveness of the alternative, 2) are there cost efficiencies in implementing the measures together, and 3) does the restoration provide a desirable mosaic of habitats that could be altered by the SDR measure?

For the breach response plans, there are several types of restoration measures that would fall into these categories, and complement the breach response alternatives.

These include:

1. Restoration of bayside habitat (bayside beach, marsh or SAV) in breach vulnerable areas in conjunction with breach closure operations to mimic habitats likely to form in the absence of breach closure, to further reduce storm damages, and provide a desirable mosaic of habitats.

2. Restoration of bayside habitats (bayside beach, marsh or SAV), through habitat restoration alone or in combination with modification of bayside structures in breach vulnerable areas to reduce bayside erosion rates and/or the potential for breaching, and increase the effectiveness of the breach response measures.

3. Restoration of ocean-front dune habitats in breach closure locations, including the acquisition of buildings to provide for continuous ocean to bay habitat connectivity, and to facilitate continued maintenance of the breach closure cross-section.

4. Adaptive Management plans, to provide for ongoing management of the area, to ensure the continuity of desirable habitats, and control invasive species.

Restoration alternatives that fall into these categories are developed in the Environmental Appendix.

7.7.3.10 Evaluation of Planning Criteria

NED Criteria. The analysis above shows that breach response plans are cost-effective storm damage reduction alternatives that contribute to reducing the damages associated with a breach remaining open. Breach response plans can be either responsive or proactive, depending upon the implementing criteria, and for the +13 ft dune plan have similar costs and benefits.

P&G Criteria. This is the evaluation of the alternatives relative to being complete, effective, efficient, and implementable. Breach response plans alone do not represent a complete solution, as they only address as small portion of the damages that arise due to a breach being open. Relative to the purpose they are accomplishing, these alternatives are effective and efficient, particularly the larger plans. These alternatives are generally implementable, although within the Federal tracts of land on Fire Island, the NPS has expressed the need to consider the timeframe for closure based upon natural resources needs and storm damage reduction needs.

Vision Criteria. The breach closure alternatives were evaluated in relationship to the planning criteria developed to reflect the Project objectives and the project approach. This systematic assessment ensures that the Vision Statement approach is fully integrated into the development and selection of the FIMP Plan. Table 7-39 provides a summary of the evaluation of breach closure alternatives relative to the established criteria.

Table 7-39. Breach Closure Alternatives		
Evaluation Criteria	**Assessment**	**Rating**
The plan or measure provides identifiable reductions in risk from future storm damage.	Provides quantified reduction in storm damage.	Full
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science considered a lower priority.	Breach closure has been the general practice in the Study Area dating back to the 1938 storm. Options to allow natural closure are less certain due to uncertainties in future storms and sediment buildup. Plans will be	Full
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	Rapid response significantly reduces the risk of increased flooding in the bays following a breach. Some closure designs may reduce the flood risk associated with repetitive breaching and overwash.	Partial
The plan or measures incorporate appropriate non-structural features provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity	Compatible with non-structural components to limit redevelopment in breach vulnerable areas and helps avoid major changes in the flood elevations used to define floodplain management regulations.	No
The plan or measure help protect and restore coastal landforms and natural habitat.	Designs restore the barrier width and provide varying levels of dune restoration. Rapid closure will reduce volumes of sand captured in flood and ebb shoals when compared to without project conditions.	Partial
The plan avoids or minimizes adverse environmental impacts	Response protocols have been developed to minimize any adverse environmental impacts.	Partial
The plan addresses long-term demands for public resources.	Because closure designs use relatively small quantities of fill, future monitoring and some profile restoration is considered necessary to prevent repetitive breaching.	Partial
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	Closure restores the littoral transport and provides storm damage reduction. Potential reduction in cross shore transport.	Full
The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures	Not Applicable	No

Table 7-39. Breach Closure Alternatives		
Evaluation Criteria	**Assessment**	**Rating**
The plan or measure incorporates appropriate alterations of inlet stabilization measures and dredging practices	Not Applicable	No
The plan or measure is efficient and represents a cost effective use of resources	Measures are highly cost effective	Full
The plan or measure reduces risks to public safety.	Closure reduces the risk of hazardous storm surges in the bay and will reduce the potential for excessive shoaling of navigation inlets.	Full

Land Use and Development Management Challenges and Opportunities.

The breach response plans introduce some land use and development management challenges that would not be realized in the without project condition. Presently, the existing land use and development management measures offer no controls that would limit rebuilding in a breach area, subsequent to a breach closure, outside of the existing CEHA area. Land management measures should consider restricting redevelopment in locations that are likely to remain as vulnerable to breaching and overwash over the project life, to reduce repeated damages to structures, facilitate the continued breach response requirements, and to provide for a desirable habitat mosaic. This could be achieved both with improvements in the land use regulations, and with acquisition alternatives.

7.7.3.11 Summary of Breach Response Alternatives.

The analysis above shows that breach response plans are cost-effective storm damage reduction alternatives that contribute to reducing the damages associated with a breach remaining open. <u>Breach response plans can be either responsive or proactive, depending upon the implementing criteria, and for the +13 ft dune plan have similar costs and benefits.</u>

Breach response plans are recommended for further evaluation with the +13 ft dune template, which maximizes net benefits. In areas where a greater amount of cross-shore transport is desirable, the breach closure at elevation +9.5 ft NGVD can be considered further.

The breach response plans partially fulfill the vision objectives. There are a number of restoration features that can be integrated with the various breach closure alternatives, that could further the vision objectives.

7.7.4 Beachfill Alternatives

General

Beachfill (berm only) and beachfill with dunes have been designed for the Atlantic Ocean shorefront as storm damage reduction features. Varying scales of protection have been developed suitable for locations across the study area. The alternative design sections are summarized as follows:

- “Small” fill template or Lower Level of Protection (LLP): a berm width of 90 ft at elevation +9.5 ft NGVD and a low dune with a crest width of 25 ft at an elevation of +13 ft NGVD;
- “Medium” level of protection template: a berm width of 90 ft at an elevation +9.5 ft NGVD and medium dune with a crest width of 25 ft at an elevation of +15 ft NGVD;
- “Large” level of protection template: design section includes a dune at an elevation of +17 to +19 ft NGVD with a 25 ft crest width. Design berm width is 90 ft or 120 ft depending on the Project Reach.

The location of the proposed dune and berm was also evaluated based on three fill alignment plans. The Unconstrained (UC) Baseline was developed to be not constrained by real estate issues or recent beach fill projects, and is the farthest landward fill alignment, and generally matches the existing topography. A Minimum Real Estate Impacts (MREI) Baseline was defined that includes a realignment of the dune farther seaward in areas where multiple structures would need to be relocated or acquired in a more landward alignment. There is a difference in alignment in most of the developed communities on Fire Island with the exception of Cherry Grove and Water Island, where no Real Estate would be impacted by the unconstrained baseline alignment. A third baseline, the Middle (MID) Baseline, aimed at optimizing the dune alignment in areas where a few structures appear to be located significantly farther seaward than adjacent ones thus pushing the whole beach fill alignment seaward.

The consideration of scale and alignment allows for optimization relative to the protection afforded, and optimization of the location of the protective feature. In order to conduct the optimization to determine the appropriate scale of protection, it was necessary to consider the three scales of alternative at the same alignment. This first analysis utilized the most seaward alignment for comparison of plan alternatives. Upon identification of a preferred scale, consideration was given for variations in alignment.

Design

In areas where there is either an insignificant risk of breaching, no oceanfront structures, or relatively few structures (areas of low damages), beach fill was not considered (e.g., Sunken Forest, Wilderness Area – West, Great Gun, Hampton Beach, and most of the shoreline between Shinnecock Inlet and Montauk with the exception of the Potato Rd. Reach and Montauk Beach). Within the Pikes and Westhampton Reaches, which cover the extent of the Westhampton Interim Storm Damage Protection Project, two plans were considered, one with dimensions equal to the Interim project (i.e., dune at +15ft and a 90 ft berm) and a *Large* template with a dune at +17 ft and a 120 ft berm. A *Small* plan was not considered within these two reaches. Figure 7-6 shows the approximate extents of proposed fill placement within the FIMP area. Table 7-40 lists the reaches where beach fill was considered as an alternative as well as the range of template dimensions under consideration. Note that this table also indicates the number of fill alignments being considered in a particular reach as well as the length of dune and/or berm fill required under baseline conditions.

Table 7-40. Reaches where Beach Fill is Being Considered

Design SubReach	Name	Subreach Length [ft]	Max. Fill Length [ft]	No. of Alignments	Design Sections (Dune height/Berm width)
GSB-1A	RMSP	25,700	16,458	1	-/90
GSB-1B	FILT	6,700	5,468	1	13/90
GSB-2A	Kismet to Lonelyville	8,900	8,880	3	13/90, 15/90, 17/90
GSB-2B	Town Beach to Corneille	5,100	4,557	3	13/90, 15/90, 17/90
GSB-2C	Ocean Beach to Seaview	3,800	3,696	3	13/90, 15/90, 17/90
GSB-2D	OBP to POW	7,400	7,267	3	13/90, 15/90, 17/90
GSB-3A	Cherry Grove	3,000	2,929	1	13/90, 15/90, 17/90
GSB-3C	Fire Island Pines	6,600	6,424	3	13/90, 15/90, 17/90
GSB-3D	Talisman to Water Island	7,300	7,076	1	13/90
GSB-3E	Water Island	2,000	1,202	1	13/90, 15/90, 17/90
GSB-3F	Water Island to Davis Park	5,500	5,445	1	13/90
GSB-3G	Davis Park	4,100	4,042	3	13/90, 15/90, 17/90
GSB-4B	Old Inlet	16,000	15,023	1	13/90
MB-1A	SPCP-TWA	6,300	1,889	1	-/90
MB-1B	SPCP	13,500	13,174	1	13/90
MB-2C	Cupsogue	7,500	2,000	1	13/90
MB-2D	WHPTIN Pikes	9,700	9,630	1	15/90, 17/120
MB-2E	WHPTIN East	18,300	10,908	1	15/90, 17/120
SB-1B	Sedge Island	10,200	4,967	1	13/90
SB-1C	Tiana	3,400	3,361	1	13/90
SB-1D	Shinnecock Inlet Park-West	6,300	6,288	1	13/90
SB-2B	WOSI	3,900	3,875	1	13/90/,15/90, 17/120
P-1G	Potato Road	4,300	3,500	1	13/90/,15/90, 17/120
M-1F	Montauk Beach	4,700	4,636	1	13/90/,15/90, 17/120

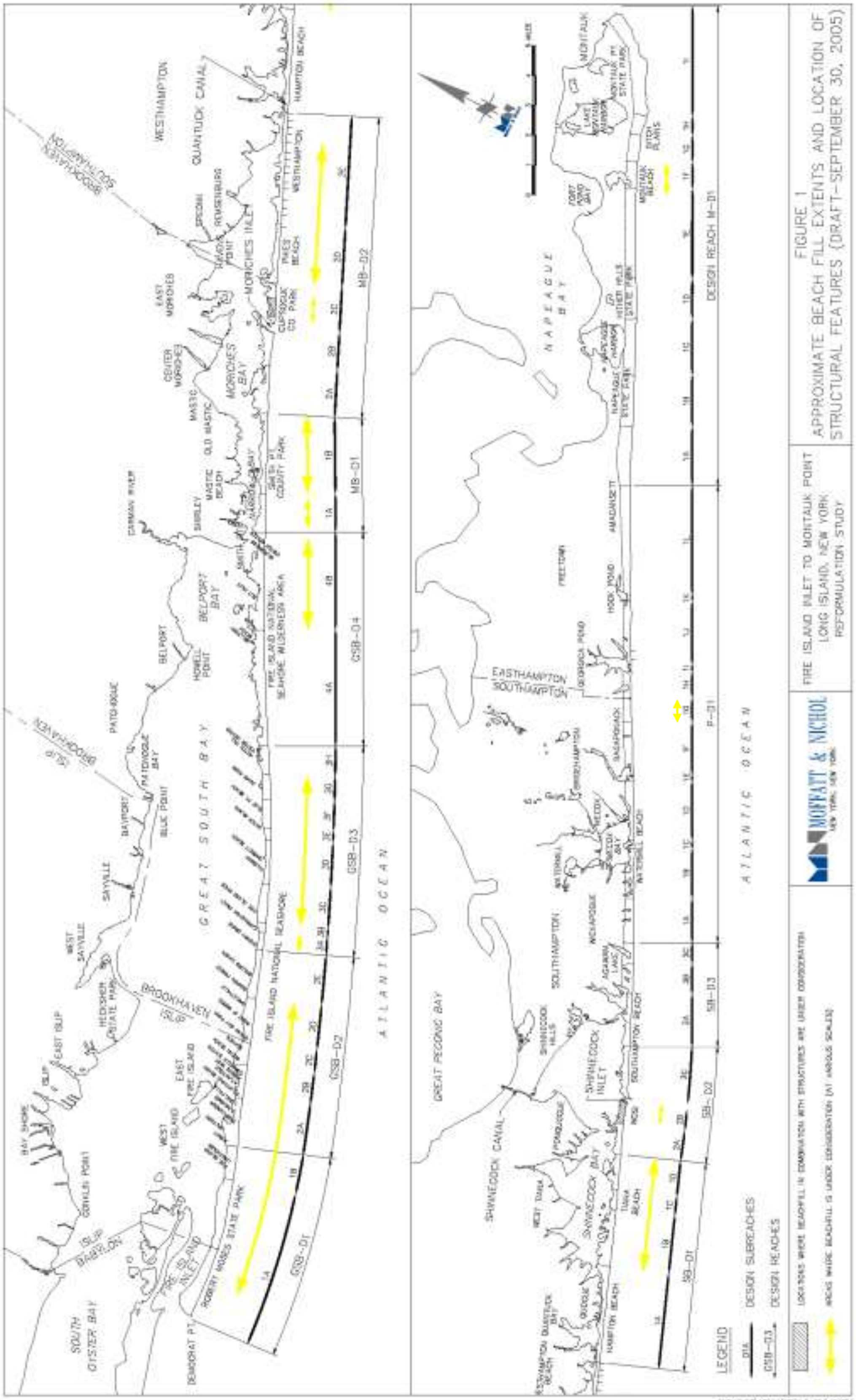


Figure 7-6 Reach Designation and Beach Restoration Locations

Figure 7-7, Figure 7-8 and Figure 7-9 show typical design sections for a few reaches considered representative of the complete set of reaches where fill placement is being considered. Specifically, Figure 7-7 shows typical profiles and design templates at Robert Moses State Park (GSB-1A) and Old Inlet (GSB-4B). Smith County Park is a unique design in that there is no dune required or proposed, only a 90 ft berm, in the area fronting the seawall that provides coastal storm damage risk reduction to the existing park facilities as well as the beach fronting the TWA memorial. Old Inlet is representative of the proposed beach fill plan in non-developed areas (including FINS tracts) subject to breaching risk.

Note that in many cases, as shown on Figure 7-8, the existing (i.e., Sept. 2000) berm and/or dune already provide the required level of coastal storm damage risk reduction along part or all of a specific reach. Nonetheless, it is necessary to have a plan in place that allows for rebuilding this minimum section in case of erosion or significant storm damage. Also, note that the figures focus on the sub-aerial and foreshore part of the profile to clearly depict the various templates and alignments being proposed. The proposed design (not construction) foreshore slope (from +9.5 to +2 ft NGVD) for the design profile is roughly 12.1 on 1. This number is based on an analysis of existing profiles in the FIMP area (based on LIDAR Sept. 2000 data) completed by M&N and CHL. Below MHW (roughly +2 ft NGVD) the submerged morphological profile representative of each specific reach is translated and used as the design profile. In other words, it is assumed that over a short period of time the fill will reach an equilibrium profile (from the edge of the berm to the depth of closure) similar to the “existing” profile.

Figure 7-8 shows a typical section and range of plans for a FI community. The figure shows design sections for two possible alignments, which are explained in detail in the next section.

Finally, Figure 7-9 and Figure 7-10 show typical profiles and the proposed range of plans for the West of Shinnecock and Montauk Beach reaches. Note that as of Sept. 2000, the berm at WOSI was relatively wide as a result of fill placement in 1998 and relatively mild weather between those two dates. Finally, note that at Montauk Beach, protection of the existing structures would require a significant amount of fill, even if a higher and narrower section was considered (i.e., 19/90). This is because the structures are very close to the seaward edge of the existing dunes and the beaches within the Ponds and Montauk reaches are relatively narrow and steep. A similar condition is observed at Potato Road.

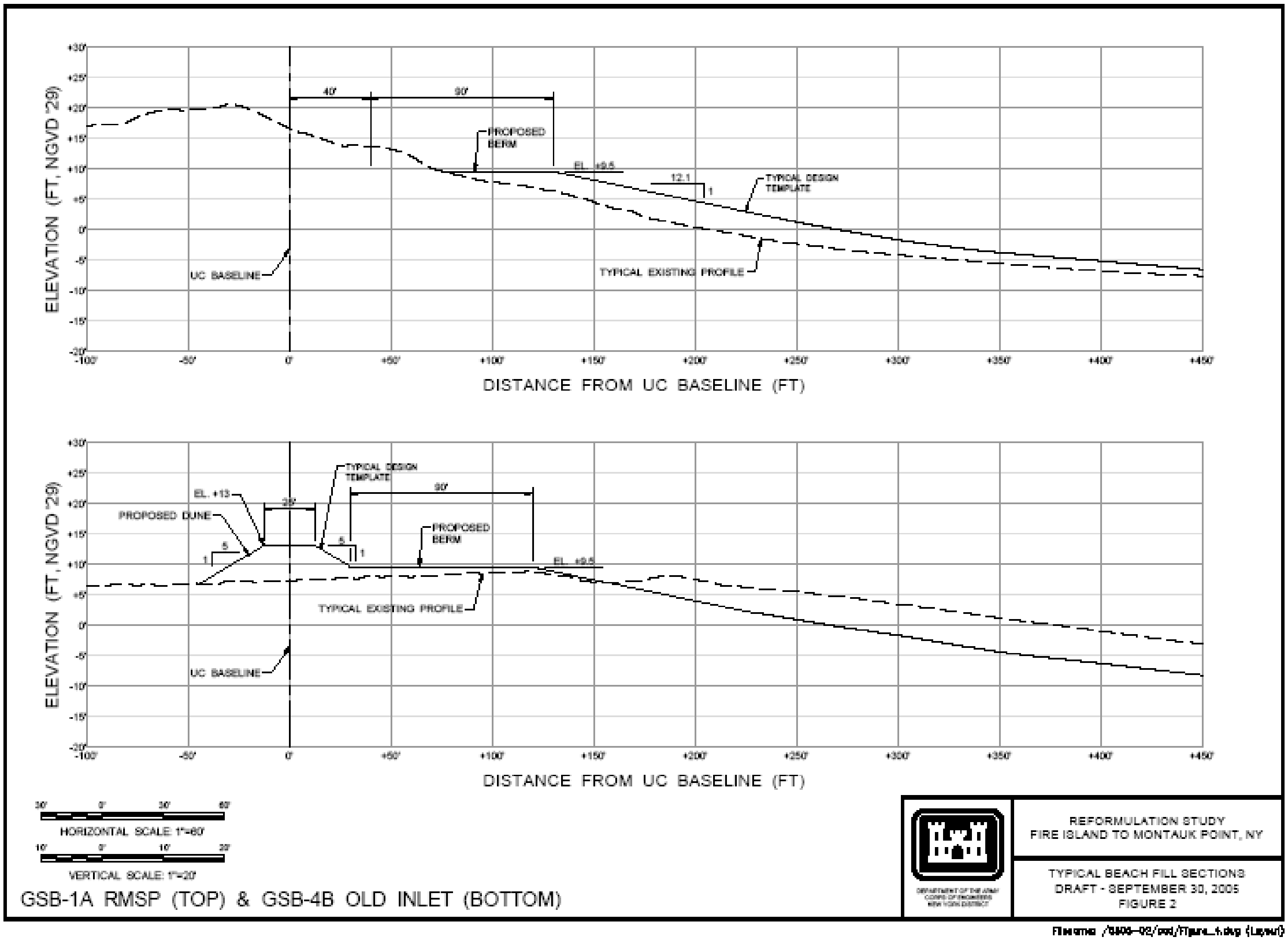


Figure 7-7. Typical Beachfill Section at GSB-1A, GSB-4B

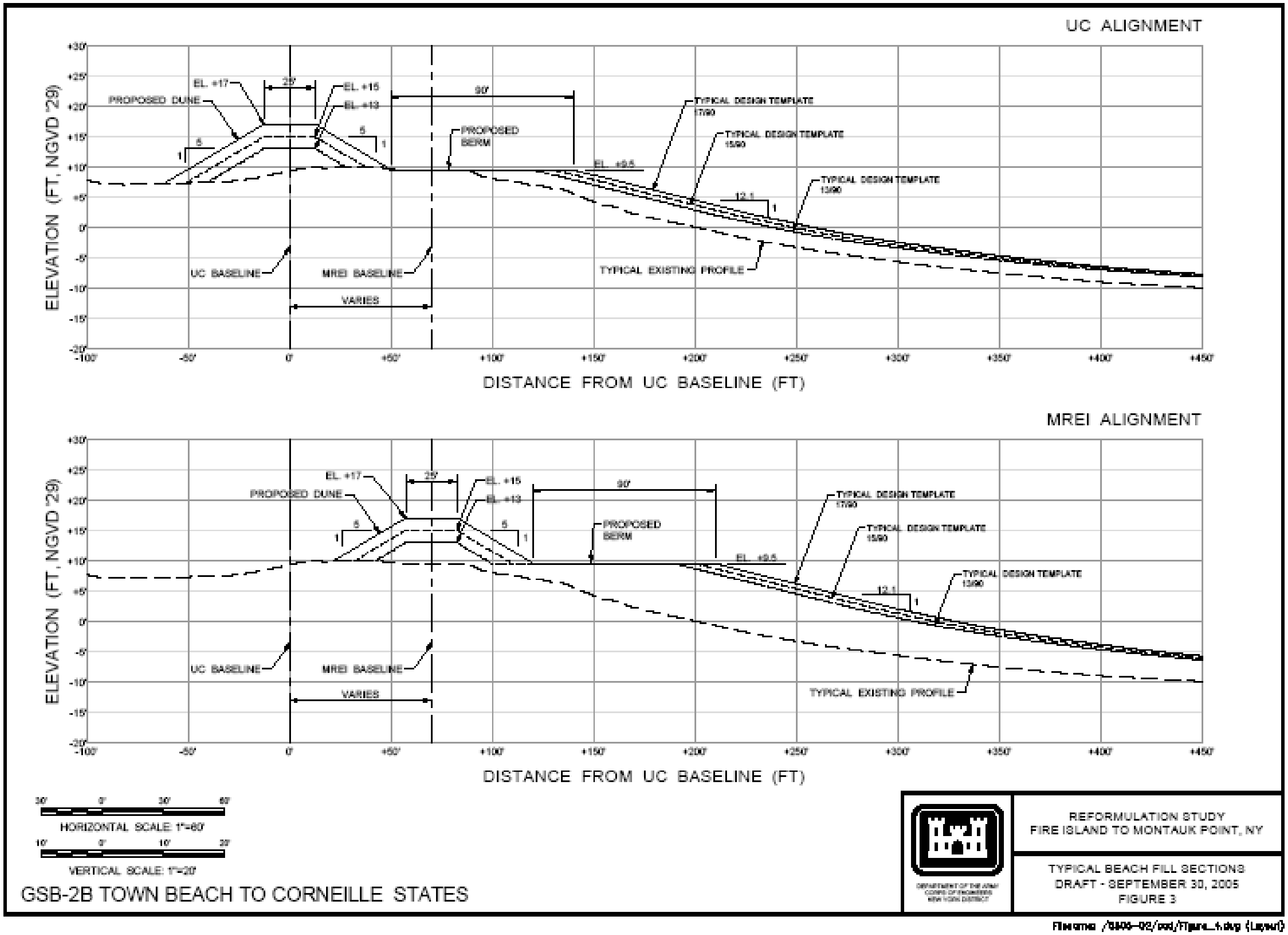


Figure 7-8. Typical Beachfill Section at GSB-2B

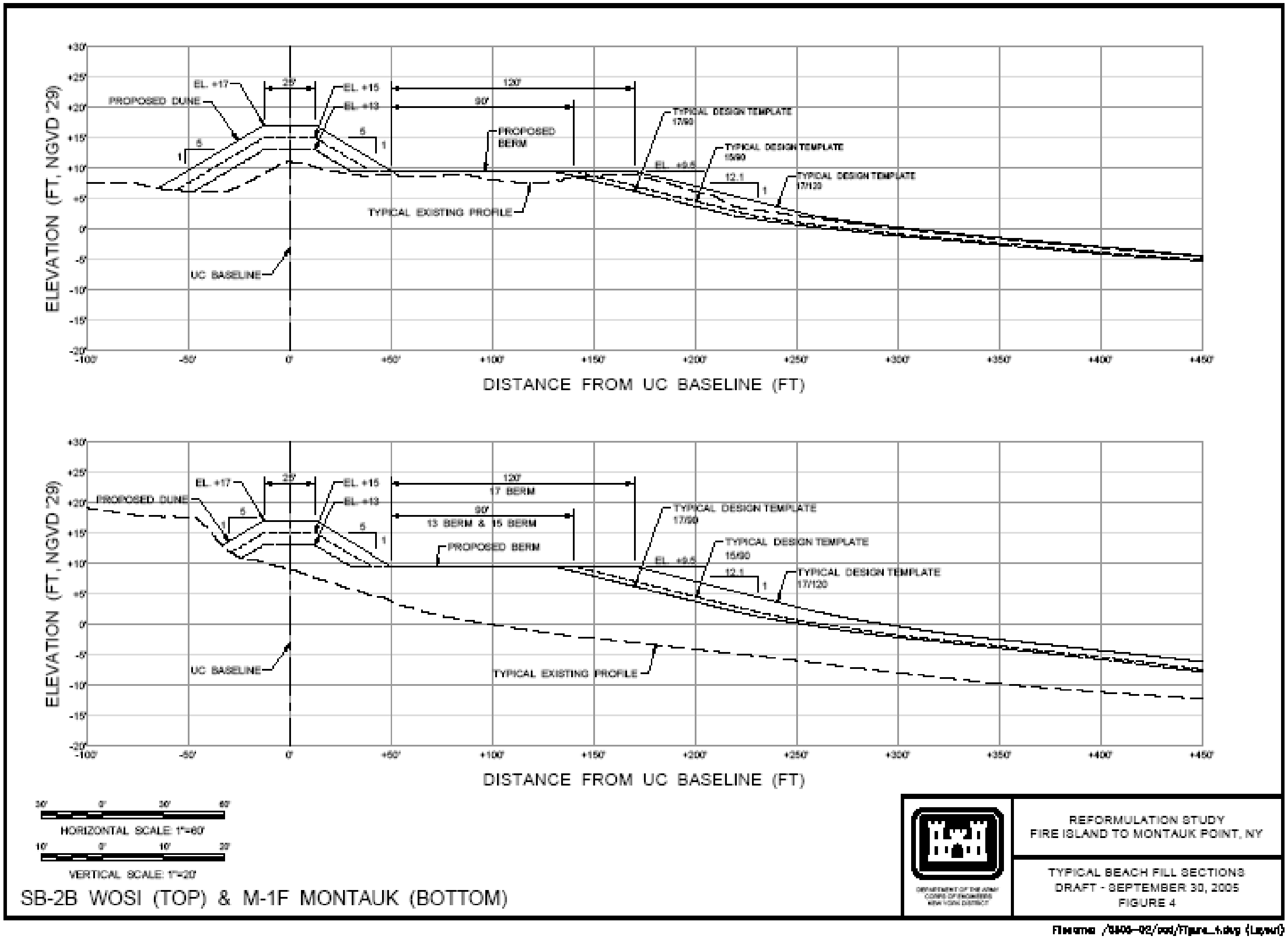


Figure 7-9. Typical Beachfill Section at WOSI

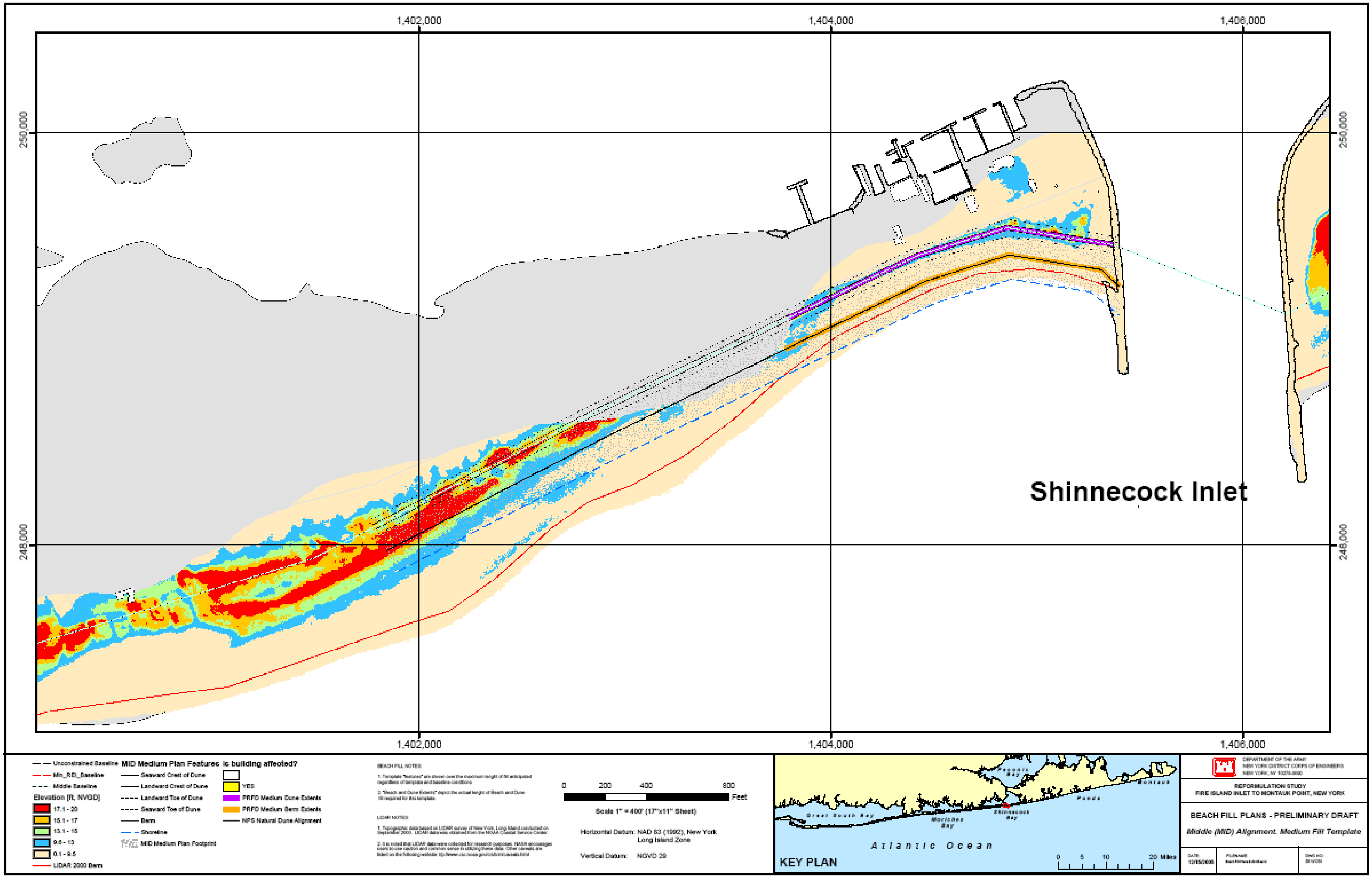


Figure 7-10. Typical Beachfill Layout at WOSI: Medium Template, MID Dune Alignment

Fill

Table 7-41, Table 7-42 and Table 7-43 summarize the length of berm and dune that would need to be placed for the three scales of alternatives at the MREI Alignment. These lengths were determined by comparing the proposed layout (including an estimate of advance fill) with the existing topography and location of the berm. For example, if the design template includes a dune at 17 ft with a 25 ft crest, only areas with lower or narrower dunes were considered. Out of a total 153,000 ft (29 miles) of shoreline where it is anticipated that beach fill may be required at some point during the project life, 43,000 ft of dune and 65,000 of berm is required for the *MREI-Large* plan and 21,000 ft of dune and 44,000 of berm for the UC-Small plan, and 31,568 ft of dune and 57,909 ft of berm is required for the *MID-Medium* plan.

Table 7-41. Required Berm and Dune Fill Lengths (MREI-Small Plan)

Design SubReach	Name	Max. Fill Length [ft]	Required Dune Length [ft]	Required Berm Length [ft]
GSB-1A	RMSP	16,458		5,795
GSB-1B	FILT	5,468	2,614	5,468
GSB-2A	Kismet to Lonelyville	8,880		8,880
GSB-2B	Town Beach to Corneille	4,557	2,100	4,555
GSB-2C	Ocean Beach to Seaview	3,696		3,151
GSB-2D	OBP to POW	7,267		7,305
GSB-3A	Cherry Grove	2,929		0
GSB-3C	Fire Island Pines	6,424		6,424
GSB-3D	Talisman to Water Island	7,076	1,492	0
GSB-3E	Water Island	1,202	262	0
GSB-3F	Water Island to Davis Park	5,445		0
GSB-3G	Davis Park	4,042		3,881
GSB-4B	Old Inlet	15,023	3,932	8,161
MB-1A	SPCP-TWA	1,889		2,366
MB-1B	SPCP	13,174	5,280	4,054
MB-2C	Cupsogue	2,000		1,845
MB-2D	WHPTIN Pikes	9,630		3,651
MB-2E	WHPTIN East	10,908		0
SB-1B	Sedge Island	4,967	801	1,057
SB-1C	Tiana	3,361	998	1,527
SB-1D	Shinnecock Inlet Park-West	6,288		1,312
SB-2B	WOSI	3,875	852	1,806
P-1G	Potato Road	3,500	1261	3,500
M-1F	Montauk Beach	4,636	1,878	4,287
Total		**152,696**	**21,470**	**79,026**

Table 7-42. Required Berm and Dune Fill Lengths (MREI-Medium Plan)

Design SubReach	Name	Max. Fill Length [ft]	Required Dune Length [ft]	Required Berm Length [ft]
GSB-1A	RMSP	16,458	0	5,795
GSB-1B	FILT	5,468	2,614	5,468
GSB-2A	Kismet to Lonelyville	8,880	2,167	8,880
GSB-2B	Town Beach to Corneille	4,557	3,700	4,555
GSB-2C	Ocean Beach to Seaview	3,696	0	3,151
GSB-2D	OBP to POW	7,267	2,397	7,305
GSB-3A	Cherry Grove	2,929	0	0
GSB-3C	Fire Island Pines	6,424	424	6,424
GSB-3D	Talisman to Water Island	7,076	1,679	0
GSB-3E	Water Island	1,202	1,097	0
GSB-3F	Water Island to Davis Park	5,445	0	0
GSB-3G	Davis Park	4,042	2,918	3,881
GSB-4B	Old Inlet	15,023	3,932	8,161
MB-1A	SPCP-TWA	1,889	0	2,366
MB-1B	SPCP	13,174	5,280	4,054
MB-2C	Cupsogue	2,000	0	1,845
MB-2D	WHPTIN Pikes	9,630	0	3,651
MB-2E	WHPTIN East	10,908	0	0
SB-1B	Sedge Island	4,967	801	1,057
SB-1C	Tiana	3,361	998	1,527
SB-1D	Shinnecock Inlet Park-West	6,288	1,034	1,312
SB-2B	WOSI	3,875	1,671	1,806
P-1G	Potato Road	3,500	1,261	3,500
M-1F	Montauk Beach	4,636	1,878	4,287
Total		**152,696**	**33,853**	**79,026**

Table 7-43. Required Berm and Dune Fill Lengths (MREI-Large Plan)

Design SubReach	Name	Max. Fill Length [ft]	Required Dune Length [ft]	Required Berm Length [ft]
GSB-1A	RMSP	16,458		5,795
GSB-1B	FILT	5,468	2614	5,468
GSB-2A	Kismet to Lonelyville	8,880	4926	8,880
GSB-2B	Town Beach to Corneille	4,557	3882	4,555
GSB-2C	Ocean Beach to Seaview	3,696	850	3,151
GSB-2D	OBP to POW	7,267	3423	7,305
GSB-3A	Cherry Grove	2,929		0
GSB-3C	Fire Island Pines	6,424	2143	6,424
GSB-3D	Talisman to Water Island	7,076	1679	0
GSB-3E	Water Island	1,202	1265	0
GSB-3F	Water Island to Davis Park	5,445		0
GSB-3G	Davis Park	4,042	3720	3,881
GSB-4B	Old Inlet	15,023	3932	8,161
MB-1A	SPCP-TWA	1,889		2,366
MB-1B	SPCP	13,174	5280	4,054
MB-2C	Cupsogue	2,000		1,845
MB-2D	WHPTIN Pikes	9,630	799	3,685
MB-2E	WHPTIN East	10,908		0
SB-1B	Sedge Island	4,967	801	1,057
SB-1C	Tiana	3,361	998	1,527
SB-1D	Shinnecock Inlet Park-West	6,288		1,312
SB-2B	WOSI	3,875	2852	1,806
P-1G	Potato Road	3,500	1950	3,500
M-1F	Montauk Beach	4,636	1878	4,287
Total		**152,696**	**42,992**	**79,060**

Beach Fill Volumes

Fill volumes were computed for each design reach for all three beach fill plans described above. Baseline Conditions were based on the September 2000 LIDAR survey for the subaerial part of the profile and the CHL representative morphological profile for the submerged portion. LIDAR survey profiles were extracted every 200 feet over the length of the project area (between 279 and 392 profiles were utilized depending on the beach fill plan). Fill was assumed only in areas where the berm and/or dune were found to be narrower and/or lower than the design template. The *Design Fill* volume per design reach was computed as the average dune or berm fill area required in each reach based on the values competed for each individual profile, multiplied by the length of berm or dune fill required in that reach. In addition to the base amount of *Design Fill* needed, *Advance Fill* volume was computed based on representative erosion rates and expected renourishment interval. The length of berm required by reach was multiplied by the active profile depth (36.5 ft) and the advance fill width (computed as the erosion rate times the renourishment interval) to come up with advance fill volume. A 15% tolerance was included based on the subtotal (design and advanced fill) as well as an overfill allowance of 1.10 to account for differences between the borrow area materials and the natural beach sand.

Initial fill volumes (i.e., design fill plus advance fill), future renourishment volumes over the project life, and total volumes for all three plans are presented in Table 7-44 through Table 7-46. Note that the future renourishment volumes are only a rough estimate based on erosion rates, renourishment interval, and, more importantly, the initial berm length. In other words, in reaches where no initial berm is required under a certain plan (e.g., SPCP or WHPTIN East), no future renourishment volume was assumed. Obviously this may result in underestimation of the total renourishment volume required over the life of the project. An alternative approach would be to assume that future renourishment will be required over the maximum length of each design subreach. This assumption, which is perhaps too conservative, would almost triple the amount of renourishment volume shown in the tables below.

Table 7-44. Required Fill Volume (MREI-Small Plan)

Design SubReach	Name	Renourish. Interval [years]	Initial Fill Volume [cy]	Renourish. Volume [cy]	TOTAL [cy]
GSB-1A	RMSP	4	546,677	4,866,667	5,413,344
GSB-1B	FILT	4	164,051	2,439,336	2,603,386
GSB-2A	Kismet to Lonelyville	4	1,953,328	3,961,467	5,914,795
GSB-2B	Town Beach to Corneille	4	1,206,756	2,032,036	3,238,792
GSB-2C	Ocean Beach to Seaview	4	426,637	1,405,696	1,832,333
GSB-2D	OBP to POW	4	1,463,368	3,258,842	4,722,209
GSB-3A	Cherry Grove	0	0	0	0
GSB-3C	Fire Island Pines	4	1,517,357	5,731,636	7,248,993
GSB-3D	Talisman to Water Island	4	3,126	312,278	315,404
GSB-3E	Water Island	4	603	107,245	107,848
GSB-3F	Water Island to Davis Park	4	0	0	0
GSB-3G	Davis Park	4	527,200	346,271	873,471
GSB-4B	Old Inlet	4	982,602	1,487,895	2,470,498
MB-1A	SPCP-TWA	4	231,138	422,218	653,356
MB-1B	SPCP	4	429,835	2,350,827	2,780,662
MB-2C	Cupsogue	4	168,112	861,866	1,029,978
MB-2D	WHPTIN Pikes	4	305,654	7,732,890	8,400,854
MB-2E	WHPTIN East	4	0	5,839,416	5,839,416
SB-1B	Sedge Island	4	101,790	471,539	573,329
SB-1C	Tiana	4	255,812	1,499,379	1,755,191
SB-1D	Shinnecock Inlet Park-West	4	192,522	585,298	777,820
SB-2B	WOSI	2	190,298	8,643,403	8,833,700
P-1G	Potato Road	4	881,839	4,684,167	5,566,005
M-1F	Montauk Beach	4	1,083,162	3,824,957	4,908,119
TOTAL		4	**12,631,865**	**62,865,328**	**75,859,503**

Table 7-45. Required Fill Volume (MREI-Medium Plan)

Design SubReach	Name	Renourish. Interval [years]	Initial Fill Volume [cy]	Renourish. Volume [cy]	TOTAL [cy]
GSB-1A	RMSP	4	546,677	4,866,667	5,413,344
GSB-1B	FILT	4	164,051	2,439,336	2,603,386
GSB-2A	Kismet to Lonelyville	4	2,138,765	3,961,467	6,100,231
GSB-2B	Town Beach to Corneille	4	1,337,322	2,032,036	3,369,358
GSB-2C	Ocean Beach to Seaview	4	485,444	1,405,696	1,891,140
GSB-2D	OBP to POW	4	1,529,389	3,258,842	4,788,231
GSB-3A	Cherry Grove	0	0	0	0
GSB-3C	Fire Island Pines	4	1,508,445	5,731,636	7,240,080
GSB-3D	Talisman to Water Island	4	3,519	312,278	315,797
GSB-3E	Water Island	4	2,849	107,245	110,094
GSB-3F	Water Island to Davis Park	4	0	0	0
GSB-3G	Davis Park	4	597,144	346,271	943,416
GSB-4B	Old Inlet	4	982,602	1,487,895	2,470,498
MB-1A	SPCP-TWA	4	231,138	422,218	653,356
MB-1B	SPCP	4	429,835	2,350,827	2,780,662
MB-2C	Cupsogue	4	168,112	861,866	1,029,978
MB-2D	WHPTIN Pikes	4	305,654	7,732,890	8,038,544
MB-2E	WHPTIN East	4	0	5,839,416	5,839,416
SB-1B	Sedge Island	4	101,790	471,539	573,329
SB-1C	Tiana	4	255,812	1,499,379	1,755,191
SB-1D	Shinnecock Inlet Park-West	4	192,522	585,298	777,820
SB-2B	WOSI	2	219,700	8,643,403	8,863,102
P-1G	Potato Road	4	893,031	4,684,167	5,577,198
M-1F	Montauk Beach	4	1,167,966	3,824,957	4,992,922
TOTAL		**n/a**	**13,261,765**	**62,865,328**	**76,127,093**

Table 7-46. Required Fill Volume (MREI-Large Plan)

Design SubReach	Name	Renourish. Interval [yrs]	Initial Fill Volume [cy]	Renourish. Volume [cy]	TOTAL [cy]
GSB-1A	RMSP	4	546,677	4,866,667	5,413,344
GSB-1B	FILT	4	164,051	2,439,336	2,603,386
GSB-2A	Kismet to Lonelyville	4	2,354,098	3,961,467	6,315,565
GSB-2B	Town Beach to Corneille	4	1,452,989	2,032,036	3,485,025
GSB-2C	Ocean Beach to Seaview	4	560,674	1,405,696	1,966,370
GSB-2D	OBP to POW	4	1,783,203	3,258,842	5,042,045
GSB-3A	Cherry Grove	0	0	0	0
GSB-3C	Fire Island Pines	4	1,773,462	5,731,636	7,505,098
GSB-3D	Talisman to Water Island	4	3,519	312,278	315,797
GSB-3E	Water Island	4	10,082	107,245	117,327
GSB-3F	Water Island to Davis Park	4	0	0	0
GSB-3G	Davis Park	4	756,931	346,271	1,103,202
GSB-4B	Old Inlet	4	982,602	1,487,895	2,470,498
MB-1A	SPCP-TWA	4	231,138	422,218	653,356
MB-1B	SPCP	4	429,835	2,350,827	2,780,662
MB-2C	Cupsogue	4	168,112	861,866	1,029,978
MB-2D	WHPTIN Pikes	4	623,489	7,732,890	8,356,379
MB-2E	WHPTIN East	4	0	5,839,416	5,839,416
SB-1B	Sedge Island	4	101,790	471,539	573,329
SB-1C	Tiana	4	255,812	1,499,379	1,755,191
SB-1D	Shinnecock Inlet Park-West	4	192,522	585,298	777,820
SB-2B	WOSI	2	363,007	8,643,403	9,006,410
P-1G	Potato Road	4	1,224,602	4,684,167	5,908,768
M-1F	Montauk Beach	4	1,400,604	3,824,957	5,225,560
TOTAL		**n/a**	**15,379,199**	**62,865,328**	**78,244,526**

As expected, the Small design template results in the least fill volume required; the Large design template combined with the MREI baseline results in the most. Also worth noting are the relatively large volumes required at Potato Road and Montauk Beach despite the fact that these are relatively small reaches. This result is directly related to the fact that significant erosion is expected within these two reaches over the project life. Other reaches requiring a significant amount of fill over the project life are western Fire Island Communities, Fire Island Pines, Pikes Beach, and WOSI.

COSTS

All cost estimates are based on October 2007 price levels. A $2,000,000 mobilization/demobilization cost is assumed per dredging contract. This is larger than the $1,000,000 mobilization/demobilization cost assumed for the BCP because the beach fill contracts are larger and cover a much greater distance per contract.

The costs for the Total Project as well as per Project Reach were examined. The essential difference lies in the distribution of dredging contracts and thus, mobilization and demobilization costs. Under the Total Project plan, dredging contracts are assigned based on volumes and distances between project locations, regardless of project reach delineation. Each dredging contract required a volume of approximately 2 million cubic yards. Under the Project Reach plan, dredging contracts are assigned to individual project reaches. In this case, dredging contracts were assigned within project reaches based on a volume of approximately 2 million cubic yards. The following provides a summary of the key cost assumptions.

First Costs

First costs include dredging, mobilization, and demobilization for the initial fill volumes estimated. First cost estimates also include a 15% contingency. Engineering and design costs are assumed to be 7% of the construction cost. Supervision and administration costs are also assumed to be a percentage of the construction cost, ranging from 6.47% to 7.09%. Dredging costs per cubic yard by reach/borrow area and mobilization costs per dredging contract were provided by CENAN, using CEDEP (Corps of Engineers Dredge Estimating Program). The program assumes the use of 2500 CY hopper dredges working 24 hours per day, 7 days per week with two daily 12-hours shifts. CEDEP incorporates influencing factors such as hopper capacity and safe load, area of borrow site, distance to borrow site, and current fuel, labor, and equipment costs, etc. Due to the larger number of contracts required, first costs are always greater when using the Project Reach plan as compared to the Total Project Plan.

Renourishment Costs

Renourishment costs include dredging, mobilization, and demobilization; the same dredging unit costs are assumed for both initial fill and renourishment fill. Renourishment costs include a 15% contingency, 7% for E&D, and the S&A percentage computed as given above. Most reaches are renourished every four years; only WOSI is renourished every 2 years.

Berm and Fill Maintenance Costs

Berm maintenance cost is the cost of moving fill to address shoreline undulations and erosion hotspots. The cost is assumed to be $15 per linear foot of fill annually and is applicable to all reaches. Fill maintenance costs are the miscellaneous costs of maintaining the beach, such as tilling. Annual fill maintenance costs are assumed to be $2 per linear foot of fill for all reaches. The unit cost of berm and fill maintenance is based upon the analysis performed by CP&E in 2002.

Annual Costs

Annual costs incorporate the initial fill cost, renourishment costs, and berm and fill maintenance costs. Annual costs assume a project life of 50 years and an interest rate of 5.125%. Annual costs under the Total Project plan range from $17,500,000 per year for the UC-Small alternative to $22,600,000 for the MREI-Large alternative.

Evaluation of Storm Damage Reduction Effectiveness

The reduction in storm damages resulting from alternatives that involve the placement of beach fill along the length of the project shorefront have been modeled using the Lifecycle Damage Analysis Model, with appropriate revisions to threshold water levels for breach and overwash, and the effect of the beach fill on back bay stage-frequency relationships. The Breach Only Lifecycle Model was also used to analyze the resulting change in breach-related damages. The three beach fill alternatives evaluated represent dune crest elevations of +13', +15' and +17' NGVD, all on a baseline selected for minimum real estate impact. This is the first set of alternatives which is designed to reduce damages along the shorefront areas. Table 7-45 presents the modeled annual damages resulting from the implementation of the three beach fill alternatives. In addition to storm damage reduction benefits the beach fill alternatives will eliminate the need for the numerous local renourishment projects. The sediment budget analysis has identified that these non Federal projects have placed an average of 180,000 cubic meters per year (234,000 cubic yards per year) of beach fill in the Great South Bay Planning Unit, considered as a local beachfill cost-avoided benefit.

The reduction in storm damages resulting from alternatives that involve the placement of beach fill along the length of the project shorefront have been modeled using the Lifecycle Damage Analysis Model, with appropriate revisions to threshold water levels for breach and overwash, and the effect of the beach fill on back bay stage-frequency relationships. The Breach Only Lifecycle Model was also used to analyze the resulting change in breach-related damages. The three beach fill alternatives evaluated represent dune crest elevations of +13', +15' and +17' NGVD, all on a baseline selected for minimum real estate impact. This is the first set of alternatives which is designed to reduce damages along the shorefront areas. Table 7-47 presents the modeled annual damages resulting from the implementation of the three beach fill alternatives.

Table 7-47. Annual Damages: Beach Fill Alternatives

Damage Category	Beach Fill +13'	Beach Fill +15'	Beach Fill +17'
Total Project			
Tidal Inundation			
Mainland	$65,154,300	$62,179,600	$62,179,600
Barrier	$11,279,800	$10,497,600	$10,497,600
Total Inundation	*$76,434,000*	*$72,677,200*	*$72,677,200*
Breach			
Inundation	$59,000	$3,000	$0
Structure Failure	$37,500	1,600	$0
Total Breach	*$96,500*	*$4,600*	*$0*
Shorefront	$3,718,800	$3,204,000	$2,946,600
Public Emergency			
Other			
Total Storm Damage	***$80,249,300***	***$75,885,800***	***$75,623,800***

Interest Rate 5.125%, Project Life 50 years

These damages have been compared with those associated with the without-project condition to generate the project benefits, which are presented in Table 7-48. In addition to storm damage reduction benefits the beach fill alternatives will eliminate the need for the numerous local renourishment projects. The sediment budget analysis has identified that these non Federal projects have placed an average of 180,000 cubic meters per year (234,000 cubic yards per year) of beach fill in the Great South Bay Reach. Eliminating the need for these efforts will provide annual savings estimated at $2,400,000 (shown as a local beachfill cost-avoided benefit).

Table 7-48. Annual Benefits: Beach Fill Alternatives

Benefit Category	Beach Fill +13'	Beach Fill +15'	Beach Fill +17'
Total Project			
Tidal Inundation			
Mainland	$9,081,200	$12,055,900	$12,055,900
Barrier	$1,718,800	$2,501,100	$2,501,100
Total Inundation	*$9,628,000*	*$14,557,000*	*$14,557,000*
Breach			
Inundation	$9,183,500	$9,239,400	$9,242,500
Structure Failure	$358,200	$394,100	$395,700
Total Breach	*$9,541,700*	*$9,633,500*	*$9,638,200*
Shorefront Damage	$3,670,000	$4,184,800	$4,442,200
Public Emergency			
Other			
Total Storm Damage Reduction	*$22,839,700*	*$28,375,300*	*$28,637,400*
Costs Avoided			
Breach Closure	$2,159,900	$2,159,900	$2,159,900
Local Beach Fill	$2,400,000	$2,400,000	$2,400,000
Other			
Recreation			
Land Loss			
Total Benefits	***$27,399,600***	***$32,935,200***	***$33,197,300***

Interest Rate 5.125%, Project Life 50 years

The summary costs associated with the beach fill alternatives are presented in Table 7-49. The total investment costs include real estate costs, contingencies, and allowances for Engineering and Design, Supervision and Administration. The total investment costs also reflect opportunity costs associated with interest during construction.

Table 7-49. Annual Costs: Beach Fill Alternatives

Cost Category	Beach Fill +13'	Beach Fill +15'	Beach Fill +17'
Total Project			
Total First Cost	$188,203,700	$197,689,400	$220,024,700
Total IDC	$15,675,100	$16,470,900	$18,347,900
Total Investment Cost	*$203,878,800*	*$214,160,300*	*$238,372,600*
Interest and Amortization	$11,384,200	$11,958,300	$13,310,265
Operation & Maintenance	$2,883,000	$2,883,000	$2,883,000
BCP Maintenance	0	0	0

Cost Category	Beach Fill +13'	Beach Fill +15'	Beach Fill +17'
Monitoring			
Renourishment	$18,535,300	$18,544,800	$18,512,360
Total Budgeted Cost	*$32,802,500*	*$33,386,000*	*$34,705,600*
Annual Breach Closure Cost	0	0	0
Major Rehabilitation	Pending	Pending	Pending
Total Additional Cost	*$0*	*$0*	*$0*
Total Annual Cost	***$32,802,500***	***$33,386,000***	***$34,705,60***

Interest Rate 5.125%, Project Life 50 years

Table 7-50, which presents the net benefits and benefit-cost ratios of the three beach fill alternatives, indicates that when considered over the full length of the study area shoreline, all three alternatives would be cost-effective in reducing storm damage with the +15 ft Plan as the Alternative which maximizes net benefits. However, on closer inspection it is apparent that beach fill alternatives do not approach cost-effectiveness for some individual component areas of the project. Only those alternatives involving beach fill along the Great South Bay and Moriches Bay Project Reaches return benefits in excess of costs when considered on an individual basis. Therefore the most cost-effective beach fill alternatives would not include the placement of fill in the Shinnecock Bay, Ponds, or Montauk Project Reaches. Hence, the beach fill alternative to be carried forward for further consideration is that including fill to a +15' NGVD crest elevation in Great South Bay and Moriches Bay Project reaches.

Table 7-50. Net Benefits and BCRs: Beach Fill Alternatives

	Beach Fill +13'	Beach Fill +15'	Beach Fill +17'
Total Project			
Total Annual Cost	$32,802,494	$33,386,047	$34,705,592
Total Benefits	$28,990,046	$33,412,259	$33,703,635
Net Benefits	-$3,812,449	$26,212	-$1,001,958
Benefit-Cost Ratio	0.88	1.00	0.97
Great South Bay			
Total Annual Cost	$18,278,991	$18,768,383	$19,580,150
Total Benefits	$21,293,935	$24,292,757	$24,498,020
Net Benefits	$3,014,944	$5,524,374	$4,917,871
Benefit-Cost Ratio	1.16	1.29	1.25
Moriches Bay			
Total Annual Cost	$6,242,411	$6,242,104	$6,556,257
Total Benefits	$5,717,182	$6,551,623	$6,572,147
Net Benefits	-$525,229	$309,519	$15,890
Benefit-Cost Ratio	0.92	1.05	1.00
Shinnecock Bay			
Total Annual Cost	$5,035,565	$5,068,009	$5,126,690
Total Benefits	$1,443,115	$1,955,522	$1,982,837
Net Benefits	-$3,592,450	-$3,112,487	-$3,143,853

	Beach Fill +13'	Beach Fill +15'	Beach Fill +17'
Benefit-Cost Ratio	0.29	0.39	0.39
Ponds			
Total Annual Cost	$2,327,357	$2,332,877	$2,505,470
Total Benefits	$268,523	$306,882	$326,063
Net Benefits	-$2,058,834	-$2,025,994	-$2,179,407
Benefit-Cost Ratio	0.12	0.13	0.13
Montauk			
Total Annual Cost	$2,191,690	$2,233,898	$2,344,466
Total Benefits	$267,291	$305,474	$324,567
Net Benefits	-$1,924,399	-$1,928,423	-$2,019,899
Benefit-Cost Ratio	0.12	0.14	0.14

Interest Rate 5.125%, Project Life 50 years

Alignment

As mentioned above, this analysis was undertaken for alternative alignments located on the most-seaward alignment. In terms of economic analysis, the benefits provided from a similar scale project located further landward would be comparable. Therefore in evaluating the cost-effectiveness of various alignments it is possible to simply compare the annual costs of the alternate alignments with the alternative costs presented above.

In addition to developing alternatives along the MREI alignment, alternatives were also developed for the unconstrained and middle alignments. To do a comparison of costs for comparable coastal storm damage risk reduction (i.e. the medium-scale plan), the volumes and costs for this medium-scale plan were developed along the unconstrained alignment, and the middle alignment. The associated volume and material costs are provided in Table 7-51.

Table 7-51. Volume and Fill Costs for Beachfll Alternatives

Design Reach	Reach Name	UC Small	Mid Medium	MREI Medium	MREI Large
GSB-1A	RMSP	502,580	502,581	502,580	502,580
GSB-1B	FILT	117,705	117,705	117,705	117,705
GSB-2A	Kismet to Lonelyville	657,997	1,239,987	1,932,004	2,137,202
GSB-2B	Town Beach to Corneille Estates	239,393	882,642	1,194,991	1,306,581
GSB-2C	Ocean Beach to Seaview	0	86,366	438,078	509,797
GSB-2D	OBP to POW	481,606	847,987	1,458,417	1,613,662
GSB-3A	Cherry Grove	0	0	0	0
GSB-3C	Fire Island Pines	840,961	1,114,379	1,504,322	1,631,764
GSB-3D	Talisman to Water Island	3,977	4,230	4,919	4,917
GSB-3E	Water Island	305	3,193	3,193	8,516
GSB-3F	Water Island to Davis Park	0	0	0	0
GSB-3G	Davis Park	74,720	262,029	609,481	714,220
GSB-4B	Old Inlet	693,505	693,507	693,507	693,505
MB-1A	SPCP-TWA	127,908	127,908	127,908	127,908

Design Reach	Reach Name	UC Small	Mid Medium	MREI Medium	MREI Large
MB-1B	SPCP	24,881	24,881	24,881	24,881
MB-2C	Cupsogue	45,458	45,458	45,458	45,458
MB-2D	WHPTIN Pikes	152,144	152,144	242,969	345,400
MB-2E	WHPTIN East	0	0	0	0
SB-1B	Sedge Island	131,461	131,461	131,461	131,461
SB-1C	Tiana	260,987	260,987	260,987	260,987
SB-1D	Shinnecock Inlet Park- West	234,248	234,248	234,248	234,248
SB-2B	WOSI	4,529	189,440	191,710	288,155
P-1G	Potato Road	774,617	837,847	837,847	1,085,586
M-1F	Montauk Beach	1,016,285	1,106,488	1,142,115	1,339,345
Total		**6,385,268**	**8,865,469**	**11,698,780**	**13,123,879**

Real Estate Impacts of Alternative Beach Fill Plans

The approximate number of structures impacted by each alternative plan is summarized in Table 7-52. Note this table includes structures impacted in Fire Island as well as Planning Units farther east all the way to Montauk Beach. This estimate is based on a structures database based on the 1995 base maps, updated by visual inspection based upon 2004 aerials. In some instances, there may be additions or deletions which are not captured completely, but should be a reasonable estimate of the number of structures impacted, with the acknowledgement that a thorough inventory would still be required for final design. The following table shows the number of structures under two acquisition scenarios – acquiring all structures on the dune, or not acquiring structures located on the landward slope of the dune.

In identifying the Real Estate Impacts associated with each of these alignments, consideration was given to the footprint necessary to construct the project. Typically, it is the Corps' practice to identify the entire dune footprint as the necessary real estate to be acquired for construction, often an additional buffer of 25 ft landward of the landward toe of the dune is included, to provide a buffer consistent with the State's CEHA definition of a dune. In the development of these plan alternatives, these requirements were re-examined to determine if there would be other options available to reduce the necessary real estate acquisition for these alternatives to see if there is more cost-effective means to implement the more landward alternatives. The two considerations are the necessary real estate in the alongshore extent, and in the cross-shore extent. For each of the alternatives considered, the plans do not require construction of a continuous dune, in many areas the existing dune meets or exceeds the design template. For identifying necessary real estate it is possible to only identify the locations where dune fill is required. Alternately, it is possible to show the necessary acquisition along what would be identified as the dune, regardless of the current condition. Since the beach and dune conditions are so variable in the alongshore extent, it was determined that it is necessary to identify the dune in its entire alongshore extent as the necessary real estate. In the cross-shore extent, consideration was given if the beach and dune could be constructed with some houses remaining on the dunes. It was identified that it would be preferable to acquire all buildings that fall within the dune and beach footprint as far landward as the landward toe of the dune. Recognizing that this could be an enormous amount of buildings, consideration was also given to a scenario which acquired only the structures which are located on the dune crest or seaward, and would allow houses to remain on the back slope of the dune. This approach would not be preferable, but as shown in Table 7-52, this approach can dramatically reduce the number of structures impacted by the various project alternatives.

Table 7-52 Real Estate Impacts (number of structures)

Structures on the Back Dune Slope?	Number of Structures Impacted by Beach Fill Plan		
	UC-Small	MID-Medium	MREI-Large
NO	256	199	66
YES	262	62	22

The alternative costs were developed for each of these plans, considering these two different assumptions. The Real Estate costs along Fire Island were developed using a gross method for mass valuation that took into consideration comparable sales in the area, adjusted to current price levels. This approach is a reasonable estimate of costs when differentiating between alternatives on this scale, but is not sufficient for providing the accuracy necessary for supporting a final, recommended plan. A gross appraisal will be conducted for the selected alternative.

A summary of the annual costs is shown in Table 7-53, which indicates that for the 15 ft dune alternative, at a middle alignment, the annual costs are comparable if structures can remain on the back slope of the dune. Costs are not comparable, if all structures on the dune would be required to be acquired.

Table 7-53 Real Estate Impacts (costs)

Cost Category	Beach Fill +15' MREI'	Beach Fill +15' MID seaward	Beach Fill +15' Mid - All
Total Annual Cost	***$33,386,000***	***$30,556,600***	***$31,400,000***

<u>Compatibility with Restoration Measures</u>

In general, the majority of the proposed restoration measures are compatible with the beach renourishment alternatives. In many instances the proposed restoration would help contribute to the SDR effectiveness, would take advantage of reduced costs associated with the construction of the two measures together, and lastly would ensure that a desirable mosaic of habitats exists.

The restoration measures that could be implemented in conjunction with beachfill include:

1) restoration of bayside habitats (bay beach, wetland, SAV) as stand-alone measures, or in conjunction with the addition or removal of shoreline stabilization structures.

2) Restoration of ocean-front beach and dune habitat, either stand-alone, with the removal of coastal structures, or through the removal of buildings and infrastructure to restore dune habitat, or allow for more natural dune functioning.

Evaluation of Beachfill Alternatives.

Beachfill Alternatives were evaluated in relationship to the planning criteria developed to reflect the Project objectives. This systematic assessment ensures that the Vision Statement approach is fully integrated into the development and selection of the FIMP Plan. Table 7-54 provides a summary of the evaluation of these measures relative to the established criteria.

Table 7-54. Evaluations

Evaluation Criteria	Assessment	Rating
The plan or measure provides identifiable reductions in risk from future storm damage.	Reduces potential for breach and overwash; reduces the risk of damages to structures directly on the shorefront	Full
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science are considered a lower priority.	Beach fill has been widely used on south shore of Long Island and other locations. It is based on sound science and is readily reversible.	Full
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	Addresses open coast storm surge and periodic overwash and breaching of barrier islands.	Partial
The plan or measures incorporate appropriate non-structural features provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity	While it is not a non-structural measure, it does help to restore littoral transport.	N/A
The plan or measure help protect and restore coastal landforms and natural habitat.	At selected locations, reduces erosion and thus protects adjacent habitat.	Partial
The plan avoids or minimizes adverse environmental impacts	The selection of borrow areas, limits in dredging windows and other mitigation measures will reduce impacts.	Partial
The plan addresses long-term demands for public resources.	Plan will require renourishment and future expenditure.	Full

Evaluation Criteria	Assessment	Rating
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	Promotes dune formation and longshore transport. In some areas, it reduces cross-shore transport because of higher dunes. Significant environmental effects will be minimized by selective implementation and avoidance of certain areas.	Partial
The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures	(See discussion of Groins). Use of beach nourishment likely to be a prerequisite for alteration of existing shoreline stabilization structures.	Partial
The plan or measure incorporates appropriate alterations of inlet stabilization measures and dredging practices	(See discussion of Inlets)	N/A
The plan or measure is efficient and represents a cost effective use of resources	The benefit/cost ratio has been established, and the alternatives are cost-effective in certain section of study area, but not the entire area.	Partial
The plan or measure reduces risks to public safety.	The plan reduces breaching and overwash; reduces damages to shorefront buildings; reduces debris volumes; and eliminates potential hazard of buildings on public beach (by moving the beach shoreward of existing structures).	Full

Areas for Sediment Management Consideration

As described in the sediment management section, there could be additional areas, where consideration of sediment management measures may be warranted. The results of the analysis of beachfill alternatives shows that beachfill is not supported in areas along Shinnecock Bay, the Ponds, or Montauk.

Knowing this, a last added analysis was considered to determine if there are any areas of high damage in the without project condition, where sediment management measures would be warranted to ensure the long-term continuity of longshore sediment transport. With this criteria, 2 locations were evident, the area of downtown Montauk and the area of Potato Road, which were evaluated for beachfill alternatives, based upon the high damages that occur in these areas.

The Littoral Sediment Transport (LST) material for regional sediment budget balance assumes the continued bluff erosion at Montauk to supply material to the west. As the bluff at both Montauk Point and eastern Atlantic shoreline are gradually stabilized, the constant source of littoral material will diminish within the life of NED plan. The LST rate is estimated at 120,000 CY/year based on the recent (c.2001) regional sediment. It is proposed that approximately 80% of the LST rate be supplemented on Montauk Beach as a feeder beach. This supplemental sediment source would provide a constant LST source east of Shinnecock Inlet and, therefore, erosion control benefit in this region. This feeder beach would include an advance fill of 450,000 CY placed during initial construction and approximately 400,000 CY placed every four years in concert with future renourishment operation.

In this area, a feeder beach would offset the long-term erosion trend, maintain the current shore protection in these areas, and prevent conditions from getting worse. This feature was evaluated in the economics model to determine the economic effect of reducing the long-term erosion trend. The results of this analysis shows that in this area, sediment management measures is economically viable.

Land and Development Management Challenges and Opportunities.

The beachfill plans introduce a number of land use and development management challenges, and also land use and development management opportunities that could be considered in conjunction with these alternatives.

Along the shorefront area, the existing land management regulations that limit the investment in this high risk area have not proven to be effective. The stabilization of the shoreline with a beachfill and dune plan could increase the need for these land management measures to function properly, to avoid an increase in the level of infrastructure that is at risk in these areas. The focus of these efforts would be to ensure the existing regulations are functioning as intended to limit the level of investment in these high hazard areas.

Also in conjunction with these beachfill plans, there is the opportunity to address existing development that is at risk, and opportunities for reducing the amount of infrastructure at risk, over time.

There are several locations where beach nourishment is included to reduce risk to public infrastructure, most notably in Robert Moses State Park, and Smith Point County Park. Opportunities exist to provide for relocation of public infrastructure in these areas to reduce the long-term requirement for renourishment.

Similarly, the beachfill alternatives have been developed to consider different beachfill alignments. To build these more landward alignments would require acquisition of buildings, prior to construction. The possibility exists for alternatives which could acquire structures over time, to reduce the level of infrastructure at risk along the shorefront.

Summary of Beachfill Alternatives.

The analysis above shows that beachfill alternatives are cost-effective storm damage reduction alternatives that contribute to reducing the damages associated with shorefront damages, and flooding along the backbay that occurs due to barrier island breaching.

Beachfill alternatives are recommended for further evaluation with the Medium fill plan at the MREI alignment, along the Great South Bay Reach and Moriches Bay Reach. If locally supported, the Medium Plan along the middle alignment could also be developed further.

In Downtown Montauk, although a traditional beachfill plan is not supported, a sediment management measure, which offsets the long-term erosion rate, would be supported. The long shore transport (LST) material for regional sediment budget balance depends on the assumption that bluff erosion at Montauk Point would supply necessary source. As the bluff at both Montauk Point and eastern Atlantic shoreline are gradually stabilized, the constant source of littoral material will diminish within the life of NED plan. The LST rate is estimated at 120,000 CY/year based on the recent (c.2001) regional sediment budget It is proposed that approximately 80% of the LST rate be supplemented on Montauk Beach as feeder beach. This supplemental sediment source would provide a constant LST source east of Shinnecock Inlet and, therefore, erosion control benefit in this region. An advance fill of 450,000 CY will be placed during initial construction and approximately 400,000 CY placed every four years in concert with future renourishment operation.

The beachfill alternatives partially fulfill the vision objectives. The vision objectives could be better accomplished with the inclusion of restoration measures, and further consideration of locations along the Great South Bay Reach and Moriches Bay Reach where beachfill could be eliminated and replaced with a breach response plan.

7.7.5 Sediment and Inlet Management Measures

General.

At each of the three inlets, multiple alternatives were identified to evaluated in addition to the existing authorized project to increase sediment bypassing, increase stability to adjacent shorelines and maintain navigability. The analysis of alternatives utilized a fatal flaw analysis, and a screening analysis to focus on alternatives to be developed more fully (which is presented in the Secondary Screening). This resulted in the consideration of eight alternatives for Shinnecock Inlet, four alternatives for Moriches Inlet and four alternatives at Fire Island Inlet. These alternatives were modeled and priced to identify the optimal means to accomplish the objectives identified above. The result of this analysis is the recommendation that the most cost-effective means to achieve bypassing is through additional dredging of the ebb shoal, outside of the navigation channel, with downdrift placement. This operation would be undertaken in conjunction with the scheduled Operations and Maintenance (O&M) dredging of the inlets.

List of Screened Alternatives

As presented in the Secondary Screening, based on the rankings of the alternatives from the MCDA screening process, and comments and discussions with state agencies, a final shortlist of screened alternatives has been developed. The list of alternatives for each inlet is shown in Table 7-55. The alternatives are listed in order from most-flexible, including soft-structural measures to measures that include both hard and soft structural features. The detailed design includes estimates of restoration of longshore sediment transport, local and regional improvements and impacts, impacts or improvements on the navigation system, and estimated annual costs for alternative comparisons.

Table 7-55. Inlet Modification Alternatives

LOCATION	INLET ALTERNATIVES
Shinnecock Inlet	Alt 1. Authorized Project (AP) + Dredging the Ebb Shoal
	Alt 2. AP + Dredging the Flood Shoal
	Alt 3. AP + Offshore Dredging for West Beach
	Alt 4. AP with Reduced Dimensions of Deposition Basin
	Alt 5. AP + Semi-fixed Bypass System (Either Stationary or Truck-Mounted)
	Alt 6. AP + Shortening the East Jetty
	Alt 7. AP + West Jetty Spur
	Alt 8. AP + Nearshore Structures
Moriches Inlet	Alt 1: Authorized Project (AP)
	Alt 2: AP + Dredging the Ebb Shoal
	Alt 3: AP + Dredging the Flood Shoal
	Alt 4: AP + Semi-fixed Bypass System (Either Stationary or Truck-Mounted)
Fire Island Inlet	Alt 1: Authorized Project (AP) / Existing Practice (EP)
	Alt 2: AP/EP + Dredging the Ebb Shoal
	Alt 3: AP/EP + Existing Practice plus Dredging the Flood Shoal
	Alt 4: Existing Practice Plus Discharge Farther West
	Alt 5: Optimized Channel and/or Deposition Basin Configuration

Shinnecock Inlet

Table 7-56 summarizes the costs for each alternative developed further. Costs associated with existing conditions (dredging the authorized project dimensions every four years) are also shown for reference. According to this table, the least expensive alternatives are those that maintain the Authorized Project features and offset the existing sediment deficit (40,000 m^3/yr or 52,000 cy/yr) by dredging the ebb shoal or the flood shoal on a 4 year cycle. In fact, dredging the inlet shoals appears to be the only effective and reliable way to completely eliminate this deficit. Other alternatives do not achieve a 100% reduction (i.e., semi-fixed bypassing plant) or carry too much uncertainty (i.e., shortening the east jetty). This alternative is also adaptable, in that it provides the ability to implement a full range of alternatives over the project life without dictating a future course of action that would be difficult to change.

Table 7-56. Summary of Costs for Shinnecock Inlet Alternatives

Plan	First Costs ($1,000's)	Average Annual Costs ($1000's)
Existing Conditions (dredging every 4 years)	-	*$2,646*
Alt 1A. AP + Ebb Shoal Dredging every 4 years	-	$2,059
Alt 1B. AP + Ebb Shoal Dredging every 2 years	-	$2,851
Alt 2A. AP + Flood Shoal Dredging every 4 years	-	$2,059
Alt 2B. AP + Flood Shoal Dredging every 2 years	-	$2,888
Alt 3. AP + Offshore Dredging for West Beach	-	$3,978
Alt 4A. -18 ft MLW Deposition Basin	-	$2,911
Alt 4B. -16 ft MLW Deposition Basin	-	$3,459
Alt 5. AP + Semi-fixed Bypass System	$4,633	$3,462
Alt 6. AP + Shortening the East Jetty	$2,167	$5,085
Alt 7. AP + West Jetty Spur	$6,629	$3,042
Alt 8. AP + Nearshore Structures (T-groins)	$25,642	$3,868

Overall, dredging the shoals outside the limits of the channel and deposition basin would entail very little risk and uncertainty as compared to others since it involves continuation of existing practice under the authorized project dimensions and bypassing would be improved using proven dredging technology with relatively well known costs, schedules, performance, and environmental effects. Uncertainty regarding accurate estimates of ebb shoal growth and its effects on the sediment budget and long shore sediment transport processes could be managed through regular monitoring surveys of the ebb shoal and dredging in areas of observed growth. Potential impacts on nearshore waves and littoral processes, which modeling results suggest are insignificant, can be also be minimized by monitoring future morphological changes and managing the dredging program accordingly.

The Authorized Project combined with dredging the inlet shoals also offers the advantage of being reversible, particularly in the case of the ebb shoal. Morphological modeling simulations suggest that the shoals would recover over time, and neither alternative requires a new capital improvement or significant upfront costs. Of the two shoals, dredging the ebb shoal is the preferred option because it reduces uncertainty and potential environmental impacts. Dredging the ebb shoal would offset the existing longshore sediment transport deficit and restore (in terms of average volume per year) longshore sediment transport processes downdrift of the inlet. Continued dredging of the deposition basin would be used to mitigate local erosion of the West Beach. Depending on future performance, which would be assessed by regular monitoring surveys, part of the sediment from the deposition basin could be placed farther downdrift beyond the ebb shoal attachment point. Conversely, ebb shoal material could be occasionally placed on the West Beach if necessary. Continued dredging of the deposition basin to -20 ft MLW would maintain navigation reliability through the inlet.

One potential disadvantage of dredging the shoals is lack of bypassing continuity, particularly on a 4 year cycle. However, a 2-year cycle could be combined with a shallower deposition basin (at -16 ft MLW) to provide for cost effective solution that would improve continuity and eliminate the LST deficit across the inlet. Only shortening the east jetty (dredging on 1 year cycle) or a bypassing plant could provide for more continued bypassing. However, both would be more expensive, less reliable, and irreversible. A two-year dredging cycle would also be much closer to the 1.5-year cycle originally anticipated in the current project authorization. This trade-off between more continues bypassing and slightly increased average annual costs could be managed and modified, if necessary, in the future depending on actual performance and costs.

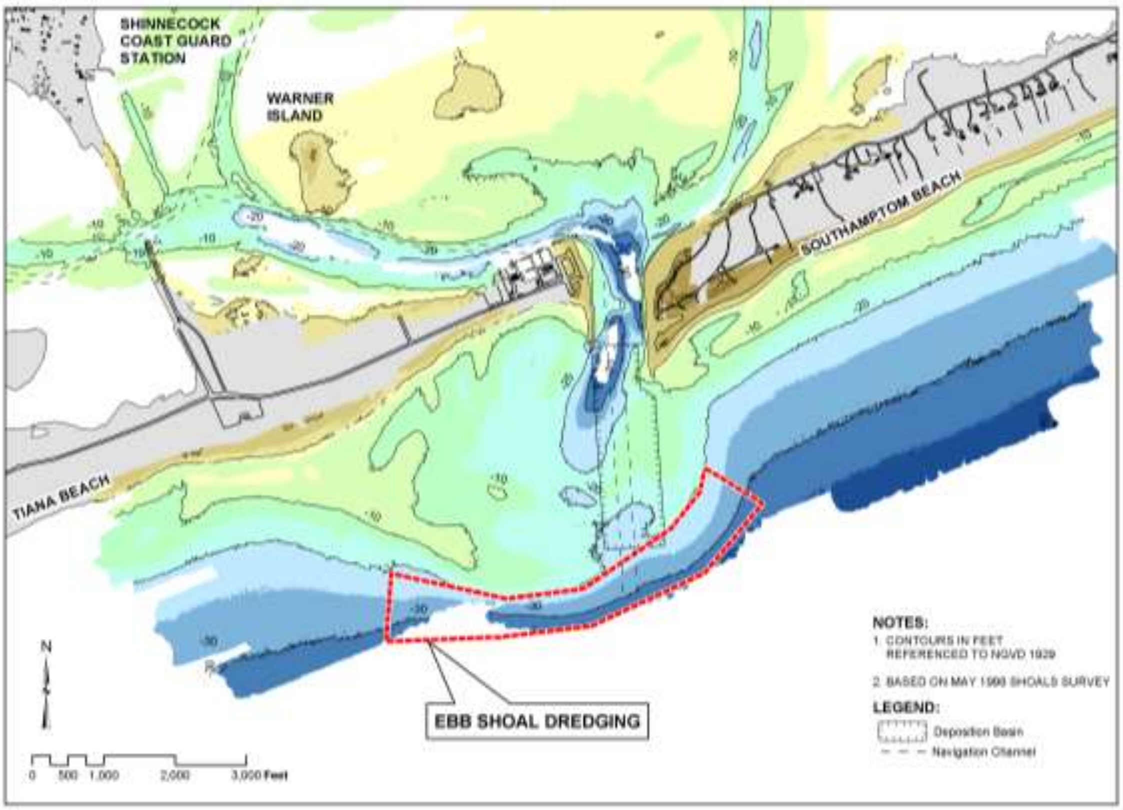


Figure 7-11. Recommended Alternative for SI: -16 ft MLW DB + Ebb Shoal Dredging

Costs for this recommended alternative combining dredging of the ebb shoal and a shallower deposition basin are presented in Table 7-57. Note that dredging both the deposition basin and the ebb shoal at the same frequency (i.e., one mobilization) and eliminating the costs of the deficit in longshore sediment transport brings the cost of this alternative below that of Existing Conditions, despite doubling the dredging frequency.

Table 7-57. Costs for SI Recommended Alternative: -16 ft MLW DB + Ebb Shoal Dredging

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($1000's)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($1000's)	E&D and S&A ($1000's)	Total Cost Per Operation ($1000's)	Average Annual Cost ($1000's)
Channel & Deposition Basin Dredging	2	130 (170)	$2,500	$6.90 ($5.30)	$3,911	$644	$4,555	$2,445
Ebb Shoal Dredging	2	80 (105)	Same Contract	$6.90 ($5.30)	$640	$116	$756	$406
							Grand Total	$2,851

Other potentially negative issues associated with the other alternatives aside from the increases annual costs are summarized as follows.

Dredging the flood shoal (Alt. 3) would be very similar in terms of meeting the stated goals to the selected alternative, however, it does have increased uncertainty with regards to morphodynamics, optimum dredging rates, and its effects on the sediment budget may be more difficult to understand and manage than in the case of dredging the ebb shoal. In addition, modeling results show that flood shoal dredging, if significant in extent and depth, may induce some hydrodynamic impacts that extend beyond the dredging footprint potentially affecting navigation and increasing tidal prism through the inlet (i.e., potentially increased flood elevations). There is also more uncertainty regarding sediment compatibility. Typically, ebb shoal sediments are very compatible with the beach material, whereas the flood shoal sands tend to be finer. Finally, flood shoal dredging would have to be performed closer to environmentally sensitive areas.

Offshore dredging (Alt. 4) combination with continued dredging of the deposition basin would mitigate local erosion of the West Beach, offset the existing longshore sediment transport deficit but accumulation of sand in the shoals and adjacent beaches would continue. Therefore, unlike Alternative 1 (ebb shoal dredging), this alternative does not “balance” the sediment budget by reducing accumulation within the inlet.

A semi-fixed bypassing plan (Alt. 5) in combination with continued dredging of the deposition basin would mitigate local erosion of the West Beach and partially offset the existing longshore sediment transport deficit. However, some accumulation of sand in the shoals and adjacent beaches would continue and downdrift erosion would not be fully mitigated unless there is also placement from offshore. Continued accumulation in the ebb shoal is consistent with experience at Indian River Inlet, where recent surveys suggest that the ebb shoal has continued to grow despite continuous bypassing.

Capacity would be a potential issue for this alternative. The actual bypassing rate for the plant at Indian River Inlet between 1990 and 2006 has been somewhat lower than anticipated (approximately 60,000 m3/yr), and although lessons learned at this facility could be applied at Shinnecock Inlet and equipment improvements could be made, it is clear that capacity will be more of an issue for this alternative than for dredging alone. Source flexibility may also be a problem in that the area that can accessed by the crane and jet pump is limited. Finally, the initial investment required would not be recoverable.

Shortening the east jetty (Alt. 6) offsets the LST deficit and partially mitigates local erosion of the West Beach through increased dredging and placement frequency. On the other hand, navigation through the inlet would be likely to deteriorate because of the increased influx of sediments from the east. Modeling results indicate that under large storm conditions channel depths could be reduced rapidly. The jetty could obviously be shortened a smaller distance to better balance navigation and dredging/bypassing needs. However, a similar result could be accomplished by reducing the depth of the deposition basin and increasing dredging frequency. Moreover, the latter would be easily reversible while the former would not. There is also a significant amount of uncertainty regarding the actual effect that shortening the jetty would have on shoaling and navigation conditions within the channel and deposition basin.

A spur of the west jetty (Alt. 7) would completely stabilize the West Beach, sand placement in this area is likely to be required in the future. More importantly, modeling results show that accumulation in the deposition basin would be reduced as compared to existing conditions. Some of the material (approximately 10,000 m3/yr) previously deposited in the deposition basin appears to be carried farther offshore and deposited on the seaward edge of the downdrift ebb shoal lobe. Model results suggest that the slightly increased training of the ebb jet as a result of spur construction is the cause of this change. Finally, this alternative is worse than other with regards to environmental impacts because it requires a structure.

Constructing the T-groins (Alt. 8) would essentially eliminate the chronic erosion problem along the West Beach and it would free up the most of the sand now being placed there to be directly bypassed to the beaches downdrift of the inlet. However, it is uncertain what their net effect would be on the sediment budget and whether or not the existing longshore sediment transport deficit would be reduced. More importantly, like Alt. 7 (Spur), the T-groins are considered to have a significantly greater environmental impact.

Moriches Inlet

Table 7-58 summarizes the costs for each alternative. Costs associated with existing conditions (dredging the authorized project dimensions every four years) are also shown for reference. Similarly to Shinnecock Inlet, the least expensive alternatives are those that maintain the Authorized Project features and offset the existing LST deficit (56,000 m^3/yr or 73,000 cy/yr) by dredging the ebb shoal or the flood shoal. *Existing Conditions,* which include dredging to the authorized project dimensions every four years on average (instead of the yearly dredging frequency established in the authorized project), is actually the least costly alternative, but it does not meet the goal of reliable navigation.

Table 7-58. Summary of Costs for Moriches Inlet Alternatives

Plan	First Costs ($1,000s)	Average Annual Costs ($1,000s)
Existing Conditions (dredging every 4 years)	-	*$2,983*
Alt 1. Authorized Project (1 yr cycle)	-	$5,709
Alt 2. AP + Ebb Shoal Dredging (1 yr cycle)	-	$4,966
Alt 3. AP + Flood Shoal Dredging (1 yr cycle)	-	$4,966
Alt 4. AP + Semi-fixed Bypass System	$4,633	$6,320

Maintaining reliable navigation would require more frequent dredging, as anticipated in the design of the authorized project, which recommended a one year dredging cycle. Recent data confirms that the deposition basin can be completely filled within months of dredging. For example, by October 2004 (i.e., 8 months after dredging in February 2004) the shoal had formed again over the channel and deposition basin and dredging was required. Only the dredging in 1998 seemed to last a little longer, although a survey in the summer of 2000 already showed the ebb shoal bar encroaching on the channel from the east, with depths shallower than -10 ft MLW.

Similarly to Shinnecock Inlet, dredging the ebb shoal on a regular cycle (1 year) to increase bypassing has less risk and uncertainty as compared to other alternatives since it involves continuation of existing practice under the authorized project dimensions and bypassing would be improved using proven dredging technology with relatively well known costs, schedules, performance, and environmental effects. A more detailed breakdown of the costs associated with this recommended alternative is presented on Table 7-59.

Table 7-59 Costs for MI Recommended Alternative: AP + Ebb Shoal Dredging (1 yr cycle)

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($1,000s)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($1,000s)	E&D and S&A ($1,000s)	Total Cost Per Operation ($1,000s)	Ave. Annual Cost ($1,000s)
Channel & Deposition Basin Dredging	1	75/ (98)	2,500	7.80/ (6.00)	3,551	588	4,139	4,341
Ebb Shoal Dredging	1	60/ (73)	Same contract	7.80/ (6.00)	504	92	596	625
							Grand Total	4,966

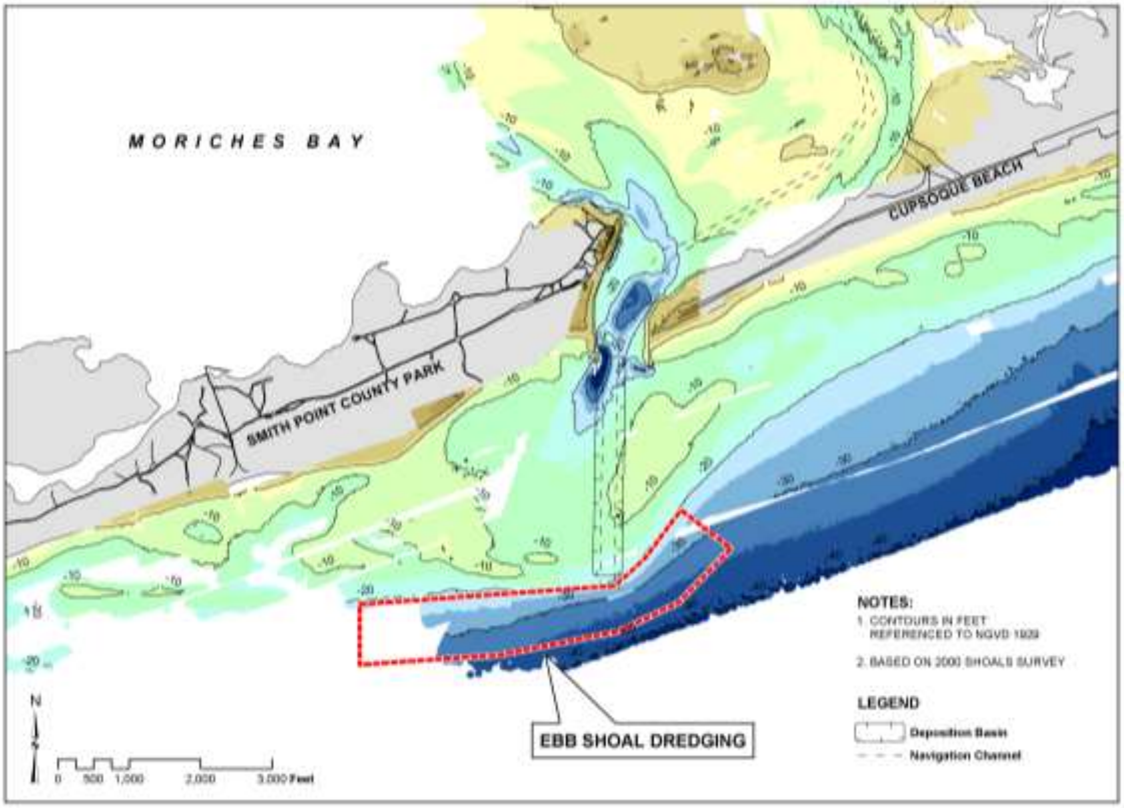


Figure 7-12. Recommended Alternative for MI: AP + Ebb Shoal Dredging

Arguably, increasing the deposition basin dimensions could be used to maintain a channel for at least one year or perhaps even two and thus to reduce average annual costs and improve navigation. However, a larger deposition basin may have unintended effects on the sediment budget for the inlet. Nonetheless, actual performance of the project on a 1-year dredging cycle should be monitored and, if needed, the dimensions and/or layout of the deposition basin could be reassessed. Dredging the flood shoal (Alt. 3) instead of the ebb shoal has similar drawbacks at Moriches than Shinnecock Inlet: increased uncertainty with regards to morphodynamics, optimum dredging rates, effects on the sediment budget and potential impacts on hydrodynamics and flooding.

The main drawbacks of the semi-fixed bypass system (Alt. 4) are capacity and costs. At Moriches Inlet the net westerly longshore sediment transport immediately updrift of the inlet is 238,000 m^3/yr, which is more than double the capacity of this type of bypassing systems (estimated at 100,000 m^3/yr). Therefore, with a semi-fixed bypassing plant annual dredging in the channel and deposition basin will continue to be required, albeit at a reduced rate. More importantly, sediment would continue to accumulate in the inlet shoals since the system would not capture and transfer 100% of the littoral drift. The resulting deficit, also somewhat reduced from existing conditions, would still have to be offset by periodic dredging from other sources (e.g., offshore). Note that combining ebb shoal dredging with a semi-fixed bypassing plant would also offset the LST deficit, but at a higher cost than dredging alone. A semi-fixed bypassing plant would provide for more continuous bypassing. However, continuity is not as much of issue for the dredging alternatives in this case given the recommended yearly dredging cycle. Dredging also allows for flexibility by, for example, potentially extending the interval between dredging events during relatively calm wave years such as the 1998 to 2000 period. Finally, it provides the ability to implement a full range of alternatives at throughout the project life.

Fire Island Inlet

Table 7-60 summarizes the costs for each shortlisted alternative. Note that Alternative 1 essentially represents continuation of the existing practice under the current, multi-purpose, project authorization. According to the table, all four alternatives have similar costs although 1 and 4 area slightly more costly because the need to offset the estimated LST deficit (145,000 m^3/yr or 190,000 cy/yr) by means offshore dredging instead of dredging the ebb shoal or flood shoal.

Table 7-60. Summary of Costs for Fire Island Inlet Alternatives

Plan	First Cost ($1,000s)	Average Annual Costs ($1,000s)
Alt 1: Authorized Project Dimensions (APD) / Existing Practice (EP)	-	$11,648
Alt 2: APD/EP + Dredging the Ebb Shoal	-	$10,054
Alt 3: APD/EP + Dredging the Flood Shoal	-	$10,054
Alt 4: Optimized Channel and/or Deposition Basin Configuration	-	$11,648

Available morphological data, model simulations, and sediment budget analyses do not suggest any significant benefits (e.g., increased bypassing, reduced maintenance dredging or improved navigation) associated with a complete realignment of the channel and/or deposition basin. However, a slightly wider deposition basin at the western tip of the existing sand spit will limit encroachment of this feature into the navigation channel at the end of each dredging cycle. Therefore, the recommended plan for Fire Island Inlet consists of combining Alternatives 1 and 4 (see Figure 7-13) and continuing the recent practice of placing all of the dredged material at least three miles west of Democrat Point.

Future placement of some of the dredged material along Robert Moses State Park (i.e., backpassing) on as needed basis depending on future shoreline changes and infrastructure protection requirements. A more detailed breakdown of the costs for this recommended plan is presented in Table 7-61. Note that the slight change in the deposition basin will not change the costs compared to Alternative 2 initial dredging in the expansion area will likely be offset with less dredging along the deposition basin farther offshore.

Table 7-61. Costs for FII Recommended Alternative: AP + Ebb Shoal Dredging & DB Expansion

Plan Component	Dredging Interval (years)	Quantity (1000x m^3 (cy))	Mob/ Demob ($1000s)	Unit Cost ($/$m^3$ ($/cy))	Subtotal Cost ($1000s)	E&D and S&A ($1000s)	Total Cost Per Operation ($1000s)	Average Annual Cost ($1000s)
Channel & Deposition Basin Dredging – Updrift Placement	2	124 (162)	$668	$6.10	$1,644	$284	$1,927	$1,035
Channel & Deposition Basin Dredging – Downdrift Placement	2	626 (819)	$2,916	$10.50	$10,983	$1,704	$12,687	$6,811
Ebb Shoal Dredging	2	290 (379)	Same contract	$10.50	$3,530	$584	$4,115	$2,209
							Grand Total	$10,054

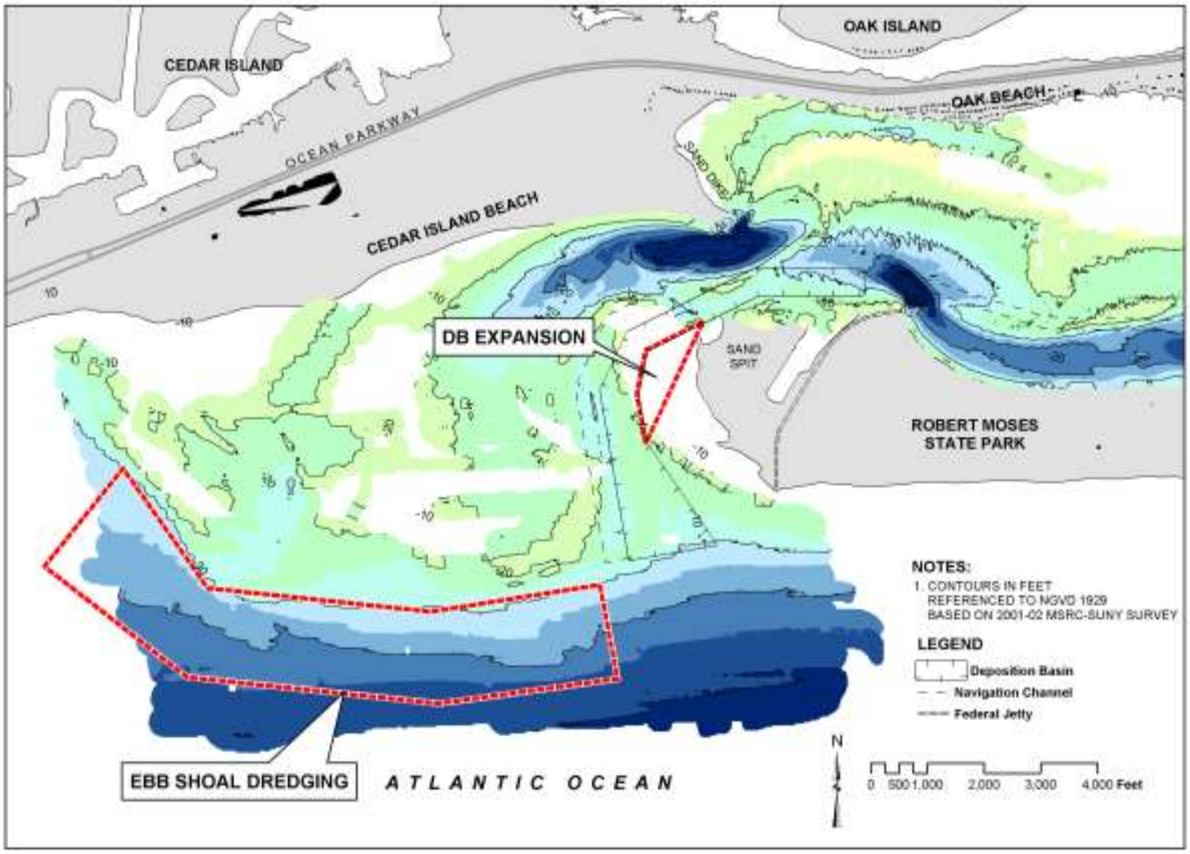


Figure 7-13. Recommended Alternative for FII: AP + Ebb Shoal Dredging & DB Expansion

As in the case of Shinnecock and Moriches Inlet, this alternative provides the most reliable, flexible, and cost-effective means for maintaining navigation and offsetting the existing LST deficit. Given the volumes and distances involved the only other feasible alternative would be to dredge the flood shoal or offshore. Dredging offshore would be more expensive and would not directly eliminate the existing sediment sink at Fire Island Inlet. Dredging the flood shoal may be technically feasible, but its dynamics are poorly understood at this time due to lack of comprehensive bathymetry data, and geomorphic, hydrodynamic, and environmental impacts associated with dredging this feature may be significant. Moreover, dredging the flood shoal, particularly in areas east of the Robert Moses Causeway, would be more costly than dredging the ebb shoal because of the increased transport distance.

Evaluation of Storm Damage Reduction Effectiveness

The reduction in storm damages arising from modifying the existing management practices at Fire Island, Moriches, and Shinnecock Inlets was modeled using the Lifecycle Damage Analysis Model to quantify back bay inundation damages, and the Breach Only Lifecycle Model to analyze the resulting change in breach-related damages. Changes to inlet management have been modeled by varying the rate of long-term erosion (through changes in profile recovery) from erosion and renourishment regimes at locations downdrift of the inlets.

As presented above, the inlet management measures were screened to identify the most cost-effective means to accomplish the desired objective at each inlet. As such, damages have been quantified for one inlet management alternative, and the results are presented in Table 7-62.

Table 7-62. Annual Damages: Inlet Management Alternative

Damage Category	**Inlet Management**
Total Project	
Tidal Inundation	
Mainland	$73,957,400
Barrier	$12,956,100
Total Inundation	*$86,913,500*
Breach	
Inundation	$9,114,600
Structure Failure	$395,700
Total Breach	*$9,510,300*
Shorefront	$7,388,900
Public Emergency	
Other	
Total Storm Damage	$103,812,700

Interest Rate 5.125%, Project Life 50 years

These damages have been compared to those associated with the without-project condition to generate the project benefits, which are presented in Table 7-63.

Table 7-63. Annual Benefits: Inlet Management Alternative

Benefit Category	**Inlet Management**
Total Project	
Tidal Inundation	
Mainland	$278,100
Barrier	$42,500
Total Inundation	*$320,600*
Breach	
Inundation	$127,900
Structure Failure	$0
Total Breach	*$127,900*
Shorefront Damage	$0
Public Emergency	
Other	
Total Storm Damage Reduction	*$448,500*
Costs Avoided	
Breach Closure	$336,900
Beach Maintenance	
Other	
Land Loss	
Total Benefits	***$785,400***

Interest Rate 5.125%, Project Life 50 year

Table 7-64. Incremental Annual Cost: Inlet Management Alternative

	Annual Cost	Incremental Annual Cost
Fire Island Inlet		
Existing Practice (dredging every2 years)	*$7,077,000*	
Ebb Shoal & Deposition Basin (expanded) dredging on 2-yr cycle	$9,077,000	$2,220,000
Moriches Inlet		
Existing Practice (dredging every 4 years)	*$1,022,000*	
Ebb Shoal & Deposition Basin (AP dimensions) dredging on 1-yr cycle	$2,803,000	$3,353,000
Shinnecock Inlet		
Existing Practice (dredging every 4 years)	*$1,033,000*	
Ebb Shoal & Deposition Basin (at -16 ft MLW) dredging on 2-yr cycle	$1,726,000	$1,221,000
Project Total		**$6,794,000**

Table 7-65. Detailed Incremental Annual Cost: Inlet Management Alternative

	Inlet Management – Incremental Annual Costs			
	Fire Island Inlet	Moriches Inlet	Shinnecock Inlet	Total Cost
First Cost	*$0*	*$0*	*$0*	*$0*
IDC	$0	$0	$0	$0
Total Investment Cost	*$0*	*$0*	*$0*	*$0*
Interest & Amortization	$0	$0	$0	$0
O&M	$0	$0	$0	$0
BCP Maintenance	$0	$0	$0	$0
Monitoring	$0	$0	$0	$0
Inlet Management	$2,220,000	$3,353,000	$1,221,000	$6,794,000
Renourishment	$0	$0	$0	$0
Total Budgeted Cost	*$2,220,000*	*$3,353,000*	*$1,221,000*	*$6,794,000*
Annual Breach Closure Cost	$0	$0	$0	$0
Major Rehabilitation	$0	$0	$0	$0
Total Annual Cost	***$2,220,000***	***$3,353,000***	***$1,221,000***	***$6,794,000***

The benefits presented in Table 7-63 do not reflect the full merits of this alternative, since the benefits associated with modifications to inlet management at Fire Island Inlet are known to extend beyond the study area. In addition, since inlet management modifications represent the restoration of longshore processes, there are also significant NER benefits associated with this alternative.

For this reason, when evaluating sand bypassing, to determine if this alternative should be carried forward as an element of combined alternative plans, the following were considered:

1. There are institutional requirements that suggest the inclusion of bypassing as an alternative.
2. There are habitat restoration benefits which when considered would make bypassing more favorable, and
3. The benefits of bypassing can be greater when considered in conjunction with other storm damage reduction alternatives.

There are a number of institutional requirements that suggest that bypassing be included as a common element of all plans. The Corps' RSM initiative recognizes the scarcity of sand as resource, and the need to efficiently use this material to achieve multiple purpose objectives. Sand bypassing also inherently advances the "Actions for Change" in that it promotes the natural sustainability of the system. There are additional institutional factors external to the Corps that point even more direct in the need for bypassing. The existing General Management Plan for Fire Island National Seashore states that bypassing must be implemented at Shinnecock and Moriches Inlet, as a precursor to any storm damage reduction plan being implemented on Fire Island. Additionally, the State CZM policies require consideration of alternatives to restore natural protective features to offset the impacts of existing coastal structures, prior to considering other storm damage reduction alternatives.

With respect to habitat benefits, sand bypassing is shown to be an integral element of the restoration of coastal processes. The restoration benefits that arise from bypassing are presented further in the Environmental Appendix, and add further credence for inclusion of bypassing when considering combined alternatives

Finally, it is acknowledged that including bypassing in combination with other alternatives can make the other Storm Damage Reduction Alternatives more cost-effective. As presented above, bypassing alone is limited in its capacity to reduce damages. When bypassing is taken into consideration as an element of other alternatives, it can often be considered as either 1) a more cost-effective source of material for renourishment of a project (as compared to offshore sand sources), 2) an element of the overall project which can reduce the frequency of renourishment by addressing areas prone to accelerated erosion, due to sediment deficits. Finally, using the inlet as a source of material can also be considered as a more environmentally acceptable source, relative to offshore borrow areas.

With the above considerations, it is recommended that bypassing should be included as an element common to all storm damage reduction alternatives as an element of the overall plan. It is also acknowledged that conditions are extremely variable at each of these inlets in terms of sediment trapping and bypassing efficiency, which influences the degree to which bypassing is required. The final consideration for a bypassing plan will include an extensive monitoring plan to evaluate the requirements for bypassing at each inlet, which can also be applied to evaluate the need for any adaptation to the proposed bypassing method, to achieve the objectives in a more efficient manner.

Additional Sediment Management Measures

In addition to sediment management measures at the inlets, there may be additional locations where sediment management measures are desirable, to provide for a balanced longshore sediment transport (feeder beaches). Since these alternatives would largely be dependent upon the results of the beachfill alternative analysis, the presentation of sediment management alternatives is included in the beachfill section, as a last-added analysis.

Compatible Restoration Measures

There are a number of restoration alternatives that are compatible with sediment management. In fact, sediment management itself is an alternative that can be characterized both as a storm damage reduction alternative, and a habitat restoration alternative, in that it restores longshore sediment transport.

Restoration measures that are compatible with this approach include:

1. restoration of bayside habitat (bay beach, marsh, SAV) in proximity to inlet management alternatives to provide the desired habitat mosaic, and to complement the SDR effectiveness of the sediment management alternatives.
2. Restoration of ocean dune habitat, in conjunction with sediment management alternatives, to provide optimal beach and dune habitat
3. Restoration of Ocean Beach and Dune habitat through removal or modification of coastal structures, to increase the extent of longshore transport restoration.

Evaluation of Sediment and Inlet Management Measures.

NED Criteria. The analysis does not conclusively shows that sediment management alternatives are cost-effective storm damage reduction alternatives. Sediment management measures at the inlets are recommended to be carried forward for further development. There are institutional reasons for the inclusion of bypassing as an alternative, along with habitat restoration considerations which would make bypassing more favorable. Lastly the benefits of bypassing can be greater when considered in conjunction with other storm damage reduction alternatives.

P&G Criteria. This is the evaluation of the alternatives relative to being complete, effective, efficient, and implementable. Inlet bypassing plans alone do not represent a complete solution, as they only address as small portion of the damages that arise due to the interruption of longshore transport. Relative to the purpose they are accomplishing, these alternatives are not particularly effective or efficient, when considered as a stand-alone option. These alternatives are implementable, supported by all parties, and in some instances are recommended to be alternatives included in all plans.

Vision Criteria. The Sediment and inlet management measures were evaluated in relationship to the planning criteria developed to reflect the Project. This systematic assessment ensures that the Vision Statement approach is fully integrated into the development and selection of the FIMP Plan. Table 7-66 provides a summary of the evaluation of these measures relative to the established criteria.

Table 7-66. Inlet Management Measures		
Evaluation Criteria	**Assessment**	**Rating**
The plan or measure provides identifiable reductions in risk from future storm damage.	Measures help to avoid excessive erosion in areas affected by inlets. Some of these affects have been quantified as reduced flooding.	Full
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science are considered a lower priority.	The inlet management measures are based on the observed historical inlet responses and extensive modeling of inlet dynamics and morphology. The historic records and modeling are considered less reliable for alternatives incorporating significant structural modifications of the inlets.	Full
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	Measures to improve sediment management may reduce flooding by preventing local areas of accelerated erosion, thus reducing flooding associated with periodic overwash or breaching of barrier islands.	Partial
The plan or measures incorporate appropriate non-structural features provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity	The measures modify sediment management procedures to restore transport and will help maintain both storm damage risk reduction and ecosystem integrity.	Full
The plan or measure help protect and restore coastal landforms and natural habitat.	The measures help to reduce or eliminate deficits in longshore sediment transport and are important for the protection of landforms and habitat.	Full
The plan avoids or minimizes adverse environmental impacts	Construction activities are scheduled to avoid or minimize impacts	Full
The plan addresses long-term demands for public resources.	The measures will require continued maintenance into the future to provide both safe navigation and coastal process restoration.	Partial
Dune and beach nourishment measures consider both storm damage reduction,	Locations for placement of bypassed sediment provide both	Full

Table 7-66. Inlet Management Measures		
Evaluation Criteria	**Assessment**	**Rating**
restoration of natural processes, and environmental effects.	storm damage reduction and restoration.	
The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures	NA	NA
The plan or measure incorporates appropriate alterations of inlet stabilization measures and dredging practices	Measures to alter dredging practices were considered more appropriate than structural changes to the inlets.	Full
The plan or measure is efficient and represents a cost effective use of resources	The measures provide significant economic and process restoration	Partial
The plan or measure reduces risks to public safety.	The measures maintain navigation safety and reduce storm damage risks	Full

Summary of Sediment Management Alternatives.

The recommended sediment management at each of the inlets is the continuation of the authorized project, plus the additional bypassing with material from the ebb shoal. It is recommended that this plan be continually monitored to allow for adaptive management changes in the future.

The breach response plans advance, and partially fulfill the vision objectives.

The sediment management alternatives do not introduce any specific land use and development management challenges.

7.7.6 Groin Modification Alternatives

General.

The screening of alternatives recommended further evaluation of groin modifications, as storm damage reduction alternatives. Groin modifications were considered at Georgica Pond in Easthampton, the existing groin field at Westhampton, and the existing State Groins at Ocean Beach, Fire Island. Groin modifications to shorten the groins were considered to first determine the influence that shortening of the structures would have on the release of sediment, and the resulting change in long-term erosion in adjacent areas. In analysis of these alternatives, altering the groins at Georgica Pond and at Ocean Beach do not appear as favorable for storm damage reduction. Modification of the groins at Westhampton, by shortening 12 groins between 70 and 100 feet could introduce upwards of 2,300,000 CY of sand, which could be cost-effective if shown to significantly reduce expected renourishment requirements for the interim project at Westhampton. The analysis of these three areas is presented below.

7.7.6.1 Georgica Pond, East Hampton

There exist four rubble mound groins east of Georgica Pond along the shoreline of East Hampton. The State of New York constructed two 275 ft long groins, one 700 ft east of Georgica Pond and the other 12,000 feet east of Georgica Pond, in the vicinity of Hook Pond. These two groins were constructed in 1959. The Army Corps of Engineers constructed two additional groins east of the state groin at Georgica Pond in 1964 and 1965. These groins were 480 ft long from the landward crest, elevation +14.0 MSL to the seaward crest at elevation +1.5 MSL (NGVD). Fill was placed by the state in 1960, 370,000 cubic yards over a 9800 ft length of beach at Georgica Pond

The state and federal groins at Georgica Pond have not had any maintenance since their construction. The structures have lost their trapezoidal shape and armor stone interlocking, but are still functioning. The East Hampton Town Trustees regularly open and close the inlet to Georgica Pond, for environmental and flood control purposes. In some years, the inlet is breached naturally by a storm event, and can also close naturally due to littoral transport of sand. The full impact to the coastal processes and littoral transport of material due to the opening and closing of the inlet, and the attendant creation of the flood and an ephemeral ebb shoal is not fully known at this time.

Various parties have studied the area of shoreline in the vicinity of Georgica Pond in the past. Multiple sediment budgets exist with the most recent thorough sediment budget incorporating shoreline changes up to 1995. These sediment budgets show that the gross littoral transport is three to four times larger than the net littoral transport. While average net transport is westward, single storm events and seasonal or yearly trends can set the net transport into a reversal, or to the east.

The shoreline erosion rates, up to 1995, are lower in the Southampton and East Hampton area compared to the rates of other locations in the FIMP study area. The Existing sediment budget erosion rates also shows erosion in the regional sediment budget, could not describe specifically the erosion rates in the immediate vicinity of the groins at Georgica Pond. An erosion rate of 15 feet per year is assigned to the area for use in estimating renourishment volumetric requirements and placement intervals. The objectives of the recommended alternative in the vicinity of Georgica pond is to provide storm damage prevention benefits in a cost-effective manner, reduce adverse impacts, and encourage the restoration of coastal littoral processes. Alternatives proposed already include:

- No-action
- Beach fill placement
- Removal of groins
- Modification of groins
- Change in management practices of Georgica Pond opening and closing
- Combinations of these alternatives

As presented in the Alternative Screening, a conceptual level analysis was conducted on the costs and benefits of groin removal compared to beach nourishment. For that conceptual screening, only the complete removal of the groins at Georgica Pond was examined. The report noted that a complete investigation into the feasibility or impacts of groin removal would require (1) historical shoreline and volumetric changes east and west of the structures before and after construction, (2) the contribution of the groins toward any irregularities in the existing beach layout, and (3) the groin impacts determined by the implementation of the GENESIS shoreline change model. The report also notes that it must be determined that existing storm protection in areas where groin removal would occur will not be adversely affected. The study concluded, based on a comparison with beachfill, groin removal results in increased annualized costs with no readily identifiable benefit in terms of beachfill performance. Total groin removal will not be further considered as an alternative.

Thorough engineering analyses of historical and recent shoreline change trends and their relation to the updrift groin field, the periodic tidal inlet at Georgica Pond, and the nearshore remnant shoal features must be completed in order to determine the appropriate type(s) and level of design required. As part of a legal dispute ongoing between Suffolk County and private landowners, Suffolk County acquired from Woods Hole Group such an engineering study for this area. This study is summarized in the technical report titled "Historical evaluation of shoreline change for the Georgica Pond region, Suffolk County, Long Island, New York." The engineering study conducted by Woods Hole Group included all pertinent components needed to make a quantitative assessment of coastal engineering issues upon which preliminary engineering design recommendations may be based. Specifically, this study included the following components:1) Bathymetric data collection; 2)Historical shoreline change analysis; 3)Wave climatology and wave transformation evaluation, including numerical modeling; 4)Engineering assessment of causes of erosion. Conclusions cited in the report include:

Federal groins in the vicinity of Georgica Pond do not significantly contribute to erosion well downdrift of the Pond. Instead, long-term background erosion most significantly contributes to erosion observed well downdrift of the Pond.

Wave-driven sediment transport patterns in the vicinity and downdrift of Georgica Pond are as influenced by natural offshore bathymetric features as they are by the groin field updrift of the Pond.

Based on the conclusions of this report, a no-action alternative is recommended. However, a monitoring program will be included as part of the recommended plan to determine the long-term effect of the groins at Georgica Pond and possible future modification.

7.7.6.2 Westhampton Groin Field

Provisions of the original Fire Island to Montauk Point Beach Erosion and Hurricane Protection (FIMP) Project provided for the construction of 23 rubble mound groins at Westhampton Beach, east of Moriches Inlet. Eleven groins were constructed in 1965 - 1966 and an additional 4 groins were constructed in 1969 - 1970. The remaining 8 groins, as provided for in the original FIMP project, were never constructed. The groins, spaced approximately 1250 ft apart, function as intended and continue to provide coastal storm damage risk reduction to a once vulnerable reach of barrier island shoreline approximately 2.8 miles in length. Construction of the Westhampton groin field had, however, resulted in accelerated erosion directly west of the westernmost groin, culminating in two breaches, Pikes Inlet and Little Pikes Inlet, during the Northeaster of December 1992.

The Westhampton Interim Project was designed to mitigate erosion problems occurring downdrift of the Westhampton groin field. The Interim Project provides for beachfill placement, dune construction west of the groin field, periodic beachfill renourishment, the shortening and lowering of the final two groins on the western edge and the construction of one additional groin. A tapered groin system was implemented to promote littoral drift between the wide beaches within the groin field and the areas downdrift. Groins 14 and 15, originally 480 ft in length were shortened to 417 ft and 337 ft, respectively. Groin 14A, constructed between groins 14 and 15 in 1997, is 417 ft in length. Groins 1 through 13 are 480 ft long.

The Westhampton Interim Project provides for renourishment within the groin field and the western beach and dune portion, contingent upon the condition of a design cross-section. A renourishment cycle of three years was originally planned and has been recently only been required every four years. Renourishment material placed within the groin field plays two roles: (1) decrease impoundment capacity within the groin field to allow littoral transport to bypass the groin field; and (2) supplies additional renourishment material to downdrift beaches as it erodes from the groin field and enters the littoral system

When considering the area within the groin field, the performance of the constructed groins has exceeded expectations, resulting in an accretive beach and well-protected dunes. Similarly, the Westhampton Interim Project has exceeded performance expectations, as indicated by the accretive dunes west of the groin field, the lengthening of the renourishment cycle and the decrease in needed renourishment volume.

Restoration of longshore transport alternatives in the vicinity of the Westhampton groin field was considered. Possible alternatives include:

- a no-action alternative,
- beach fill placement,
- removal of groins,
- modification of groins,
- and combinations of these alternatives.

The objective of the selected alternative will be to provide storm damage prevention benefits in a cost-effective manner, reduce adverse impacts, and encourage the restoration of coastal littoral processes for both the areas contained within the groinfield as well as the vulnerable areas directly downdrift. Given the relative and proven consistent health of the beach contained within the groin field and the beneficial performance of the groin tapering and renourishment provisions of the Westhampton Interim Project, a combined alternative that incorporates the shortening of groins in the eastern and middle portions of the groin field, the tapering of groins on the western edge of the groinfield, in addition to continued renourishment was analyzed to evaluate the plan as a cost-effective solution. The specific elements of this possible alternative are as follows:

- Shortening of groins 1 through 8 to 380 ft
- Shortening of groins 9 through 13 to 386, 392, 398, 402, and 410 ft respectively
- Continued renourishment through the tapered section and westward as needed

Shortening of groins 1 through 13 has the potential to release a substantial amount of sediment back into the littoral system, providing a one-time release of sediment as the shoreline within the confines of the well-filled groin-compartments retreats in response to the modified groin lengths. In addition, groin shortening would provide an opportunity to repair the seaward end of these groins, which have not received maintenance since original construction, thereby maintaining functional stability. Finally, tapering along the western mid-portion of the groin field (groins 9 to 13) will improve transport between the feeder beach and downdrift areas.

To analyze the benefits of this proposed alternative, an estimate of the amount of sediment that would be released through groin shortening was developed. Considered from an elevation of –15 ft NAVD88, it is estimated that this alternative has the potential to release 150,000 cu yd into the littoral system. Considered from an elevation of –30 ft NAVD88, it is estimated that this alternative has the potential to release 5,00,000 cu yd into the littoral system.

The above alternative involves the removal of 70 to 100 ft of stone from the seaward end of 13 groins. Total length of removal considered is equal to 1210 ft. The cross-sectional area of the seaward head (which is approximately 100 ft in length) is approximately 560 sq ft. This alternative therefore entails the removal of approximately 675,000 cu ft of 16-ton armor stone. Removal of this quantity of armor stone would require a 25-ton capacity crane and attendant equipment to remove the stone from the beach to an approved disposal location. If the removal of the stone is conceptually priced at $400,000 per groin, the total construction cost for the shortening of 12 groins is approximately $5,000,000. The amount of sediment estimated to be released, 500,000 cu yd, can be purchased at an approximate cost of $12 cu yd, yielding a total cost of $6,000,000. The benefit of sediment released to downdrift beach is higher than the estimated construction cost. It is, therefore, concluded that the modification (shortening) of the existing grins represent the most cost effective strategy for the protection of the beaches within and downdrift of the Westhampton groin field.

7.7.6.3 Ocean Beach Groins

Two shore perpendicular structures were constructed in the winter of 1970 within the Village of Ocean Beach, on Fire Island. Ocean Beach and the State of New York built two groins at the western end of this community. Originally these groins were only constructed of tetrapods, which are concrete armor units, with five lower legs and one upper leg. The tetrapods have a base width of approximately 10 feet and a total height of approximately eight feet. The groins were constructed in an area of higher erosion, to add stability to the ocean shoreline seaward of the Ocean Beach water tower and pumping stations (wells). The water tower has been moved north in the Village, within Village owned land, however the three wells remain just landward of the eastern groin, within three village owned facilities. A separate Village maintenance facility is also located in the same Village property containing the wells. The groins are also in a location of the Fire Island shoreline that makes a change in orientation and has a higher background erosion rates than areas to the east.

The existing groins consist of two rows of tetrapods, spaced approximately 10 feet apart in the nearshore portion of the western groin, and 20 feet apart in the offshore portions of both groins. The nearshore portion of the eastern groin consists of only armor stone, while the space between the offshore portion of the western groin has been filled with armor stone. Both groins are 200 feet long from landward crest to seaward crest, with the offshore portion about 85 feet of the total length. The landward crest of the eastern groin is approximately 130 feet from the seaward limit of Ocean View Walk, and the landward crest of the western groin is approximately 50 feet from the seaward limit of Ocean View Walk. Ocean View Walk was eroded in the western area before the groins were constructed. The groins are approximately 660 feet apart along the shoreline, and the western groin is about 200 ft from the border of the Village of Ocean Beach and Corneille Estates. Based on 2006 aerial photography, the beach width, measured updrift of the groins, from the dune toe to the approximate mean high water line varies from 132 to 142 feet (the beach width is fairly stable). Generally, beach widths farther east of the two groins are larger, and farther west of the two groins are considerably narrower. Over a shoreline length of 1000 ft. from west and east of the two groins, the dune toe moves, in relationship to the seaward limit of Ocean View Walk, approximately 140 feet, for a change in shoreline alignment relative to Ocean View Walk of about 14 degrees. This follows a general change in alignment of the shoreline and dune toe along this section of the Fire Island shoreline.

Several historical shoreline datasets (1933, 1979, 1995 and 2001) were analyzed to determine the effect that these structures have had on adjacent shorelines and to assess the feasibility of removing them as part of this project. Shoreline comparisons suggest that shoreline downdrift of the groins between Corneille States and Kismet (2.5 miles which is the approximate extent of the alongshore groin impacts, as explained below) eroded at an average rate of roughly 3 ft/yr between 1979 and 2001 despite the placement of 1.3 million cubic yards of fill during that period. The shoreline updrift of the groins has been relatively stable or even accreting. In addition to the direct comparison between shoreline datasets, an even-odd function analysis was performed to determine the alongshore extent of the groin impacts. This analysis separates the shoreline position change data into symmetric (even) and anti-symmetric (odd) functions. In theory, the even function represents changes due to background erosion and sea level rise that occur symmetrically on both sides of the groins while the odd function account for anti-symmetric changes due updrift structure impoundment and downdrift erosion. Application of this method to the available shoreline change datasets and interpretation of the results suggest that the groins extent of influence is between 1.5 and 2.5 miles both updrift and downdrift of the structures. The analysis also suggests that background erosion in this area (i.e., what the erosion rate would be in absence of the groins) is on the order of 2 ft/yr.

From this analysis and a general understanding of shoreline behavior in the presence of this type of coastal structures it follows that, should the groins be removed, erosion rates downdrift would be reduced to background levels. However, erosion along the stable/accreting shoreline to the east would also increase, particularly the areas immediately adjacent to the groins (i.e., Ocean Beach), increasing the uncertainty in shoreline location, and therefore increasing the risk of storm damage to the Village-owned pumping facilities. Although the cost to modify the Ocean Beach groins is relatively inexpensive, the cost to relocate the Village's three pumping facilities would be over 5.0 million dollars assuming the property is available at no cost to move the facilities. Therefore, removing the groins at this point would not result in a net reduction in the cost of providing coastal storm damage risk reduction to the western Fire Island communities, from Oakleyville to Kismet. Moreover, visual inspection of the structures suggests that they are in relatively poor functional condition (i.e., relatively short, low and permeable) and are not as effective in trapping longshore sediment transport as first constructed. As a result, it is recommended that the two groins at Ocean Beach will not be modified for purposes of Storm Damage Reduction.

If there is a desire to remove or modify these structures in order to achieve other objectives, such as achieving habitat restoration benefits, to advance Vision objectives, or advance the objectives of the National Park Service, the following would need to be considered. The removal or modification of these groins would need to be implemented in conjunction with a more comprehensive storm damage reduction alternative, and would need to include the removal, relocation, or replacement of the existing well-field. With any of the proposed beachfill alternatives, the existing groinfield would be largely covered. As a result, the effect of removing the groin field would largely come into play in the future after the cessation of renourishment. In this scenario, groin modification could be accomplished in the future, subsequent to the relocation of the water supply.

Table 7-67 presents the costs for groin modification of the Westhampton Groin field.

Table 7-67. Modification of Westhampton Groins

Construction Cost	$5,000,000
Contingency	$1,500,000
E&D	$455,000
S&A	$585,000
Total First Cost	*$7,500,000*
IDC	$142,441
Total Investment Cost	*$7,642,441*
Interest & Amortization	$426,754
O&M	$0
BCP Maintenance	$0
Monitoring	$0
Renourishment	$0
Total Budgeted Cost	*$426,754*
Annual Breach Closure Cost	$0
Major Rehabilitation	$0
Total Annual Cost	***$426,754***

<u>Compatibility with Restoration Measures</u>

There are several types of restoration measures that are compatible with the groin modification alternatives. It should be recognized that groin modification itself can be considered as a restoration alternative, which can help restore the longshore transport. The restoration measures that are compatible with groin modifications include the following:

1) Restoration of dune habitat in conjunction with groin modification, and in conjunction with building removal

2) Restoration of bayside habitat (bay beach, marsh, sav), that would require stabilization, and would allow for a beneficial re-use of the stone.

Evaluation of Groin Modification Alternatives.

Groin Modification Alternatives were evaluated in relationship to the planning criteria developed to reflect the Project. This systematic assessment ensures that the Vision Statement approach is fully integrated into the development and selection of the FIMP Plan. Table 7-68 provides a summary of the evaluation of these measures relative to the established criteria.

Table 7-68 - Evaluations

Evaluation Criteria	Assessment	Rating
The plan or measure provides identifiable reductions in risk from future storm damage.	Plan will reduce risk in certain locations. There is a potential tradeoff in risk levels between locations.	Partial
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science are considered a lower priority.	Groin modifications are fairly well understood and were successfully implemented at western limit of Westhampton groin field. Physical changes are not easily reversed. Continued monitoring and beach fill may be required.	Partial
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	Plan addresses open coast storm surge and flow into the bays due to periodic overwash or breaching of the barrier islands. Upon shortening of the groin in Ocean Beach, sand would move to fill scour at the potential breach location at Robins Rest. Shortening the groin in Westhampton would reduce risk and renourishment requirements in Fire Island Interim Project (FIIP) study area.	Partial
The plan or measures incorporate appropriate non-structural features provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity	N/A	N/A
The plan or measure help protect and restore coastal landforms and natural habitat.	Would help restore natural landforms	Partial
The plan avoids or minimizes adverse environmental impacts	No significant impacts	Full
The plan addresses long-term demands	May reduce need for long-term	Full

Evaluation Criteria	Assessment	Rating
for public resources.	renourishment	
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	N/A	N/A
The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures	Yes	Full
The plan or measure incorporates appropriate alterations of inlet stabilization measures and dredging practices	N/A	N/A
The plan or measure is efficient and represents a cost effective use of resources	It appears to be cost-effective in certain areas	Partial
The plan or measure reduces risks to public safety.	Reduces erosion risk	Partial

Summary of Groin Modification Findings

The analysis above shows that groin modification alternatives for the Westhampton Groin field are cost-effective storm damage reduction alternatives that can reduce the long-term volumes of sand required for the areas to the west of the groins, without compromising the coastal storm damage risk reduction that is provided to homes within the groin field.

Groin modification alternatives are not recommended for storm damage reduction at Georgica Pond or for the Ocean Beach Groins. Modification of the groins at Ocean Beach could help restore alongshore transport, and could have NER benefits that should be evaluated. Any removal or modification of groins at ocean beach would need to include an alternative storm damage reduction measures for the Village of Ocean Beach, and under any modification scenario would require the relocation of the Village water supply.

The groin modification alternative partially fulfills the vision objectives, but offers limited reduction in storm damages when considered as a stand alone alternative. Groin modification itself, can be considered as a restoration alternative. Opportunities exist for beneficial reuse of the stone, which may be needed for any habitat restoration alternative.

The groin modification alternatives do not directly present land management or development management challenges, but as presented, to implement the groin modification alternative, specifically in the vicinity of Ocean Beach would require measures to reduce the risks to existing development.

7.7.7 Land and Development Management

General

Land and development management alternatives include land use regulations and acquisition alternatives that could be implemented to reduce the risk of storm damages to existing development in high risk areas, and to reduce development pressure in those areas. These at-risk areas generally include areas vulnerable to flooding, and also areas that are vulnerable to erosion.

As presented in the with-out project conditions section of this report, the existing land use regulations are not effective in addressing development and redevelopment in these at-risk areas, particularly in areas that are vulnerable to erosion. There is a concern that alternatives implemented under this Project could exacerbate this problem. The following is provided as a review of the land-use regulations, the additional challenges and opportunities inherent with the different alternatives, and opportunities to more effectively address the development and redevelopment concerns in the hazard areas.

Existing Programs

The following is a summary of the existing land-use regulations with a focus on the major programs including NYS CEHA, FIIS – Dune District, and FEMA floodplain management.

While the federal, state and county governments each have regulatory authority, the local governments have regulatory jurisdiction with respect to land management, principally through zoning and also through management of environmental features (e.g., freshwater and tidal wetlands). In addition, FIIS is administered by the NPS under the DOI, a federal agency with land use and environmental management authority.

In New York State, the primary responsibility for zoning land use regulations rests with local municipalities, including towns and incorporated cities or villages, under the system known as "home rule". However, in the case of shorefront areas potentially subject to flooding or coastal erosion, and for Fire Island in particular, a number of other federal and state zoning and other land use regulations pertain, as described below.

<u>Fire Island National Seashore</u>

When Congress enacted FIIS-enabling legislation, the law mandated the Secretary of the Interior to establish federal zoning regulations. These regulations provide standards for local zoning to protect and preserve Fire Island, and they exist solely as an overarching law to which local ordinances must conform.
Federal zoning regulations provide a set of standards for the use, maintenance, renovation, repair, and development of property within FIIS. NPS has established three districts within its boundary, which are: 1) the Community Development District; 2) the Seashore District; and 3) the Dune District. The Community Development District comprises 17 communities and encompasses the existing communities and villages. In the Community Development District, existing uses and development of single-family houses are allowed. The Seashore District includes all land in FIIS that is not in the Community District. No new development is allowed in the Seashore District, but existing structures may remain.

The Dune District extends from Mean High Water (MHW) to 40 feet landward of the primary natural high dune crest which has been mapped by NPS. This district overlaps the other two districts. Only pedestrians, and necessary vehicles such as ambulances, are allowed in the Dune District. Like the Seashore District, existing legal structures may remain and may be repaired and maintained. The existing dune district was established based upon the dune condition in 1976 and adopted by Congress. The dune district has not been re-mapped, and presently is not an accurate representation of the existing dune. NPS developed federal zoning standards that became effective September 30, 1991 under 36 CFR Part 28. These set standards that local zoning must meet to be exempt from the condemnation authority of the Secretary of the Interior.
These standards include controlling population density and protecting natural resources, limiting development to single-family homes, and prohibiting any new commercial or industrial uses. NPS is not responsible for enforcing the federal zoning standards in the communities and villages, despite the presence of federal regulations. It is the responsibility of the local governments to maintain regulatory jurisdiction. The federal government ensures local compliance with the federal law by maintaining the power of condemnation; in cases where the law is not met, FIIS has statutory authority to purchase and condemn the non-compliant building. While local zoning ordinances conform to standards issued by the Secretary of the Interior, the federal power of condemnation is suspended. In practice, this authority has been seldom exercised, and Congress has not given funding to FIIS for this purpose in recent years.

FEMA

Other agencies also have responsibility to affect land use regulation in the project area. An organization that indirectly affects land use regulation is the Federal Emergency Management Agency (FEMA). Any community seeking to register with the Federal Insurance Association, which allows homeowners to obtain flood insurance, must join FEMA's National Flood Insurance Program (NFIP). Participation in the NFIP requires a municipality to adopt a local floodplain management ordinance that regulates floodplain development and redevelopment following damage. The intent of the local ordinance is to reduce damage to buildings and property through the establishment of base flood elevations, building code requirements, and restrictions on allowable development in floodplain areas. Specific provisions include the requirement that the first finished floor or new construction must be elevated above the base flood elevation. All municipalities within the study area participate in the NFIP.

USFWS

The Coastal Barrier Resources Act of 1990 established the Coastal Barrier Resources System (CBRA), which consists of specifically identified undeveloped coastal barriers on the United States coastline. The U.S. Fish and Wildlife Service (USFWS) is the responsible agency for administering CBRA. Coastal barriers include barrier islands, bay barriers, and other geological features that protect landward aquatic habitats from direct wind and waves. CBRA units are prohibited from receiving federal monies or financial assistance or insurance for new development in CBRA in areas. The CBRA, however, identifies exceptions to this restriction, including non-structural shoreline stabilization similar to natural stabilization systems; the maintenance of channel improvements, jetties, and roads; necessary oil and gas exploration and development; essential military activities; and scientific studies.

NYS CEHA

Due to the erosion-prone nature of parts of the New York coastline, the Coastal Erosion Hazard Areas Act (CEHA) (Article 34 of the Environmental Conservation Law) regulates construction in areas where buildings and structures could be damaged by erosion and flooding. NYCRR Part 505 provides procedural requirements for development, new construction, and erosion protection structures. The responsibilities for NYSDEC regarding towns, counties, and regulation of coastal erosion hazard areas are defined by these regulations. These regulations restrict development in the primary dune, which is defined as 25 ft landward of the landward toe of the dune. Since these regulations were more recently adopted, and since the locations of the dunes have changed over time, there are a number of pre-existing, non-conforming structures within the CEHA area.

NYS CMP

In 1981, the New York State Legislature enacted the Waterfront Revitalization and Coastal Resources Act (Article 42 of the Executive Law) to implement the State Coastal Management Program (CMP) at the state level. The CMP and Article 42 establish a balanced approach for managing development and providing for the protection of resources within the state's designated coastal area by encouraging local municipalities to prepare Local Waterfront Revitalization Programs (LWRPs) in accordance with state requirements.

Land Use and Development Challenges

It is acknowledged that within the study area this existing collection of land use regulations is not adequate to address the development pressures, nor to effectively address building and rebuilding in the high hazard areas along the coast.

As presented throughout this Chapter, there is a concern that certain alternatives could create additional land and development challenges or intensify the existing challenges that exist. Alternately, there are alternatives that provide opportunities for reducing these pressures. Throughout this Chapter, each alternative presents the land-use challenges and opportunities. The following is a summary of the alternatives, and land-use challenges and opportunities associated with them.

Breach Response. The breach response plans introduce some land use and development management challenges that would not be realized in the without project condition. Existing land management measures do not address rebuilding in breach locations, or locations that are likely to remain vulnerable to breaches in the future. Land and development management measures should consider the need for restricting redevelopment in locations that are likely to remain as vulnerable to breaching and overwash. Not only will this address reducing development at risk, but is also important to facilitate continued breach response requirements, and can help provide a desirable habitat mosaic by maintaining an open bay to ocean connection.

Inlet Management. The inlet management plans do not introduce any specific land use and development management challenges.

Non-Structural. The non-structural plans do not introduce land use and development management challenges, but instead introduce additional land use and development management opportunities that could be considered in conjunction with these alternatives. As has been presented, there could be a larger benefit obtained by acquiring rather than retrofitting structures in some situations, including instances where 1) buildings are in sparsely developed areas, where habitat connectivity could be achieved, or 2) buildings located at such low ground elevations that under future sea level rise conditions would be in the intertidal zone. If there is a local desire for structure acquisition rather than retrofit alternatives, these alternatives could be considered if the additional costs for acquisition would be warranted to provide restoration of habitat to the underlying area.

Beachfill. Beachfill plans introduce a number of land use and development management challenges as well as opportunities that could be considered in conjunction with these alternatives.

Along the shorefront area, the existing land management regulations that limit the investment in the primary dune have not proven to be effective. There a number of existing structures within the dune, partially due to structures that existed prior to the implementation of these regulations, and also partially due to long-term changes in the dune position; and development continues to occur in the primary dune. In the absence of a project, it is likely that the number of pre-existing, non-conforming structures would be reduced as a result of storms that would destroy these buildings beyond repair, with the acknowledgement that additional buildings would be at risk, due to the long-term evolution of the dune position. With a beachfill project in place, it is much less likely that the structures in the CEHA would be destroyed, and would likely persist.

Additionally, there is a concern that there could be increased incentive to develop these areas if there is a beachfill and dune project that reduces the likelihood of storm damages. The stabilization of the shoreline with a beachfill and dune plan would increase the need for effective land management measures which function properly to avoid an increase in the level of infrastructure that is at risk in these areas.

It must be noted that these beachfill plans also create opportunities to address existing development that is at risk, and opportunities for reducing the amount of development and infrastructure at risk, over time.

There are several locations where beach nourishment is included to reduce risk to public infrastructure, most notably in Robert Moses State Park, and Smith Point County Park. Opportunities exist to provide for relocation of public infrastructure in these areas to reduce the long-term requirement for renourishment.

As presented in this chapter, the beachfill alternatives have also been developed to consider different beachfill alignments. The construction of a beachfill and dune project requires real estate easements to be obtained to construct and maintain the beach and dune. These easements would preclude development in the footprint of the project. As presented previously, the construction of a more landward alignment would require acquisition of buildings, prior to construction, and would effectively achieve the goal of reducing the number of structures in the high-risk area. This, however, would likely require extensive condemnation to achieve this. Rather than trying to acquire structures up-front, at project initiation, the possibility exists for alternatives which improve land management regulations, or could acquire structures over time to reduce the level of development at risk along the shorefront.

Groin modification. The groin modification alternatives do not directly present land management or development management challenges. However, the implementation of the groin modification alternative in the vicinity of Ocean Beach could increase the vulnerability of the existing development and would require measures to reduce the risks to existing development, and would require the relocation of public infrastructure which is at risk.

Land and Development Management Opportunities

Section 7.5.1 presents a table that highlights all of the possible land and development management alternatives that could be implemented to address the existing land use challenges, and the challenges that may become more apparent with a plan resulting from this study. This table, with supporting information, was used as the basis for meetings with local municipalities and stakeholder groups to develop recommendations on alternatives that could be implemented to address these challenges.

These meetings have identified that the biggest challenge is addressing building and rebuilding in erosion-prone areas. These discussions have resulted in a framework to address these concerns, which generally consider solutions that improve upon or modify the existing set of regulations that are presently in place, rather than the introduction of new land-use regulations. This approach considers:

Step 1: Improving the effectiveness of the existing regulatory program, by establishing a common funding source, establishing common and clearly communicated boundaries for regulated hazard areas, increasing training of local officials, and coordination to ensure consistent implementation across regulatory boundaries.

Step 2: Modification of statutes to allow for more effective implementation of the existing laws.

Step 3: Establishing a funding mechanism to acquire vacant parcels, or buildings that are at risk

Step 4: The establishment of a regional entity that would be responsible for various aspects related to land management and acquisition, and to fulfill the requirements of the local sponsor.

Step 5: Establishment of post-storm response plans to guide recovery following major, catastrophic events.

Step 1. Improving the effectiveness of the existing land-use regulations through establishment of common funding, and improved implementation of the law, generally includes the following:

Update the Existing Dune District in FIIS

The FIIS enabling legislation set the established dune location in 1978; this line does not reflect the current dune location. Effective implementation of the regulation would benefit from a common definition of the dune, and a common regulatory jurisdiction with the CEHA Program. The federal law should be revised to create the same definition of a dune and the same requirement as contained in CEHA for a 10-year remapping process. This common mapping would require the identification of and agreement on a common defining feature. Presently, the CEHA program is based upon the landward toe of the primary dune, plus 25 feet. The federal dune district is based upon the dune crest plus forty feet. Furthermore, the NYS process provides for a public hearing as input into the process, which is not a provision of the Federal dune district. Since the CEHA serves as the primary regulatory mechanism, has been applied throughout the state, and is more current than the dune district, it is recommended that the provisions within the FIIS enabling legislation be changed to identify that the dune district be coterminous with the CEHA line, and allowed to change with changes in the CEHA designation.

<u>CEHA Improvements.</u>

CEHA improvements include map updates, funding to adequately implement the program, and provisions for improved DEC monitoring of local implementation of CEHA. These improvements are described below:

.

<u>Updating CEHA Maps Across the FIMP Area</u>. CEHA requires review and remapping of dune locations every 10 years. Fire Island was completed 10 years ago and no remapping is scheduled. Other areas of the study were mapped even earlier. Dune positions change in response to storm activity. The routine remapping of CEHA is necessary to effectively implement the program, and should be scheduled on a routine 10-year basis.

<u>Improve DEC monitoring and support of local implementation of CEHA and establish adequate funding for effective implementation of CEHA</u>. DEC has delegated the implementation of CEHA to local communities in many instances. By regulation, DEC must conduct regular annual monitoring reviews for compliance by all delegated programs so that missteps are addressed, monitoring, management and communication can improve, consistent implementation can be acknowledged, and, where necessary, delegation can be withdrawn. At its current funding level, DEC cannot provide oversight and conduct adequate training for local implementation by municipalities that have assumed direct management, nor oversee and properly implement the law elsewhere. Effective funding of the program at the state level would allow for technical and legal support for municipalities who administer their program, and improve their effectiveness. Effective funding of this program is necessary regardless of any alternative implemented under FIMP, and is presumed to be a responsibility of the local sponsor.

Step 2. Modification of statutes to allow for more effective implementation of the existing laws.

<u>CEHA Statutory changes.</u> Make statutory and rule changes to enhance enforcement authority and provide indemnification by New York State for properly-administered local CEHA programs against takings claims (e.g.; Pine Barrens § 57-0123.6) to reduce the influence of potential litigation costs, including potential takings claims, on local program decision making. Presently, local municipalities are responsible for providing the legal defense in the instance where CEHA variance requests are taken to court. Often the cost of defending these lawsuits is comparable to the costs associated with acquiring properties, and beyond the means of the municipalities. State indemnification for properly administered CEHA programs would mitigate this issue.

Step 3: Establishing a funding mechanism to acquire vacant parcels, or buildings that are at risk

Improved implementation of the land use regulations can help address inappropriate building and rebuilding in the primary dune. It is acknowledged however, that even with such improvements, these programs would benefit from a funding mechanism made available to purchase vacant developable property, or for acquisition of vulnerable shorefront structures. This could serve as a means to acquire properties when enforcement of the regulations establishes a "takings", or in a broader application could be applied to reduce the number of structures within the CEHA area that would be vulnerable to storm damages.

Acquisition of structures as a stand-alone alternative was evaluated as a possible alternative along the shorefront. Analyses were undertaken to identify buildings falling within different hazard areas, and also at risk from storm damages. It should be noted that since CEHA maps the dune, regardless of the size and height that there may be structures within the CEHA (on the back crest of a high, wide dune) that are less vulnerable to damages than a similar structure on a low, narrow dune. In conjunction with this analysis, an extensive Real Estate analysis was undertaken to identify an approximate acquisition cost for structures which fall within the CEHA. In evaluating the acquisition alternatives, it became clear that acquisition could not be supported on NED analysis alone. The NED analysis evaluates the potential damages to a building, whereas the costs to acquire a building must consider the value of the structure and the property.

Within the study area, the Real Estate cost to acquire a structure was on average 4 to 5 times the value of the structure, which means that 20 - 25% of the real estate value is derived from the building. This cost differential makes it impossible to support acquisition on purely NED criteria, since it is impossible for the building to be damaged enough to offset the Real Estate costs. It is acknowledged that if there are additional benefits that could be realized, it could be possible to justify these efforts. It is possible that acquisition would also:

1. Provide additional habitat values by restoring the beach and dune to more natural condition,
2. Provide cost savings if the volume of material required for renourishment could be lowered,
3. Provide benefits associated with having a sustainable solution that would effectively reduce the need for long-term maintenance beyond the project life.

Recognizing this, and recognizing that environmental benefits could accrue from acquisition of buildings and restoration of the land, selective acquisition is considered further in the context of restoration alternatives. Recognizing the benefits of providing a more sustainable, long-term plan for the area, this is also something that could be considered further as a measure to be implemented as part of the overall collaborative plan.

It is acknowledged that the scope of the acquisition plan could range from a plan to acquire properties when there is a determination of a taking, to a broader scope that would allow for the acquisition of structures from willing-sellers in high-risk areas, and could also include an acquisition plan for breach vulnerable areas. With this larger concept, there are a number of acquisition scenarios that could be developed as an incentive for increased participation. These are presented below.

Voluntary sales with retained occupancy or lease-back programs. In the past, FIIS has purchased noncommercial residence at fair market value, reduced by up to 25% allowing for the right to no more than 25 years of retained occupancy, unless the house is destroyed. Federal leaseback programs are generally very restrictive but state, county or local programs may have provisions for retained occupancies or less restrictive lease-back arrangements. This type of program could encourage voluntary participation by landowners. Landowners who recognize the hazards presented by their location may find such programs attractive as it provides them a fixed sum upfront based upon a pre-storm appraisal and the opportunity to continue to use the structure for the term, or until it is destroyed. It allows homeowners to spread their risks, as a post-storm value for a destroyed and eroded parcel would be far less. The advantage for the public is that while structures will remain on the dunes and continue to inhibit natural dune growth, this voluntary approach could substantially reduce the controversies around immediate condemnation, reduces acquisition costs by at least 25%, and particularly for the secondary line of houses, will facilitate dune advancement over time, ultimately achieving a more sustainable dune.

Step 4. The establishment of a regional entity that would be responsible for various aspects related to land management and acquisition, and to fulfill the requirements of the local sponsor.

With the proposed alternatives identified in Steps 1-3, there would be a benefit to having a single regional entity who would be capable of addressing these needs, as well as fulfilling the requirements of a non-State, local sponsor. The formation of a Suffolk County Coastal Commission with authority to implement land management and authority (and sufficient funding) to acquire property, could ensure the following:

1. The local, non-State sponsor will be responsible for acquisition of lands necessary for construction of the project, and providing funds necessary, in excess of the Real Estate costs to meet the local share. A County-wide entity with the ability to undertake this would facilitate project sponsorship, and could address concerns expressed previously from Suffolk County regarding liability for the Project.
2. As described in the CEHA provisions, this entity could serve as a group who would be responsible for CEHA variances, and in defending legal challenges arising from CEHA.
3. this entity could be responsible for the acquisition of properties in the instance of regulatory takings,

4. This entity could also be responsible for implementing a willing-seller program to address structures that are at-risk in the erosion prone areas.

Step 5. Establishment of post-storm response plans to guide recovery following major, catastrophic events. It is acknowledged that no plan will reduce all risks. It is likely that over the project life, a storm will occur which will compromise the design, and result in damages. This could occur in areas that are protected, or areas that are not protected as a result of this project. New York State has suggested that they will require, as part of their Local Cooperation Agreements the development and implementation of local post-storm redevelopment plans. It is expected that these plans would be in place, and would provide direction for the rebuilding of communities in a more sustainable manner, which recognizes the storm risks. It is expected that New York State will oversee the creation of such plans, including their expected content and rationale.

While there is a limited role for the Corps' in the implementation of the land and development management measures, it is acknowledged that this is an integral component of any plan. It is important to ensure that adequate provisions are in place for the project to perform as expected, and does not result in increased development that is at risk. It is advised that the above land and development management measures be considered further in conjunction with the alternative plans, to ensure the functioning of the project, and to consider the longer-term sustainability of the project.

7.7.8 Design and Evaluation of Alternative Findings

The analysis of each of these alternatives as stand-alone alternatives, and their effectiveness in reducing storm damages in the current framework has been used to identify which of these alternatives are to be carried forward for consideration in developing comprehensive alternative plans. These alternative plans are developed to consider combining alternatives, and allowing for a range of solutions along a Project Reach.

Based upon the results of these analyses, there are a number of alternatives that could be recommended to be carried forward for consideration as input into combined alternative plans.

<u>The alternatives recommended for further consideration include the following:</u>

- Breach Response Plan – +13 ft dune
- Breach Response Plan – + 9.5 ft cross-section (primarily for environmentally sensitive areas)
- Inlet bypassing
- Nonstructural Alternative 2
- Nonstructural Alternative 3
- Nonstructural Alternative 2 with Road Raising
- Nonstructural Alternative 3 with Road Raising
- Beachfill Alternative +15 ft for Great South Bay and Moriches Bay Reaches
- Sediment Management Measures in the Ponds and Montauk Reach
- Groin Modification Alternatives at Westhampton

The project evaluation criteria for all the plans are shown in

Table 7-69, which illustrates that while no one measure meets all of the objectives, a careful combination of the project measures can be identified to satisfy the objectives.

Table 7-69. Combined Alternatives

Evaluation Criteria	Breach Closure	Inlet Management	Non-Structural Retrofit	Beach Fill	Groin Modification
The plan or measure provides identifiable reductions in risk from future storm damage.	Full	Full	Full	Full	Partial
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science are considered a lower priority.	Full	Full	Full	Full	Partial
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	Partial	Partial	Full	Partial	Partial
The plan or measures incorporate appropriate non-structural features provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity	No	Full	Partial	N/A	N/A
The plan or measure help protect and restore coastal landforms and natural habitat.	Partial	Full	No	Partial	Partial
The plan avoids or minimizes adverse environmental impacts	Partial	Full	Full	Partial	Full
The plan addresses long-term demands for public resources.	Partial	Partial	Full	Full	Full
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	Full	Full	No	Partial	N/A
The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures	N/A	N/A	No	Partial	Full
The plan or measure incorporates appropriate alterations of inlet stabilization measures and dredging practices	No	Full	No	N/A	N/A

Evaluation Criteria	Breach Closure	Inlet Management	Non-Structural Retrofit	Beach Fill	Groin Modification
The plan or measure is efficient and represents a cost effective use of resources	Full	Partial	Full	Partial	Partial
The plan or measure reduces risks to public safety.	Full	Full	No	Full	Partial

7.8 Alternative Plan Evaluation

The previous sections presented the results of the screening of alternatives, and the evaluation of the detailed design alternatives. This section of the appendix presents the integration of the alternatives and the effects of combining these measures together, considering the integration of different solutions for different reaches. This analysis focuses on the integration of alternatives from the subset of alternatives identified as feasible in the prior sections.

7.8.1 Identification of Storm Damage Reduction Alternatives by Reach

The NED analyses presented previously identified that a wide range of the individual alternatives are cost effective options for Storm Damage Reduction. Section 7.7 also illustrates that there is not one alternative that addresses all the storm damage reduction problems; it highlights that addressing multiple problems requires multiple solutions. In this respect, many of the alternatives compliment each other, and Alternative Plans benefit from combinations of alternatives. In addition, the NER evaluation has identified that various restoration alternatives are complimentary to, or compatible with each of the Storm Damage Reduction Plans.

The combinations of Alternative Plans have been developed in accordance with the the FIMP Project Vision Statement. The approach gives first priority to management options, particularly options that restore natural processes. The second priority is to include non-structural alternatives, with beach nourishment or other structural alternatives considered last. This formulation approach ensures that Plans are consistent with the NY State Coastal Zone Management policies, and also places a priority on avoiding or minimizing any negative environmental impacts.

Based on the evaluation of the individual alternatives, combined plans were developed. First, Second and Third added plans were developed by incrementally adding Management Alternatives (Plan 1), Non-Structural Alternatives (Plan 2), and Structural Alternatives (Plan 3). The scale of the alternatives selected for inclusion was based on the results of the optimization of individual alternatives and the potential for the combined alternatives to more fully satisfy the project objectives and evaluation criteria.

7.8.2 Plan 1

The first series of plans (Plans 1.a and 1.b) reflect combinations of Management Alternatives and have combined the Inlet Management and BCP Alternatives. The Inlet Management Alternative includes continuation of the authorized project at the inlet, plus additional bypassing of sand from the ebb shoal to offset the erosion deficit. Inlet Management Alternatives are included because they meets both Restoration and Storm

Damage Reduction objectives. Inlet Management is compatible with all plans in the Great South Bay, Moriches Bay and Shinnecock Bay reaches. Two of the BCP alternatives have been selected for evaluation in the combined Plans. The 13 ft NGVD BCP Closure Alternative is selected because it maximizes the BCP Storm Damage Reduction Benefits. The 9.5 ft NGVD BCP Closure Alternative is selected because it maximizes opportunities to restore cross shore transport. Plan 1 is illustrated in Figure 7-14.

Plan 1.a is based on the combination of the economically optimum Inlet Management Alternative and BCP Alternative (13 ft NGVD BCP). Plan 1.b combines the optimum Inlet Management Alternative with the 9.5 ft NGVD BCP Alternative. Table 7-70 through Table 7-72 provide summaries of the Storm Damage Reduction Benefits, Costs and Benefit Cost Ratios for the Management Only Plans. Plans are presented for both comprehensive plans covering the Great South Bay (GSB), Moriches Bay (MB) and Shinnecock Bay (SB), and for each of the three bays separately.

Table 7-70. Annual Benefits Plan 1 – Management Only

	Plan 1.a	**Plan 1.b**
Benefit Category	**Inlet Management BCP 13**	**Inlet Management BCP 9.5**
Inundation	$0	$0
Mainland	$280,000	$280,000
Barrier	$40,000	$40,000
Total Inundation	*$320,000*	*$320,000*
Breach		
Inundation	$8,980,000	$8,840,000
Structure Failure	$230,000	$240,000
Total Breach	*$9,210,000*	*$9,080,000*
Shorefront	$0	$0
Total Storm Damage Reduction	*$9,530,000*	*$9,400,000*
Costs Avoided		
Breach Closure	$2,160,000	$2,160,000
Beach Maintenance	$0	$0
Total Benefits	***$11,690,000***	***$11,560,000***

Interest Rate 5.125%, Project Life 50 years

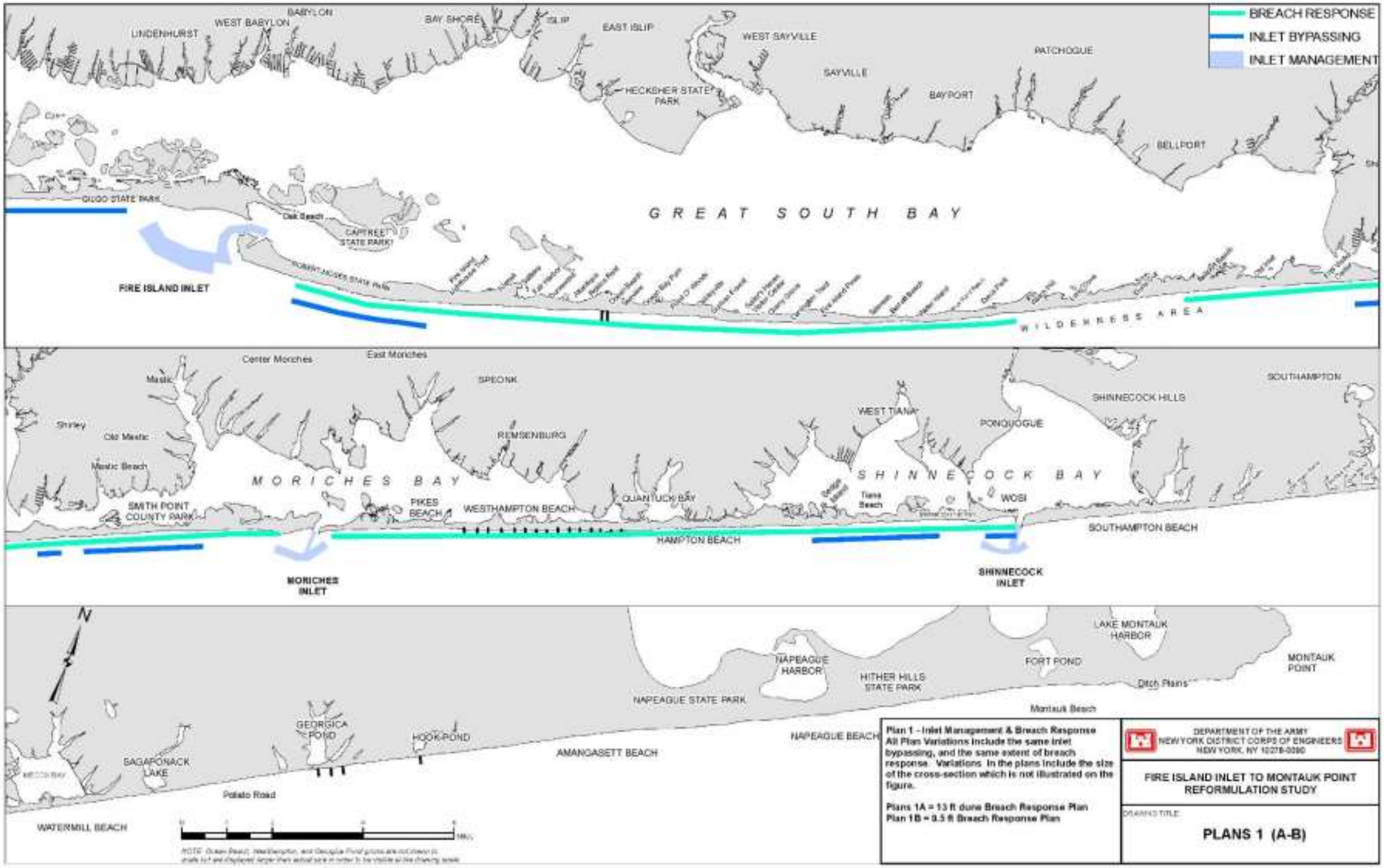


Figure 7-14. Plan 1 Overview

Table 7-71. Annual Cost Plan1 – Management Only

	Plan 1.a	Plan 1.b
Cost Category	**Inlet Management BCP 13**	**Inlet Management BCP 9.5**
Beach Fill	$0	$0
Nonstructural	$0	$0
Road Raising	$0	$0
Total First Cost	$0	$0
Total IDC	$0	$0
Total Investment Cost	$0	$0
Interest and Amortization	$0	$0
Operation & Maintenance	$7,000,000	$7,300,000
Renourishment	$0	$0
Subtotal	*$7,000,000*	*$7,300,000*
Annual Breach Closure Cost	$800,000	$1,100,000
Major Rehabilitation	$0	$0
Total Annual Cost	***$7,800,000***	***$8,400,000***

Interest Rate 5.125%, Project Life 50 years

Table 7-72. Net Benefits and BCR, By Project Reach Plan 1 – Management Only,

	Plan 1.a	Plan 1.b
Component	**Inlet Management BCP 13**	**Inlet Management BCP 9.5**
Total Project		
Total Annual Cost	$7,800,000	$8,400,000
Total Benefits	$11,700,000	$11,600,000
Net Benefits	$3,900,000	$3,200,000
Benefit-Cost Ratio	1.5	1.4
Project Reaches		
Great South Bay		
Total Annual Cost	$2,800,000	$3,200,000
Total Benefits	$9,100,000	$8,900,000
Net Benefits	$6,300,000	$5,700,000
Benefit-Cost Ratio	3.2	2.8
Moriches Bay		
Total Annual Cost	$3,700,000	$3,800,000
Total Benefits	$2,100,000	$2,100,000
Net Benefits	-$1,600,000	-$1,700,000
Benefit-Cost Ratio	0.6	0.6
Shinnecock Bay		
Total Annual Cost	$1,400,000	$1,400,000

Component	Plan 1.a Inlet Management BCP 13	Plan 1.b Inlet Management BCP 9.5
Total Benefits	$500,000	$500,000
Net Benefits	-$900,000	-$900,000
Benefit-Cost Ratio	0.4	0.3

Interest Rate 5.125%, Project Life 50 years

<u>NED Evaluation</u>

The Management Plans provide Storm Damage Reduction by increasing longshore sediment transport, which reduces erosion on the barrier islands, and by reducing the potential impact of breaches. The reduction in shoreline erosion associated with increased longshore sediment transport will provide a wide range of benefits to both the natural and built environments including a reduction in storm damage due to breaching, increases in future backbay flooding and reduced erosion and wave damage to shorefront development. The management alternatives will also have a positive impact on maintaining future beach widths at several important recreation sites including Robert Moses State Park, Smith Point County Park, and Tiana Beach, including Shinnecock County Park and Town Park. Overall this plan is economically viable; however, when excluding the impact of recreation, the economic analysis of the Management Plans indicates that at some locations the Plans provide a Benefit to Cost Ration (BCR) of less than 1. This is generally a result of the high cost of the increased bypassing relative to the measurable Storm Damage Reduction Benefits. Because bypassing is such a critical component to restoring physical processes in the study area, it has been incorporated into the remaining plans.

<u>P&G Evaluation.</u>

The existing Principles and Guidelines establish that alternative plans should be complete, effective, efficient and implementable. This evaluation discusses how well these alternative plans meet these objectives. The alternatives that combine inlet bypassing and breach response plans are not complete solutions. These plans address the storm damage problems associated with a breach being open, and help to address the chronic erosion in the vicinity of inlets, but only address 10% of the damages that are likely to occur in the study area, and have a high level of residual damages. Under this alternative there would still remain a high level of damages to the shorefront, a high likelihood of recurring breaches, and a high likelihood of damages due to flooding along the bayside shoreline. Based purely on the storm damage reduction these plans are marginally effective, and marginally efficient. These alternative plans are implementable. NYS, through the Governor's Coastal Erosion Task Force supports bypassing and breach closure. The specific details related to breach closure will need to be coordinated with the USFWS, and FIIS, to ensure that the closure procedures are consistent with their requirements.

<u>Vision Evaluation of Plan 1 Alternatives</u>

The Plan 1 alternatives (Plan 1a and Plan 1b) were evaluated in relationship to the planning criteria developed to reflect the Project objectives. This systematic assessment ensures that the approach is fully integrated into the development and selection of the FIMP Plan, and builds on the evaluation of individual plan components provided in Section 7.7. Table

7-73 provides a summary of the evaluation of these measures relative to the established criteria.

Table 7-73. Plan 1 (Plan 1.a and 1.b): Inlet Management Measures + BCP		
Evaluation Criteria	**Assessment**	**Rating**
The plan provides identifiable reductions in risk from future storm damage.	The Plans help to avoid excessive erosion in areas affected by inlets. This provides reduced risk of bayside flooding and reduced erosion at beaches downdrift of the Inlet or breach locations.	Full
The plan is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science are considered a lower priority.	The selected sediment management measures are based on the observed historical inlet responses and extensive modeling of inlet dynamics and morphology. Breach closure has been the general practice in study area since the response to the 1938 Hurricane.	Full
The plan addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	Sediment management may reduce flooding by preventing local areas of accelerated erosion, thus reducing flooding associated with periodic overwash or breaching of barrier islands.	Partial
The plan incorporates appropriate non-structural features to provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity	The Plan represents enhanced management of existing resources. The inlet and sediment management measures maintain both storm damage risk reduction and directly restores longshore sediment transport, contributing to ecosystem integrity. The BCP provides enhanced breach response decision making. In some cases the more rapid breach closure will reduce cross shore sediment transport.	Partial
The plan or measure help protect and restore coastal landforms and natural habitat.	Sediment management helps to reduce or eliminate deficits in longshore sediment transport and is important for the protection of landforms and habitat. The BCP decision process help protect some existing barrier and bayside habitats, but may reduce the	Partial

Table 7-73. Plan 1 (Plan 1.a and 1.b): Inlet Management Measures + BCP		
Evaluation Criteria	**Assessment**	**Rating**
	extent of bayside spit or shoal formation.	
The plan avoids or minimizes adverse environmental impacts	The use of improved sediment and breach management reduces the volume of breach closure or other dredging, reducing impacts. Construction activities for inlet management are scheduled to avoid or minimize impacts. For breach closure, response protocols have been developed to minimize any adverse impacts.	Full
The plan addresses long-term demands for public resources.	The plan incorporates required navigation maintenance to provide future cost efficiencies. Future monitoring and restoration to maintain the beach profile to prevent repetitive breaching and limit future expenses..	Full
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	Locations for placement of bypassed sediment provide both storm damage reduction and restoration. The BCP decision process balances SDR needs and environmental effects.	Full
The plan incorporates appropriate alterations of existing shoreline stabilization structures	NA	NA
The plan incorporates appropriate alterations of inlet stabilization measures and dredging practices	Measures to alter dredging practices were considered more appropriate than structural changes to the inlets.	Full
The plan is efficient and represents a cost effective use of resources	The measures provide significant economic and process restoration. BCP measures are highly cost effective in providing SDR.	Partial
The plan reduces risks to public safety.	Inlet management measures maintain navigation safety and contribute to increased storm damage risk reduction, and BCP reduces risk of hazardous storm surge in the bay and excessive shoaling of navigation inlets.	Full

Plan 1 includes breach response plans along the barrier island, and inlet bypassing at the inlets achieved by continuation of the authorized projects at the inlets, and the additional bypassing of sand through dredging of the ebb shoal in the amount of 100,000 CY per year at each inlet. The results of the above analysis, shows that plan 1 (both 1a, and 1b) is marginally effective.

This plan is not a complete solution, in that it only addresses damages that occur due to a breach remaining open, and as a result reduce only a small percentage of the overall damages. This plan only addresses 10% of the damages. The remaining damages that arise due to a combination of breach occurrence, bayside flooding, and shorefront damages remain unaddressed.

When considering this Plan in comparison with the Vision Criteria, it has its strengths and it shortcomings. The areas where this plan falls short in comparison with the vision objectives, are in the following areas:

1. The plan doesn't address all the contributors to damages.
2. The plan doesn't fully address the need for non-structural measures to provide both storm damage reduction and ecosystem integrity
3. The plan does not fully address the needs for protection and restoration of natural landforms and habitat
4. The plan does not meet the objective for including appropriate modification of existing structures

The shortcomings that exist with Plan 1 highlight the need to consider additional plan elements. The shortcomings are addressed in the following alternative plans, with the inclusion of additional plan elements.

7.8.3 Plan 2

The second series of plans (Plan 2.a through Plan 2.h) reflect the addition of non-structural protection to Plan 1.a and Plan 1.b. The inclusion of non-structural protection is considered essential to address flooding from storm surge propagating through inlets into the bays and wind and wave setup within the bays. The Non Structural Alternatives selected for consideration in Plan 2 include both the economically optimum Alternative NS2, which provides coastal storm damage risk reduction to 3,400 structures, and Alternative NS2-r, which supplements the non-structural features by raising selected roadways. In addition, the NS3 and NS3-r Alternatives, which cumulatively provide Storm Damage Reduction Benefits to an additional 2,000 buildings over NS2, have also been included. These plans are shown in Figure 7-15.

Table 7-74 through Table 7-76 present the Storm Damage Reduction Benefits, Annual Costs and Benefit Cost Ratios for Plans 2.a through 2.h. Plans 2.a through 2.d include combinations of the Management and Non-structural Alternatives without the Road Raising features, while plans 2.e through 2.h include the same combinations but with the addition of

Road Raising at four locations as described in Section 7.7. Each of the overall Plans provides a BCR of 1.3 or higher, and each of the Project Reaches has a BCR of greater than 1.1

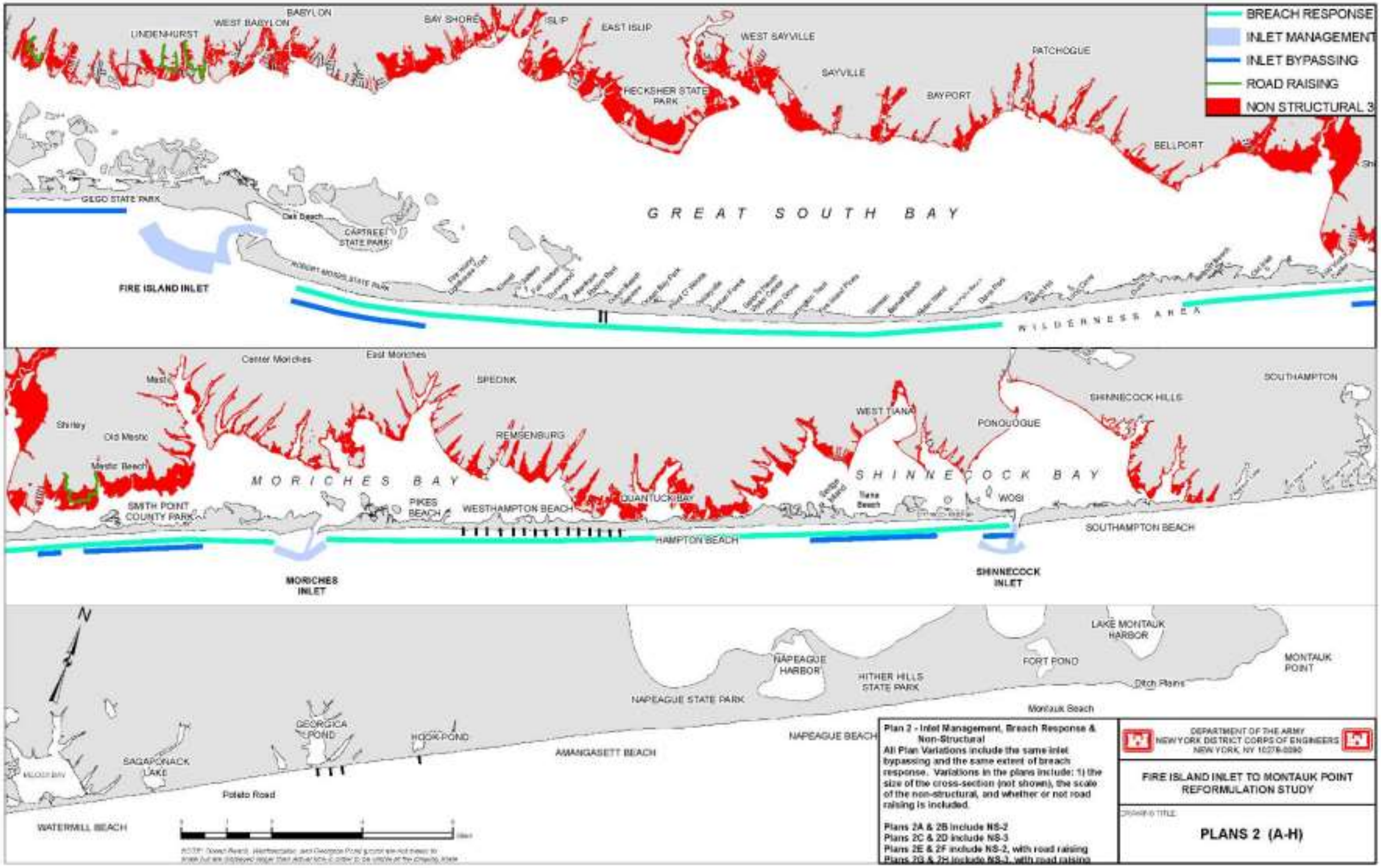


Figure 7-15. Alternative Plan 2 Overview

Table 7-74: Annual Benefits Plan 2 – Management & Non-Structural Plans

	Plan 2.a	Plan 2.b	Plan 2.c	Plan 2.d	Plan 2.e	Plan 2.f	Plan 2.g	Plan 2.h
Benefit Category	**Inlet Managemen t BCP 9.5, NS 2**	**Inlet Managemen t BCP 13, NS 2**	**Inlet Managemen t BCP 9.5, NS 3**	**Inlet Managemen t BCP 13, NS 3**	**Inlet Management BCP 9.5, NS2, Road Raising**	**Inlet Management BCP 13, NS2, Road Raising**	**Inlet Management BCP 9.5, NS 3, Road Raising**	**Inlet Management BCP 13, NS 3, Road Raising**
Inundation								
Mainland	$38,410,000	38,410,000	$45,270,000	$45,270,000	$40,020,000	$40,020,000	$46,500,000	$46,500,000
Barrier	$40,000	$40,000	$40,000	$40,000	$40,000	$40,000	$40,000	$40,000
Total Inundation	*$38,450,000*	*$38,450,000*	*$45,310,000*	*$45,310,000*	*$40,060,000*	*$40,060,000*	*$46,540,000*	*$46,540,000*
Breach								
Inundation	$8,840,000	$8,980,000	$8,840,000	$8,980,000	$8,840,000	$8,980,000	$8,840,000	$8,980,000
Structure Failure	$240,000	$230,000	$240,000	$230,000	$240,000	$230,000	$240,000	$230,000
Total Breach	*$9,080,000*	*$9,210,000*	*$9,080,000*	*$9,210,000*	*$9,080,000*	*$9,210,000*	*$9,080,000*	*$9,210,000*
Shorefront	$0	$0	$0	$0	$0	$0	$0	$0
Total Storm Damage Reduction	*$47,530,000*	*$47,660,000*	*$54,390,000*	*$54,520,000*	*$49,140,000*	*$49,270,000*	*$55,620,000*	*$55,750,000*
Costs Avoided								
Breach Closure	$2,160,000	$2,160,000	$2,160,000	$2,160,000	$2,160,000	$2,160,000	$2,160,000	$2,160,000
Beach Maintenance	$0	$0	$0	$0	$0	$0	$0	$0
Total Benefits	***$49,690,000***	***$49,820,000***	***$56,550,000***	***$56,680,000***	***$51,300,000***	***$51,430,000***	***$57,780,000***	***$57,910,000***

Interest Rate 5.125%, Project Life 50 years

Table 7-75. Annual Cost Plan 2 – Management & Non-Structural Plans

	Plan 2.a	Plan 2.b	Plan 2.c	Plan 2.d	Plan 2.e	Plan 2.f	Plan 2.g	Plan 2.h
Cost Category	**Inlet Management BCP 9.5, NS 2**	**Inlet Management BCP 13, NS 2**	**Inlet Management BCP 9.5, NS 3**	**Inlet Management BCP 13, NS 3**	**Inlet Management BCP 9.5, NS2, Road Raising**	**Inlet Management BCP 13, NS2, Road Raising**	**Inlet Management BCP 9.5, NS 3, Road Raising**	**Inlet Management BCP 13, NS 3, Road Raising**
Beach Fill	$0	$0	$0	$0	$0	$0	$0	$0
Nonstructural	$435,400,000	$435,400,000	$590,500,000	$590,500,000	$407,000,000	$407,000,000	$550,600,000	$550,600,000
Road Raising	$0	$0	$0	$0	$15,000,000	$15,000,000	$15,000,000	$15,000,000
Total First Cost	*$435,400,000*	*$435,400,000*	*$590,500,000*	*$590,500,000*	*$422,000,000*	*$422,000,000*	*$565,600,000*	*$565,600,000*
Total IDC	$13,800,000	$13,800,000	$18,700,000	$18,700,000	$13,300,000	$13,300,000	$17,800,000	$17,800,000
Total Investment Cost	*$449,300,000*	*$449,300,000*	*$609,300,000*	*$609,300,000*	*$435,300,000*	*$435,300,000*	*$583,500,000*	*$583,500,000*
Interest and Amortization	$25,100,000	$25,100,000	$34,000,000	$34,000,000	$24,300,000	$24,300,000	$32,600,000	$32,600,000
Operation & Maintenance	$7,100,000	$7,300,000	$7,300,000	$7,100,000	$7,300,000	$7,100,000	$7,300,000	$7,100,000
Renourishment	$0	$0	$0	$0	$0	$0	$0	$0
Subtotal	*$32,400,000*	*$32,100,000*	*$41,300,000*	*$41,100,000*	*$31,600,000*	*$31,400,000*	*$39,900,000*	*$39,600,000*
Annual Breach Closure Cost	$1,100,000	$800,000	$1,100,000	$800,000	$1,100,000	$800,000	$1,100,000	$800,000
Major Rehabilitation	$0	$0	$0	$0	$0	$0	$0	$0
Total Annual Cost	***$33,500,000***	***$32,900,000***	***$42,400,000***	***$41,800,000***	***$32,700,000***	***$32,100,000***	***$41,000,000***	***$40,400,000***

Interest Rate 5.125%, Project Life 50 years

Table 7-76. Net Benefits and BCR, By Project Reach Plan 2 – Management & Non-Structural Plans

Component	Plan 2.a Inlet Managemen t BCP 9.5, NS 2	Plan 2.b Inlet Managemen t BCP 13, NS 2	Plan 2.c Inlet Managemen t BCP 9.5, NS 3	Plan 2.d Inlet Managemen t BCP 13, NS 3	Plan 2.e Inlet Managemen t BCP 9.5, NS2, Road Raising	Plan 2.f Inlet Management BCP 13, NS2, Road Raising	Plan 2.g Inlet Management BCP 9.5, NS 3, Road Raising	Plan 2.h Inlet Management BCP 13, NS 3, Road Raising
Total Project								
Total Annual Cost	$33,500,000	$32,900,000	$42,400,000	$41,800,000	$32,700,000	$32,100,000	$41,000,000	$40,400,000
Total Benefits	$49,700,000	$49,800,000	$56,500,000	$56,700,000	$51,300,000	$51,400,000	$57,800,000	$57,900,000
Net Benefits	$16,200,000	$16,900,000	$14,100,000	$14,800,000	$18,600,000	$19,300,000	$16,800,000	$17,500,000
Benefit-Cost Ratio	1.5	1.5	1.3	1.4	1.6	1.6	1.4	1.4
Project Reaches								
Great South Bay								
Total Annual Cost	$19,200,000	$18,800,000	$24,000,000	$23,500,000	$19,100,000	$18,700,000	$23,800,000	$23,400,000
Total Benefits	$30,000,000	$30,100,000	$33,800,000	$33,900,000	$31,100,000	$31,200,000	$34,900,000	$35,000,000
Net Benefits	$10,800,000	$11,300,000	$9,900,000	$10,400,000	$11,900,000	$12,500,000	$11,100,000	$11,600,000
Benefit-Cost Ratio	1.6	1.6	1.4	1.4	1.6	1.7	1.5	1.5
Moriches Bay								
Total Annual Cost	$10,700,000	$10,500,000	$12,800,000	$12,700,000	$9,900,000	$9,800,000	$11,500,000	$11,400,000
Total Benefits	$13,100,000	$13,100,000	$14,500,000	$14,500,000	$13,600,000	$13,600,000	$14,700,000	$14,700,000
Net Benefits	$2,400,000	$2,600,000	$1,700,000	$1,800,000	$3,700,000	$3,800,000	$3,200,000	$3,300,000
Benefit-Cost Ratio	1.2	1.2	1.1	1.1	1.4	1.4	1.3	1.3
Shinnecock Bay								
Total Annual Cost	$3,600,000	$3,500,000	$5,500,000	$5,500,000	$3,600,000	$3,500,000	$5,500,000	$5,500,000
Total Benefits	$6,600,000	$6,600,000	$8,200,000	$8,200,000	$6,600,000	$6,600,000	$8,200,000	$8,200,000
Benefit-Cost Ratio	$3,000,000	$3,100,000	$2,700,000	$2,700,000	$3,000,000	$3,100,000	$2,700,000	$2,700,000
Benefit-Cost Ratio	1.8	1.9	1.5	1.5	1.8	1.9	1.5	1.5

Interest Rate 5.125%, Project Life 50 years

As seen in Table 7-71 through Table 7-73, combining Inlet Management and Non-structural Alternatives to develop Alternative Plans does not alter which Breach Closure design and which Non-structural Alternative provide the most Storm Damage Reduction Benefits in excess of costs. The primary Storm Damage Reduction Benefits of Plans 2.a through 2.h are the reduction of structure and content damage due to high frequency flooding of residential development within the bays. This high frequency flooding is generally a result of surge through the inlets and wind setup within the bays. With the exception of the locations proposed for road raising, Plans 2.a through 2.h will have very little impact on actual water levels, and will not provide substantial reductions in emergency response & evacuation costs or car damage.

Evaluation of Plan 2 Alternatives

<u>NED Evaluation.</u>

The analysis of these alternatives show that all of the alternatives that include breach response, inlet modifications, and mainland non-structural measures are cost-effective, with a BCR greater than 1. The plans that provide the greatest net benefits are Alternative 2f and 2h. Alternative 2f, which includes NS-2 with road raising may appear to be the preferred plan, but as discussed in Section 7.7, Alternative 2h includes a significantly larger number of structures to be protected with a design water elevation that is 0.5 ft larger than NS-2. Since these plans are so close in scale, and provide such similar results, Alternative 2h represents the best plan from this collection of alternative Plan 2.

<u>P&G Evaluation.</u>

The existing Principles and Guidelines establish that alternative plans should be complete, effective, efficient and implementable. This evaluation discusses how well these alternative plans meet these objectives. These alternatives that combine inlet bypassing, breach response plans, and mainland non-structural alternatives are still not complete solutions. These plans address the storm damage problems associated with a breach being open, address the chronic erosion in the vicinity of inlets, and address damages due to flooding along the bayside shoreline. Combined, these plans address approximately 50% of the damages that are likely to occur in the study area. While these plans are better, they still have a relatively high level of residual damages. Under this alternative there would still remain a high level of damages to the shorefront, and a high likelihood of recurring breaches. Based purely on the storm damage reduction these plans are effective, and efficient. These alternative plans are implementable. As discussed previously, there is general support for bypassing and breach closure, with specific details that need to be coordinated with the USFWS, and FIIS, to ensure that the closure procedures are consistent with their requirements. There are no institutional limitations in implementing Non-structural measures. It must be recognized however, that non-structural plans to retrofit 5,000 buildings, is a difficult undertaking, which requires voluntary participation, and would likely require multiple decades to implement

Vision Criteria Evaluation.

The alternatives of Plan 2 (2a to 2h) were evaluated in relationship to the planning criteria developed to reflect the Project objectives. This systematic assessment ensures that the Vision Statement approach is fully integrated into the development and selection of the FIMP Plan. Table 7-77 provides a summary of the evaluation of these measures relative to the established criteria.

Table 7-77. Plan 2 Alternatives (Inlet Management and BCP plus Non-Structural Retrofit)

Evaluation Criteria	Assessment	Rating
The plan or measure provides identifiable reductions in risk from future storm damage.	Inlet management helps avoid excessive erosion in areas affected by inlets. Breach closure provides quantified reduction in storm damage. Non-structural retrofit provides quantifiable reductions in storm damage to the specific structures and contents.	Full
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science considered a lower priority.	The sediment management and BCP components are based on proven application within the Project area. Non-Structural building retrofits are a standard method for flood mitigation. Some individual structures may present design challenges, requiring a comparatively large cost contingency.	Full
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	The sediment management and BCP components will reduce some flooding from direct ocean storm surge and from periodic overwash or breaching. The non-structural retrofit and road-raising components address bayside flooding from all causes except open coast storm surge, including storm surge propagating through the inlets and wind and wave setup within the bays.	Partial
The plan incorporates appropriate non-structural features to provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity	The plan provides management and non-structural components that contribute to SDR and help to restore coastal processes and ecosystem integrity.	Partial
The plan helps protect and restore coastal landforms and natural habitat.	The plan will reduce or eliminate deficits in longshore sediment transport and will restore the barrier island landform after a breach. As noted in Table 7-66, more rapid breach closure could reduce the	Partial

Table 7-77. Plan 2 Alternatives (Inlet Management and BCP plus Non-Structural Retrofit)		
Evaluation Criteria	**Assessment**	**Rating**
	volume cross island transport contributing to the formation of spits and shoals.	
The plan avoids or minimizes adverse environmental impacts	The use of improved sediment and breach management reduces the volume of breach closure or other dredging, reducing impacts. The use of non-structural retrofits may reduce the need for reliance on structural measures that have larger impacts.	Full
The plan addresses long-term demands for public resources.	The plan incorporates required navigation maintenance to provide future cost efficiencies. Future monitoring and restoration to maintain the beach profile to prevent repetitive breaching and limit future expenses. The non-structural features require no long term public involvement beyond monitoring. The benefits of the non-structural features will minimize the need for structural features.	Full
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	Locations for placement of bypassed sediment provide both storm damage reduction and restoration. The BCP decision process balances SDR needs and environmental effects. Non-structural retrofit has no effect.	Full
The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures	N/A	No
The plan incorporates appropriate alterations of inlet stabilization measures and dredging practices	Measures to alter dredging practices were considered more appropriate than structural changes to the inlets. Non-structural retrofit has no effect.	Full.
The plan is efficient and represents a cost effective use of resources	The sediment management measures provide significant economic benefit and environmental process restoration. BCP measures are extremely cost-effective. Non-structural measures are highly cost-effective when targeted to frequently flooded structures.	Full
The plan reduces risks to public safety.	Inlet management measures maintain navigation safety and contribute to increased storm protection, while the BCP reduces risk of hazardous storm	Full

Table 7-77. Plan 2 Alternatives (Inlet Management and BCP plus Non-Structural Retrofit)		
Evaluation Criteria	**Assessment**	**Rating**
	surge in the bay and excessive shoaling in navigation inlets. Non-structural measures reduce damage only. It is important to maintain evacuation plans so that residents do not remain in homes that are inaccessible during a flood event. (Note: Plans 2.e through 2.f contain road-raising in limited areas, which may improve evacuation and access by reducing inundation of roads within protected areas and providing means of egress.)	

Plan 2 Summary.

Plan 2 includes breach response, inlet modifications, and mainland non-structural measures. All of the alternative plans are cost-effective, with a BCR greater than 1. The plans that provide the greatest net benefits are Alternative 2F and 2H. Alternative 2F, which includes NS-2 may appear to be the preferred plan, but as discussed in Section 7.7, Alternative 2H includes a significantly larger number of structures to be protected with a design water elevation that is 0.5 ft larger than NS-2. Since these plans are so close in scale, and provide such similar results, the recommendation would be that Alternative 2H represents the best plan from this collection of alternative plan 2. Alternative 2H includes inlet management at the inlets (consistent with each alternative), a breach response plan with the +13 ft cross-section, non-structural plan 3, which addresses structures in the existing 10-yr floodplain, and road raising at 4 locations.

When these Plans are considered relative to the Vision, they all provide similar results, since the features are similar in all plans. These plans, with the inclusion of the non-structural measures along the mainland advance a greater number of Vision Objectives, than plan 1, but still have some shortcomings when compared with the Vision criteria.

The areas where this plan falls short in comparison with the vision objectives, are in the following areas:

1. The plan doesn't address all the contributors to damages. While the plan now does address the increased flooding due to breaching, and the flooding in the back-bay, this alternative does not address coastal damages that would occur along the ocean shorefront.
2. While this plan includes a tremendous amount of non-structural efforts along the mainland, the plan doesn't fully address the need for non-structural

measures to provide both storm damage reduction and ecosystem integrity along the barrier island system.

3. The plan does not fully address the needs for protection and restoration of natural landforms and habitat
4. The plan does not meet the objective for including appropriate modification of existing structures

These shortcomings suggest the need to include the next increment of alternatives. These short-comings can be addressed with the inclusion of the next increment of effort.

7.8.4 Plan 3

The third series of plans (Plan 3.a through Plan 3.g) reflects the addition of Beach Nourishment to Plans 2.e through Plan 2.h. The inclusion of Beach Nourishment will more fully address the various sources of flooding and will also address any significant erosion resulting from alterations of the existing shoreline stabilization structures. The Non-structural Alternatives selected for inclusion in these Plans include the Road Raising feature, which provides significant benefits above Plans 2.a through 2.d that exclude this feature.

The Beach Nourishment Alternative included in these Plans is the 15 ft dune/ 90 ft berm width design with the minimum real estate alignment. This design and alignment were identified as having the highest net benefits in Section 7.7. Although the minimum Real Estate alignment was selected for alternative comparison, since the costs and benefits of the Middle alignment are close, it is expected that an evaluation including the middle alignment would offer similar results. The analysis in Section 7.7 also identified that the Beach Nourishment Alternatives are not cost effective in reducing storm damage in the Shinnecock Bay, Ponds, and Montauk Reaches. Plans 3.a through 3.g, therefore, have excluded beach nourishment in these reaches. Within the Shinnecock Bay reach the Breach Contingency Plan with the +13 ft design section has been included. For Reaches protected by Beach Nourishment, breaches would be closed to the design section as part of the project maintenance or major rehabilitation.

Within the Great South Bay and Moriches Bay Reaches, there are several environmentally sensitive areas along the barrier island that present a risk of future breaching with significant damage to back bay development, but with little or no human development on the barrier. These locations include the Otis Pike Wilderness Area (OPWA), areas designated as Major Federal Tracts (MFT) by the Fire Island National Seashore (FIIS), and the Smith Point County Park (SPCP). Plans were developed to evaluate the impact of excluding these locations on Storm Damage Reduction Benefits, Costs and BCRs. For Plans 3.b through 3.g, at any location in the Great South Bay and Moriches Bay Reaches where beachfill has been excluded due to environmental concerns, the Breach Contingency Plan with a 9.5 ft closure design has been included. The lower level closure design has been selected for these locations as the alternative most compatible with special environmental concerns. Figure 7-16 illustrates the conceptual layout of alternative plans 3a to g.

Table 7-78 through Table 7-80 present the Storm Damage Reduction Benefits, Annual Costs and Benefit Cost Ratios for Plans 3.a through 3.g

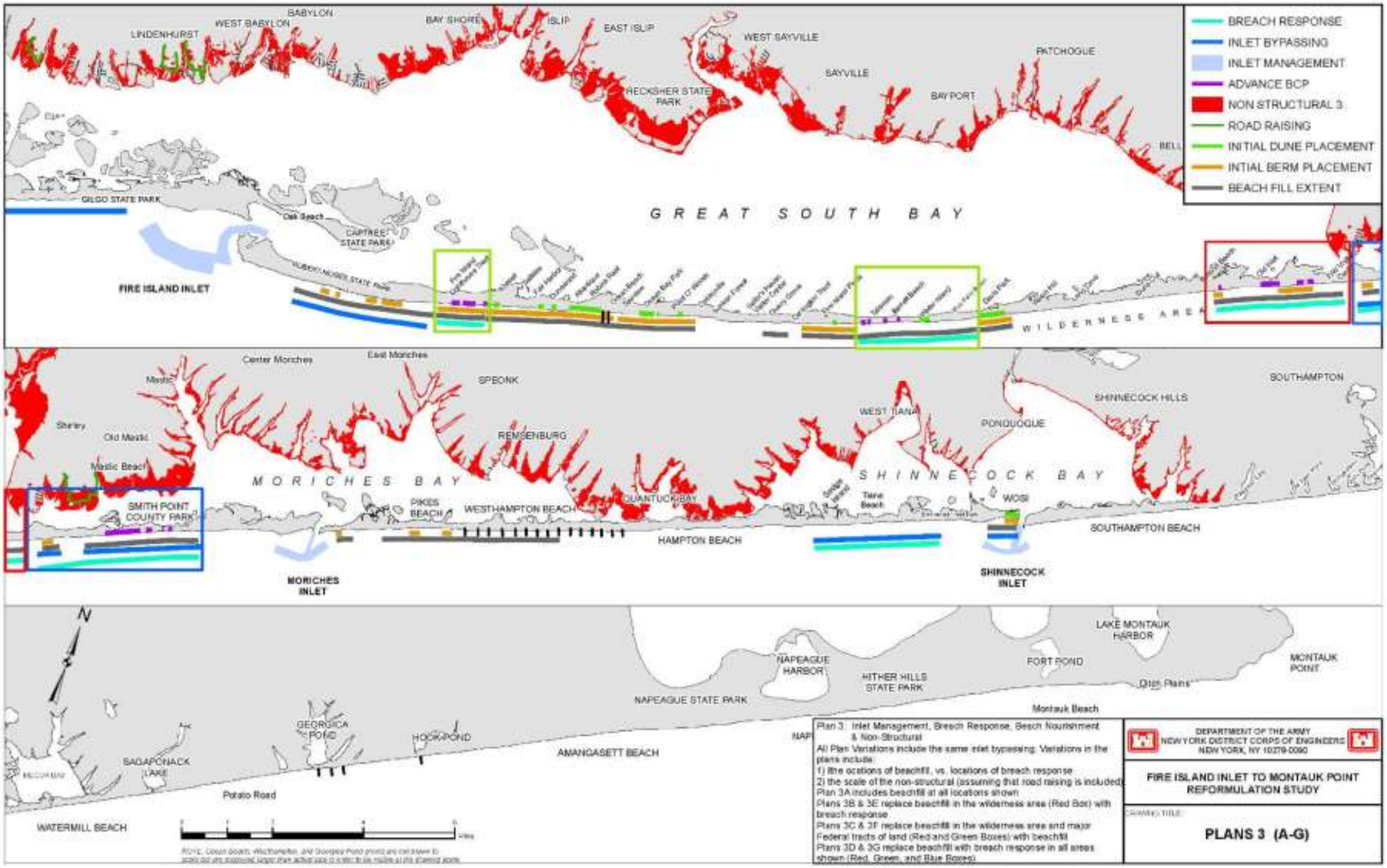


Figure 7-16. Alternative Plan 3 Overview

Table 7-78. Annual Benefits Plan 3 – Management, Non-Structural and Beach Nourishment Plans

	Plan 3.a	Plan 3.b	Plan 3.c	Plan 3.d	Plan 3.e	Plan 3.f	Plan 3.g
Benefit Category	Inlet Mgmt, BCP 13 @SB, NS2R, 15ft Dune @ GSB & MB	Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, NS2R, 15 ft Dune @ GSB & MB	Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA & MFT, NS2R, 15 ft Dune @ GSB & MB	Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, MFT, & SPCP, NS2R, 15 ft Dune @ GSB & MB	Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, NS3R, 15 ft Dune @ GSB & MB	Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA & MFT, NS3R, 15 ft Dune @ GSB & MB	Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, MFT, & SPCP, NS3R, 15 ft Dune @ GSB & MB
Inundation							
Mainland	$49,020,000	$48,340,000	$46,390,000	$43,260,000	$54,320,000	$52,560,000	$49,600,000
Barrier	$2,510,000	$2,460,000	$1,960,000	$1,890,000	$2,460,000	$1,960,000	$1,890,000
Total Inundation	*$51,540,000*	*$50,800,000*	*$48,350,000*	*$45,160,000*	*$56,780,000*	*$54,510,000*	*$51,500,000*
Breach							
Inundation	$9,230,000	$9,040,000	$8,990,000	$8,920,000	$9,040,000	$8,990,000	$8,920,000
Structure Failure	$370,000	$370,000	$360,000	$360,000	$370,000	$360,000	$360,000
Total Breach	*$9,600,000*	*$9,410,000*	*$9,350,000*	*$9,280,000*	*$9,410,000*	*$9,350,000*	*$9,280,000*
Shorefront	$3,260,000	$3,260,000	$3,250,000	$3,180,000	$3,260,000	$3,250,000	$3,180,000
Total Storm Damage Reduction	*$64,770,000*	*$63,470,000*	*$60,950,000*	*$57,620,000*	*$69,450,000*	*$67,110,000*	*$63,960,000*
Costs Avoided							
Breach Closure	$2,160,000	$2,160,000	$2,160,000	$2,160,000	$2,160,000	$2,160,000	$2,160,000
Beach Maintenance	$2,400,000	$2,400,000	$2,400,000	$2,400,000	$2,400,000	$2,400,000	$2,400,000
Total Benefits	***$68,960,000***	***$68,040,000***	***$65,500,000***	***$62,180,000***	***$74,020,000***	***$71,760,000***	***$68,520,000***

Interest Rate 5.125%, Project Life 50 years

Table 7-79. Annual Cost Plan 3 – Management, Non-Structural and Beach Nourishment Plans

Cost Category	Plan 3.a Inlet Mgmt, BCP 13 @SB, NS2R, 15ft Dune @ GSB & MB	Plan 3.b Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, NS2R, 15 ft Dune @ GSB & MB	Plan 3.c Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA & MFT, NS2R, 15 ft Dune @ GSB & MB	Plan 3.d Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, MFT, & SPCP, NS2R, 15 ft Dune @ GSB & MB	Plan 3.e Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, NS3R, 15 ft Dune @ GSB & MB	Plan 3.f Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA & MFT, NS3R, 15 ft Dune @ GSB & MB	Plan 3.g Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, MFT, & SPCP, NS3R, 15 ft Dune @ GSB & MB
Beach Fill	$160,200,000	$148,700,000	$146,000,000	$139,200,000	$148,700,000	$146,000,000	$139,200,000
Nonstructural	$407,200,000	$407,200,000	$407,200,000	$407,200,000	$550,800,000	$550,800,000	$550,800,000
Road Raising	$14,900,000	$14,900,000	$14,900,000	$14,900,000	$14,900,000	$14,900,000	$14,900,000
Total First Cost	*$582,400,000*	*$570,800,000*	*$568,000,000*	*$561,400,000*	*$714,500,000*	*$711,800,000*	*$705,000,000*
Total IDC	$26,600,000	$25,700,000	$25,400,000	$24,900,000	$30,200,000	$30,000,000	$29,400,000
Total Investment Cost	*$609,000,000*	*$596,500,000*	*$593,400,000*	*$586,300,000*	*$744,700,000*	*$741,800,000*	*$734,400,000*
Interest and Amortization	$34,000,000	$33,300,000	$33,100,000	$32,700,000	$41,600,000	$41,400,000	$41,000,000
Operation & Maintenance	*$9,300,000*	*$9,200,000*	*$9,100,000*	*$8,900,000*	*$9,200,000*	*$9,100,000*	*$8,900,000*
Renourishment	*$12,900,000*	*$12,500,000*	*$11,600,000*	*$11,000,000*	*$12,500,000*	*$11,600,000*	*$11,000,000*
Subtotal	*$56,200,000*	*$55,000,000*	*$53,800,000*	*$52,600,000*	*$63,300,000*	*$62,100,000*	*$60,900,000*
Annual Breach Closure Cost	*$0*	*$500,000*	*$600,000*	*$1,000,,000*	*$500,000*	*$600,000*	*$1,000,,000*
Major Rehabilitation	*$0*	*$0*	*$0*	*$0*	*$0*	*$0*	*$0*
Total Annual Cost	*$56,200,000*	*$55,600,000*	*$54,500,000*	*$53,600,000*	*$63,800,000*	*$62,800,000*	*$61,900,000*

Interest Rate 5.125%, Project Life 50 years

Table 7-80. Net Benefits and BCR, By Project Reach Plan 3 – Management, Non-Structural and Beach Nourishment Plans

Component	Plan 3.a Inlet Mgmt, BCP 13 @SB, NS2R, 15ft Dune @ GSB & MB	Plan 3.b Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, NS2R, 15 ft Dune @ GSB & MB	Plan 3.c Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA & MFT, NS2R, 15 ft Dune @ GSB & MB	Plan 3.d Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, MFT, & SPCP, NS2R, 15 ft Dune @ GSB & MB	Plan 3.e Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, NS3R, 15 ft Dune @ GSB & MB	Plan 3.f Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA & MFT, NS3R, 15 ft Dune @ GSB & MB	Plan 3.g Inlet Mgmt, BCP 13 @ SB, BCP 9.5 @ OPWA, MFT, & SPCP, NS3R, 15 ft Dune @ GSB & MB
Total Project							
Total Annual Cost	$56,200,000	$55,600,000	$54,500,000	$53,600,000	$63,800,000	$62,800,000	$61,900,000
Total Benefits	$69,000,000	$68,000,000	$65,500,000	$62,200,000	$74,000,000	$71,700,000	$68,500,000
Net Benefits	$12,800,000	$12,500,000	$11,000,000	$8,600,000	$10,200,000	$8,900,000	$6,600,000
Benefit-Cost Ratio	1.2	1.2	1.2	1.2	1.2	1.1	1.1
Project Reaches							
Great South Bay							
Total Annual Cost	$36,900,000	$36,200,000	$35,200,000	$35,200,000	$40,900,000	$39,900,000	$39,900,000
Total Benefits	$44,900,000	$44,300,000	$41,800,000	$41,300,000	$47,800,000	$45,500,000	$45,000,000
Net Benefits	$8,100,000	$8,100,000	$6,600,000	$6,200,000	$6,800,000	$5,600,000	$5,200,000
Benefit-Cost Ratio	1.2	1.2	1.2	1.2	1.2	1.1	1.1
Moriches Bay							
Total Annual Cost	$15,700,000	$15,700,000	$15,700,000	$14,800,000	$17,300,000	$17,300,000	$16,400,000
Total Benefits	$17,400,000	$17,100,000	$17,100,000	$14,200,000	$18,100,000	$18,000,000	$15,300,000
Net Benefits	$1,700,000	$1,400,000	$1,400,000	-$600,000	$800,000	$700,000	-$1,100,000
Benefit-Cost Ratio	1.1	1.1	1.1	1.0	1.0	1.0	0.9
Shinnecock Bay							
Total Annual Cost	$3,500,000	$3,500,000	$3,500,000	$3,500,000	$5,500,000	$5,500,000	$5,500,000
Total Benefits	$6,600,000	$6,600,000	$6,600,000	$6,600,000	$8,200,000	$8,200,000	$8,200,000
Benefit-Cost Ratio	$3,100,000	$3,100,000	$3,100,000	$3,100,000	$2,700,000	$2,700,000	$2,700,000
Benefit-Cost Ratio	1.9	1.9	1.9	1.9	1.5	1.5	1.5

NED Analysis.

The analysis of plans with beach nourishment reveals that all of the plans are economically viable. Plan 3.a provides greater storm damage reduction benefits than Plan 2f, but the net Storm Damage Reduction Benefits that are less than those of Plan 2.f. Although beach nourishment is cost-effective in providing storm damage reduction as a first-added or stand-alone measure, there is some duplication in benefits between the BCP and non-structural measures of Plan 2.f, and the additional beach fill in Plan 3.

The results of this analysis also indicate that eliminating sections of beach nourishment from the Great South Bay and Moriches Bay reaches, and replacing these features with breach response, results in increases in damages that are greater than the reductions in cost. Plan 3.d, for example results in an approximately $7,000,000 increase in annual damage and a $3,000,000 decrease in annual cost relative to Plan 3.a. These breach response alternatives were evaluated considering a responsive plan, and a breach maintenance plan that requires a significant amount of dune lowering and beach loss, prior to action being taken (and no maintenance in the wilderness area). If the triggers for implementation were adjusted to establish action being taken when the beach and dune contains a greater volume of material than presently considered, the costs for breach response would be higher but less than for beach nourishment. Similarly, as the trigger for breach response gets larger, the benefits would increase, and eventually approach the benefits for beachfill. Therefore, the costs and benefits are bracketed by the alternatives that have been evaluated. This illustrates that the breach triggers could be increased in scale to account for a larger breach threshold trigger and remain economically viable, so long as the annual costs are less than the beachfill plan.

An additional important result of this analysis is that when the non-structural and beach nourishment components of the project are combined, the overall project remains economically justified for all combinations evaluated. This result was anticipated because the non-structural plan is targeted to the structures that flood most frequently, meaning that most of the damage reduced by the non-structural components is caused by flow through the inlets and local wind and wave setup, not by overwash or breaching of the barrier islands. In contrast, the back bay damage reduction for the beach nourishment component is related to damage from more extreme events that cause overwash or breaching. The results are plans that are highly complimentary in addressing damage from both high frequency repetitive flooding, and the potential for elevated water levels during larger, less frequent events.

There are concerns regarding the rate at which the non-structural measures could be constructed, and the overall time required for full construction. Practical constraints include the availability of funding, availability of trained construction workforce, and development of effective implementation strategies. Thus, full implementation of the non-structural measures is expected to take a significant period of time.

The BCP and beachfill measures are typically considered to be constructible more rapidly. When these factors are considered, this would further emphasize the relative benefits, in comparison to the other alternatives.

<u>P&G Evaluation.</u>

The existing Principles and Guidelines establish that alternative plans should be complete, effective, efficient and implementable. This evaluation discusses how well these alternative plans meet these objectives. These alternatives that combine inlet bypassing, breach response plans, mainland non-structural alternatives, and shorefront solutions are not complete solutions, but are as complete as any of the alternatives evaluated. These plans address the storm damage problems associated with a breach being open, address the chronic erosion in the vicinity of inlets, address damages due to flooding along the bayside shoreline, address damages that occur due to breach formation, and address shorefront damages. Combined, these plans address approximately 75% of the damages that are likely to occur in the study area. While these plans are the most effective in reducing damages, they still have residual damages. Under this alternative there would still remain the potential for damages due to events that exceed the design, and damages in areas where there are no project features. Based on the storm damage reduction these plans are effective, and efficient. These alternative plans vary in being implementable. As discussed previously, there is general support for bypassing and breach closure, with specific details that need to be coordinated with the USFWS, and FIIS, to ensure that the closure procedures are consistent with their requirements. There are no institutional limitations in implementing Non-structural measures, although the size and voluntary nature of the alternative makes implementing the alternative more difficult. The beachfill component introduces challenges regarding implementibility. Generally in community areas, beachfill is accepted. Along Fire Island, particularly in areas fronting the Wilderness Area and Major Federal Tracts of Lands there are park service policies which dissuade this practice. In general, alternatives which do not place beachfill in these areas would be considered as more implementable.

Evaluation of Plan 3 Alternatives

The alternatives of Plan 3 (3a to 3g) were evaluated in relationship to the planning criteria developed to reflect the Project objectives. This systematic assessment ensures that the Vision Statement approach is fully integrated into the development and selection of the FIMP Plan. Table 7-81 provides a summary of the evaluation of these measures relative to the established criteria.

Table 7-81. Plan 3 Alternatives (Inlet Management and BCP, Non-Structural Retrofit, Beach Nourishment)		
Evaluation Criteria	**Assessment**	**Rating**
The plan or measure provides identifiable reductions in risk from future storm damage.	Inlet management helps avoid excessive erosion in areas affected by inlets. Breach closure provides quantified reduction in storm damage. Non-structural retrofit provides quantifiable reductions in storm damage to the specific structures and contents. Beach nourishment reduces risks to structures directly on the shorefront and reduces overwash and breaching.	Full
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science considered a lower priority.	The sediment management and BCP components are based on proven application within the Project area. Non-Structural building retrofits are a standard method for flood mitigation. Some individual structures may present design challenges, requiring a comparatively large cost contingency. Beach fill has been widely used in the project area and other locations, and is readily reversible.	Full
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	The sediment management and BCP components will reduce some flooding from direct ocean storm surge and from periodic overwash or breaching. , The non-structural retrofit and road-raising components address bayside flooding from all causes except open cast storm surge, including storm surge propagating through the inlets and wind and wave setup within the bays. Beach nourishment addresses open coast storm surge and flow into the bays due to periodic overwash and breaching of barrier islands.	Full
The plan incorporates appropriate non-structural features to provide both storm damage and to restore coastal processes and ecosystem integrity	The plan provides management and non-structural components that contribute to SDR and help to restore coastal processes and	Partial

Table 7-81. Plan 3 Alternatives (Inlet Management and BCP, Non-Structural Retrofit, Beach Nourishment)		
Evaluation Criteria	**Assessment**	**Rating**
	ecosystem integrity. The beach nourishment measures help restore littoral transport by reducing sediment deficits. Some alternatives provided beach nourishment only in selected locations, allowing significant cross-shore transport where appropriate.	
The plan helps protect and restore coastal landforms and natural habitat.	The plan will reduce or eliminate deficits in longshore sediment transport and will restore the barrier island landform after a breach. As noted in Table 7-66, more rapid breach closure could reduce the volume of cross island transport contributing to the formation of spits and shoals. The non-structural measures have no direct impact on coastal landforms or natural habitat. At selected locations, beach nourishment will reduce erosion and thus protect adjacent habitat.	Partial
The plan avoids or minimizes adverse environmental impacts	The use of improved sediment and breach management reduces the volume of breach closure or other dredging, reducing impacts. The use of non-structural retrofits may reduce the need for reliance on structural measures that have larger impacts. Some plans avoid renourishment impacts to the Major Federal Tracts on Fire Island, Otis G. Pike Wilderness Area, and/or Smith Point County Park. The selection of borrow areas, limits in dredging windows, and other mitigation measures will reduce impacts of renourishment.	Full
The plan addresses long-term demands for public resources.	The plan incorporates required navigation maintenance to provide future cost efficiencies. Future monitoring and restoration to maintain the beach profile to prevent repetitive breaching and limit future expenses. The non-structural features require no long	Partial

Table 7-81. Plan 3 Alternatives (Inlet Management and BCP, Non-Structural Retrofit, Beach Nourishment)		
Evaluation Criteria	**Assessment**	**Rating**
	term public involvement beyond monitoring. The benefits of the non-structural measures will minimize the need for structural features. The assessment of beach renourishment in Table 7-69 considers periodic renourishment over the project life. Future levels of renourishment, including the profile design and level of maintenance, could be reduced to account for the benefit of non-structural retrofits and remain cost-effective.	
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	Locations for placement of bypassed sediment provide both storm damage reduction and restoration. The BCP decision process balances SDR needs and environmental effects. Non-structural retrofit has no effect. Beach nourishment promotes dune formation and longshore transport. It may reduce the frequency of breach closure because of higher dunes. Significant environmental effects will be minimized by selection and avoidance of certain areas.	Partial
The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures	Use of beach nourishment likely to be a prerequisite for alteration of existing shoreline stabilization structures.	Partial
The plan incorporates appropriate alterations of inlet stabilization measures and dredging practices	Measures to alter dredging practices were considered more appropriate than structural changes to the inlets. Non-structural retrofit and beach nourishment have no effect.	Full.
The plan is efficient and represents a cost effective use of resources	The sediment management measures provide significant economic benefit and environmental process restoration. BCP measures are extremely cost-effective. Non-structural measures are highly cost-effective when targeted to frequently flooded structures. Beach	Full

Table 7-81. Plan 3 Alternatives (Inlet Management and BCP, Non-Structural Retrofit, Beach Nourishment)		
Evaluation Criteria	**Assessment**	**Rating**
	nourishment is cost-effective in certain sections of the study area. The combination plan has a net positive benefit-cost ratio.	
The plan reduces risks to public safety.	Inlet management measures maintain navigation safety and contribute to increased storm protection, while the BCP reduces risk of hazardous storm surge in the bay and excessive shoaling in navigation inlets. Non-structural measures reduce damage only. It is important to maintain evacuation plans so that residents do not remain in homes that are inaccessible during a flood event. (Note: Plans 2.e through 2.f contain road-raising in limited areas, which may improve evacuation and access by reducing inundation of roads within protected areas and providing means of egress). Beach nourishment reduces breaching and overwash; reduces damage to shorefront buildings; reduces debris volumes; and eliminates potential hazard of buildings on the public beach (by moving the beach shoreward of existing buildings. Adequate beach width is needed to allow access for school buses, firefighting trucks and construction vehicles. The beachfront is their primary route to access the community areas.	Full

Plan 3 Summary.

As discussed in the text above, a review of the analysis of these alternatives shows that the plans of combined inlet management, breach response, non-structural retrofits, and beachfill are economically viable, and to different degrees satisfy the P&G criteria and the Vision criteria. The analysis shows that the relative effectiveness of the beachfill alternative plans is reduced, with each reduction in the alongshore extent of fill (replaced with breach response plans), corresponding with environmentally sensitive areas. This analysis does show that plans that do not include fill in the Federal tracts of land are economically viable. The plans that provide the greatest net benefits are the alternatives that include fill in the environmentally sensitive areas.

The plans, with the inclusion of beachfill advance a greater number of Vision Objectives, than plan 2, (particularly in addressing all the contributors to storm damages) but still have shortcomings when compared with the Vision criteria. When these Plans are considered relative to the Vision, they provide results that vary depending upon the extent of fill that is proposed, particularly as it relates to the criteria to balance storm damage reduction considerations with ecosystem restoration considerations. Plan 3A is the alternative which best addresses the Storm Damage Reduction needs, but includes beachfill throughout, and as a result does not rank highly with respect to the Vision criteria for balancing storm damage reduction needs and environmental needs, and also does not rank highly with consideration of the P&G criteria for implementibility, since it is contrary to NPS policies for fill within undeveloped tracts of land. Alternative 3G includes beachfill in the developed areas and replaces beachfill within the major public tracts of land with breach response plans. While this plan is less effective in reducing storm damages, it is a plan which is economically viable, is better aligned with the P&G criteria, as being more consistent with the NPS policies, and better achieves the Vision objectives in that this plan balances storm damage reduction needs and ecosystem restoration needs. It is also acknowledged that the breach response plans evaluated as part of this plan represent a scenario that introduces the greatest risk. As part of the final design, the breach response protocols can be adjusted to consider opportunities for further reducing the risk, by the establishment of a higher threshold at which action is taken.

The areas where this plan falls short in comparison with the vision objectives, are in the following areas:

1. While this plan includes a tremendous amount of non-structural retrofits along the mainland, the plan doesn't fully address the need for non-structural measures to provide both storm damage reduction and ecosystem integrity along the barrier island system.
2. The plan does not fully address the needs for protection and restoration of natural landforms and habitat.
3. The plan does not meet the objective for including appropriate modification of existing structures.
4. The extent to which the plans balance the need for storm damage reduction and habitat restoration, depends largely upon the alongshore extent of the dune fill. As discussed above, eliminating fill in the environmentally sensitive areas and focusing on protection within the community areas balances this consideration.
5. This plan does not fulfill the Vision objective of addressing the long-term demand for public resources, in that the plan requires a continued commitment to beach renourishment over the life of the project.

It is clear that the alternatives that were developed to meet the storm damage reduction efforts are not sufficient to address these Vision criteria. Addressing these criteria requires the consideration of additional alternatives that are described in the following Section.

7.8.5 Summary of NED Alternative Integration

A comparison of Alternative Plans 1, 2 and 3 are included in Table 7-82 below, which shows that Alternative Plan 3 is the plan that more completely addresses the NED criteria, the P&G criteria and the Vision Criteria. From the Alternative Plans evaluated within the framework of Plan 3, Plan 3A is the plan that best accomplishes the storm damage reduction objectives, while plan 3G is identified as the plan that best balances the storm damage reduction objectives, the P&G criteria, and the Vision Criteria.

Based upon this analysis of this evaluation, Plan 3A is identified as the plan that best accomplishes the storm damage reduction objectives, as measured by the NED, based upon the integration of the alternatives. Plan 3G is identified as the plan that best meets the three objectives of NED, the P&G and the Vision.

While these plans address the issues of storm damage reduction, and Plan 3G also advances the P&G requirements, and the Vision Criteria, these plans still do not achieve all of the objectives of the Vision Statement. The following short-comings are identified and used as the basis for considering additional alternatives in the next Section. In the following Section, alternative 3A is included for comparison, but Alternative 3G is used to establish the point of departure for considering plan variations to consider the following.

1. The plan doesn't fully address the need for non-structural measures to provide both storm damage reduction and ecosystem integrity along the barrier island system,
2. The plan does not fully address the need for protection and restoration of natural landforms and habitat,
3. The plan does not meet the objective for including appropriate modification of existing structures.

This plan requires a continued commitment to beach renourishment over the life of the project and does not fulfill the Vision objective of addressing the long-term demand for public resources.

Table 7-82. Summary of NED Alternative Integration Analysis

Evaluation Criteria	Plan 1	Plan 2	Plan 3A	Plan 3G
NED Criteria	Marginal	Full	**BEST**	Full
P&G Criteria				
- Complete	No	Partial	Yes	Yes
- Effective	Marginal	Yes	Yes	Yes
- Efficient	Marginal	Yes	Yes	Yes
- Implementable	Yes	Yes	Marginal	Yes
Vision Criteria				
The plan or measure provides identifiable reductions in risk from future storm damage.	Full	Full	Full	Full
The plan or measure is based on sound science and understanding of the system. Measures that may have uncertain consequences should be monitored and be readily modified or reversed. Measures that could have unintended consequences, based upon available science are considered a lower priority.	Full	Full	Full	Full
The plan or measure addresses the various causes of flooding, including open coast storm surge, storm surge propagating through inlets into the bays, wind and wave setup within the bays, and flow into the bays due to periodic overwash or breaching of the barrier islands.	Partial	Partial	Full	Full
The plan or measures incorporate appropriate non-structural features provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity	Partial	Partial	Partial	Partial
The plan or measure help protect and restore coastal landforms and natural habitat.	Partial	Partial	Partial	Partial
The plan avoids or minimizes adverse environmental impacts	Full	Full	Partial	Full
The plan addresses long-term demands for public resources.	Full	Full	Partial	Partial
Dune and beach nourishment measures consider both storm damage reduction, restoration of natural processes, and environmental effects.	Full	Full	Partial	Full
The plan or measure incorporates appropriate alterations of existing shoreline	N/A	N/A	Partial	Partial

Evaluation Criteria	Plan 1	Plan 2	Plan 3A	Plan 3G
stabilization structures				
The plan or measure incorporates appropriate alterations of inlet stabilization measures and dredging practices	Full	Full	Full	Full
The plan or measure is efficient and represents a cost effective use of resources	Partial	Full	Full	Full
The plan or measure reduces risks to public safety.	Full	Full	Full	Full

7.9 Integration of Features for Recommended Alternative

The results of the integration of the NED Features identifies Plan 3a as the plan that best accomplishes the storm damage reduction objectives, while Plan 3g is identified as the plan that best balances the storm damage reduction objectives, the P&G criteria, and the Vision Criteria. This analysis also shows that none of these alternative plans, standing alone, fully meet the Vision Criteria.

A Summary of these two plans is as follows:

Plan 3A, is the plan that functions optimally in reducing storm damages, (the plan that maximizes NED benefits). Plan3a includes inlet bypassing at the three inlets, NS-3 with road raising, continuous (as needed) beachfill along Great South Bay, and Moriches bay, and a breach response plan along Shinnecock Bay.

Plan 3G, is the combination of storm damage reduction alternatives that balances the objectives of storm damage reduction, P&G criteria, and Vision Criteria. This plan includes inlet bypassing at the 3 inlets, NS-3 with road raising, beachfill fronting the communities along Great South Bay, and Moriches Bay, and a breach response plan along unprotected areas of Great South Bay, Moriches Bay, and Shinnecock Bay

These plans accomplish much of the Vision Objectives, but fall short in the following Vision Criteria:

- The plan or measure incorporates appropriate alterations of existing shoreline stabilization structures
- The plan helps protect and restore coastal landforms and natural habitat.
- The plan incorporates appropriate non-structural features to provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity
- The plan addresses long-term demands for public resources.

This chapter considers the integration of additional alternatives to satisfy these Vision requirements. This chapter considers the following.

1. Integration of groin modification alternatives.

2. Integration of restoration alternatives
3. Integration of appropriate land use and development management alternatives
4. Consideration of the life cycle management of these plans.
   Consideration of Climate Change
   Considerations for Adaptive Management

7.9.1 Integration of Groin Modification Alternatives

In Section 7.7 and the Environmental Appendix of this report, groin modification alternatives were evaluated in the context of both storm damage reduction and habitat restoration. As described above, the Vision Statement advocates appropriate modification of coastal structures.

<u>Groin modifications for SDR.</u>

As presented in Section 7.7, the evaluation of groin modifications for purposes of storm damage reduction concluded that the existing groin field at Westhampton could be modified by shortening the groins and providing for increased sediment transport to the west, which in turn reduces the need for renourishment in this area. This groin modification would be considered as a storm damage reduction feature. For the groins at Georgica Pond, this analysis determined that the groins should not be modified because studies have shown that they have little measured impact on the downdrift shoreline. Instead, an intensive monitoring plan could be adopted to confirm the effect that the groins are having on the downdrift shorelines, to allow for consideration of future modification. At Ocean Beach, the findings for purposes of SDR was to not modify the Ocean Beach groins, because of the critical infrastructure located immediately landward of the dune. This analysis acknowledged that modification of the groins at Ocean Beach could help restore alongshore transport and could have NER benefits that should be evaluated. Any removal or modification of groins at Ocean Beach would need to include an alternative for the Village of Ocean Beach that would compensate for any negative effects of removal, and under any modification scenario would require the relocation of the Village water supply. Lastly it was recognized that groin modification would have limited effectiveness under any beachfill plans in alternative 3, because the groins would be largely buried.

<u>Groin modifications for Habitat Restoration.</u>

As discussed in the Environmental Appendix, groin modification alternatives were considered for both Georgica Ponds and Ocean Beach. Based upon input from the restoration team, these alternatives focused on structure removal, which would achieve the largest habitat outputs. In the evaluation of these structures, these alternatives were not selected for further consideration, primarily due to cost-effectiveness analysis and lack of land-owner support.

In order to improve the effectiveness of plans 3A and 3G in meeting the Vision Criteria, specifically to accomplish the objective of “integrating appropriate modification of shoreline stabilization structures”, the following should be included, and could be considered in both Plans 3A and 3G.

1) The groinfield at Westhampton be modified by shortening the groins a length of 70-100 ft for reducing the renourishment commitment to areas to the west.
2) The groins in the area of Georgica should continue to be monitored to determine if any structure modification is warranted.
3) Modification of the groins at Ocean Beach be undertaken upon relocation of the water-supply. This alternative becomes a factor when considered in conjunction with the desire to reduce the long-term need for renourishment.

7.9.2 Integration of Restoration Alternatives

Alternative Plans 3a and 3g, developed through the combination of storm damage reduction alternatives does not meet the Vision objectives that "The plan helps protect and restore coastal landforms and natural habitat." The plans partially fulfill these requirements, because sand-bypassing is considered as a restoration alternative that restores the alongshore transport. Plan 3G is also better than plan 3A in that it includes provisions for minimal intervention in the public tracts of lands along Fire Island.

The restoration planning focused on development of alternatives that would be complementary to the storm damage reduction objectives of the Reformulation Study. The criteria used in considering the complementary nature of the restoration were: 1) does the restoration increase the SDR effectiveness of the alternative, 2) are there cost efficiencies in implementing the storm damage reduction and habitat restoration alternatives together, and 3) does the restoration provide a desirable mosaic of habitats that could be altered by the SDR measure?

Alternative Plans 3A and 3G includes all the components of the storm damage reduction alternatives, including inlet bypassing, breach response, non-structural, and beach nourishment. The restoration alternatives were developed with a linkage to one or more of these storm damage reduction alternatives to demonstrate the appropriateness for inclusion. Because all of the storm damage reduction measures are included, it is a logical extension that each of the restoration alternatives that were supported in the Incremental Coast Analysis of NER benefits be included as a component of a plan which seeks to accomplish the Vision objectives.

It is established that the following restoration alternatives be included as a component of Alternatives 3A and 3G. These measures include:

- Sand bypassing, as identified in the NED portion of the plan
- Bayside restoration alternatives, at the two locations of high breach potential (Tiana Beach and Smith Point Park)
- Bayside habitat restoration in conjunction with breach closure alternatives where determined to be appropriate
- Shorefront habitat restoration alternatives (12) that were selected through the HEP process

- Bayside Habitat Restoration Alternatives (26) that were selected through the HEP process.
- Additional features developed for specifically meeting the needs for endangered species, as developed through the ESA process.

The restoration alternatives have been developed, and evaluated in a manner where the alternatives can be expanded spatially, or replicated in similar locations, to achieve similar results, at a similar expense. Based upon this, it is our determination that the above findings are supported and can be further scalable to meet the overall restoration objectives.

As discussed in the Environmental Appendix, the implementation of these restoration alternatives must be undertaken in a phased approach that embraces the concept of monitoring and adaptive management. Many of these types of alternatives have not been constructed in this area, and the nuances associated with these restoration alternatives are not completely recognized. The phased approach will allow for refinements in the overall magnitude of the effort. It will also allow for monitoring and adaptive implementation of the restoration alternatives, based upon the success or failures of the alternatives that have been implemented. These restoration alternatives should be integrated with both plans 3A and 3G.

7.9.3 Integration of Appropriate Land Use and Development Measures

Alternative Plans 3A and 3G that solely combine the storm damage reduction alternatives do not fully meet the Vision Criteria that “the plan incorporates appropriate non-structural features to provide both storm damage risk reduction and to restore coastal processes and ecosystem integrity”. Plans 3A and 3G partially fulfill this requirement in that they include a significant non-structural component to reduce storm damages along the mainland shoreline. These plans, however, do not include non-structural measures along the shorefront, which can reduce the potential for storm damages, and help to restore ecosystem integrity.

As discussed in Section 7.7, the land and development management alternatives generally include: 1) land management alternatives, and 2) acquisition alternatives. The implementation of these land use regulations is the responsibility of the local municipalities in conjunction with New York State, and within the FINS, the National Park Service.

As discussed in Section 7.7, there are existing challenges in implementing the land management regulations that exist in the study area, and Alternative Plans 3A and 3G could make it more difficult to implement these regulations, or in some instances could reduce the challenges in implementing these regulations (most notable in this connection is the requirement that for construction of the beach and dune that all properties in the footprint of the project be in public ownership or permanent easement).

The existing land use regulations were reviewed; and based upon that review, it is recommended that the following alternatives be included and considered an incremental component of this overall project in order for Alternative Plans 3A and 3G to function as intended.

Step 1. Improving the effectiveness of the existing land-use regulations through establishment of common funding, and improved implementation of the law, generally includes the following:

Update the Existing Dune District in FIIS

The federal law should be revised to create the same definition of a dune and the same requirement as contained in CEHA for a 10-year remapping process. It is recommended that the provisions within the FIIS enabling legislation be changed to identify that the dune district be coterminous with the CEHA line, and allowed to change with changes in the CEHA designation.

CEHA Improvements.

CEHA improvements include map updates, funding to adequately implement the program, and provisions for improved DEC monitoring of local implementation of CEHA.

Step 2. Modification of statutes to allow for more effective implementation of the existing laws.

CEHA Statuatory changes.

Make statutory and rule changes to enhance enforcement authority and provide indemnification by New York State for properly-administered local CEHA programs against takings claims to reduce the influence of potential litigation costs, including potential takings claims, on local program decision making. .

Step 3: Establishing a funding mechanism to acquire vacant parcels, or buildings that are at risk

This should serve as a means to acquire properties when enforcement of the regulations establishes a “takings”, and in a broader application could be applied to reduce the number of structures within the CEHA area that would be vulnerable to storm damages.

Step 4: The establishment of a regional entity that would be responsible for various aspects related to land management and acquisition, and to fulfill the requirements of the local sponsor.

The formation of a Suffolk County Coastal Commission with authority to implement land management and authority (and sufficient funding) to acquire property, could ensure the following:

1. The acquisition of lands necessary for construction of the project, and providing funds necessary, in excess of the Real Estate costs to meet the local share.
2. Responsible for CEHA variances, and in defending legal challenges arising from CEHA.
3. Responsible for the acquisition of properties in the instance of regulatory takings,
4. Responsible for implementing a willing-seller program to address structures that are at-risk in the erosion prone areas.

Step 5: The Establishment of post-storm response plans to guide recovery following major, catastrophic events. This includes the development and implementation of local post-storm redevelopment plans to provide direction for the rebuilding of communities in a more sustainable manner.

As discussed in Section 7.7, land use management is the first tool available to address new development, and is not a responsibility of the Corps. Acquisition is the second tool that is available to address existing and proposed development. The acquisition of shorefront properties was evaluated for purposes of both storm damage reduction and habitat restoration. In both instances, the relatively high price of the real estate results in these alternatives not being cost-effective. That being said, it is acknowledged that alternatives which acquire properties for purposes of a more landward beachfill alignment are cost-effective but have the downside of requiring condemnation in order for the project to be constructed.

New York State and the National Park Service have indicated their interest in an acquisition program along the shorefront, which over time, with willing sellers could remove the most at-risk structures from the shoreline. While this alternative does not meet the NED or NER criteria for Corps cost-sharing participation, an acquisition plan along the shorefront would accomplish the Vision objectives and would help with the implementation of the land use regulations.

Overall, these changes in the land use regulations, and acquisition plans are critical for the Corps to make a determination that the proposed project will not induce development. The Corps will look for New York State as the sponsor to advance these floodplain management regulations and be able to certify that sufficient land management regulations are in place, to avoid induced development as a result of the project. Construction of the project, and continued renourishment of the project would be dependent upon this certification from New York State

7.9.4 Consideration of the life cycle management of these plans

Alternative Plans 3A and 3G, were developed with a 50-year project life, and 50 years of renourishment. These plans do not meet the Vision objectives that “the plan addresses long-term demands for public resources.” These plans do not include provisions that would change the need for continued renourishment within the project life or alter the conditions so that a different solution could be expected following the 50-year project life.

In order to achieve a reduction in the long-term commitment for renourishment, alternatives would need to be implemented that would reduce the development that is at risk or remove development to allow for a more efficient use of resources. The integration of land and development management regulations identifies improvements in the application of land use regulations, acquisition planning, and post-storm response planning that could help to reduce the infrastructure at risk along the shorefront.

With this as a component of the overall plan, there are several approaches which could be undertaken in the life-cycle management of the project to achieve this. The options that have been identified include:

1. A scheduled reduction in the scale of protection for the beachfill in a timeframe that coincides with the real estate acquisition planning. Under this scenario a beachfill plan would be maintained for a shorter period of time, during which period purchase offers would be made to owners of property on which shorefront structures at risk are situated. After this period of time, the scale of protection would be reduced or eliminated, thus reducing the commitment of resources for continued renourishment. The benefit of this approach is that the reduction in protection is not dependent upon the acquisition actually occurring.

2. A scheduled relocation of the proposed line of protection that coincides with the implementation of the acquisition. Under this scenario, the beachfill plan would be linked with the proposed acquisition plan. After a period of time, the footprint of the project would be maintained in a more landward location on a scheduled timeframe. The difficulty with this initiative is that the movement of the dune on a prescribed timeframe would require guaranteed acquisition, which could not be guaranteed with a willing-seller program, and would require condemnation.

3. Adaptive Management. Under this scenario, the beachfill plan and the acquisition plan could proceed independently, on parallel tracks. Adaptive Management would not dictate a defined timeframe for implementation, but would provide for a process, where on a periodic basis, coinciding with the

scheduled renourishment, the constructed project would be revisited to identify whether opportunities exist for adjustment of the maintained profile based upon the relative success in implementing the acquisition plan.

Under any of these scenarios, it is important to 1) identify the time scale that would be necessary for the implementation of alternatives, and 2) identify the effect that these changes would have on project benefit realization and implementation costs.

It is recognized that the acquisition of shorefront property through a willing-seller program is not an instantaneous action, particularly with consideration for acquisition strategies that could allow for a homeowner to sell their property but be allowed to use the property for some period of time. The timeframes necessary for implementation of these measures tend to be estimated in decades, not in years. Along the shorefront, consideration must be given for: the funding availability for acquisition, the timing of interest in selling, and the staffing to process these acquisitions. When consideration was given for the time necessary to implement the non-structural alternatives along the mainland, accounting for staffing this effort, and funding these programs, it was estimated that implementation of the mainland non-structural program would require 25 to 30 years. Discussions have also been held with agencies responsible for the relocation of public infrastructure along the shorefront. Input from these agencies indicates that major public works improvements, whether relocation or otherwise typically require 10 to 20 years from conception to execution.

These timeframes suggest that if there is interest in reducing the long-term commitment to public investment in renourishment, a beachfill with a duration of 20 to 30 years could be considered in conjunction with an acquisition plan. As the project duration is shortened, it impacts the project economics. A sensitivity analysis was conducted which established that Alternative 3, built and maintained for 30 years, and subsequently replaced with a breach response plan, would have little effect on the project economics. Achieving this objective, however, would require a larger investment in Real Estate to provide an alternative form of risk reduction for houses along the shorefront (these costs were not considered in the cost).

7.9.5 Adaptive Management

The challenge with developing a plan that integrates the land management, acquisition, and scheduled renourishment of the project is the uncertainty that exists. These elements introduce uncertainty to a situation that is already uncertain due to the complexities of evaluating the system, projecting renourishment, projecting the functioning of the inlets, and the unknowns regarding future climate change. With all these uncertainties it is suggested that the implementation of the project adopt an incremental adaptive management approach. This approach would establish 1) data collection that would be implemented to have an improved understanding of the physical, social and environmental setting, 2) modeling efforts (engineering and formulation) to analyze the data, and 3) an adaptive management framework that would establish the overall objectives, decision rules, and identify the adaptations to the plan that could be accomplished with the project. This adaptation strategy is based upon the concept that with the passage of time the trends become established and more appropriate strategies can be executed. It is expected that this adaptation strategy would require a periodic review of the project execution (10-yr basis) and recommendations for the adaptation of the project, based upon the findings.

It is expected that the adaptive management plan would integrate the lifecycle management of the project, as it relates to the following elements:

- *Inlet Management*: Improved understanding of inlet functioning, the volume and frequency of bypassing, and the optimal alternatives for achieving the long-term objectives for inlet management.
- *Breach Response*. Improved understanding of breaching processes and consequences, refinement of the breach triggers and the implementing procedures, optimization of maintenance requirements, and the improved integration of habitat improvements.
- *Beachfill*. Improved understanding of beachfill performance, refinement of renourishment triggers and allowable variability in design, accounting for alignment changes based upon non-structural plan implementation, consideration of durations.
- *Non-Structural*. Improved delineation of structure vulnerability, and identification design details, identification of implementation effectiveness, identification of acquisition effectiveness, identification of the effectiveness of land management regulations
- *Restoration*. Identification of relative effectiveness of alternatives, identification of deisgn improvements, and better definition of overall restoration success objectives.
- *Climate Change*. As presented in the without project damages section, damages are likely to increase in the future without the project. Under historic or moderate increases in sea level rise, it is likely that adaptive management measures could accommodate these changes. Under more extreme rates of sea level rise, or more dramatic climate change conditions, adaptive management would allow for consideration in the relative effectiveness of the different solutions.

7.9.6 Summary of Alternative Plan Comparison

The above analysis demonstrates a number of key factors:

1. There are a number of Alternative Plans that meet the objective of cost-effective storm damage reduction,
2. The plan that functions optimally in reducing storm damages, that is, the plan that maximizes net NED benefits, is Plan 3A, which includes inlet bypassing, NS-

3 with road raising, continuous (as needed) beachfill along Great South Bay, and Moriches bay, and a breach response plan along Shinnecock Bay.

3. Alternative 3 G, which include inlet bypassing, NS-3 with road raising, beachfill fronting the communities along Great South Bay, and Moriches Bay, and a breach response plan along unprotected areas of Great South Bay, Moriches Bay, and Shinnecock Bay is the combination of storm damage reduction alternatives that best balances the objectives of storm damage reduction, P&G criteria, and Vision Criteria.
4. Plans 3A and 3G do not meet all the objectives of the Vision.
5. The plan that maximizes the objectives of the Vision Statement is:
   a. Plan 3G, modified as follows
      i. Inclusion of the groin modification plan at Westhampton, and Ocean Beach
      ii. Inclusion of the recommended restoration alternatives
      iii. Inclusion of Land Management Measures
      iv. Inclusion of an acquisition program along the barrier island
      v. Includes an incremental adaptive management strategy over the project life to address the uncertainties in project implementation

A plan consisting of the above features is identified as the plan that meets the objectives of the Vision Statement.

8.0 POST-SANDY TFSP MODIFICATIONS

Since the 2009 Feasibility Report, Federal Agencies including USACE, New York State (NYS) and local municipalities had been working toward consensus on a finalized FIMP plan. On March 11, 2011, USACE and the Department of the Interior agreed to a Tentatively Federally Supported Plan (TFSP) that included all of the measures listed in Section 7.1, and a few additional items. The tentatively agreed-on plan was moving forward toward approval until October 29, 2012, when Hurricane Sandy made landfall in the New York Bight. The entire FIMP study area was impacted, inflicting severe damage to homes, infrastructure and beaches.

As a result of the impacts of Hurricane Sandy, USACE, National Parks Service (NPS), and NYS agreed that the selected plan needed to be revisited to determine if changes to the TFSP were warranted in light of the changes in the beach and dune condition of the study area.

One of the first items accomplished, following the storm, was collection of a new set of LIDAR topography and aerials of the study area. This was accomplished in November 2012 and provided insight into the damage and the study path forward. Of particular interest was how the barrier island beaches responded to the storm. Prior to Hurricane Sandy, it was accepted that the barrier island condition would degrade over time creating the likelihood of a higher probability of barrier island breaching. However, it was unknown exactly what would occur as a result of a major storm. Several smaller nor’easters have impacted the FIMP coastline in recent years, but nothing near the 100-year storm design level simulated in the hydrodynamic numerical modeling. When examining still water level records of Hurricane Sandy, it became clear that it was much closer to the 100-year level. Table 8-1 shows the peak still water level during Hurricane Sandy and how it compares to historic return periods at each project location. The return period was rounded to the nearest 5-year interval.

Table 8-1. Hurricane Sandy Peak Still Water Levels

Project	Nearest Tide Gage	SWL NGVD29 (ft)	SWL NAVD88 (ft)	Return Period* (yrs)	Stage-Frequency source document
Fire Island, NY	Proportional by distance between Battery (NOAA) and Montauk Ft. Pond (NOAA)	9.6	8.6	110	FIMP Station 23 (Great South Bay - Ocean)
Westhampton, NY	Proportional by distance between Battery (NOAA) and Montauk Ft. Pond (NOAA)	8.6	7.7	65	Jul 1995 Westhampton Interim Report
West of Shinnecock Inlet, NY	Proportional by distance between Battery (NOAA) and Montauk Ft. Pond (NOAA)	8.2	7.2	65	Mar 1999, Vol 1, West of Shinnecock Draft Decision Doc.

Project	Nearest Tide Gage	SWL NGVD29 (ft)	SWL NAVD88 (ft)	Return Period* (yrs)	Stage-Frequency source document
Montauk Lighthouse	Montauk Ft. Pond (NOAA)	6.8	5.8	30	Montauk Point, NY Feasibility Study, 2005, table A-2

The primary area for project area beachfill will be on Fire Island and Sandy corresponded to a 110-year water level in this stretch of the project. Therefore, Sandy provided tangible evidence of the consequences of a 100-year storm within the project area.

To further evaluate Sandy's damage, a qualitative analysis was performed to determine how the LIDAR/aerial photos compared to the hydrodynamic numerical model cross-sectional alternatives described in Section 4.6. Ten locations were selected along the most affected areas (Fire Island Inlet to Shinnecock Inlet). Because the primary focus of the beachfill projects will be on Fire Island, the LIDAR data were used to compare with the modeling while only aerials and damage reports were used to analyze the stretch between Moriches Inlet and Shinnecock Inlet. These results are shown in Table 8-2.

Table 8-2. Barrier Island Elevation Conditions: Post Sandy vs. Model Cross-Sections

Reach	Location	FVC minimum dune height Simulated (ft NGVD)	Approximate minimum dune height from LIDAR Nov. 2012 (ft NGVD)	2013 Conditions	Data source for analysis
GSB	FI Lighthouse	8	8	FVC	LIDAR profiles
GSB	Kismet/Corneille	8	8	FVC	LIDAR profiles
GSB	Talisman/Blue Pt.	10	12.5	BLC	LIDAR profiles
GSB	Davis Park	10	12	Interpolation BLC/FVC	LIDAR profiles
GSB	Old Inlet W	8	OPEN	BOC	LIDAR profiles
GSB	Old Inlet E	8	5	FVC	LIDAR profiles
MOR	SPCP	8	5	FVC	LIDAR profiles
SHN	Sedge I.	10	*NA*	FVC	aerials & damage estimates

Reach	Location	FVC minimum dune height Simulated (ft NGVD)	Approximate minimum dune height from LIDAR Nov. 2012 (ft NGVD)	2013 Conditions	Data source for analysis
SHN	Tiana	8	*NA*	FVC	aerials & damage estimates
SHN	WOSI	10	*NA*	FVC	aerials & damage estimates

Seven out of the ten locations were consistent with the Future Vulnerable Condition (FVC) with areas at Old Inlet East and Smith Point County Park showing much worse conditions. Talisman/Blue Point and Davis Park better correlate with the Baseline Condition (BLC) and the breach at Old Inlet West, obviously compares to the Breach Open Condition (BOC). While this analysis was only qualitative, it did show that the damage of Hurricane Sandy resulted in a condition best described by the FVC.

8.1 FIMI Beachfill Alignment and Real Estate

In the absence of oceanfront structures, the most cost effective alignment is one that ties into the existing dune line and extends seaward from the existing shoreline only the distance necessary to achieve the required level of protection. The beachfill alignment also affects costs, as beachfill losses caused by “spreading out” or diffusion of beachfill will be greater the farther seaward an alignment is located.

Prior to Hurricane Sandy, the selected beachfill alignment, Minimum Real Estate Impacts (MREI), generally followed the existing dune alignment except within the communities where it was aligned seaward of the existing buildings to minimize real estate costs. Because of the extensive morphological changes observed during Hurricane Sandy, a landward shift in the beachfill alignment was evaluated and is required to account for, as much as possible, the new existing (Post-Sandy) dune alignment.

The beachfill alignment, Updated Middle Alignment (MIDU), preserves as much as possible the existing (Post-Hurricane Sandy) dune alignment while balancing the cost of acquiring or relocating oceanfront structures versus increased beachfill needs. The selected plan requires approximately 3 million cubic yards less of initial beachfill. However, the selected alignment requires 41 real estate acquisitions, 6 real estate relocations and over 600 permanent easements for construction.

Lifecycle cost estimates for the MIDU and Minimum Real Estate Alignment (MREI) indicate that reduced annual costs in the MIDU due to the reduced initial fill volumes ($2.0 million per year) exceed the additional expense of the real estate acquisitions and relocations in the MIDU. This more landward alignment, which requires less sand is also more sustainable, and environmentally preferred, as it requires fewer sand resources.

In addition to the plan comparisons described above informing the costs of the different plans, this information also served as a tangible measure of the environmental impacts of the proposed plans, and the long-term sustainability of these plans. The plan which is constructed farther south, that requires a greater volume of sand for both construction and long-term renourishment, has a greater environmental impact and is less sustainable than the plan placed farther landward. Factoring sustainability and environmental effects into the decision-making, the more landward alignment is clearly preferable.

8.1.1 Initial Construction Quantities

Beachfill quantities, costs and locations have evolved since the TSFP plan. Hurricane Sandy produced record storm tides and wave heights in the New York Bight. As a result, several breaches occurred and significant overwash and beach erosion was observed along Fire Island. Aerial images and LIDAR data from 2000 to 2012 are presented below for Fire Island Pines to illustrate the aforementioned beach changes (e.g. dune erosion, increased beach width) that are reflected in the initial beachfill volume estimates presented in this section.

Aerial images of Fire Island Pines from March 2001, March 2012, and November 2012 are shown in Figure 8-1. The +2 NGVD contour derived from the LIDAR data is shown in red (2000) and cyan (2012 Post-Sandy). LIDAR elevations from c. 2000, 2011 (Post-Irene), and 2012 (Post-Sandy) are shown in Figure 8-2. Once again, the +2 NGVD contour derived from the LIDAR data is shown in red (2000) and black (2012 Post-Sandy). The MREI baseline is shown in purple.

Representative cross-shore beach profiles cut from the 2000, 2011, and 2012 LIDAR are shown in Figure 8-2 and Figure 8-3. The design profiles for the MREI plan, as well as the calculated dune and berm fill volumes, are also shown.

The aerial images and LIDAR data tell the same story at Fire Island Pines: the beach width increased considerably from 2000 to 2011 and Hurricane Sandy caused significant dune erosion from 2011 to 2012. Some of the sediment eroded from the dune face and berm top during Hurricane Sandy appears to have been transported seaward and deposited along the seaward edge of the berm, resulting in a wider dry beach and shoreward migration of the +2 NGVD contour. The trends observed at Fire Island Pines are similar, although perhaps more exaggerated, to other communities along Fire Island.

There is some concern that the 2012 LIDAR dataset overestimates the existing berm width, as a result of dune and berm elevation losses. The approach used to compute volumes assumes that a wider subaerial beach corresponds to an equally wide subaqueous profile. If this were not the case, this methodology could result in erroneously low estimates of beachfill requirements. However, the beachfill volumes increased nearly 1 MCY from 2011 to 2012, thereby making the exaggerated berm width unlikely. Table 8-3 shows the estimated design fill volumes for the various LIDAR sets. The quantities represent the MREI alignment. Historic beachfill volumes for each design reach from 2000 to 2012 are presented in the last column.
The required initial beachfill volumes decrease by approximately 2.78 MCY from 2000 to 2011. This decrease in beachfill is attributed, partly, to the 3.37 MCY of

beachfill placed along Fire Island between 2000 and 2011. A 0.91 MCY increase in the required initial fill volumes from 2011 to 2012 was observed and is mostly attributed to the effects of Hurricane Sandy. Figure 8-5 shows the spatial distribution of the dune and berm fill across the project area.

Table 8-3. MREI Design Fill Volumes

Design Reach	Name	Dune Height (NGVD29)	2000 Design Fill Volume (cy)	2011 Design Fill Volume (cy)	2012 Design Fill Volume (cy)	Historic Beachfill Placement (cy)
GSB-1A	RMSP	+13	352,646	322,593	536,289	
GSB-1B	FILT	+13	71,045	107,323	194,591	
GSB-2A	Kismet to Lonelyville	+15	1,392,014	349,664	311,847	1,238,471
GSB-2B	Town Beach to Corneille	+15	835,023	333,461	379,541	68,039
GSB-2C	Ocean Beach to Seaview	+15	295,080	346,056	259,361	349,422
GSB-2D	OBP to POW	+15	890,365	201,006	387,187	159,463
GSB-3A	Cherry Grove	+15	8,347	3,459	20,167	
GSB-3C	Fire Island Pines	+15	877,823	266,102	280,206	1,070,098
GSB-3E	Water Island	+15	3,113	2,585	19,742	
GSB-3G	Davis Park	+15	478,079	274,880	367,957	313,804
MB-1A	SPCP-TWA	+15	212,850	135,891	231,948	
MB-1B	SPCP	+13	301,321	317,626	543,488	172,000
MB-2A	MB-2A	+13	174,388	451,923	490,342	
Total			**5,892,094**	**3,112,569**	**4,022,666**	**3,371,297**

Figure 8-1. Aerial Images from 2001, 2012 Pre-Sandy, 2012 Post-Sandy. MHW contour from 2000 LIDAR shown in Red, MHW contour from 2012 LIDAR shown in Cyan.

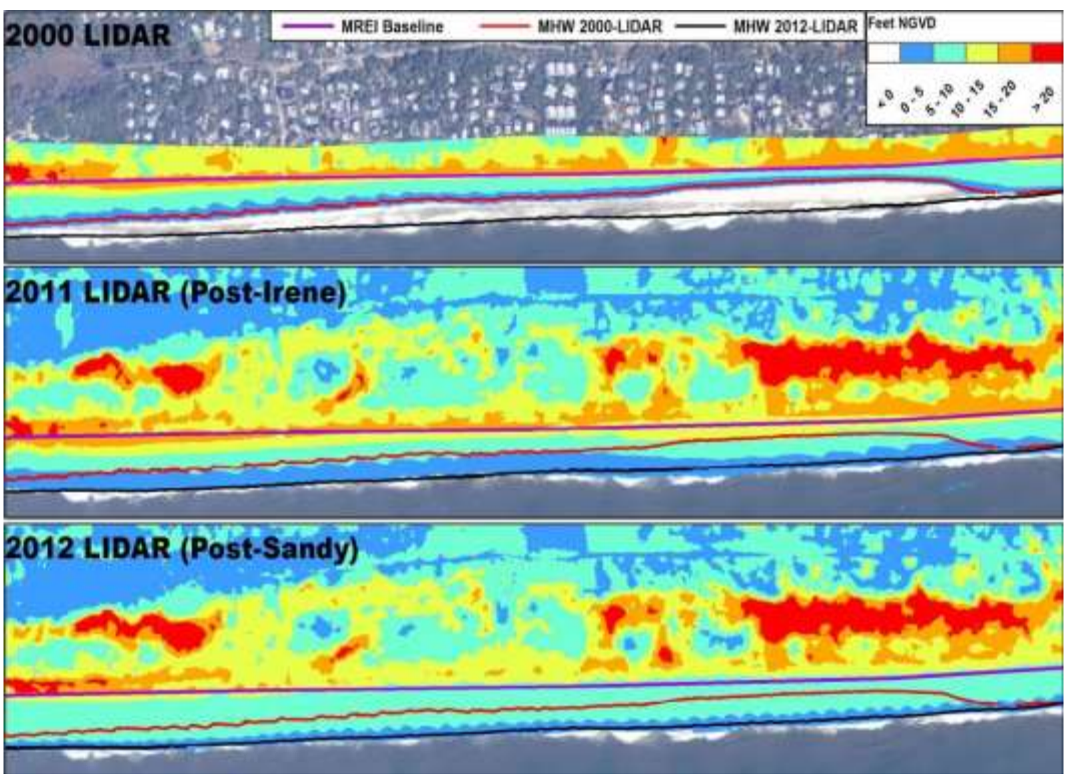


Figure 8-2: LIDAR Data from 2000, 2011, and 2012 (Fire Island Pines).

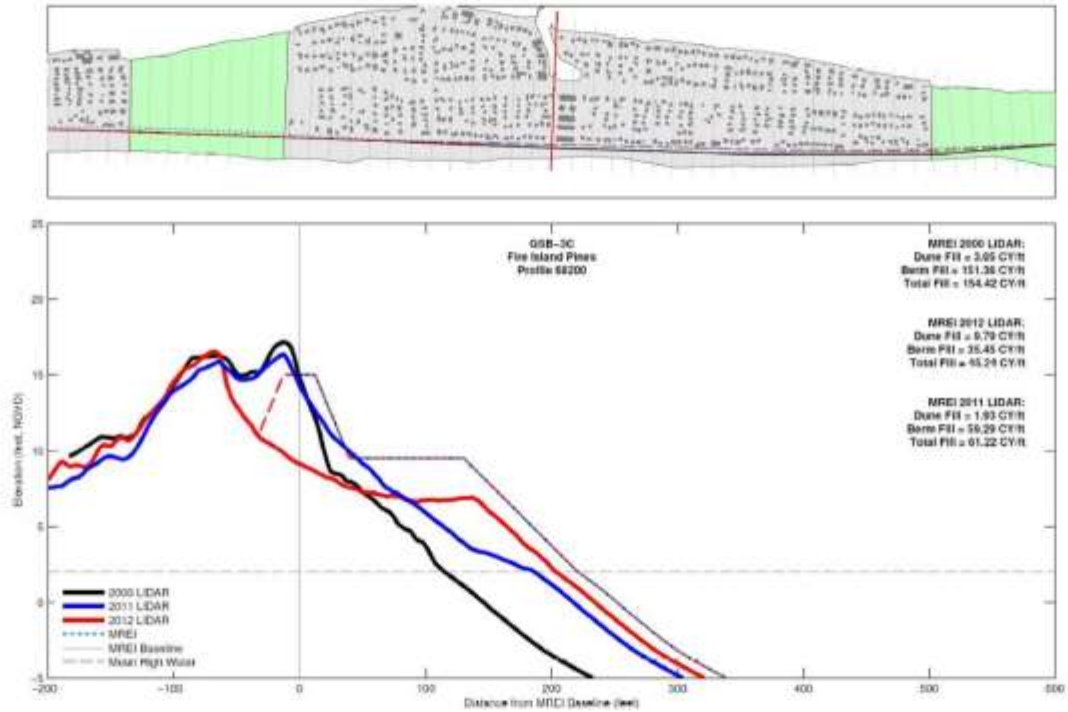


Figure 8-3. Comparison of Cut LIDAR Profiles at Fire Island Pines (Profile 68200)

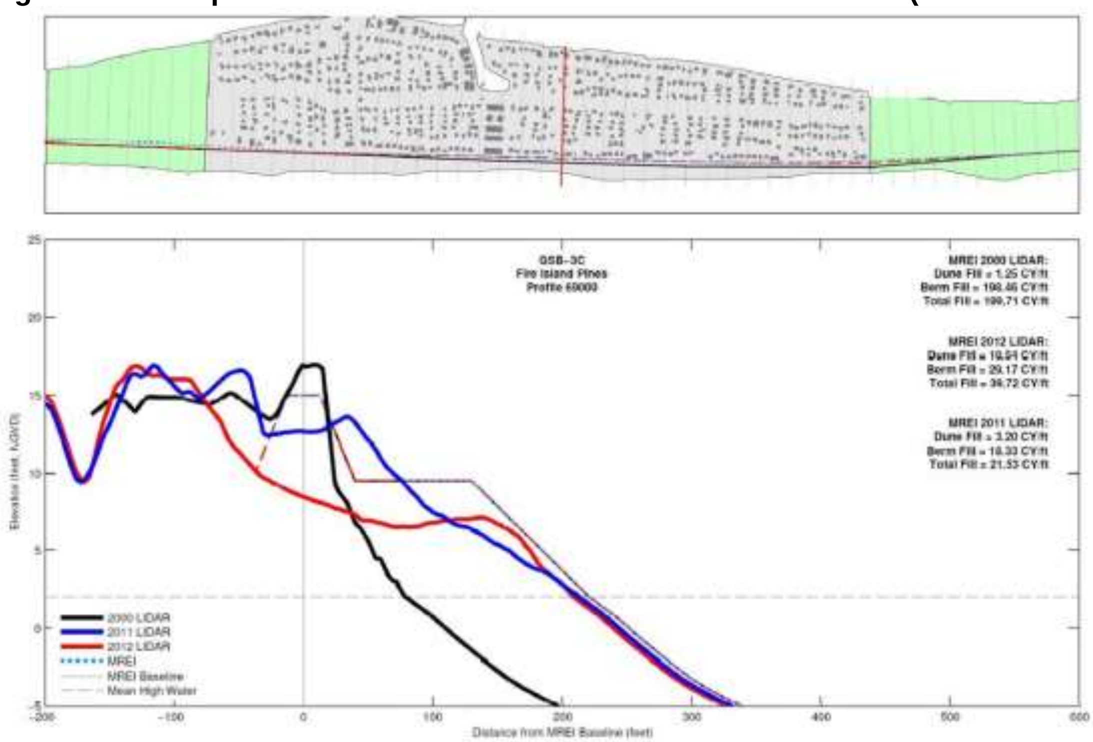


Figure 8-4. Comparison of Cut LIDAR Profiles at Fire Island Pines (Profile 69000)

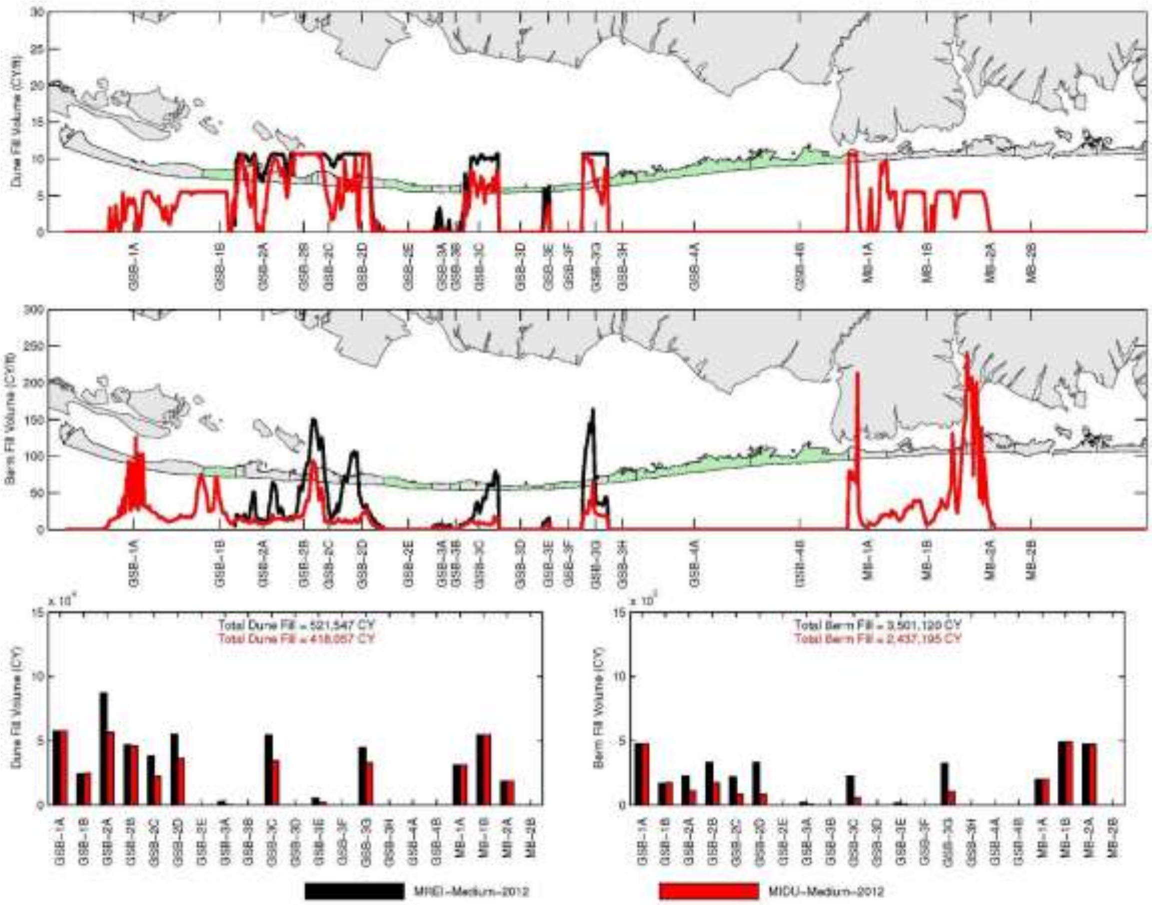


Figure 8-5. Spatial distribution of Dune and Berm Fill

8.1.2 Advance Fill and Renourishment

To ensure the design profile remains intact over the entire project life, periodic renourishment is required. However, the goal is to limit the number of renourishment cycles because the costs to mobilize equipment for placing is cost prohibitive when it is considered annually. Therefore, a volume of advance fill was also determined. The following subsections outline the approach used to determine these advanced fill/renourishment quantities.

8.1.2.1 Representative Erosion Rates

The advance fill berm width and renourishment volumes are determined based on the representative erosion rates for each design reach. The representative erosion rate accounts for:

- "Spreading out" or diffusion of sand resulting from the shoreline anomaly or "bump" created by the beachfill;
- Background erosion due to ongoing coastal processes before the project was constructed.

Beachfill diffusion is a function of the longshore length of the beachfill, cross-shore width of the beachfill, and longshore diffusivity. The rate of beachfill diffusion is particularly sensitive to longshore length of the beachfill project. Shorter projects (e.g. Fire Island Pines) will generally experience a much higher rate of diffusion than longer projects (e.g. Western Fire Island). Analytical solutions to the diffusion equation (i.e. Pelnard Considere, 1956) are applied below to determine the rate of beachfill diffusion along Fire Island. Generally, it is assumed that the background shoreline erosion will continue at the same rate as before project. Background erosion rates were determined from the FIMP sediment budget and Most Vulnerable Conditions Report.

8.1.2.2 Previous Work (c. 2008)

Representative erosion rates applied in the earlier estimates of renourishment volumes were based on the FIMP sediment budget, Most Vulnerable Conditions Report, and the performance of historical beachfill projects. These rates are shown in Table 8-4. The representative erosion rates essentially accounted for both the historical background erosion rate and beachfill diffusivity. However, a specific beachfill diffusion analysis was not performed and the relative contribution of the two processes was not identified. It was also assumed that the representative erosion rates were the same for all three project baselines (Minimum Real Estate, Middle, and Unconstrained).

Table 8-4. Previous (c. 2008) Representative Erosion Rates

Design Reach	Name	Reach Length[1] (ft)	Representative Erosion Rate (ft/yr)
GSB-1A	RMSP	23,200	5
GSB-1B	FILT	5,400	5
GSB-2A	Kismet to Lonelyville	9,000	5
GSB-2B	Town Beach to Corneille Estates	4,400	5
GSB-2C	Ocean Beach to Seaview	3,800	5
GSB-2D	OBP to POW	7,200	5
GSB-3A	Cherry Grove	3,000	0
GSB-3C	Fire Island Pines	6,400	10
GSB-3E	Water Island	2,000	1
GSB-3G	Davis Park	4,200	1
MB-1A	SPCP-TWA	6,400	2
MB-1B	SPCP	13,000	2
MB-2A	MB-2A	7,800	2

Notes: [1]Distances are approximate (rounded to 200 ft)

8.1.2.3 Recently Measured Erosion Rates (2009-2012)

From January to April of 2009, a total of 1.9 MCY of sand was placed in eleven communities along Fire Island. The 2009 project consisted of four continuous sections of beachfill placement: Western Fire Island, Central Fire Island, Fire

Island Pines, and Davis Park. An overview of the 2009 beachfill project is provided in Figure 8-6. The performance of the beachfill project has been monitored by collecting beach profile surveys in May 2009, March 2011, and Dec 2012. These beach profile surveys were used by Coastal Planning & Engineering to determine the volumetric changes along the 2009 project extents. Volumetric losses were converted for this study to erosion rates by dividing the total volumetric loss over each project reach by the length of the project reach, and by the active beach height (36.5 feet, depth of closure plus berm elevation). Table 8-5 presents the volumetric losses and erosion rates for Western Fire Island, Central Fire Island, Fire Island Pines, and Davis Park in the 3.6 years following the 2009 beachfill project.

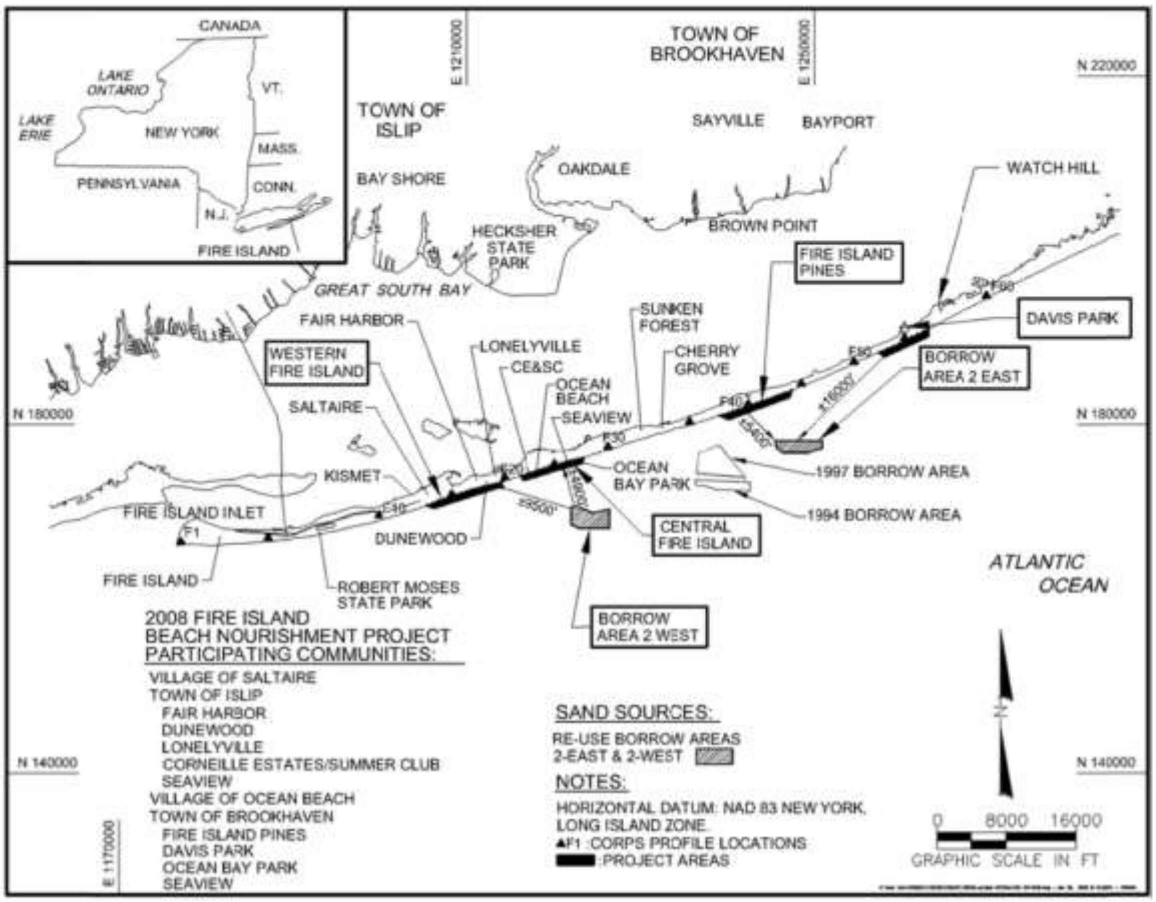


Figure 8-6. 2009 Beachfill Project Location Map

The observed erosion rates for Western and Central Fire Island are significantly greater than the previously applied representative erosion rates for these design reaches (5 ft/yr). One possible explanation for the relatively high erosion rates is that the alongshore beachfill lengths in the 2009 project were significantly shorter (9,351 and 8,115) than in the Federal plan (41,800 ft). It will be shown below that the rate of beachfill diffusion is very sensitive to the alongshore length of the beachfill project. Another possible explanation is that the rate of background erosion and beachfill diffusion were above average from 2009 to 2013 due to the occurrence of several extreme storm events including several nor'easters, Hurricane Irene, and Hurricane Sandy.

Table 8-5. Summary of 2009 Beachfill Project Performance

Project	Length (ft)	Placed Volume (cy)	May 2009 to Dec 2012 (cy)	Erosion Rate (ft/yr)
Western Fire Island	9,351	520,743	-462,446	-10.2
Central Fire Island	8,115	594,398	-733,873	-18.7
Fire Island Pines	6,785	491,784	-671,791	-20.4
Davis Park	4,125	291,352	-257,218	-12.9

As noted earlier, the rate of beachfill diffusion is also affected by the cross-shore width of the beachfill project. Therefore, it is important to consider the width of the 2009 beachfill project and compare it to the proposed Federal alignments. The location of the design or adjusted seaward berm crest is used here to represent the relative cross-shore width of the beachfill projects. Figure 8-7 to Figure 8-10 show the location of the design berm for the 2009 beachfill project and Federal plans at Western Fire Island, Central Fire Island, Fire Island Pines, and Davis Park. Visual analysis of design berm alignments indicates that the cross-shore width of the 2009 beachfill projects are similar to the TFSP except at Davis Park where the cross-shore width of the 2009 beachfill is similar to the MIDU plan. This simple analysis indicates that the measured erosion rates in the 3.6 years following the 2009 beachfill project may be used to predict the representative erosion rates for the TFSP at Fire Island Pines and MIDU plan at Davis Park.

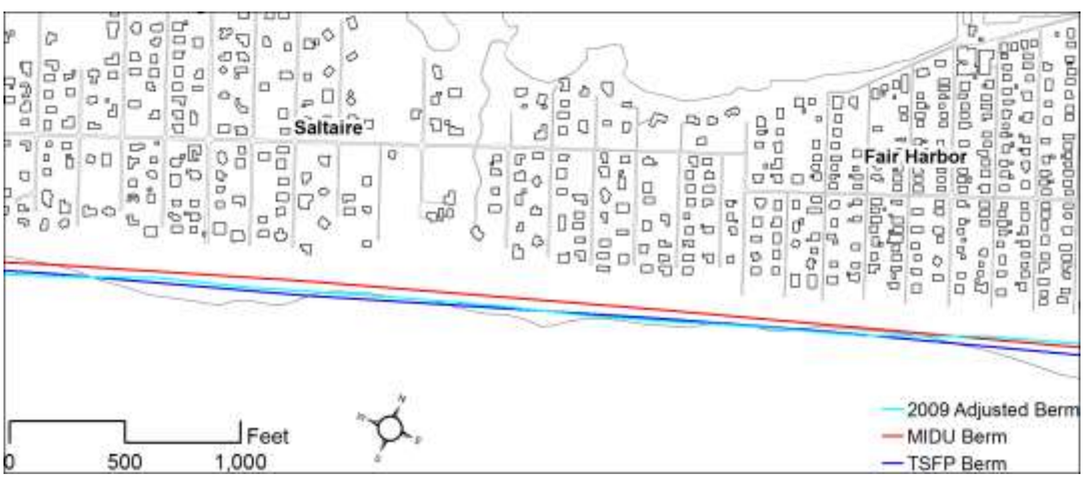


Figure 8-7. 2009 Design Berm at Western Fire Island

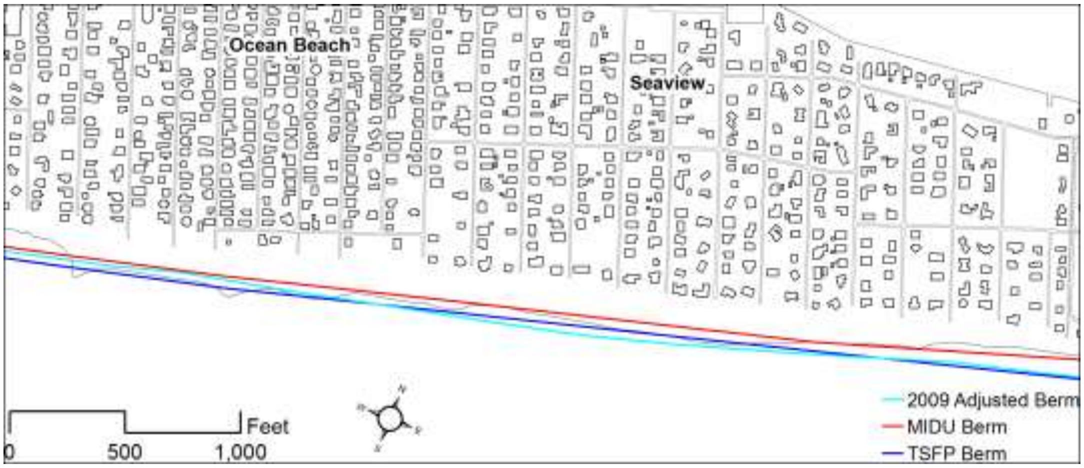


Figure 8-8. 2009 Design Berm at Central Fire Island

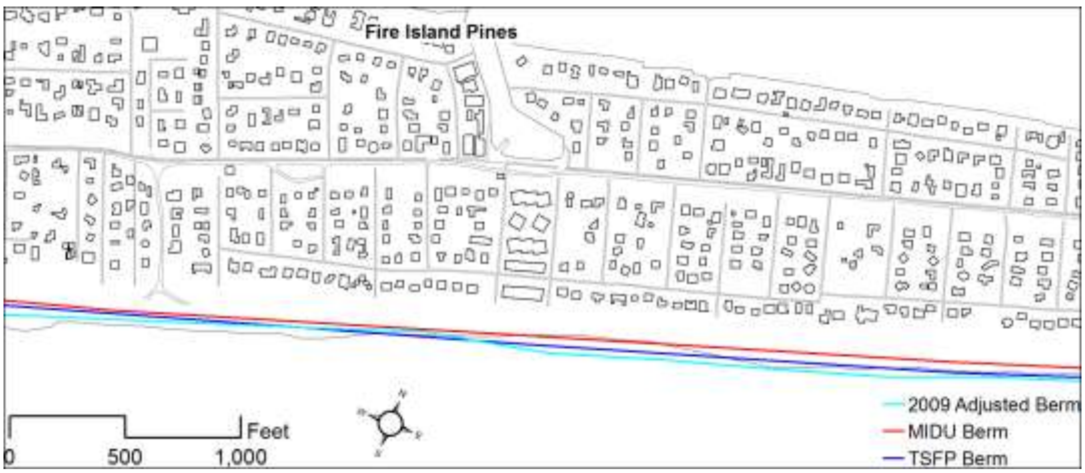


Figure 8-9. 2009 Design Berm at Fire Island Pines

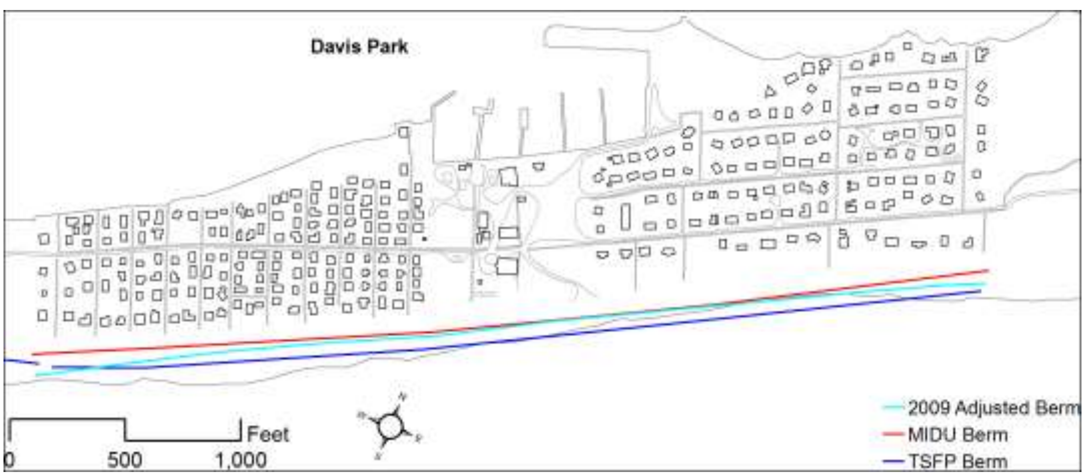


Figure 8-10. 2009 Design Berm at Davis Park

8.1.2.4 Beachfill Diffusion

A beach nourishment project constructed on a long beach represents a perturbation, which under wave action will spread out along the shoreline. If the wave action is small, then the rate at which the anomaly resulting from the beach nourishment is spread out from the placement area will likewise be small. It is important to remember that beachfill diffusion is a separate process from background shoreline erosion, which is generally caused by gradients in the net longshore sediment transport.

8.1.2.4.1 Theoretical Background

The one-dimensional diffusion equation or Pelnard-Considere equation for planform evolution may be derived from combining the conservation of sediment equation with the total longshore sediment transport equation.

The conservation of sediment equation:

$$\frac{\partial Q}{\partial x} + \left(h_* + B\right)\frac{\partial y}{\partial t} = 0$$

Where Q is the total longshore sediment transport, y is the shoreline, and h* and B are the depth of closure and berm height respectively.

The total longshore sediment transport, Q, equation or CERC formula is given by:

$$Q = C' H_b^{5/2} \sin 2\theta_b$$

$$C' = \frac{K\sqrt{g/\delta_b}}{8(S-1)(1-p)}$$

Where H_b is the breaking wave height, θ_b is breaking wave angle relative to shore normal (10 deg), g is acceleration of gravity, δ_b breaking wave index (0.78), S specific gravity of sand (2.65), p is the porosity of sand (0.35) and K sediment transport coefficient (0.77). This value for K is consistent with a medium sand as shown in Figure III-2-6 of the Coastal Engineering Manual (CEM).

For an undulating shoreline, with small values of $\partial y / \partial x$ the sediment transport equation may be re-written as follows:

$$Q = C' H_b^{5/2} \sin 2\theta_b - G\left(h_* + B\right)\frac{\partial y}{\partial x}$$

The first term above represents the background sediment transport rate for shoreline parallel to the x-axis, and the second term represents the transport induced by the shoreline undulations ($\partial y / \partial x$). Parameter G is the longshore diffusivity and is equal to

$$G = \frac{2C' H_b^{5/2} \cos 2\theta_b}{(h_* + B)}$$

Taking the derivative of the sediment transport equation (assuming $\partial y / \partial x$ << 1) and combining with the conservation of sediment equation yields the final form of the Pelnard-Considere equation

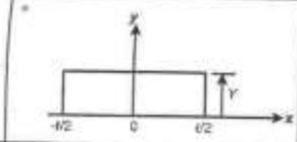
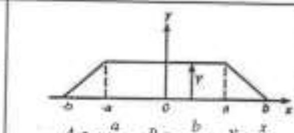
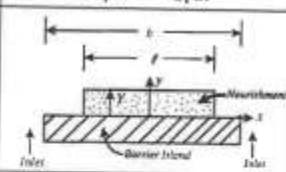
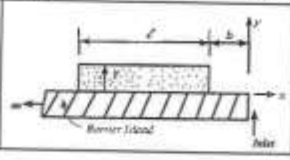
$$\frac{\partial y}{\partial t} \cong G \frac{\partial^2 y}{\partial x^2}$$

There are many solutions to the equation, of interest here are the solutions for a rectangular and trapezoidal beachfill (e.g. with tapers) on a long straight beach. Consideration was given to solutions to the Pelnard-Considere equation for a barrier island with inlets; however, the distance between the inlets and limits of beachfill are sufficiently large to result in very small differences.

Rectangular Beachfill

The solution to the Pelnard-Considere equation for a rectangular beachfill project on a long straight beach is shown in panel "a" of Figure 8-11. The non-dimensional results for a rectangular beachfill project with alongshore length l, cross-shore width Y, and time t are shown in Figure 8-12 illustrating that the planform location after some time "t" is proportional to $1/l^2$. As a result, the performance of the beachfill is very sensitive to the alongshore length.

Figure 8-13 further demonstrates the sensitivity of the performance of a beachfill project to the alongshore length by plotting the fraction of volume remaining, M(t), versus non-dimensional time, $\sqrt{Gt}/l$. The solid black line shows the solution to the Pelnard-Considere equation, the dashed black line presents the results for exponential decay, and the four markers present the volume remaining after 4 years for beachfill projects at Western Fire Island (41,800 feet), Fire Island Pines (6,400 feet), Davis Park (4,200), and Eastern Fire Island (19,400 feet). It is important to note, that the results in Figure 7-30 are in the absence of background erosion. However, the implications are clear, in that shorter beachfill projects will experience a much higher rate of diffusion. Therefore, it is expected that the representative erosion rates at Fire Island Pines and Davis Park will be much higher than at Western and Eastern Fire Island because the alongshore length of the beachfill project is significantly smaller.

Description	Illustration	Solution
(a) Initially rectangular planform on a long straight beach	* 	$y(x,t)=\frac{Y}{2}\left\{\begin{array}{l}\operatorname{erf}\left[\frac{\ell}{4\sqrt{Gt}}\left(\frac{2x}{\ell}+1\right)\right]\\-\operatorname{erf}\left[\frac{\ell}{4\sqrt{Gt}}\left(\frac{2x}{\ell}-1\right)\right]\end{array}\right\}$
(b) Initially trapezoidal planform on a long straight beach	 $A=\frac{a}{2\sqrt{Gt}},\ B=\frac{b}{2\sqrt{Gt}},\ X=\frac{x}{a}$	* $y(x,t)=\frac{Y}{2(B-A)}\left\{\begin{array}{l}(A-AX)\operatorname{erf}(AX-A)-(A+AX)\operatorname{erf}(AX+A)\\+(B+AX)\operatorname{erf}(AX+B)-(B-AX)\operatorname{erf}(AX-B)\\+\frac{2}{\sqrt{\pi}}[e^{-(A^2X^2+B^2)}\cosh(2AXB)-e^{-(A^2X^2+A^2)}\cosh(2A^2X)]\end{array}\right\}$
(d) Initially rectangular planform centered on a barrier island		$y(x,t)=4Y\left(\frac{\ell}{b}\right)\sum_{n=1}^{\infty}\frac{1}{\mu_n}\sin\left(\frac{\mu_n}{2}\right)\cos\left(\mu_n\frac{x}{\ell}\right)e^{-\mu_n^2Gt/\ell^2}$ $\mu_n=(2n-1)\frac{\ell}{b}\pi$
(d) Initially rectangular planform near inlet on a long barrier island		$y(x,t)=\frac{Y}{2}\left\{\begin{array}{l}\operatorname{erf}\left(\frac{x+b+\ell}{\sqrt{4Gt}}\right)-\operatorname{erf}\left(\frac{x+b}{\sqrt{4Gt}}\right)\\+\operatorname{erf}\left(\frac{x-b-\ell}{\sqrt{4Gt}}\right)-\operatorname{erf}\left(\frac{x-b}{\sqrt{4Gt}}\right)\end{array}\right\}$

*Walton, 1997

Figure 8-11. Solutions to Pelnard-Considere Equation

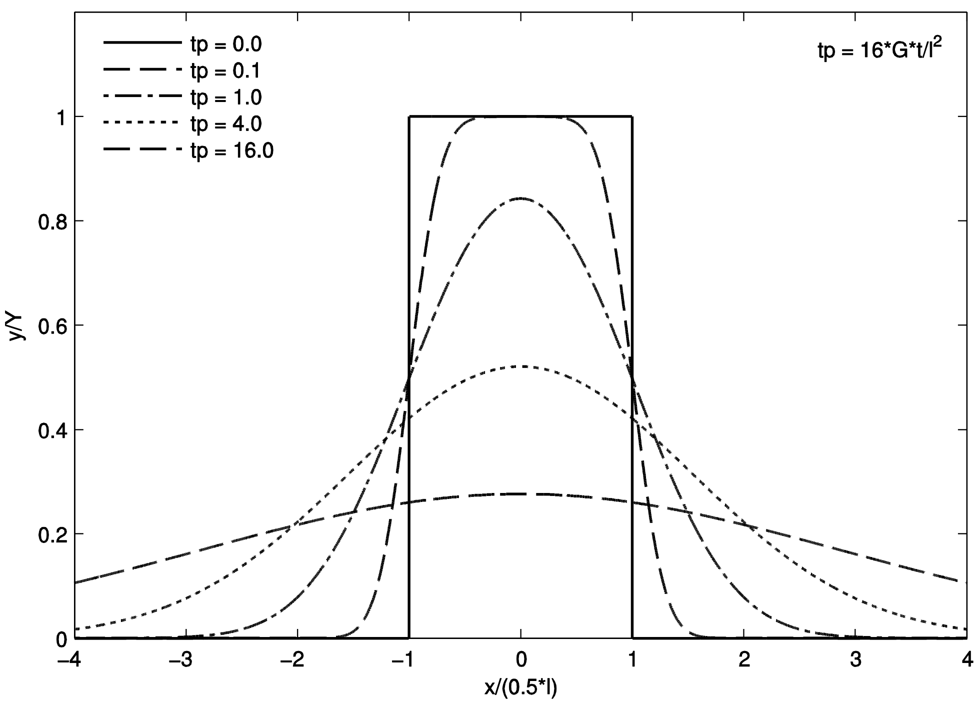


Figure 8-12. Nondimensional Beachfill Evolution Based on Diffusion Equation

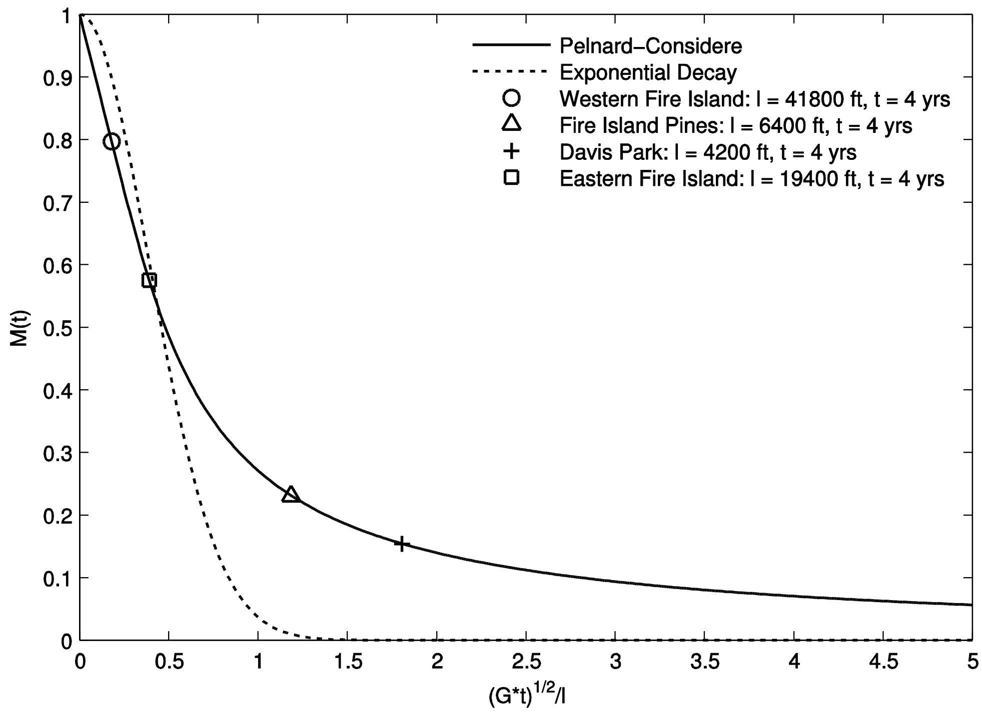


Figure 8-13. Theoretical Longevity of Beachfill (Excluding Background Erosion)

Trapezoidal Beachfill

The solution to the Pelnard-Considere equation for a trapezoidal beachfill project on a long straight beach is shown in panel "b" of Figure 8-11. The results for a trapezoidal beachfill project are similar to the results for a rectangular beachfill project with the exception that end losses are slightly lower due to the tapers. The trapezoidal beach solution was applied to Fire Island because tapers are expected to be considered in the final design. As in previous efforts, a six (6) degree taper was assumed for this study.

Incorporating Background Erosion

The combined effect of diffusion and background erosion, $\partial E / \partial t$, can be accounted for by adding an additional term to solutions for a rectangular or trapezoidal beachfill:

$$y(x,t) = \ldots - \frac{\partial E}{\partial t}$$

8.1.2.4.2 Application to Fire Island

Federal Tracts along Fire Island prevent the construction of a continuous beachfill project. Instead, the Fire Island project consists of several individual segments of beachfill that are sandwiched between Federal Tracts. The alongshore length of the individual segments varies from 1,200 feet at Water Island to 41,800 feet at Western

Fire Island. For the simple analytical approach applied here, each beachfill segment is treated as a stand-alone project. In practice, the individual beachfill project may have positive impacts on each other. However, a more sophisticated shoreline modeling approach (e.g. GENESIS) would be required to simulate the combined performance of all the beachfill projects. The simple analytical approach taken here is conservative, because it does not account for sediment transport from adjacent fill areas, and believed to be suitable for determining the relative differences in the representative erosion rates between the MREI and MIDU baselines.

Table 8-6 presents the six individual beachfill projects, the design reaches they encompass, their respective length, and associated background erosion rate. It is assumed that the background erosion rates will continue at the same rate as before the project. Background erosion rates were determined from the FIMP sediment budget and Most Vulnerable Conditions Report.

Table 8-6. Individual Beachfill Segments

Project	Design Reaches	Length (ft)	Background Erosion Rate (ft/yr)
Western Fire Island	GSB-1A, GSB-1B, GSB-2A, GSB-2B, GSB-2C, GSB-2C	41,800	3
Cherry Grove	GSB-3A	3,000	0
Fire Island Pines	GSB-3C	6,400	0
Water Island	GSB-3E	1,200	0
Davis Park	GSB-3G	4,200	0
Eastern Fire Island	MB-1A, MB-1B	19,400	1

Alongshore Diffusivity

The alongshore diffusivity, *G*, controls the rate at which "spreading" or diffusion of the beachfill project occurs. The alongshore diffusivity is proportional to the breaking wave height raised to the 5/2 power. Since the wave conditions at a site vary over time, so too does the alongshore diffusivity. Therefore, the alongshore diffusivity can be determined by integrating *G* over time or by determining an effective wave breaking height.

If the gross sediment transport rate at a site is known, then it is possible to back-calculate the effective breaking wave height, H_b, from the CERC sediment transport formula and use H_b to determine the alongshore diffusivity, *G*. It is important to use the gross sediment transport rates because it reflects the true diffusivity of project site. For example, if a study area had a very high gross sediment transport potential but virtually zero net sediment transport, one would still expect the alongshore diffusivity to be high.

Based on a gross sediment transport rate of 2.25 million m^3/yr (2.94 MCY), along Fire Island (1999 FIMP Reformulation Study), an effective breaking wave height of 3.65 feet (1.10 m), and alongshore diffusivity of 0.15 ft^2/s was found. The alongshore diffusivity was reduced by 60% to account for stabilizing effect of wave refraction around the beachfill project. This is a fair assumption when considering the work of Dean in 2005 in "Beach Nourishment Theory and Practice."

Approach to MREI and MIDU Baselines

In order to apply the beachfill diffusion analysis, the cross-shore width, Y, of the beachfill project must be known. In this application, the cross-shore width represents the distance that the design berm (plus advance nourishment) protrudes from the adjacent shoreline where no beachfill placement is planned. It is not a straightforward task to determine this cross-shore width. The cross-shore width, Y, can be further broken down into three components:

$$Y = Y_o + Y_{baseline} + Y_a$$

Where Y_o is the initial cross-shore distance that the design MIDU shoreline protrudes from the adjacent shoreline, Y_a is the advance nourishment width, and $Y_{baseline}$ is equal to:

$Y_{baseline} = 0$ for the MIDU Plan;

$Y_{baseline} = \overline{MREI_{baseline} - MIDU_{baseline}}$ for the MREI Plan.

Y_o is the same for both baselines, but Y_a will differ for two baselines since it is a function of the representative erosion rate and renourishment interval.

The approach adopted here to determine the representative erosion rates is as follows:

Assume the representative erosion rates in Table 7-83 (c. 2008) are valid for the MIDU plan except at Davis Park where recent monitoring data indicates that the erosion rate is closer to 12 ft/yr.

With length, $Y_{baseline}$, Y_a and representative erosion rates assumed, iteratively run the diffusion analysis for the MIDU plan to determine the value of Y_o. by iterating until the erosion rates converge to equal the corresponding assumed representative erosion rates from 2008.

With length, $Y_{baseline}$ and background erosion assumed and Y_o determined from the MIDU analysis, iteratively run the diffusion analysis for the MREI plan to determine the required value of Y_a. From this, diffusion erosion rates can be determined for the MREI plan.

The representative erosion rate in the diffusion analysis is measured as the average shoreline position over the initial beachfill extents. In all cases the background erosion rates were included in the beachfill diffusion analysis.

A closer examination of the 2012 LIDAR profiles at Cherry Grove and Water Island indicate that both the MIDU and MREI baseline are set back far enough that beachfill design does not extend the width of the existing beach. Therefore, the representative erosion rates from Table 7-83 (c. 2008) are applied to both the MIDU and MREI plan at these two locations.

Diffusion Results for MIDU Baseline

The results of the diffusion analysis for the MIDU baseline are presented in Table 8-7. The theoretical evolution at Western Fire Island and Fire Island Pines is presented in Figure 8-14 and Figure 8-15.

Table 8-7. Diffusion Results for MIDU Baseline

Project	Length (ft)	Y_o (ft)	$Y_{baseline}$ (ft)	Y_a (ft)	Y (ft)	Background Erosion (ft/yr)	Diffusive Erosion (ft/yr)	Representative Erosion (ft/yr)
Western Fire Island	41,800	50.5	0.0	20.0	70.5	3	2.0	5.0
Fire Island Pines	6,400	28.2	0.0	40.0	68.2	0	10.0	10.0
Davis Park	4,200	20.4	0.0	48.0	68.4	0	12.0	12.0
Eastern Fire Island	19,400	6.8	0.0	8.0	14.8	1	1.0	2.0

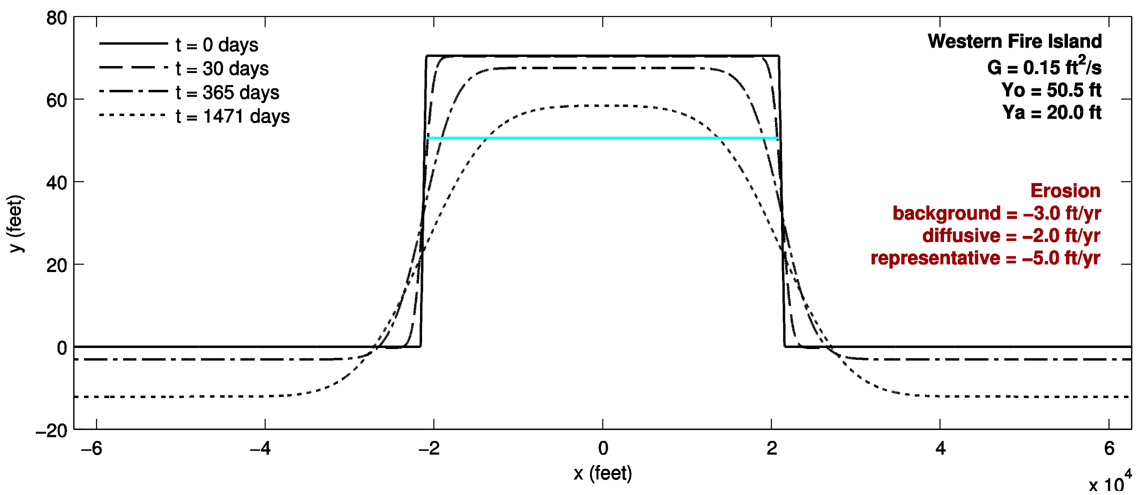


Figure 8-14. Beachfill Evolution at Western Fire Island (MIDU)

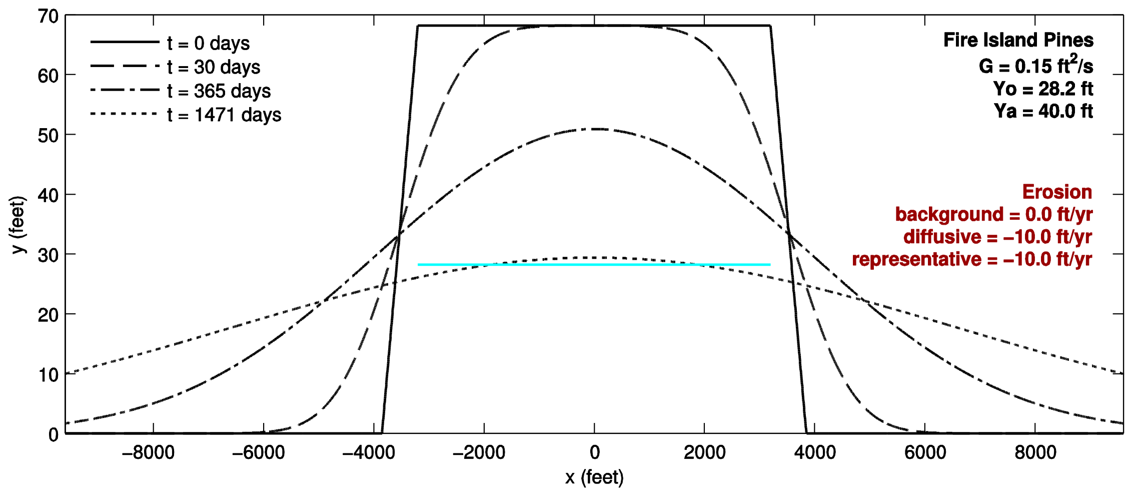


Figure 8-15. Beachfill Evolution at Fire Island Pines (MIDU)

Diffusion Results for MREI Baseline

The results of the diffusion analysis for the MREI baseline are presented in Table 8-8. The theoretical evolution at Western Fire Island and Fire Island Pines is presented in Figure 8-16 and Figure 8-17. It is worth noting that CP&E measured erosion rates of approximately 20 ft/yr at Fire Island Pines in the 3.5 years following the 2009 beachfill project so numbers in Table 7-87 seem reasonable. The results also highlight the sensitivity of the beachfill diffusion to the alongshore length. The MREI representative erosion rate at Fire Island Pines increases by 100% whereas the MREI representative erosion rate at Western Fire Island increases by about 20% even though the baseline offset is nearly the same (34 feet). A significant increase in the representative erosion rate at Davis Park is predicted because the alongshore length is relatively short (4,200 feet) and the difference in the MREI and MIDU baseline is 72 feet, nearly twice as much as at Fire Island Pines.

Table 8-8. Diffusion Results for MREI Baseline

Project	Length (ft)	Y_o (ft)	$Y_{baseline}$ (ft)	Y_a (ft)	Y (ft)	Background Erosion (ft/yr)	Diffusive Erosion (ft/yr)	Representative Erosion (ft/yr)
Western Fire Island	41,800	50.5	34.8	24.3	109.6	3	3.1	6.1
Fire Island Pines	6,400	28.2	34.4	77.1	139.8	0	19.3	19.3
Davis Park	4,200	20.4	72.6	145.7	238.7	0	36.4	36.4
Eastern Fire Island	19,400	6.8	0.0	7.9	14.6	1	1.0	2.0

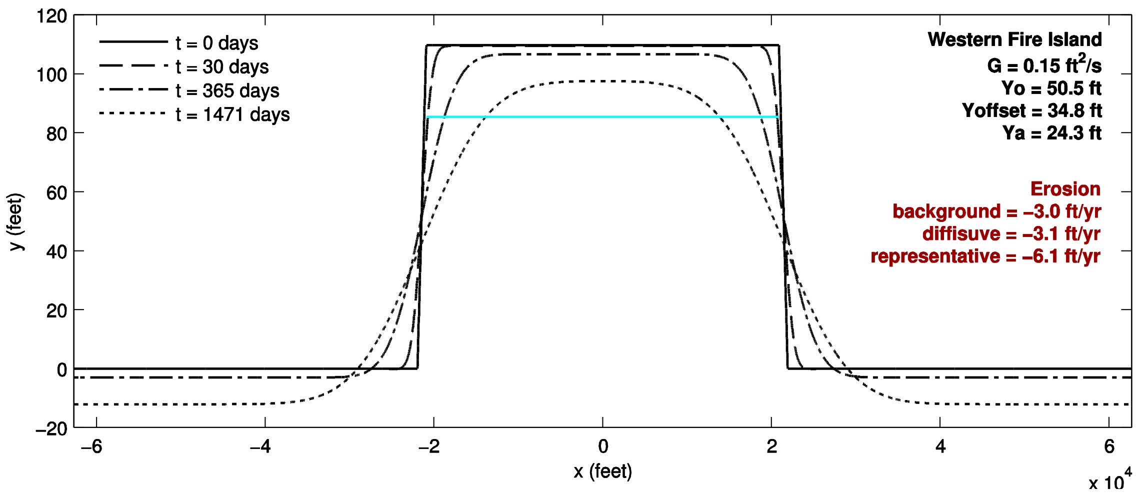


Figure 8-16: Beachfill Evolution at Western Fire Island (MREI)

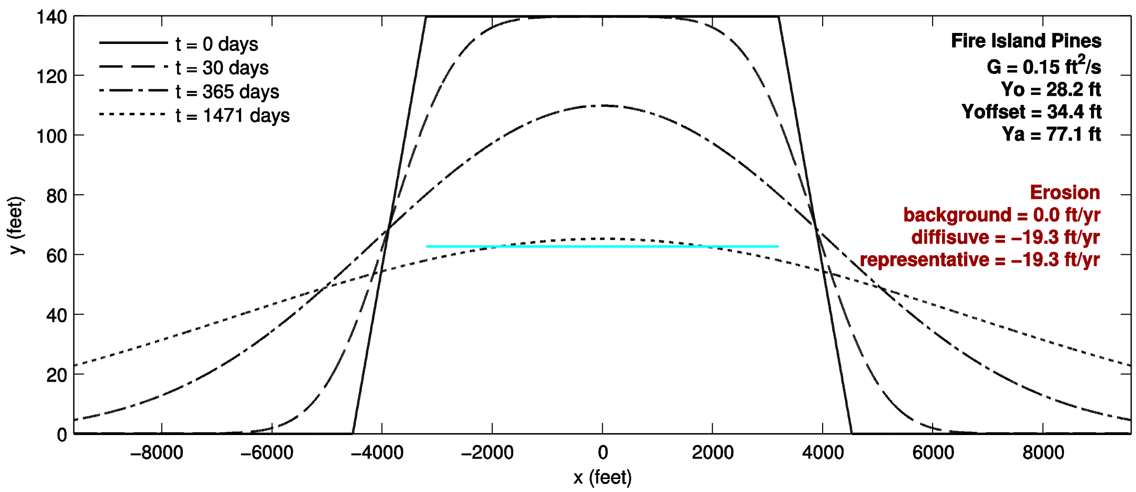


Figure 8-17. Beachfill Evolution at Fire Island Pines (MREI)

Summary of Applied Representative Erosion Rates

The beachfill diffusion analysis provides an analytical technique to predict the anticipated higher renourishment volumes for the MREI plan. The analysis indicates that representative erosion rates at Fire Island Pines and Davis Park will increase by 100% and 300% respectively. However, engineering judgment must be applied to Davis Park as the predicted increases in representative erosion rates seems excessively high. The results from the beachfill diffusion analysis have been rounded off and adjusted based on engineering judgment to determine the final representative erosion rates to be used in the re-nourishment volume estimates. These are shown in Table 8-9.

Table 8-9. Individual Beachfill Segments

Project	Design Reaches	Length (ft)	MIDU Representative Erosion Rate (ft/yr)	MREI Representative Erosion Rate (ft/yr)
Western Fire Island	GSB-1A, GSB-1B, GSB-2A, GSB-2B, GSB-2C, GSB-2C	41,800	5	6
Cherry Grove	GSB-3A	3,000	0	0
Fire Island Pines	GSB-3C	6,400	10	20
Water Island	GSB-3E	1,200	1	1
Davis Park	GSB-3G	4,200	12	25
Eastern Fire Island	MB-1A, MB-1B	19,400	2	2

8.1.3 Cost Basis Summary-MREI/MIDU Comparison

Dredging costs per cubic yard and mobilization/demobilization costs per dredging contract were determined using CEDEP (Corps of Engineers Dredge Estimating Program). CEDEP incorporates influencing factors such as hopper capacity and safe load, area of borrow site, distance to borrow site, and current fuel, labor, and equipment costs, etc. All of these items are combined with a history of recent bid prices for comparable work to determine the final cost. The cost estimates also include Engineering and Design (E&D) costs were assumed to be 7% of the construction cost. Supervision and administration (S&A) costs were calculated as a percentage of the construction costs.

Mobilization and demobilization costs for each contract were assumed to be shared between design reaches. The cost of Mob/Demob was estimated to be $4 million and was distributed proportionately to each design reach based on the volume of fill within each reach. The same construction schedule and Mob/Demob costs was assumed for the Renourishment Costs.

First costs include dredging, mobilization, and demobilization for the initial fill volumes. 15% contingency was also included. Engineering and design (E&D) costs were assumed to be 7% of the construction cost. Supervision and administration (S&A) costs were also assumed to be a percentage of the construction cost, which was computed according to the Corps formula:

$$\% = \frac{17 - \log\left(\dfrac{subtotal}{1000}\right)}{100}$$

Where *subtotal* is the total construction cost for the entire project. Note that the total construction cost does not include contingency, E&D costs, or S&A costs.

Renourishment costs include dredging, mobilization, and demobilization. Dredging unit costs are assumed to be the same for both initial fill and renourishment fill. Renourishment costs include a 15% contingency, 7% for E&D, and the S&A percentage computed as shown in above.

Berm maintenance cost is the cost of moving fill to address shoreline undulations and erosion hotspots. The cost was assumed to be $15 per linear foot of fill annually and is applicable to all reaches. Fill maintenance costs are the miscellaneous costs of maintaining the beach, such as tilling. Annual fill maintenance costs are assumed to be $2 per linear foot of fill for all reaches.

Every effort was made to keep real estate acquisition to a minimum. However, to keep benefits and costs optimized, acquiring some real estate was required. Real estate costs associated with acquiring the necessary real estate for construction of the beachfill project vary based on the alignment. The MREI alignment minimized real estate requirements and does not include any real estate acquisitions. The MIDU alignment is landward of the MREI alignment and has higher real estate costs. 42 homes were identified that interfere with the MIDU alignment. The market value of these homes was obtained from a market gross appraisal completed by NAN on June 10, 2013. The market gross appraisal reflects the value of the real estate post Hurricane Sandy. The estimated market Gross Appraisal value for the 42 properties is $47,105,000.

Annual costs incorporated initial fill, renourishment, berm and fill maintenance, and real estate. They also assumed a project life of 50 years and an interest rate of 3.75%, consistent with 2013 price levels. The annualized cost for the MREI is $24,846,059. The annualized cost for the MIDU-Medium Plan is $21,724,553.

8.2 Breach Response Costs

8.2.1 2007 Price Levels

Previous BCP cost estimates were based on an assumed daily revenue and calculated production rate. The production rate varies at each location based on the distance to the placement site, assumed work day efficiency and weather efficiency. In the past, the same daily revenue was assumed at all BCP locations:

$126,527 per day for 30" Cutter Head Dredge
$89,623 per day for 6,500 CY Hopper Dredge
$52,720 per day for 3,500 CY Hopper Dredge

The cost estimate also depends on the “effective” production rate, which accounts for washout losses before the breach is choked. Washout losses have typically been assumed to about 60% before choking and 5% after the breach is choked.

As an example, the daily production rate at Sedge Island, 1.4 nautical miles from borrow site, was determined to be 35,280 CY/day for the 30” Cutter Head Dredge. The unit price for “cut” was $3.60 per CY. However, due to washout losses, the “effective” production rate was much lower and the unit price for “placed” was $8.05 per CY.

8.2.2 2013 Price Levels

Breach closures following Hurricane Sandy and recent CEDEP unit cost estimates of beachfill indicated that the 2007 price levels need to be escalated. The unit price for "cut" quoted by Great Lakes Dock and Dredge was $17.93 per CY for Cupsogue Park, which is significantly higher than the 2007 unit cost estimates at similar locations.

CEDEP unit costs of beachfill were converted to a daily revenue cost estimate to evaluate the differences with the 2007 price levels. The CEDEP unit prices are based on a 3,800 CY Hopper Dredge and correspond to a daily revenue between $78,000 and $89,000 per day. The majority of the CEDEP daily revenue rates are $79,000 which represents a 50% increase from the 2007 price levels ($52,720 per day).

Based on this information dredging daily revenues were increased by 50% resulting in:

$190,000 per day for 30" Cutter Head Dredge;
$134,500 per day for 6,500 CY Hopper Dredge;
$79,000 per day for 3,500 CY Hopper Dredge.

The cost of mobilization and demobilization is also increased from $1.0 million to $2.5 million based on the recent estimates provided by CENAN for the Fire Island Interim Project. The discount rate was updated to 3.75%, consistent with 2013 price levels. The BCP costs were escalated from Aug 2013 price levels to Jan 2015 based on the Civil Works Construction Cost Index System (CWCCIS). No changes were made to washout losses, production rates, etc. Only the daily revenue, Mob/Demob costs, and discount rate were updated.

8.3 Storm Surge Modeling

Additional storm surge numerical modeling simulations were performed to validate the integrity of the previously completed modeling efforts and examine the applicability of the numerical model to the post-Hurricane Sandy breach open conditions at Old Inlet. The following tasks were completed and are documented in Sub Appendix A4:

- Re-validation of model to breach closed conditions
- Validation of model to breach open conditions at Old Inlet
- Impact on tides of breach open conditions at Old Inlet
- Impact on storm tides of breach open conditions at Old Inlet
- Stage frequency curves representing breach open conditions at Old Inlet

9.0 RECOMMENDED PLAN

9.1 Overview

Section 7.0 summarized alternative development and screening process originally documented in the 2009 “Fire Island to Montauk Point New York Reformulation Study Draft Formulation Report”. This analysis recommended a modified version of Alternative 3G. Based on post-Hurricane Sandy conditions the modifications described in Section 8.0 have been incorporated into a Recommended Plan, which is summarized below. The Recommended Plan also reflects modifications and refinements to the Tentatively Selected Plan (TSP) that was proposed in the June 2016 Draft HSGRR/EIS, based on public and agency review comments, and subsequent discussions to identify the USACE/DOI mutually acceptable plan, and subsequent coordination with the local sponsor.

An overall map of the Recommended Plan is shown in Figure 9-1 and Figure 9-2. An overview of the shorefront Recommended Plan features is provided in Table 9-1 and Table 9-2. More details on each measure are provided in the following sections.

9.1.1 Inlet Sand Bypassing

Continuation of authorized navigation projects, and scheduled O&M dredging with beneficial reuse of sediment at Fire Island, Moriches and Shinnecock Inlets.

Additional dredging of equivalent of 73,000 to 379,000 cy from the ebb shoals of each inlet, outside of navigation channel, with downdrift placement undertaken in conjunction with scheduled O&M dredging of the inlets.

Placement of a +13 ft. dune and berm, as needed in identified placement areas.

Monitoring to facilitate adaptive management changes in the future.

9.1.2 Mainland and Non-Structural

Addresses approximately 4,432 structures within the 10 year floodplain using nonstructural measures, primarily, structural elevations and building retrofits, based upon structure type and condition.

Includes localized acquisition in areas subject to high frequency flooding, and reestablishment of natural floodplain function .

9.1.3 Barrier Islands

9.1.3.1 Breach Response

Proactive Breach Response - is triggered when the breach and dune are lowered below a 4% AEP level of performance and provides for restoration to the design condition (+13 ft. dune and 90 ft. berm).

Reactive Breach Response - is triggered when a breach has occurred, e.g. the condition where there is an exchange of ocean and bay water during normal tidal conditions. It will be utilized as needed when a breach occurs.

Conditional Breach Response - applies to the large, Federally-owned tracts within Fire Island National Seashore where the Breach Closure Team determines whether the breach is closing naturally, and if found not to be closed at Day 60, that closure would begin on Day 60. Conditional Breach closure provides for a 90 ft. wide berm at elevation +9.5 ft. and no dune.

Wilderness Conditional Breach Response – is a response plan that applies to the Wilderness Federally-owned tracts within Fire Island National Seashore, where the Breach Closure Team determines whether a breach should be closed, based upon whether the breach is closing naturally and whether the breach is likely to cause significant damage.

9.1.3.2 Beach and Dune Fill on Shorefront

Provides for a continuous 90 ft. width berm and +15 ft. dune along the developed shorefront areas fronting Great South Bay and Moriches Bay on Fire Island and Westhampton barrier islands.

On Fire Island the alignment follows the post-Sandy optimized alignment that includes overfill in the developed locations and minimizes tapers into Federal tracts.

Renourishment: up to 30 years, placed approximately every four years.

Provides for construction of a feeder beach every 4 years for up to 30 years at Montauk Beach.

9.1.4 Groin Modifications

The Recommended Plan provides for removal of existing Ocean Beach groins.

9.1.5 Coastal Process Features (CPFs)

Provides for 12 barrier island locations and two mainland locations as coastal process features

Includes placement of approximately 4.3 M CY of sediment in accordance with the Policy Waiver for a Mutually Acceptable Plan between the Department of the Army and the Department of the Interior. Sediment will be placed along the barrier island bayside shoreline over the period of analysis that reestablishes the coastal processes consistent with the reformulation objective of no net loss of habitat or sediment. The placement of sediment along the bay shoreline will be conducted in conjunction with other nearby beach fill operations undertaken on the barrier island shorefront.

The CPFs will compensate for reductions in cross-island transport and sediment input to the Bay, offset Endangered Species Act impacts from the placement of sediment along the barrier island shorefront, augment the resiliency and enhance the overall barrier island and natural system coastal processes.

9.1.6 Adaptive Management

Will provide for monitoring and the ability to adjust specific project features to improve effectiveness.

Climate change will be accounted for with the monitoring of climate change parameters, identification of the effect of climate change on the project design, and identification of adaptation measures that are necessary to accommodate climate changes as it relates to all the project elements.

9.1.7 Integration of Local Land Use Regulations and Management

Upon project completion, the U.S. Army Corps of Engineer's Project's Annual Inspection of Completed Works (ICW) program provides for monitoring and reporting of any new development within the project area to the appropriate federal, state, and local entities responsible for enforcing applicable land use regulations.

A detailed description of each of the plan components follows.

Table 9-1. FIMP Recommended Plan Shorefront Reach Features – GSB to MB

				Subreach Recommended Plan			Breach Response Plan		Coastal Process Features		Lifecycle Plan		
Project Reach	**Design Subreach**	**Sub-Reach Name**	**Length (ft)**	**Plan**	**Berm (Ht. and width)**	**Dune**	**Breach Response**	**Breach Response Plan**	**CPF located in Sub-reach**	**Purpose (CSRM, ESA)**	**Lifecycle Response Years 1-30**	**Lifecycle Response Years 31-50**	**Years 31-50 Dune Height**
GSB (Great South Bay)		Fire Island Inlet and Gilgo Beach	N/A	Inlet Dredging and bypassing (FI)	+9.5 ft berm section	No Dune	NA	NA			FI Inlet bypassing, 2 yr cycle	FI Inlet bypassing, 2 yr cycle	No dune
	1A	Robert Moses State Park - West (need Plate -from Parkway to Jetty)	6,700	No Action	+9.5 ft, 90 ft wide	No Dune	Reactive	9.5 ft berm, 90 ft wide	1 Democrat Point West 2 Democrat Point East	ESA ESA	Reactive Breach Response	Reactive Breach Response	No dune
	1A	Robert Moses State Park - East	19,000	Beach, Dune, Berm, Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide	3 Dunefield West of Field 4	ESA	Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	1B	FI Lighthouse Tract	6,700	Proactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune, no planting	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide			Proactive Breach respone	Proactive Breach respone	13 ft dune
	2A	Kismet to Lonelyville	8,900	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide	4 Clam Pond	CSRM, ESA	Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	2B	Town Beach to Corneille Estates	5,100	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide	5Atlantique to Corneille	CSRM, ESA	Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	2C	Ocean Beach & Seaview	3,800	Beach, Dune, Renourish, Groin Modification	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide			Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	2D	OBP to Point O' Woods	7,400	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide		CSRM, ESA	Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	2E	Sailors Haven	8,100	Conditional Breach Response	+9.5 ft closure section (max berm ht.)	No Dune	Conditional	No dune. Berm closure width to taper to adjacent area.		CSRM, ESA	Conditional Breach Closure	Conditional Breach Closure	No dune
	3A	Cherry Grove	3,000	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide			Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	3B	`Carrington Tract	1,500	Conditional Breach Response	+9.5 ft closure section (max berm ht.)	No Dune	Conditional	No dune. Berm closure width to taper to adjacent area.		CSRM, ESA	Conditional Breach Closure	Conditional Breach Closure	No dune
	3C	Fire Island Pines	6,600	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide			Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	3D	Talisman to Water Island	7,300	Reactive Breach Response	+9.5 ft, 90 ft wide	No Dune	Reactive	No dune. Maximum berm height 9.5 ft. Berm closure width to taper to adjacent area.	6 Talisman	CSRM, ESA CSRM, ESA	Reactive Breach Closure	Reactive Breach Closure	No dune
	3E	Water Island	2,000	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide			Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	3F	Water Island to Davis Park	4,700	Conditional Breach Response	+9.5 ft closure section (max berm ht.)	No Dune	Conditional	No dune. Berm closure width to taper to adjacent area.			Conditional Breach Closure	Conditional Breach Closure	No dune
	3G	Davis Park	4,100	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide			Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	3H	Watch Hill	5,000	Conditional Breach Response	+9.5 ft closure section (max berm ht.)	No Dune	Conditional	No dune. Berm closure width to taper to adjacent area.			Conditional Breach Closure	Conditional Breach Closure	No dune
	4A	Wilderness Area - West	19,000	Wilderness Conditional Breach Response	+9.5 ft closure section (max berm ht.)	No Dune	Conditional	No dune. Berm closure width to taper to adjacent area.			Wilderness Conditional Closure	Wilderness Conditional Closure	No dune
	4B	Old Inlet	16,000	Wilderness Conditional Breach Response	+9.5 ft closure section (max berm ht.)	No Dune	Conditional	No dune. Berm closure width to taper to adjacent area.			Wilderness Conditional Closure	Wilderness Conditional Closure	No dune

Table 9-2. FIMP Recommended Plan Shorefront Reach Features – SB to M

				Subreach Recommended Plan			Breach Response Plan		Coastal Process Features		Lifecycle Plan		
Project Reach	**Design Subreach**	**Sub-Reach Name**	**Length (ft)**	**Proposed Plan**	**Berm (Ht. and width)**	**Dune**	**Breach Response**	**Breach Response Plan**	**CPF located in Sub-reach**	**Purpose (CSRM, ESA)**	**Lifecycle Response Years 1-30**	**Lifecycle Response Years 31-50**	**Years 31-50 Dune Height**
MB (Moriches Bay)	1A	Smith Point CP- West	6,300	Reactive Breach Response and nourishment	+9.5 ft closure section (max berm ht.)	No Dune	Reactive	No dune. Berm closure width to taper to adjacent area.			Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	1B	Smith Point CP - East	13,500	Proactive Breach Response, sand bypassing	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide	7 Pattersquash Reach 8 New Made Is. Reach	CSRM, ESA; CSRM, ESA	Moriches Inlet sand bypassing placement- 1-yr cycle, and proactive response	Moriches Inlet sand bypassing placement- 1-yr cycle, and proactive response	13 ft dune
	2A	Great Gun	7,600	Proactive Breach Response, sand bypassing	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide	9 Smith Point County Park Marsh	CSRM	Moriches Inlet sand bypassing placement- 1-yr cycle, and proactive response	Moriches Inlet sand bypassing placement- 1-yr cycle, and proactive response	13 ft dune
	2B	Moriches Inlet - West	6,200	Proactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide	10 Great Gun	ESA	Proactive Breach respone (actual dimentsions to conform with Great Gunn FIMI CPF)	Proactive Breach respone (actual dimentsions to conform with Great Gunn FIMI CPF)	13 ft dune
		Moriches Inlet		Inlet Dredging and bypassing - 1-yr cycle	+9.5 ft, 90 ft wide						Inlet Dredging and bypassing - 1-yr cycle	Inlet Dredging and bypassing - 1-yr cycle	
	2C	Cupsogue Co Park	7,500	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide		ESA	Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	2D	Pikes	9,700	Beach, Dune and Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide			Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
	2E	Westhampton	18,300	Beach, Dune, Renourishment	+9.5 ft, 90 ft wide	15 ft dune	Reactive	15 ft dune, 9.5 ft berm, 90 ft wide			Periodic renourishment (approx. 4 year cycle)	Proactive Breach respone	13 ft dune
SB (Shinecock Bay)	1A	Hampton Beach	16,800	Proactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide			Proactive Breach respone	Proactive Breach respone	13 ft dune
	1B	Sedge Island	10,200	Shinnecock Inlet bypassing placement; Proactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide	11 Dune Road, East Quogue	CSRM	Shinnecock sand bypassing placement - 2 yr cycle, and proactive breach respone	Shinnecock sand bypassing placement - 2 yr cycle, and proactive breach respone	13 ft dune
	1C	Tiana Beach	3,400	Shinnecock Inlet bypassing placement; Proactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide	12 Tiana Bayside Park	CSRM	Shinnecock sand bypassing placement - 2 yr cycle, and proactive breach respone	Shinnecock sand bypassing placement - 2 yr cycle, and proactive breach respone	13 ft dune
	1D	Shinnecock Inlet Park West	6,300	Shinnecock Inlet bypassing placement; Proactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide			Shinnecock sand bypassing placement - 2 yr cycle, and proactive breach respone	Shinnecock sand bypassing placement - 2 yr cycle, and proactive breach respone	13 ft dune
	2A	Ponquogue	5,300	Proactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide			Proactive Breach respone	Proactive Breach respone	13 ft dune
	2B	WOSI	3,900	Shinnecock Inlet bypassing placement; Proactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Proactive	13 ft dune, 9.5 ft berm, 90 ft wide			Shinnecock sand bypassing placement - 2 yr cycle, and proactive breach respone	Shinnecock sand bypassing placement - 2 yr cycle, and proactive breach respone	13 ft dune
		Shinnecock Inlet		Inlet Dredging and bypassing - 2-yr cycle							Inlet Dredging and bypassing - 2-yr cycle	Inlet Dredging and bypassing - 2-yr cycle	13 ft dune
	2C	Shinnecock Inlet - East	9,800	Reactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Reactive	13 ft dune, 9.5 ft berm, 90 ft wide			Reactive breach response, initial 30 yrs	Reactive breach response, Years 31-50	13 ft dune
	3A	Southampton Beach	9,200	Reactive Breach Response	+9.5 ft, 90 ft wide	13 ft dune	Reactive	13 ft dune, 9.5 ft berm, 90 ft wide			Reactive breach response, initial 30 yrs	Reactive breach response, Years 31-50	13 ft dune
	3B	Southampton	5,300	No Federal Action									
	3C	Agawam	3,800	No Federal Action									

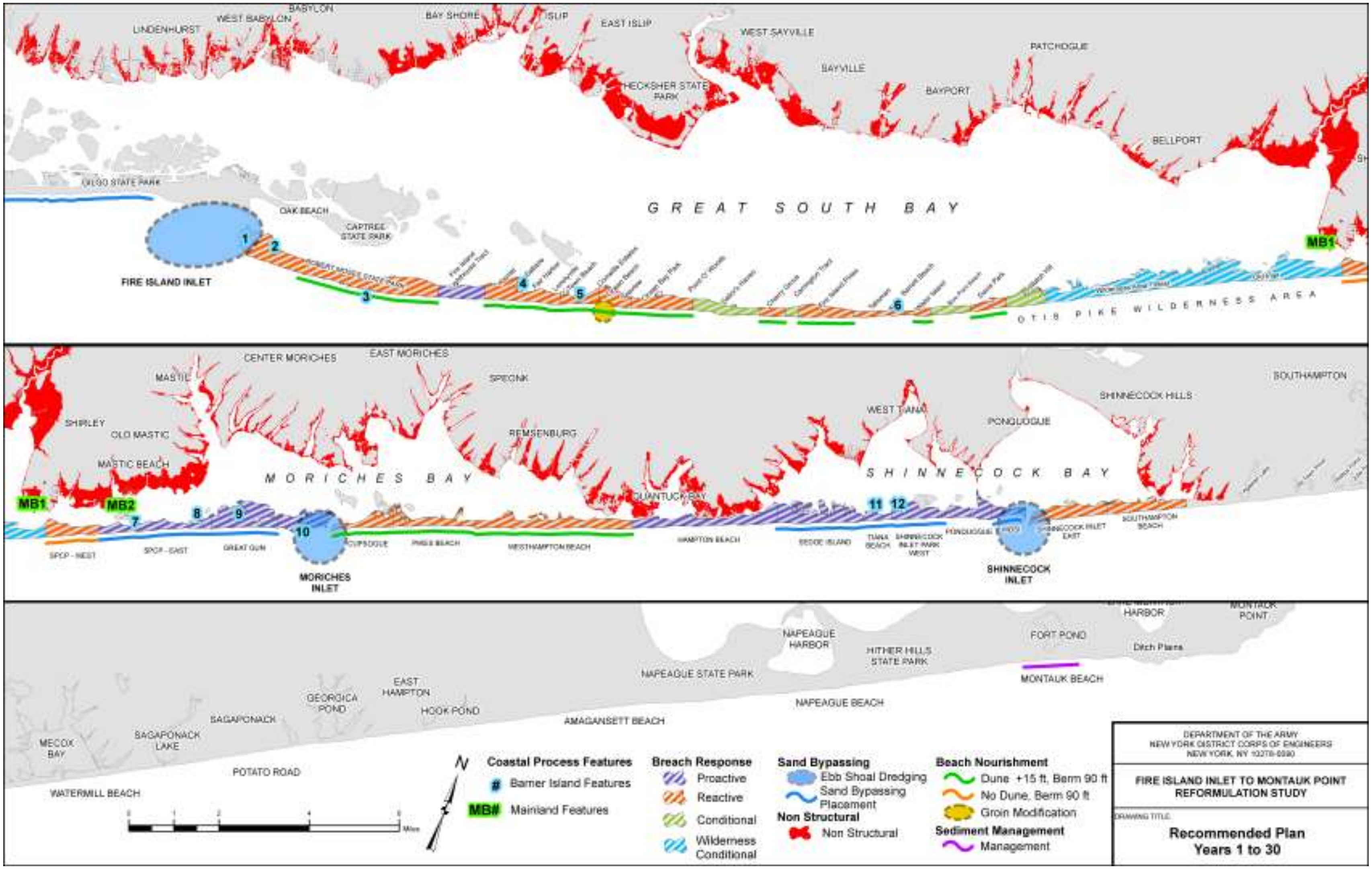


Figure 9-1. Recommended Plan: Year 1 to Year 30

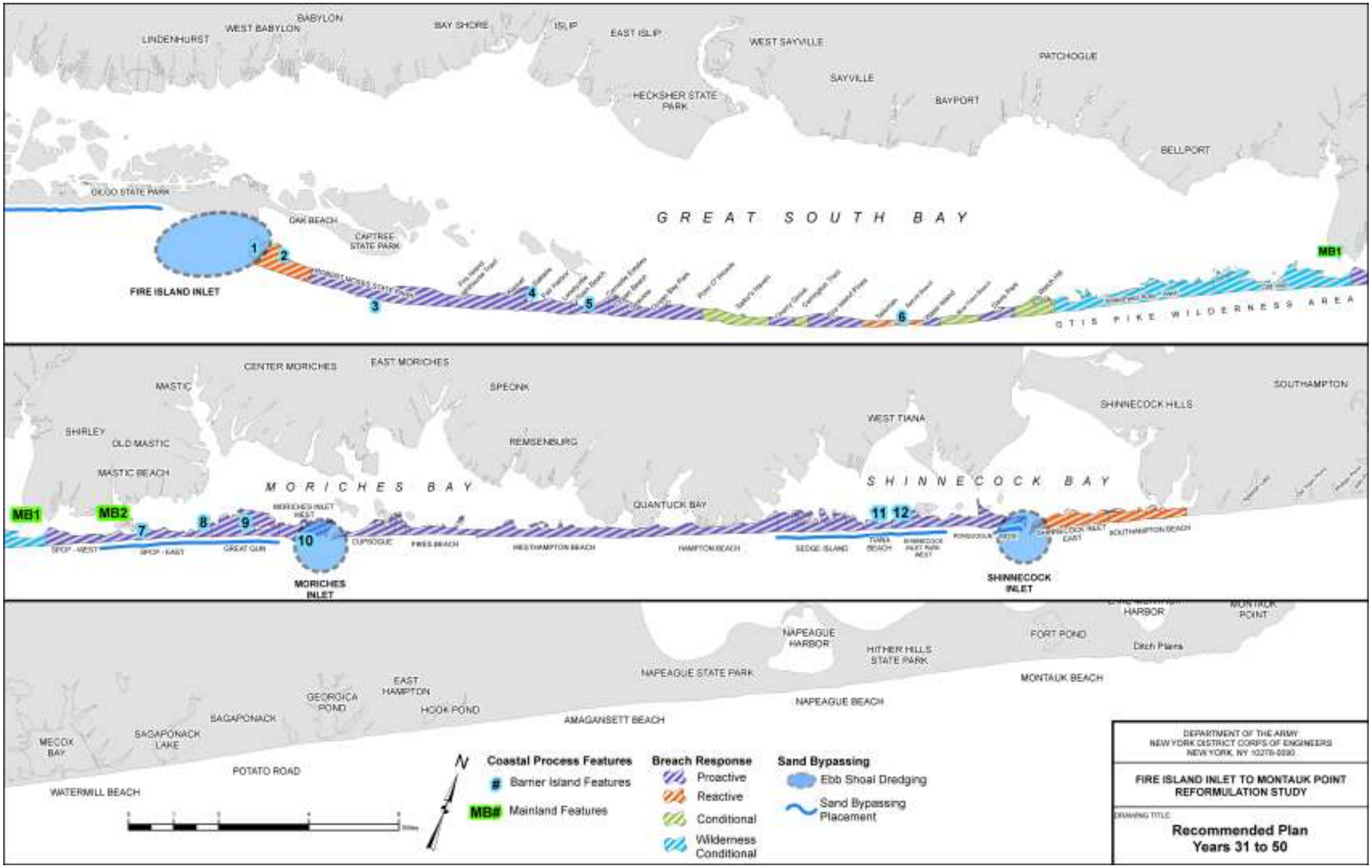


Figure 9-2. Recommended Plan: Year 31 to Year 50

9.2 Inlet Sand Bypassing

The selected inlet sand bypassing plans at the three Federal navigation inlets consist of continuation of the existing authorized projects and additional dredging of the ebb shoal, outside of the navigation channel, with downdrift placement in the quantities needed to restore littoral transport of sediment across the inlets for 50 years. Bypassing would be undertaken in conjunction with scheduled Operations and Maintenance (O&M) dredging of the inlets and would increase sediment bypassing and reduced future renourishment fill requirements.

<u>Fire Island Inlet</u>

O&M maintenance dredging of authorized channel and deposition basin to take place on a 2 year interval, as authorized;

379,000 CY (per O&M event) dredged from the ebb shoal (as needed to offset sediment deficit) and placed downdrift at Gilgo Beach;

<u>Moriches Inlet</u>

O&M maintenance dredging of authorized channel to take place on a 1-year interval (as authorized);

Approximately 73,000 CY (per O&M event) dredged from the from ebb shoal (as needed to offset sediment deficit) and placed downdrift at Smith Point County Park;

<u>Shinnecock Inlet</u>

O&M maintenance dredging of authorized channel to take place on a 2- year interval as authorized);

105,000 CY (per O&M event) dredged from channel/deposition basin, and from ebb shoal (as needed to offset sediment deficit) and placed downdrift at Sedge Island, Tiana Beach, and West of Shinnecock (WOSI);

9.2.1 Inlet Management – Initial Construction

Initial construction quantities for the Inlet Management measures include the estimated quantity to restore the channel to its authorized dimensions as well as dredging of the ebb shoal for bypassing. Initial construction quantities were estimated based on expected sedimentation in the authorized channel over the period between the last anticipated O&M dredging operation prior to start of FIMP construction. Table 9-3 shows the anticipated date of the last O&M dredging event prior to the start of FIMP and the number of years in which sedimentation may occur.

Table 9-3. Number of Years between Last Inlet O&M Dredging Operation and FIMP Start

Inlet	Sedimentation (years)	Anticipated Dredging Event prior to FIMP Start
Fire Island Inlet	1.75	Q2 2019
Moriches Inlet	2.5	Q1 2019
Shinnecock Inlet	7.25	Q2 2014

Expected initial construction dredging volumes at each inlet are presented in Table 9-4. As noted on the table, sediment will be used for beachfill, Proactive Breach Response Plan (PBRP), and Coastal Process Features (CPFs). Sedimentation rates at the three inlets are based on the Existing Conditions sediment budget at each inlet as document in the 2007 Inlet Modifications Report (see Sub-Appendix A3). Actual dredging volumes and distribution of the fill placement will be refined during PED based surveys of the inlets and beach prior to construction.

Table 9-4. Inlet Management (Initial Construction)

Location	Subreach	Fill Length (ft)	Volume per Operation (cy)
Fire Island Inlet – 2 year Dredging Cycle			
Gilgo Beach (Bypassing)		12,700	701,048
RMSP (Beachfill)	GSB-1A	12,000	536,327
			1,237,375
Moriches Inlet – 1 year Dredging Cycle			
SPCP-West (Beachfill-Bypassing)	MB-1A	6,900	129,317
SPCP-East (PBRP and CPFs)	MB-1B	13,100	188,683
			318,000
Shinnecock Inlet – 2 year Dredging Cycle			
WOSI (PRBP)	SB-2B	2,700	700,000
			700,000

9.2.2 Inlet Management – Life Cycle

Following the initial dredging of the inlets to authorized depths, future bypassing quantities are expected to on average equal the values outlined above. A summary of the dredging quantities and placement locations for all future dredging operations is shown in Table 9-5.

Table 9-5. Inlet Management (Life Cycle)

Location	Subreach	Fill Length (ft)	Volume per Operation (cy)
Fire Island Inlet – 2 year Dredging Cycle			
Gilgo Beach (Bypassing)		12,700	1,145,469
RMSP (Beachfill) (only Y1 to Y30)	GSB-1A	12,000	214,531
			1,360,000
Moriches Inlet – 1 year Dredging Cycle			
SPCP-West (Beachfill-Bypassing)	MB-1A	6,900	40,959
SPCP-East (PBRP and CPFs)	MB-1B	13,100	96,261
Great Gun (PBRP and CPFs)	MB-2A	4,500	33,780
			171,000
Shinnecock Inlet – 2 year Dredging Cycle			
Sedge Island (PBRP and CPFs)	SB-1B	5,600	45,296
Tiana Beach (PBRP and CPFs)	SB-1C	3,400	41,699
SPW (PBRP)	SB-1D	3,400	18,005
WOSI (PBRP)	SB-2B	2,700	170,000
			275,000

9.3 Non-Structural and Road Raising

The plan for the mainland provides for coastal storm risk management for a total of 4,432 structures that are located within the existing 0.1% exceedance floodplain. Of these 3,675 would be elevated, 650 would receive flood proofing, 93 would receive ringwalls, and 14 would be bought out. The design elevation level includes 2 ft. of freeboard consistent with State of New York Building Code, and Hurricane Sandy Recovery guidelines.

It is noted that following Hurricane Sandy, multiple post storm recovery programs have proposed nonstructural treatments within the study area. The specific nonstructural scale and treatment will be reviewed and refined in the Preconstruction Engineering and Design (PED) phase to ensure that the treatment proposed and the applicable population is appropriately identified.

The number of non-structural treatments initially proposed by town are as follows:

Babylon	1,523
Islip	942
Brookhaven	1,269
Southampton	705

The locations are conceptually shown in Figure 9-1 in red based on the 10-year flood plain.

9.4 Breach Response Plans

9.4.1 Proactive Breach Response Plan

The Proactive Breach Response Plan (Proactive BRP) is an alternative that includes measures to prevent breaches from occurring at locations vulnerable to breaching, when a breach is imminent. This alternative provides a beach cross-section area that is comparable to the Breach Closure Alternatives, and smaller than a beach fill alternative.

The Proactive BRP is not specifically designed with the intent of protecting ocean shorefront development from overwash, wave attack or storm induced erosion losses. The Proactive BRP allows for a greater level of overwash and dune lowering during a storm, so long as the overwash extent is below the threshold that would result in breaching.

Based upon the results of the Breach Closure Alternatives analysis, this alternative considered only the plan with the +13 ft dune section. A typical Proactive BRP section is shown in Figure 9-3.

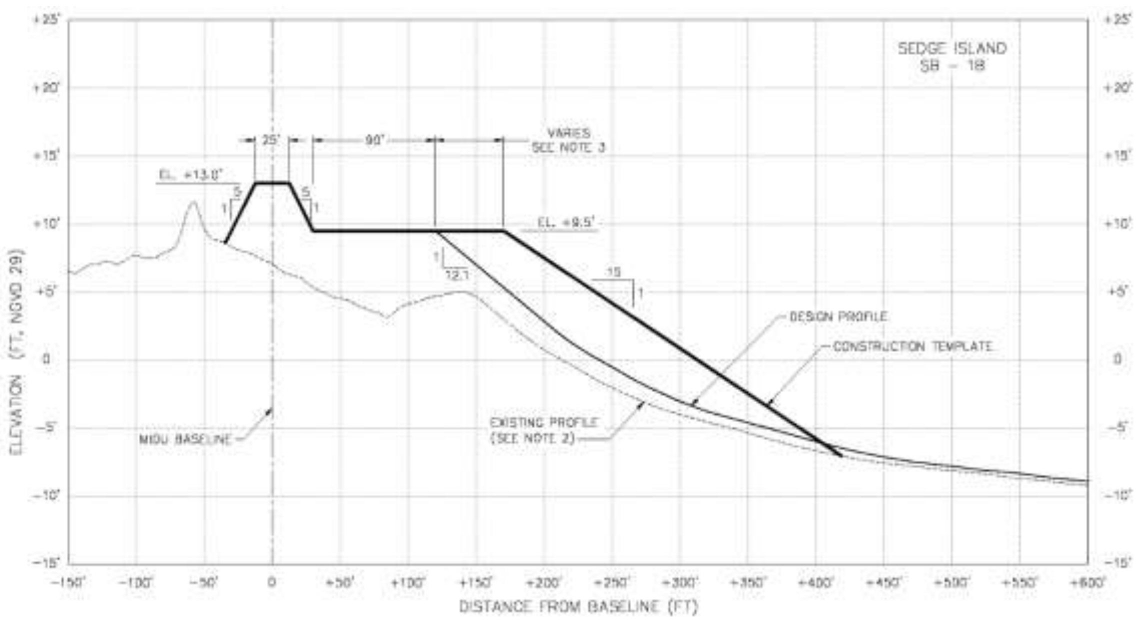


Figure 9-3. Typical Proactive BRP Section (notes are included in the Plates Appendix)

9.4.1.1 Praoctive Breach Response Triggers

Proactive Breach Response (PBR) triggers have been developed based on dimensions than can be easily measured and monitored as part of the FIMP project. These triggers are reach specific and consider historic breaching/overwash data, modeling results, and overall understanding of the hydraulic "conductivity" at each location. Breaching response is a multidimensional problem, so there is not one single measurement than can be monitored and used as threshold for action. Therefore, the following relevant dimensions are measured and considered instead:

1. Barrier island width: distance between bay and ocean MHW contours
2. Elevation: generally characterized by volume/area above +10 ft NGVD29
3. Beach width: distance between baseline (generally the natural dune alignment) and the MHW contour

Specific PBR thresholds by reach are summarized in Table 7-35. When one or more of these proposed thresholds is exceeded, the risk of a partial breach is at the 25 year return period level and proactive action should be taken to rebuild the PBR template and reduce the risk of breaching. Note that if one of these thresholds is met over a very small area but the barrier island is generally in good condition otherwise, the risk of breaching is significantly less than if the threshold is met over a large area. Therefore, the response triggers recommended in Table 7-35 are based on both widespread but not necessarily contiguous weakness within a reach and smaller, localized, but potentially weaker spots

9.4.1.2 Initial Construction (Proactive BRP)

Four of the Proactive BRP reaches were recently nourished as part of either FIMI (FILT, SPCP- East, and Great Gunn) or the WOSI Interim Project (WOSI). Due to the relatively low erosion rates at FILT and Great Gunn it is not expected that Proactive BRP would be required at any of these locations at the time of initial construction However, due to the relatively high erosion rates at WOSI, initial Proactive BRP beach fill placement is expected to be required at this location. WOSI fill would be obtained from inlet dredging, as summarized in Table 9-4 above. In addition, some sediment from Moriches Inlet would be placed as Proactive BRP fill at SPCP-East (see Table 9-4).

At the other Proactive BRP reaches along Shinnecock Bay an assessment was conducted to determine if the beach conditions were below Proactive BRP thresholds warranting beach fill placement during initial construction of FIMP. LIDAR data collected by the USACE on November 14, 2012 (two weeks following Hurricane Sandy) was used to define conditions at the time. It was determined that Sedge Island, Tiana Beach, and SPW were below the threshold. Initial construction volume estimates at these three locations are derived from quantity takeoffs based on the 2012 LIDAR data and Proactive BRP template plus additional erosion prior to the start of construction. Average-end-area calculations were completed based on profiles spaced 200 feet apart. All Proactive BCP quantities include 15% overfill and 15% contingency/tolerance. No advance fill is included in the Proactive BRP.

A summary of the initial construction quantities from offshore borrow sources for the Proactive BRP is provided in Table 9-6.

Table 9-6. Proactive BRP Initial Construction Quantities from Offshore Borrow Sources

Location	Subreach	Sediment Source	Fill Length (ft)	Volume (cy)
Sedge Island	SB-1B	BA 5Bexp	10,200	1,037,027
Tiana Beach	SB-1C	BA 5Bexp	3,400	207,199
SPW	SB-1D	BA 5Bexp	3,400	427,284
				1,671,511

9.4.2 Reactive Breach Closure

Reactive Breach Response is triggered in response to the occurrence of a breach at any locations along the barrier islands, except for most of the large federally-owned tracts within Fire Island National Seashore. Conditional and Wilderness Breach Responses typically apply to these FIIS tracts, in which the Breach Response Team will assess if the breach is closing naturally or if mechanical closure is required. Exceptions include the Fire Island Lighthouse and Talisman tracts, where Proactive and Reactive Breach Response, respectively, would be implemented (see Figure 9-1 and Figure 9-2). A typical Reactive BRP section is shown in Figure 9-4.

The Reactive BRP template would restore the design beachfill template in locations where beachfill is recommended (dune at +15 ft NGVD 29 and 90 ft wide berm at +9.5 ft NGVD 29). At Talisman, where breach response does not include a dune and the berm width would match conditions in adjacent areas. A typical breach closure section at Robert Moses State Park is shown in Figure 9-4. The design foreshore slope is 1 on 12 which is also the same slope defined for the beach fill design templates. The design profile below MHW would match the representative morphological profile corresponding to each specific location. At a minimum, bayside slopes and shorelines would generally match the preexisting adjacent shorelines. Based on the existing topography the bayside design slope was selected as 1 on 20 from the bayside crest of the berm to an elevation of +6 ft. NGVD 29. The specific layout will be developed as part of the breach closure plan at the time of the closure operation and may include more placement of sediment along the bay shoreline than existed prior to the breach in order to replicate cross-island sediment transport, and to achieve the project goals of no net loss of sediment.

9.4.3 Conditional and Wilderness Conditional Breach Closure

Conditional or Wilderness Conditional Breach Responses apply to most FIIS tracts as shown in Figure 9-1 and Figure 9-2. As part of the Conditional BRP, the Beach Closure Team may delay breach closure up to 60 days to determine whether the breach is closing naturally. Under this scenario, construction would be initiated after 60 days, if the breach does not close naturally within these first 60 days. Under the Wilderness Conditional BRP a breach would be closed only if it is determined that the breach is not closing naturally, and that significant damage is likely to occur.

This approach is consistent with the NPS recommended plan for the existing Wilderness Area breach.

The Conditional and Wilderness Conditional BRP templates do not include a dune. Both breach closure templates have a berm with height of +9.5 ft. NGVD 29. A typical breach closure section is shown in Figure 7-37. The intent of the conditional response template is to match the berm width with conditions prior to the breach and within adjacent areas. The design foreshore slope and bayside slopes and shorelines would generally match the preexisting adjacent shorelines. The specific dimensions and configuration will be developed as part of the breach closure plan at the time of the closure operation, and may include more placement of sediment along the bay shoreline than existed prior to the breach in order to replicate cross-island sediment transport, and to achieve the project goals of no net loss of sediment.

9.4.4 Breach Closure Costs

Table 9-7 presents the estimated cost of breach closure with and without a BCP for large breach sizes at Great South Bay and standard breach sizes at Moriches and Shinnecock Bay. The without project BCP assumes a 9 month delay in construction. Table 9-8 presents the estimated cost of breach closure with and without a BCP for Great South Bay and small breach size.

Table 9-7. Breach Closure Cost by BCP Location and Design Template (Large & Standard Breach)

Location	Construction Alternative Resulting in Lowest Total Cost	Without Project Closure Cost	BCP Closure Cost
FI Lighthouse Tract	Hopper Dredge	$38,987,425	$31,689,217
Town Beach to Corneille Estates	Cutterhead Dredge	$36,837,420	$18,612,316
Talisman to Water Island	Cutterhead Dredge	$28,710,076	$13,889,596
Davis Park	Cutterhead Dredge	$28,737,131	$13,899,421
Old Inlet West	Cutterhead Dredge	$31,469,134	$15,435,697
Old Inlet East	Cutterhead Dredge	$28,031,824	$14,133,247
Smith Point County Park	Hopper Dredge	$24,599,965	$18,208,062
Sedge Island	Cutterhead Dredge	$16,710,948	$10,254,929
Tiana Beach	Cutterhead Dredge	$16,194,807	$10,033,388
WOSI	Hopper Dredge	$19,159,535	$15,374,275

Table 9-8. Breach Closure Cost by BCP Location and Design Template (Small Breach)

Location	Construction Alternative Resulting in Lowest Total Cost	Without Project Closure Cost	BCP Closure Cost
FI Lighthouse Tract	Hopper Dredge	$10,919,328	$8,647,621
Town Beach to Corneille Estates	Cutterhead Dredge	$10,746,227	$7,340,820
Talisman to Water Island	Cutterhead Dredge	$9,340,158	$6,677,611
Davis Park	Cutterhead Dredge	$9,345,042	$6,679,387
Old Inlet West	Cutterhead Dredge	$9,861,252	$7,065,152
Old Inlet East	Cutterhead Dredge	$9,240,913	$6,829,861

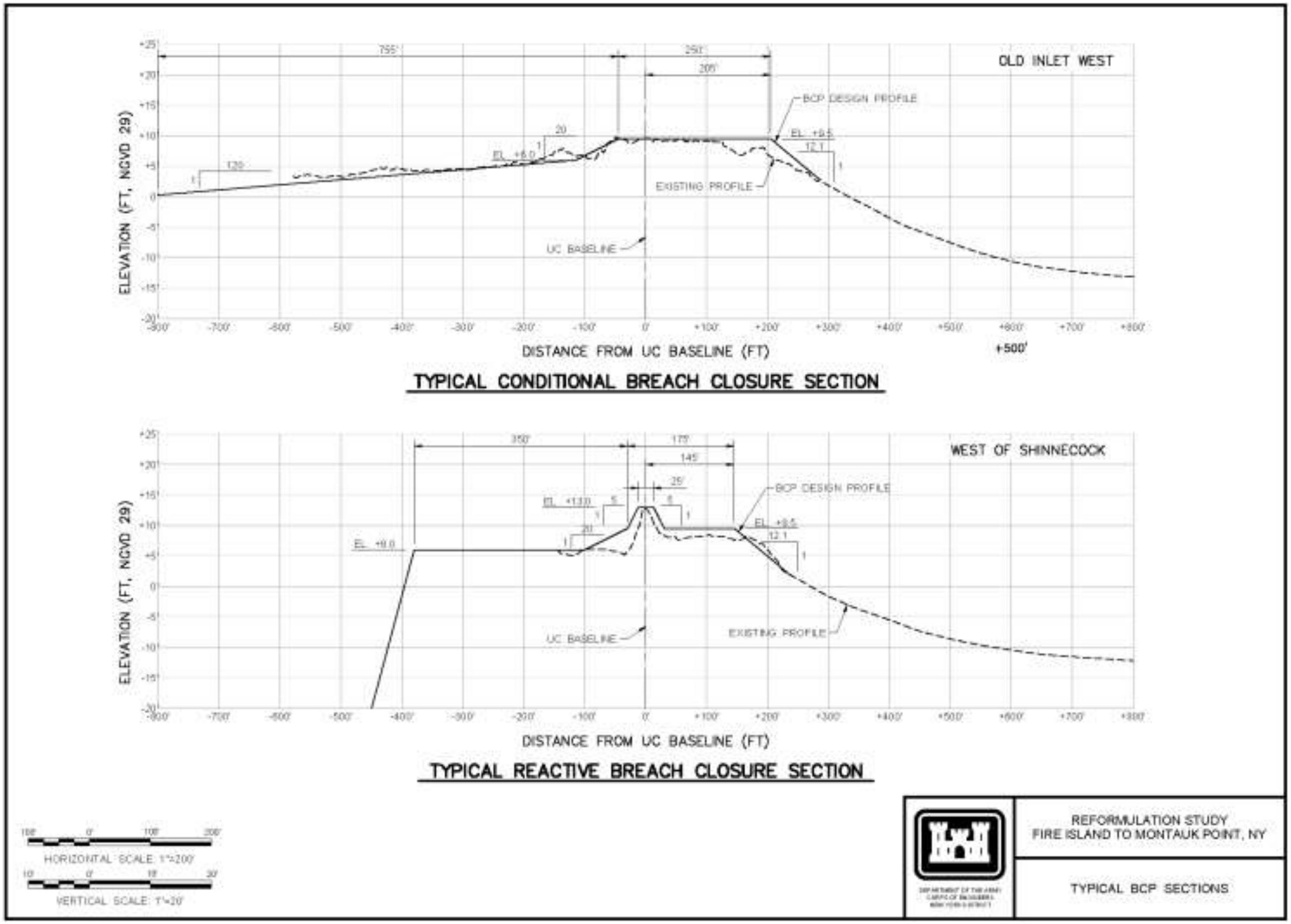


Figure 9-4. Typical Breach Closure Sections (notes are included in the Plates Appendix)

9.5 Beach Fill Plan

Specific locations of beachfill placement are outlined in Table 9-9. The three locations slated for beachfill not on Fire Island (Cupsogue County Park, Pikes Beach & Westhampton) remained consistent with the earlier TFSP.

The *Berm Only* and *Medium* design templates are used in the selected plan. The *Medium* design template has a dune with a crest width of 25 feet and dune elevation of +15 feet NGVD. Both design templates have a berm width of 90 feet at elevation +9.5 feet NGVD. The proposed design (not construction) foreshore slope (from +9.5 to +2 feet NGVD) is roughly 12.1 on 1. Below MHW (roughly +2 feet NGVD) the submerged morphological profile, representative of each specific reach, is translated and used as the design profile. Figure 7-39 shows typical design section for the *Medium* design template. Table 9-9. provides an overview of the dune elevations by location along the selected plan.

The *Berm Only* template is applicable to areas in which the existing condition dune elevation and width reduce the risk of breaching but have eroded beach berm conditions. The 90 feet design berm provides protection to the existing dunes and ensure vehicular access during emergency response and evacuation. The *Berm Only* template is applied to SPCP-West (MB-1A).

The *Medium* template was identified as having the highest net benefits and provides for approximately a 44-yr level of protection. The *Medium* template is applied to the areas with the greatest potential for damages to oceanfront structures.
Advance fill is a sacrificial quantity of sand which acts as an erosional buffer against long-term and storm-induced erosion as well as beachfill losses cause by “spreading out” or diffusion. The required advance berm width was computed based on representative erosion rates and expected renourishment interval, 4 years. The representative erosion rates were calculated based on the historical sediment budget, volumetric changes in measured profiles between 1988 and 2012, the performance of recent beach fill projects, and anticipated beach fill spreading.

The Beach Fill Plan includes taper (transition) to reduce end losses and increase the longevity of the fill. The taper lengths along Fire Island match the plans for FIMI. Tapers are accounted for in initial and renourishment volume estimates.

Table 9-9. Beach Fill Locations

Location	Subreach	Plan Component	Max Fill Length (ft)	Ren. Fill Length (ft)	Dune Elv. (ft, NGVD)
RMSP	GSB-1A	Beach Fill & Inlet Mgmt.	16,600	12,000	15
Kismet to Lonelyville	GSB-2A	Beach Fill	8,900	8,900	15
Town Beach to Corneille Est.	GSB-2B	Beach Fill	4,500	4,500	15
Ocean Beach to Seaview	GSB-2C	Beach Fill	3,800	3,800	15
OBP to POW	GSB-2D	Beach Fill	7,300	7,300	15
Cherry Grove	GSB-3A	Beach Fill	3,000	3,400	15
Fire Island Pines	GSB-3C	Beach Fill	6,500	7,000	15
Water Island	GSB-3E	Beach Fill	1,200	1,600	15
Davis Park	GSB-3G	Beach Fill	4,200	5,000	15
SPCP-West	MB-1A	Beach Fill & Inlet Mgmt.	6,300	6,300	-
Cupsogue	MB-2C	Beach Fill	4,300	2,000	15
Pikes	MB-2D	Beach Fill	9,600	9,600	15
Westhampton	MB-2E	Beach Fill	10,900	10,900	15

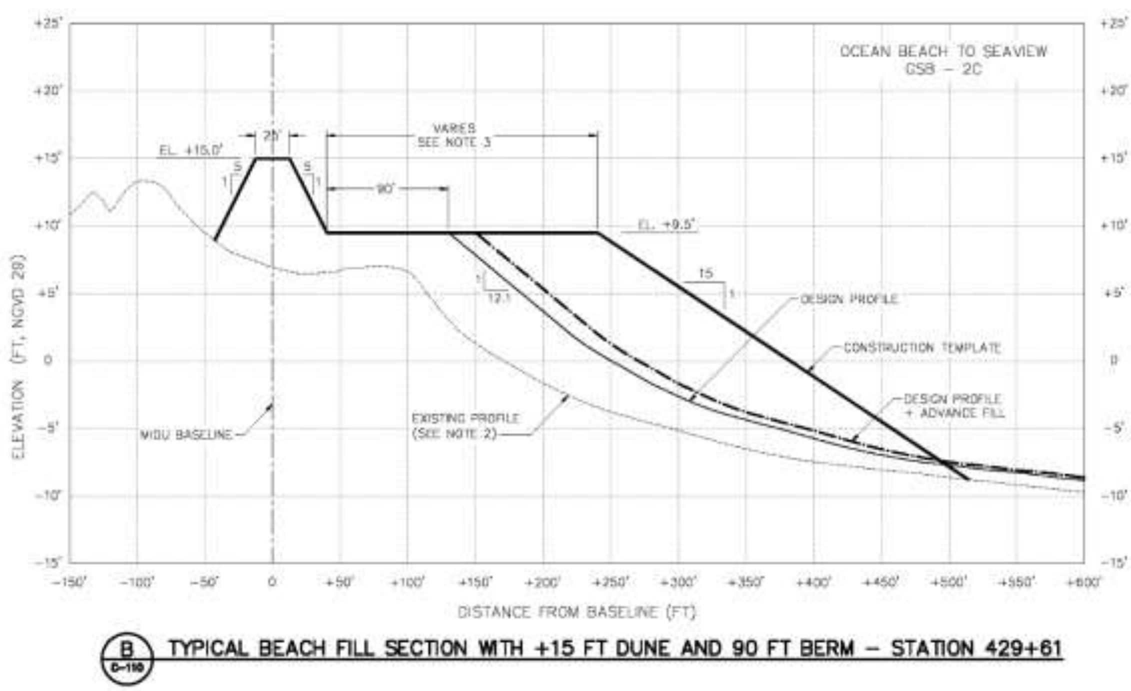


Figure 9-5. Dune and Beach Fill Design Profile (notes are included in the Plates Appendix)

9.5.1 Beach Fill Plan – Initial Construction

With the exception of Cupsogue, all of the beach fill design reaches have been recently constructed or are soon to be under construction as part of the Fire Island to Moriches Inlet (FIMI) Stabilization Project or Westhampton Interim Project. Therefore, it is not possible to use the existing beach conditions to estimate initial construction beach fill volumes at the start of the FIMP project. Instead, initial beach fill volumes were estimated based on predicted sediment losses following the completion of the FIMI and Westhampton Interim projects. The representative erosion rates are also used here to estimate initial construction volumes.

It is noted that advance fill was included in the design and construction of FIMI and the Westhampton Interim Project. Therefore, by restoring sediment losses the initial construction estimates for FIMP indirectly include advance fill. All beach fill quantity estimates include advance fill, 15% overfill, and 15% for contingency/tolerance. A summary of the initial construction quantities for the Beach Fill Plan is shown in Table 9-10.

Table 9-10. Beach Fill Plan Initial Construction Quantities

Location	Subreach	Sediment Source	Fill Length (ft)	Volume (cy)
Kismet to Lonelyville	GSB-2A	2C	8,900	458,367
Town Beach to Corneille Estates	GSB-2B	2C	4,500	192,298
Ocean Beach to Seaview	GSB-2C	2C	3,800	159,307
OBP to POW	GSB-2D	2C	7,300	163,490
Cherry Grove	GSB-3A	2H	3,400	30,294
Fire Island Pines	GSB-3C	2H	7,000	314,377
Water Island	GSB-3E	2H	1,600	23,129
Davis Park	GSB-3G	2H	5,000	240,816
Subtotal				**1,582,000**
Cupsogue	MB-2C	4C	2,000	156,429
Pikes	MB-2D	4C	9,600	232,417
Westhampton	MB-2E	4C	10,900	175,508
Subtotal				**564,000**
Total				**2,146,000**

Notes: RMSP and SPCP-West are not shown here because the required fill material is coming from inlet dredging. Initial fill along Fire Island (1,582,000 CY) will be deferred to Year 4 and coincide with first renourishment event.

9.5.2 Beach Fill Plan – Year 1 to Year 30

The required renourishment fill volumes have been computed based on representative erosion rates and expected renourishment interval of approximately every 4 years. The representative erosion rates were calculated based on the historical sediment budget, volumetric changes in measured profiles between 1988 and 2012, the performance of recent beach fill projects, and anticipated beach fill spreading. All beach fill quantity estimates include advance fill, 15% overfill, and 15% for contingency/tolerance. A summary of the renourishment quantities for the Beach Fill Plan is provided Table 9-11.

Table 9-11. Beach Fill Plan - Renourishment Quantities Per Operation

Location	Subreach	Sediment Source	Fill Length (ft)	Volume (cy)
Kismet to Lonelyville	GSB-2A	2C	8,900	318,864
Town Beach to Corneille Estates	GSB-2B	2C	4,500	161,935
Ocean Beach to Seaview	GSB-2C	2C	3,800	134,153
OBP to POW	GSB-2D	2C	7,300	261,584
Cherry Grove	GSB-3A	2H	3,400	48,470
Fire Island Pines	GSB-3C	2H	7,000	503,003
Water Island	GSB-3E	2H	2,900	41,118
Davis Park	GSB-3G	2H	5,000	428,117
Subtotal				**1,897,000**
Cupsogue	MB-2C	4C	2,000	71,510
Pikes	MB-2D	4C	9,600	619,779
Westhampton	MB-2E	4C	10,900	468,020
Subtotal				**1,159,000**
Total				**3,057,000**

Notes: RMSP and SPCP-West are not shown here because the required fill material is coming from inlet dredging.

9.6 Sediment Management at Downtown Montauk

Sediment management measures that include a feeder beach will be initiated at Downtown Montauk as summarized in Table 9-12. The construction template is a berm with a variable width at an elevation of +9.5 feet NGVD29. The berm width will be determined based on a fill volume of approximately 400,000 cy every 4 years (450,000 cy initial). A typical section of the sediment management feature is shown in Figure 9-6.

Table 9-12. Sediment Management Fill Volumes at Downton Montauk

Location	Subreach	Sediment Source	Fill Length (ft)	Volume (cy) evevry 4 years
Downtown Montauk	M-1F	BA 8D	6,000	400,000

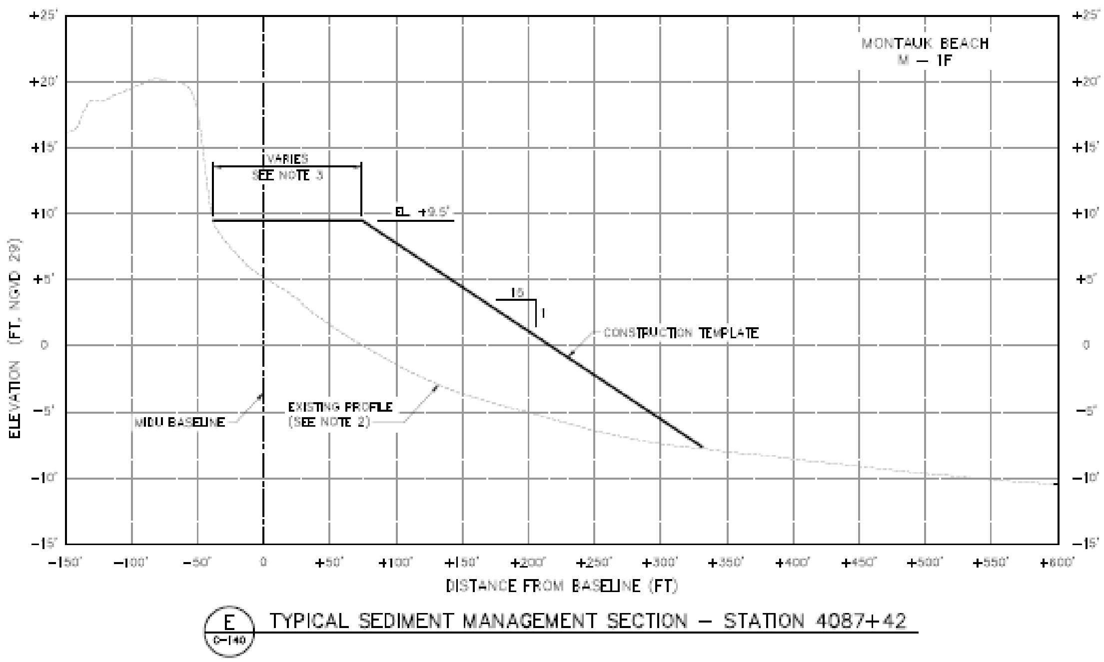


Figure 9-6: Typical Sediment Management Construction Template at Downtown Montauk *(notes are included in the Plates Appendix)*

9.7 Groin Modification Plan

The groin modification plan includes the removal of 2 groins at Ocean Beach. The final requirements for removal will be finalized during the design phase. The GRR cost estimate assumes complete removal of these structures.

9.8 Coastal Processes Features

A key objective of the FIMP project is to restore the natural coastal processes that have been impacted by past development of the barrier island, including: 1) alongshore transport, 2) cross-island transport, 3) dune growth and evolution, 4) bay shoreline processes, and 5) estuarine circulation and water quality. To achieve these objectives and to provide offsets for Endangered Species Act (ESA) impacts and Coastal Storm Risk Management (CSRM) impacts, the project provides for 12 Barrier Island CPFs and 2 Mainland CPFs that are shown in Figure 9-1 and Figure 9-2. A summary of the CPF's are provided in Table 9-13 and a detailed description of the each of the CPF's is provided in Appendix A.

Table 9-13: Description of Coastal Process Features (CPF)

CPF Number	CPF Name	CPF Purpose	CPF Description	Construction Contract	Initial Volume (CY)	Renourish volume (4-year) (CY)
1	Democrat Point West	ESA	Regrade and devegetate; modify pond to improve functionality of existing wetland/create new foraging habitat; conserve on site sand volume.	FI Inlet bypassing	n/a	n/a
2	Democrat Point East	ESA	Regrade and devegetate bay side; modify sand stockpiles to form barrier between recreation and ESA areas; conserve on site sand volume.	FI Inlet bypassing	n/a	n/a
3	Dunefield West of Field 4	ESA	Devegetate ocean side; maintain vegetation buffer with road on north side.	FI Inlet bypassing	n/a	n/a
4	Clam Pond	CSRM	Bay side fill placement to simulate cross island transport; possible living shoreline on north side per adaptive management plan.	Fire Island Renourishment	deferred to Year 4	123,000
5	Atlantique to Corneille	CSRM	Bay side fill placement to simulate cross island transport.	Fire Island Renourishment	deferred to Year 4	162,000
6	Talisman	CSRM	Bay side fill placement to simulate cross island transport.	Fire Island Renourishment	deferred to Year 4	221,000
7	Pattersquash Reach	CSRM/ESA	Devegetate bay side; shallow water bay side fill placement; south boundary follows Burma Rd alignment, includes physical barrier.	Moriches Inlet Bypassing	26,000	15,000
8	New Made Island Reach	CSRM/ESA	Devegetate bay side; shallow water bay side fill placement; south boundary follows Burma Rd alignment, includes physical barrier.	Moriches Inlet Bypassing	133,000	29,000
9	Smith Point County Park Marsh	CSRM	Bay side marsh restoration; fill placement to simulate cross island transport; regrade marsh elevation filling ditches and creating channels for tidal exchange.	Moriches Inlet Bypassing	343,000	18,000
10	Great Gun	ESA	Devegetate ocean side parcel.	Moriches Inlet Bypassing	n/a	n/a
11	Dune Rd Bayside Shoreline	CSRM	Bay side fill placement; bulkhead/groin removal; possible additional fill within offshore channel.	Shinnecock Inlet bypassing / PBRP	66,000	31,000
12	Tiana Bayside Park	CSRM	Bay side fill placement at east side of site; PED will determine fate of existing gabions.	Shinnecock Inlet bypassing / PBRP	48,000	47,000
				TOTAL VOLUME	**616,000**	**425,000**
MB 1	Mastic Beach 1	CSRM	Regrade and vegetate in conjunction with NS acquisition	Non-Structural Contract	n/a	n/a
MB 2	Mastic Beach 2	CSRM	Regrade and vegetate in conjunction with NS acquisition	Non-Structural Contract	n/a	n/a

9.9 Integration of Local Land Use Regulations and Management

The existing Land management regulations and opportunities to improve land management are described in Appendix H- Land Use and summarized below:

- The National Park Service enforces regulations regarding zoning and development within the boundaries of the Fire Island National Seashore and is committed to work with the Towns and Villages on Fire Island to ensure their compliance with the 'Federal Zoning Standards'.

- Before construction of any Corps project for coastal storm risk management (CSRM), the non-federal sponsor must agree to participate in and comply with federal floodplain management.

- Development restrictions exist within the easements for beachfill projects. These are enforceable restrictions. The proposed construction of the CSRM features, including a beach and dune will require the acquisition of permanent easements along the shorefront. These easements preclude future development on lands within the beach and dune footprint. These easements would be enforced by state and local authorities to ensure no development within the easements.

- Additionally, within the study area there are existing land use regulations to address building and rebuilding in the high hazard areas along the coast. State and local agencies have authority to restrict development within shoreline areas through zoning or special district restrictions. Efforts should be made to ensure that these zoning overlays are consistent in their geographic applicability.

- While USACE has no authority to enforce other entities' laws and regulations it does have authority to enforce FIMP project agreements, easements and other project elements. In addition, the Inspection of Completed works program provides a mechanism for monitoring and reporting of any new development within the project area to the appropriate federal, state, and local entities responsible for enforcing applicable land use regulations.

10.0 OPERATION, MAINTENANCE, AND MONITORING

10.1 Operations and Maintenance

A complete Operation, Maintenance, Repair, Replacement and Rehabilitation (OMRR&R) Manual was developed for the FIMP area and is included in Appendix E. This manual outlines the responsibilities of the non-Federal sponsor (State of New York) under the Project Cooperation Agreement (PCA) to ensure the project is maintained to perform during extreme events. Specifically, the FIMP OMRR&R outlines requirements for maintaining dunes, beaches and groins. It also outlines the expectations for periodic inspections and beach monitoring.

10.2 Monitoring

A complete description of the proposed monitoring of the FIMP area is included in Appendix D. In general, the purpose of monitoring shore protection projects can be summarized below:

- Measure project performance;
- Improve the understanding of the physical processes at work and their interaction with project performance; and
- Plan the timing and volumetric requirements of renourishment and any other required maintenance or mitigation measures.

The Physical Monitoring Plan recommends inspection, measurement and analysis of the following physical phenomena and coastal processes within the project boundary and project life:

a. General:
- Periodic site inspection of shoreline condition and structure functionality;
- Aerial photography;
- Shoreline changes and sediment budget update;
- Ocean wave height, period and direction;
- Water level measurement;
- Borrow area infilling;

b. Beach Fill:
- Beachfill/dune profile evolution;
- Sediment sample collection and analysis;
- Post-placement fill characterization;
- Fill compatibility analysis for each renourishment;

c. Inlet Management:
- Inlet morphology evolution;
- Ebb/Flood shoal evolution;
- Deposition basin in-filling rate;

d. Groin Modification:

- Shoreline and dune evolution including one mile both updrift and downdrift;
- Volume changes;
- Regional sediment budget;

e. Breach Response Plan:
- Storm, overwash and breach impacts;
- Cross-sectional volume;

f. Sediment Transport Modeling:
- Inner-shelf bathymetric changes;
- Sub aerial morphologic change;
- Wave, current, bed load and suspended sediment concentration measurements;
- Sediment transport modeling between the inner shelf and western Fire Island;

11.0 REFERENCES

Bates, R.L., and Jackson, J.A. (ed) 1987. Glossary of geology, 3rd Ed. American Geological Institute, Alexandria, VA.788p.Cardone, V. J., H. C. Graber, R. E. Jensen, S. Hasselmann, M. J. Caruso, 1995. In search of the true surface wind field in SWADE IOP-1: Ocean wave modelling perspective. The Global Atmosphere and Ocean System, 3, 107-150.

Gravens, M. B., Rosati, J. D., and Wise, R. A., 1999. "Fire Island Inlet to Montauk Point reformulation study (FIMP): Historical and existing condition coastal processes assessment," prepared for the U.S. Army Engineer District, New York

Holthuijsen, L.H., Booij, N. and T.H.C Herbers, 1989: A prediction model for stationary, short-crested waves in shallow water with ambient currents. Coastal Engineering, 13, 23-54.

Hubertz, J. M., 1992: User's guide to the Wave Information Studies (WIS) Wave Model, Version 2.0. WIS Report 27, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hubertz, J. M., Brooks, R. M., Brandon, W. A., and Tracy, B. A. 1993. "Hindcast Wave Information for the U.S. Atlantic Coast," WIS Report 30, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Intergovernmental Panel on Climate Change (IPCC), 2013: Climate Change 2013: The Physical Science Basis. Contribution of Working Group I to the Fifth Assessment Report of the Intergovernmental Panel on Climate Change [Stocker, T.F., D. Qin, G.-K. Plattner, M. Tignor, S.K. Allen, J. Boschung, A. Nauels, Y. Xia, V. Bex and P.M. Midgley (eds.)]. Cambridge University Press, Cambridge, United Kingdom and New York, NY, USA, 1535 pp.

Kraus and Wise, 1993. Simulation of January 4, 1992 Storm Erosion at Ocean City, Maryland, Shore and Beach, 61(1), 34-31.

Larson and Kraus, 1989. SBEACH: Numerical Model for Simulating Storm-Induced Beach Change; Report 1, Empirical Foundation and Model Development. TR CERC-89-9, USACE.

Larson, Kraus, and Byrnes, 1990. SBEACH: Numerical Model for Simulating Storm-Induced Beach Change; Report 2, Numerical Formulation and Model Tests, RT CERC-89-9, USACE.

Leatherman, S.P., 1981. Barrier Beach Development: A Perspective on the Problem. SHORE AND BEACH, V. 49, p. 2-9.

Lentz, E.E, Hapke, C.J., Stockdon, H.F., and Hehre, R.E., 2013. "Improving Understanding of Near-Term Barrier Island Evolution Through Multi-Decadal Assessment of Morpholigical Change". Marine Geology, 337 (2013) 125-139.

Luettich, R.A., Jr., J.J. Westerink, and N.W. Scheffner, 1992, ADCIRC: an advanced three-dimensional circulation model for shelves coasts and estuaries, report 1: theory and methodology of ADCIRC-2DDI and ADCIRC-3DL, Dredging Research Program Technical Report DRP-92-6, U.S. Army Engineers Waterways Experiment Station, Vicksburg, MS, 137p.

Moffatt & Nichol, (2000), "Barrier Island Breach and Overwash Sediment Transport", Final Draft.

Mukai A.Y., J.J. Westerink, R.A. Luettich Jr., and D. Mark, 2002, "Eastcoast 2001: a tidal constituent database for the western North Atlantic, Gulf of Mexico and Caribbean Sea", US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, Technical Report, ERDC/CHL TR-02-24, September 2002, 201p.

National Center for Environmental Prediction, National Oceanic and Atmospheric Administration, NCEP online: www.ncep.noaa.gov

National Ocean Service, National Oceanic and Atmospheric Administration, tides and currents online: tides&currents.noaa.gov/sltrends.

National Research Council, 1987. Responding to changes in sea level: engineering applications, Committee on Engineering Implications of Changes in Relative Mean Sea Level, Marine Board Commission on Engineering and Technical Systems, National Academy Press, Washington, D.C., 644 p.

Powell, M.D. and PG. Black, 1989: The relationship of hurricane reconnaissance flightlevel wind measured by NOAA's oceanic platforms. 6th National Conference on Wind Engineering.

Resio, D.T. and W. Perrie, 1989: Implication of an f--4 equilibrium range for wind-generated waves. Journal of Physical Oceanography, 19(2), 193-204.

Scheffner, N. W., Borgman, L. E., Clasusner, J. E., Edge, B. L., Grace, P. J., Militello, A., and Wise, R. A. 1997. "Users Guide to the Use and Application of the Empirical Simulation Technique," Technical Report CHL-97-00, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Scheffner, N. W., Clausner, J. E., Militello, A.,Borgman, L. E., Edge, B. L., and Grace, P. J. 1999. "Use and Application of the Empirical Simulation Technique: User's Guide," Technical Report CHL-99-21, Engineer Research and Development Center, Corps of Engineers, Vicksburg, MS.

Stevenson, J. C., L. G. Ward, and M. S. Kearney, 1986. Vertical accretion in marshes with varying rates of sea level rise, in Estuarite Variability, E. Wolf ed., Academic Press, New York, p. 241-260.

Swail, V.R. and A.T. Cox, 1999. On the use of NCEP/NCAR reanalysis surface marine wind fields for a long term North Atlantic wave hindcast. J. Atmos. Ocean. Technology.

Taney, N.E. (1961). Geomorphology of the South Shore of Long Island, New York. Beach Erosion Board, T.M. n. 128.

Thompson, E. F. and V. J. Cardone. Practical modeling of hurricane surface wind fields. ASCE J. of Waterway, Port, Coastal and Ocean Engineering. 122, 4, 195-205.

U.S. Army Corps of Engineers, 1958. Atlantic coast of Long Island, NY Fire Island Inlet to Montauk Point coperative beach erosion control and interim hurricane study (survey), U.S. Army Corps of Engineers, New York District.

U.S. Army Corps of Engineers, 1995. "Breach Contingency Plan, Fire Island Inlet to Montauk Point, Long Island, New York, Reformulation Study," U.S. Army Corps of Engineers, New York District.

U.S. Army Corps of Engineers, 2006a. Draft Memorandum. "FIMP Significant Overwash Computation-Summary of Methodology". March 23, 2006, .S. Army Corps of Engineers, New York District.

U.S. Army Corps of Engineers, 2006b. "Draft overwash/breaching volumes/areas". March 22, 2006,U.S. Army Corps of Engineers, New York District.

U.S. Atlantic Coast," WIS Report 30, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.Jarvinen, B.R., Neumann, C.J., and M.A.S. Davis, 1984.: A tropical cyclone data tape for the North Atlantic basin, 1886-1983: Contents, limitations, and uses. NOAA Technical Memorandum NWS NHC 22 (1984).

U.S. Naval Oceanographic Office, 1970. Summary of synoptic meteorological observations (SSMO), Area 006, New York.

Van Rijn, L.C., 1993. Principles of Sediment Transport in Rivers, Estuaries and Coastal Seas. Aqua Publications, Amsterdam.

Wilby, F.B., G.R. Young, C.H. Cunningham, A.C. Lieber, Jr., R.K. Hale, T. Saville and M.P. O'brien, 1939. Inspection of Beaches in Path of the Hurricane of September 21, 1938. Shore and Beach 7:43-47.

Wise and Kraus, 1993. Simulation of Beach Fill Response to Multiple Storms, Ocean City, MD, Proc. Coastal Zone '93, ASCE, 133-147.

Wise, Smith, and Larson, 1996. SBEACH: Numerical Model for Simulating Storm-Induced Beach Change; Report 4, USACE.

12.0 GLOSSARY (IMPORTANT DEFINITIONS)

12.1 Barrier Island Processes

A ***hurricane*** is an intense tropical cyclone in which winds tend to spiral inward toward a core of low pressure, with maximum surface wind velocities that equal or exceed 74 mph (33 m/sec or 64 knots) for several minutes or longer at some points. Tropical storm is the term applied if maximum winds are less than 74 mph. Tropical storms are typically fast moving and compact. Therefore, surge hydrographs peak rapidly, within a few hours, and surge varies along the coast depending on the location of landfall.

A ***Northeaster***, or ***Nor'easter***, is a large-scale storm formed by Artic cold fronts mixing with warm low pressure fronts from the Gulf of Mexico that are pulled up the Northeast coast by the northeast winds. These storms generally occur in fall, winter, and spring. The predominant wind direction during these storms is from the northeast. These storms generally are characterized by widespread area of influence and elevated surge levels lasting over one tidal cycle or more.

The severity of flooding along the mainland shoreline in the FIMP bays (Great South Bay, Moriches Bay and Shinnecock Bay) is a function of open coast storm surge, defined as the rise above normal water level due to wind-induced surface shear stress and/or atmospheric pressure reduction propagation through the inlets, storm surge in the bay (i.e., local wind and pressure effects), and barrier island overwash and breaching. These three effects plus astronomical tides combine to produce the net bay storm stage,, defined as the level of the quasi-steady state water surface above a given datum at a given location. The following definitions were adopted for this study:

Overwash is "(a) a mass of water representing the part of the uprush that runs over the berm crest (or other structure) without flowing directly back to the sea or lake and (b) the flow of water in restricted areas over low parts of barriers or spits, especially during high tides or storms," (Glossary of Geology, American Geological Institute, 1987). Overwash tends to erode or flatten dunes during a storm with an attendant deposition of eroded sediment on the landward side of the barrier island (***washover***). This terminology is commonly used in most of the relevant research in the area of barrier island morphodynamics (e.g., Leatherman, 1981) and in reports of large storm damage available in the literature (e.g., Wilby et al., 1939). More importantly, a similar terminology has been adopted in previous reports and studies relating to FIMP (e.g., USACE, 1995).

Note, however, that engineers and researchers sometimes use the term overwash to refer specifically to the intermittent volume of water that overtops the dune due solely to ***wave runup***, defined as the peak elevation of ***wave uprush*** above still-water level. ***Wave uprush*** consists of two components: super elevation of the mean water level due to wave action (***wave setup***) and fluctuations about that mean (***swash***). This intermittent flow occurs only when the ***total water level*** (tide + storm surge + wave setup) remains below the dune crest elevation. Others use the term ***overtopping*** instead to refer to this intermittent water flow and the term overwash to refer to the sediment transport associated with it. For the purposes of this study the intermittent flow due to runup will be referred to as

overtopping, whereas the continuous flow that occurs after the dune is inundated by setup will be denoted as ***overflow***. Overwash will be used according to the more general definition provided in the previous paragraph, which could include both overtopping and overflow.

The term ***overwash*** (or ***overwash area***) will also be used in this report to denote the resulting storm-induced barrier island response (topographic change) to water moving over the barrier island by overwash and overflow processes (Figure G-12-1). In this report, the term ***overwash*** when referring to storm-induced morphological change will indicate lowering of the barrier island, between its pre-storm elevation and the Mean High Water (MHW) datum. An ***overwash area*** only allows exchange of ocean and bay waters through a portion of the spring tidal cycle. While the formation of a full breach during spring-tide conditions following a storm event is possible, it is much less likely than if the same barrier island location was cut to a lower elevation and during the storm.

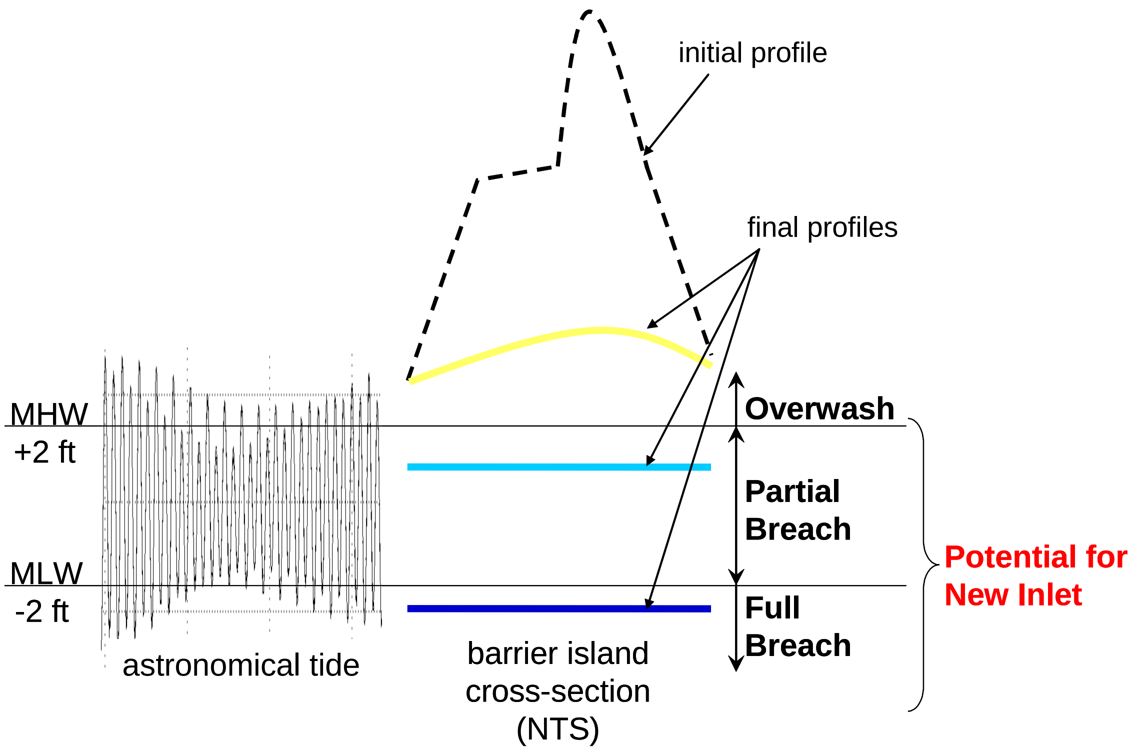


Figure G-12-1. Definition of morphological responses used in this report.

Breaching refers to the condition where overflow cuts a channel across the island that permits the exchange of ocean and bay waters under normal tidal conditions. For this report, two degrees of morphological response to breaching will be used Figure G-12-1). A ***partial breach*** is a storm-induced barrier island cut that has a scoured depth between MHW and Mean Low Water (MLW) while a ***full breach*** is a storm-induced barrier island cut that has a scoured depth at or below Mean Low Water (MLW). A partial breach will allow for water to exchange between the ocean and bay during a portion of the normal tidal cycle while a full breach will allow water exchange during the complete tidal cycle. A partial or full breach may potentially develop into a permanent breach during normal tide conditions following a storm.

Overwashing and breaching are interrelated. For example., severe overwashing can lead to breaching. The breach or overwash area may be temporary or permanent (i.e., a new inlet) depending on the size of the breach, adjacent bay water depths, potential tidal prism, littoral drift, etc.

Overwash, and particularly breaching, during a storm may contribute significantly to the storm stage in the bays and therefore the modeling approach should be capable of simulating these effects as well as open coast surge propagation through the inlets and bay storm surge.

12.2 Vertical Datums

Collected bathymetric and topographic data for the study area were referenced to various different vertical datums including Mean Sea Level (MSL), Mean Low Water (MLW), and National Geodetic Vertical Datum 1929 (NGVD29). For this study, the New York District has adopted feet NGVD29 as the vertical datum for design elements and reporting. However, the hydrodynamic model inputs must be relative to meters in mean sea level (MSL). Therefore, all available data were converted to meters, MSL. For those data sets referenced to MLW, conversions were applied based on the nearest tidal benchmark information developed from long-term water level measurements. These included a number of NOAA tidal benchmark sheets (1960-1978 tidal epoch) nearby and throughout the study area along with several LISHORE measurements offshore and within Shinnecock and Moriches Bays. Generally, available tidal benchmark information within or near the study area does not include vertical reference to NGVD29. Further, the limited information regarding NGVD29-to-MSL conversions show that conversions vary widely throughout the study area: NGVD29 is below MSL by 0.59 ft (0.18 m), 0.50 ft (0.15 m), and 0.75 ft (0.23 m), at Shinnecock, Moriches, and Fire Island Coast Guard Stations, respectively. This presented a challenge for converting measured data referenced to NGVD29 to MSL. For this study, the following conversion between NGVD29 and MSL was adopted:

$$\mathrm{Elevation}_{\mathrm{NGVD29}} = \mathrm{Elevation}_{\mathrm{MSL}} + 0.5\,\mathrm{ft}\,(0.15\,\mathrm{m})$$

This conversion was based upon that used for past New York District studies for the south shore of Long Island and approximates the average of the known conversions within the study area. Fortunately, water level predictions by hydrodynamic models are not overly sensitive to small bathymetric changes (on the order of 0.2 ft (0.1 m)). Therefore, using one conversion for the entire project is expected to have a negligible impact on the final water level simulations.

The conversion given by the equation above is also used to convert simulated peak water levels from MSL to NGVD29 for stage-frequency development and reporting. In this report, all water level comparisons between simulated storm water levels and measured water levels are presented relative to MSL. However, all stage-frequency results and comparisons are presented in NGVD29, as this is the datum required for this study.

The life cycle model, used to compute the damages and economic benefits for the various alternatives, accounts for the uncertainty in inputs such as stage-damage curves, breaching, erosion, sea level rise, timing of storms, etc. (which intrinsically account for the datum conversion uncertainty) by assuming a range of variability for each of these parameters and using Monte Carlo sampling techniques.

12.3 Tidal Constituents

Tidal constituents are components of the astronomic tidal time series computed by performing a harmonic analysis. This analysis decomposes the tide signal into diurnal (K1, O1, Q1, etc.) and semidiurnal (M2, N2, S2, K2, etc.) components, where each component is itself a sine wave defined by amplitude, phase, and speed. The most dominant tidal constituent will have the largest amplitude. In the FIMP area, the largest-amplitude constituent is M2, a semidiurnal constituent.

12.4 Observed and Measured Peak Water Levels

Two types of information exist that document historical storm water levels within the FIMP area. ***High Water Marks (HWM)*** are indirect measurements of high water level. These are namely post-storm observations of the high water line, typically on a permanent structure, and are oftentimes represented by the debris line. The HWM includes the effects of astronomical tide, storm surge, localized wave setup, and the impact of individual waves (including wave runup). Figure G-12-2 illustrates these contributions to the HWM.

Another type of peak water level measurements are ***Water Level Gage (WLG)*** measurements. These are direct measurements of water surface elevation, and they are generally more accurate and more reliable than HWM observations. A peak WLG measurement includes the effects of all quasi-steady state contributions to water level. Specifically, the WLG measures the water level contributions from astronomical tide, storm surge, and localized wave setup (Figure G-12-2). These measurements do not include the effects of individual waves; therefore, they better reflect the quasi-steady state water level conditions experienced during storm events.

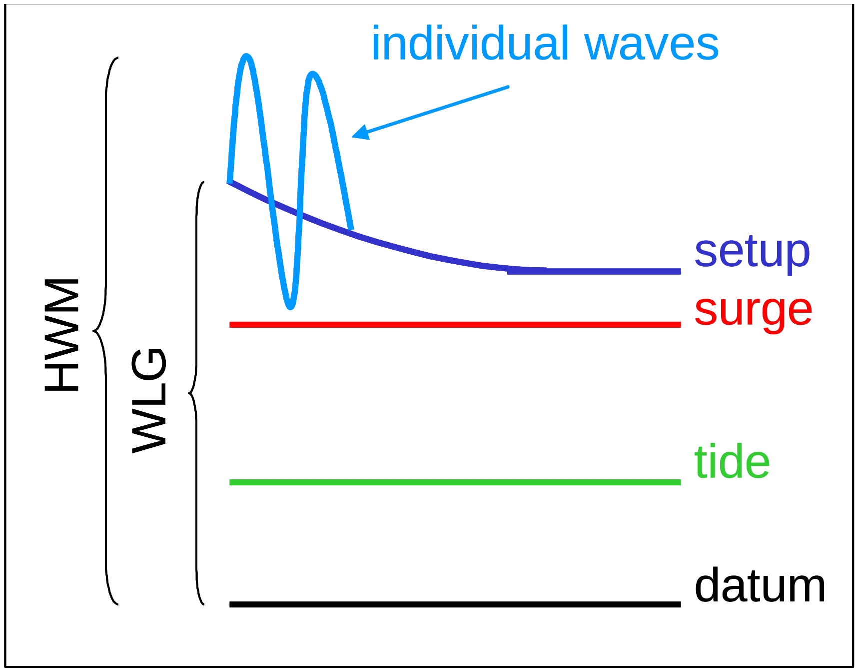


Figure G-12-2. Water level contributions to HWM and WLG peak water level records.