Savannah Harbor Beach Erosion Study

Savannah Harbor Expansion Project

PREPARED BY:

APPLIED TECHNOLOGY AND MANAGEMENT, INC.

CHARLESTON, SC

PREPARED FOR:

GEORGIA PORTS AUTHORITY

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1. INTRODUCTION

Applied Technology and Management, Inc. (ATM) was retained by the Georgia Ports Authority (GPA) to evaluate the coastal shoreline impacts of the proposed Savannah Harbor Expansion Project. The project is proposed to deepen the Federal Navigation Channel as shown in Figure 1-1. The deepening of the portion of the channel that extends from Fort Pulaski to the offshore limit (known as the Bar Channel) is of particular concern with regard to coastal processes. The authorized depth for the existing navigation channel is –44 feet mean low water (MLW) for the Bar Channel, which would be deepened by up to 6 feet with the project. The deepening of the Bar Channel will cause changes to the current, wave and sediment transport environment, which may result in impacts to the regional shorelines. This study evaluates these impacts using a combination of historical data analysis, field data collection, current and wave modeling, and sediment budget analysis.

1.1. Study Goals

In order to determine the potential impacts of the deepening project, the study will first establish an accurate description of the existing coastal processes in the study area. An analysis of the historic changes in the system will provide necessary information for establishing the existing conditions. Previous engineering modifications to the inlet system, which must be assessed, include a number of deepening projects, construction of entrance jetties, and construction of a submerged breakwater. Those modifications will be assessed together as a group to allow identification of the cumulative impacts of previous inlet modifications. After the existing (without project) conditions have been established, the study will proceed to assess the potential impacts of the deepening project. The study goals can be summarized as follows:

- Develop Numerical Models – Develop computer models to describe the waves and currents in the study area.

- Historical Analysis - Qualitatively and (to the practical extent) quantitatively define the historical bathymetry and shoreline changes that have occurred in the study area. Include the development of the federal navigation project and the construction of the jetties at the mouth of the Savannah River. Qualitatively assess changes in mechanisms that control shoreline accretion/erosion: incident wave energy, currents, sediment supply, and shoreline hardening (e.g., seawalls, groin fields, etc.).

- Establish Existing Conditions – Use the computer models to describe the existing conditions that influence sediment transport in the area, including incident wave energy and currents. Use the modeling results and historic change analysis to develop a sediment budget for the study area.

- Determine Projected Deepening Impacts - Use the computer models to determine effects to the local wave and current conditions caused by the proposed channel deepening. Utilizing the projected effects to the wave and current conditions, quantify the potential impact of the proposed channel deepening. Any effects of the proposed deepening on the nearshore and inlet sediment budget will also be identified.

1.2. Report Outline

This report will present the study methodology, analysis and results in the following sections:

- Section 2.0, Jetty and Bar Channel History
- Section 3.0, Project Environment
- Section 4.0, - Shoreline and Bathymetry Morphology
- Section 5.0, - Current Data Collection
- Section 6.0, - Hydrodynamic Modeling
- Section 7.0, - Wave Modeling
- Section 8.0, - Sediment Transport
- Section 9.0, - Conclusions

2. JETTY AND BAR CHANNEL HISTORY

2.1. Project Description

The Bar Channel is the portion of the Federal Navigation Channel that extends from the jetties out across the Tybee Knoll to the seaward end of the navigation channel. Maintenance dredging of the Bar Channel and the construction of the jetties are two elements of the navigation project that significantly altered the coastal processes in the area. This section provides a brief history of this part of the navigation project.

In the early 1700s, the lower portion of Savannah River contained a number of islands around and between which the river flowed in shallow channels to the sea. Rhodes (1949) found that the first ship to navigate the river was the JAMES, which arrived in Savannah in 1733 and found close to the town a depth of 15 feet at low water in which to anchor. The controlling depth across the bar at that time was about 13 feet at low water. In 1773, Sir James Wright reported 21 feet at low water on the bar. Black (1893) states that prior to 1763, the depth on Tybee Knoll (located 1/2 mile east of Fort Pulaski) was 15 feet, and a good, clear channel was reported as far as Savannah. Inspection of the 1855 survey conducted by the U.S. Coast Survey shows controlling depths of 15 feet MLW over Tybee Knoll and 13 feet near Oyster Bed Island. A sketch of the harbor configuration in 1855 is shown in Figure 2-1.

The major efforts to deepen the harbor began in 1873 when General Gilmore submitted a project for the formation of a channel, 22 feet deep at Mean High Water (MHW) (i.e., 15.5 feet MLW) from Savannah to the sea. This was to be accomplished by building a dam at the Cross Tides (a natural channel between Hutchinson Island and Argyle Island located just south of the existing Kings Island Turning Basin), by widening the waterway opposite the city and by dredging the lower reaches. In 1874, Congress authorized a 22-foot deep project based on this plan. The construction of the dam at the Cross Tides was completed in 1881, and the effect of the dam on Front River was immediate. The mean bottom velocity was raised from 1.75 to 2.37 feet per second, and between 1879 and 1889 the cross-sectional areas and the mean depth increased 20 percent (Black, 1893). Additionally, Black (1893) reports that the difference in stage on the two sides of the dam amounted to about 3 feet, and caused a 75 million cubic foot reduction in the tidal prism of the Front River.

This project was successfully modified and enlarged, and work under it continued until 1890. The annual report of the Chief of Engineers for 1888 states that the project for 22 feet at high water had been practically accomplished, that in Front River below Cross Tides there was a channel from 14 to 15 feet deep at low water, and in the river below and across the bar the depths ranged from 13 to 13.5 feet at low water.

In 1890 Captain Carter submitted a plan for a 26-foot MHW channel from Tybee Roads to Savannah. In its unaltered condition, a significant portion of the Savannah River tidal prism flowed between Oyster Bed Island and Turtle Island (the flow north of Oyster Bed Island, which is augmented by Wright River, was approximately half that of the flow to the south of Oyster Bed Island [Black, 1893]). The 1890 plan included authority for the district engineer to restrict the flows at the mouth of the river to a channel south of Oyster Bed Island by closing the gap between this island and Turtle island, or by closing the gap between this island and Jones Island, as was subsequently done in 1931 (Rhodes, 1949). The plan included dredging of the channel south of Oyster Bed Island (in the present location). The plan also provided for the construction of training walls extending eastward from Cockspur Island and Oyster Bed Island (now referred to as jetties). These structures were completed by 1896. The map in Figure 2-2 shows the general layout of the channels and the general depths therein at that time and the control works that had been constructed.

The construction of the jetties had consequences beyond the maintenance of the navigation channel. The jetties redirected the river entrance flows, which in turn, caused modifications to the shoal formations and adjacent shorelines that were once in some form of dynamic equilibrium with the natural inlet configuration. Figure 2-2 shows some of this effect in the reconfiguration of the north end of Tybee Island in the short time between the two surveys. The effects of the jetties on the morphological and shoreline changes are examined further in Section 4.

In the 1894 Rivers and Harbors Act, Congress instructed the Secretary of War to submit a report as to whether the works then under way in Savannah would afford safe anchorage for vessels lying in Tybee Roads. If such a safe anchorage could not be provided by the project as modified in 1890, the Engineer in Charge was to report on any changes needed in the construction. Captain Carter's 1894 report concluded that additional works would not only afford a safe anchorage for vessels in Tybee Roads, but would be of great value in developing and maintaining the channels across the Knoll and the ocean bar. He recommended building an 800-foot detached breakwater on the shoal separating Tybee Roads and Calibogue Sound to the north. Congress approved his plan and the work was funded under the Rivers and Harbors Act of 1896 (Barber and Gann, 1989). A U.S. Army Corps of Engineers (USACE) Savannah District chart titled "Mouth of Savannah River Training & Protection Works" showing works through 1937 has the breakwater listed as "Gainor's Bank Submerged Jetty, 1897." The affects of the submerged jetty on the morphology of the ebb shoal system is examined in Section 4.2. The present condition of the submerged jetty is unknown.

In June 1902, Congress adopted a project for a channel from just above the center of the city to the ocean, 28 feet deep at mean high water (i.e., 21.5 feet MLW). The River and Harbor Act of March 3, 1905 called for a re-survey of the harbor with plans and estimates of cost, with a view to securing a channel to the sea 26 feet deep at mean low water. Congress adopted this plan in 1912.

The Rivers and Harbor Act of August 30, 1935 provided for a 30-foot MLW Bar Channel out to the 30-foot contour in the ocean. The Rivers and Harbor Act of November 7, 1945 increased the Bar Channel depth to 36 feet. On October 27, 1965, the project was authorized to deepen the Bar Channel from 36 feet to 40 feet, and the Bar Channel was widened from 500 feet to 600 feet. WRDA 1992 authorized a Bar Channel depth of 44 feet MLW. The deepening was completed in April 1994.

The present federal navigation project includes the following channels: 44 feet deep and 600 feet wide across the ocean bar, a distance of about 7 miles; 42 feet deep and 500 feet wide to the Georgia Ports Authority Terminal in Garden City, Georgia; 36 feet deep and 400 feet wide to the vicinity of the Savannah Sugar Refinery of Savannah Foods and Industries, Inc., a distance of about 22.6 miles; 30 feet deep and 200 feet wide to a point 1,500 feet below the Atlantic Coastal Highway Bridge, about 1.5 miles. The total length of the Federal Project is 31.1 miles. The present harbor configuration is shown in Figure 1-1.

2.2. Dredge Volumes

Early dredging was mostly focused on removal of shoals in the inner harbor. However, this section summarizes the available information regarding dredging of the bar channel from the jetties to the sea.

In 1854 the bar at the entrance of Savannah was considered one of the best along the southern coast, having a depth of 19.5 feet MLW, and the "sea-bar" required no improvement

(Granger, 1968). The first obstruction in the navigation channel at that time was two and a half miles above Tybee Point (one half mile off Fort Pulaski), which was referred to as the "Knoll." The USACE annual report of the Chief of Engineers indicates that significant dredging in this area began in 1873. The sediments dredged from this area were almost entirely pure silica sand (Granger, 1968).

The preliminary survey of 1887 had established the depth of the outer bar as 26 feet MHW. After considering a 28-foot MHW project, a 26-foot MHW project was submitted by Captain Carter in 1890 that would avoid the necessity of dredging the outer bar. The annual reports indicate that the first significant dredging of the bar channel occurred in 1910 when 1,640,000 CY were dredged from the ocean bar, Tybee Roads, Tybee Knoll and near Quarantine Station (Oyster Bed Island). In the 1910 annual report it is stated that there was a channel 26 feet deep at MLW through the bar and Tybee Roads and 24 feet deep up to Fort Pulaski, and that above that point the old project depth of 28 feet MHW had been maintained. In 1915, 667,000 CY of maintenance dredging was done on the bar channel (Rhodes, 1949).

Dredge volumes for the bar channel obtained from the USACE Savannah District from 1920 through 1998 are tabulated in Table 2-1. The table includes maintenance dredge volumes (dredging required to maintain the authorized channel depths) and new work material (dredging of undisturbed sediments for channel enlargement or realignment). The data sources include annual reports, dredge records, log sheets, and the USACE dredging database. The period between 1929 and 1945 does not provide any data about the bar channel since these dredge quantities were grouped together with the inner harbor dredge quantities in the annual reports. Some of the sources do not compare well; Wilbur Wiggins of the USACE Savannah District (personal communication, December 12, 1999) recommended that the most reliable maintenance dredging data include the annual reports for 1921 to 1928 and 1951 to 1965 and the dredge database between 1974 and 1998.

The maintenance dredge volumes should indicate the amount of sedimentation in the navigation channel over time. Figure 2-3 is a plot of the reliable maintenance dredging data, the authorized channel depth, and new work dredging volumes versus time. The plot illustrates that the 26-foot project resulted in approximately 500,000 cubic yards of material accumulating in the bar channel annually. Data is not available for the period following the deepening to the 30-foot project. Following the increase in project depth to 36 feet, the average sedimentation rate in the bar channel increased to approximately 700,000 cubic yards per year. The 40-foot project was authorized in 1965, but the dredge records show that the new work material was not dredged until 1972, following which the maintenance dredge volumes jumped to as high as 2.8 million cubic yards. However, within a few years following the new work dredging, the maintenance volumes reduced to rates similar to the 1950s. The maintenance dredging jumped again following the 1994 deepening and quickly returned to the pre-deepening levels.

The data indicate that maintenance dredging increases substantially for only a year or two following new work dredging, most of which is the result of side slope adjustment along the channel. Following the period of adjustment of the channel side slopes after deepening work, the maintenance dredge volumes can be considered an approximation of the littoral drift intercepted by the channel. However, because of the variability in the interval between maintenance dredging and the variability in the maintenance dredging volumes, Figure 2-3 does not provide a good picture of the average annual sedimentation volumes.

A better method of studying maintenance dredging values is to integrate the dredging quantity over time. The resulting plot is cumulative dredged volume versus time (Figure 2-4). The slope of the cumulative dredging curve represents the average rate of dredging (and shoaling) in the bar channel. The slope was computed for each channel depth, assuming enough data was available. It must be noted in this plot that for the years with more than one estimate, the more reliable estimates established by Wilbur Wiggins of the USACE Savannah District (personal communication, December 12, 1999) were used. Also, years 1977 to 1979 were conservatively estimated at 1,000,000 CY. The dredge volumes reported for 1977 to 1979 are all above 2,000,000 CY; however, these volumes are anomalous and are assumed to result from construction of the tide gate. The 21-year annual average maintenance estimate for the 40-foot depth was 700,000 CY excluding these three years.

The cumulative dredging plot indicates that a rate of approximately 700,000 CY/yr has been consistent since the bar channel was deepened to the 36-foot depth. This supports the idea that the Savannah River Channel is a sand sink where no sediment naturally bypasses. Therefore, dredging volumes should not increase significantly for the entrance channel between Station 0+000 and -60+000. Some increase in dredging volume could result from the extension of the channel from Station -60+000 to -85+000. However, based on sedimentation patterns in the bar channel, the sedimentation in this additional offshore channel length is expected to be comparatively small.

It must also be noted that the dredging records used for this study were from station 0+000 to –60+000 and that approximately 100,000 CY/yr is dredged between stations 0+000 and 24+000 which borders Jones and Oysterbed Islands (USACE, 2000). Therefore, including Jones and Oysterbed Island reaches, the navigation channel represents a littoral sink of approximately 810,000 CY/yr.

Sediment samples collected by Dial Cordy (2001) between stations 0+000 and 24+000 are 85% sand (based on an average of 74 grab samples). Additionally, sediment samples collected by Dial Cordy (2001) between stations 0+000 and -60+000 are 76% sand (based on an average of 180 grab samples). Therefore, 85,000 CY/yr of sand (i.e., 85 percent of 100,000 CY/yr) is deposited in the Savannah River Channel between stations 24+000 to 0+000. 540,000 CY/yr of sand (i.e., 76 percent of 710,000 CY/yr) is deposited in the Savannah River Channel between stations 0+000 to –60+000.

Section 4 of this report analyzes subsequent surveys of the bar channel between the after dredge (AD) surveys in 1999 and the before dredge (BD) of 2000. The analysis found a sedimentation rate of 1,152,000 cubic yards for this period: more than twice the maintenance dredging reported for the previous year. However, two hurricanes passed close to Savannah in 1999 (Dennis and Floyd). The elevated waves associated with the storms undoubtedly increased the littoral transport into the channel.

The maintenance dredging and sedimentation rate data from 1998 to 2000 illustrate the variability in sedimentation rates of the bar channel. The rate is dependent on environmental conditions (such as river flooding and wave climate) and a range of sedimentation rates may occur for any given year.

For comparison, the historical dredge rates for the inner are shown in Table 2-3. The data show that the quantity of sediments required to be dredged from the harbor each year has remained near 7 MCY over the last forty years.

Table 2-1 Bar channel dredge volumes

Channel Depth (ft)	Year	By Annual Report		By Dredging Records		By Database		By Log Sheet	
		Mainten-ance (CY)	New Work (CY)	Mainten-ance (CY)	New Work (CY)	Mainten-ance (CY)	New Work (CY)	Mainten-ance (CY)	New Work (CY)
26	1921	565000							
26	1922	156690							
26	1923	270200							
26	1924	1142197							
26	1925	322794							
26	1926	502244							
26	1927	217000							
26	1928	716727							
26	1929	Grouped Qty							
30	1930	Grouped Qty	2470490						
30	1931	Grouped Qty							
30	1932	Grouped Qty							
30	1933	Grouped Qty							
30	1934	Grouped Qty							
30	1935	Grouped Qty							
30	1936	Grouped Qty							
30	1937	Grouped Qty							
30	1938	Grouped Qty							
30	1939	Grouped Qty							
30	1940	Grouped Qty							
30	1941	Grouped Qty							
30	1942	Grouped Qty							
30	1943	Grouped Qty							
30	1945	Grouped Qty							
30	1946	2380977							
30	1947	695709							
30	1948		671350						
30	1950		2830694						
36	1951	1589329		2864450					
36	1953	916454		916454					
36	1954	504022		667330					
36	1956	450636							
36	1957	1826336		2439914					
36	1958	95382		202234					
36	1959	66752							
36	1961	1368231		1219623					
36	1962	1231648		1414182					
36	1963	1017490		1339289					
36	1964	645095		903051					

(continued)

Table 2-1 Bar channel dredge volumes (continued)

Channel Depth (ft)	Year	By Annual Report		By Dredging Records		By Database		By Log Sheet	
		Mainten-ance (CY)	New Work (CY)	Mainten-ance (CY)	New Work (CY)	Mainten-ance (CY)	New Work (CY)	Mainten-ance (CY)	New Work (CY)
36	1965	504254		655518					
40	1966	711046		879518					
40	1968			458430					
40	1969			401814					
40	1970			677949					
40	1971			582442					
40	1972	304405	2667987	489687	3469633				
40	1973	648144	2151664	771923	2151664				
40	1974	96503	1146262	1207598		1415731			
40	1975					96503	1146262		
40	1976	1066024	979235			1066024	979235		
40	1977	494234	976161			2811201	1806359	2811201	1806359
40	1978	4117355	988500			2763715			
40	1979	1863477						2140873	
40	1980	741064				400282		471064	
40	1981	865736				865736		865736	
40	1982	188266				188266		188266	
40	1983	644940				644940			
40	1984	789754				490224			
40	1985	1212478							
40	1986					1166528			
40	1989					442414			
40	1990	600000							
40	1991	1104991				1104991			
40	1993	555000				554707			
44	1994						2454441		
44	1995					1993061			
44	1996					486108			
44	1997					544508			
44	1998					548044			
44	1999					508885			
44	2000					1217331			
44	2001					720006			

Source: Wilbur Wiggins, USACE Savannah District, December 1999. Volumes for 1999, 2000 and 2001 updated August 2002.

Notes: (1) Shaded cells indicate "more reliable" reports, as indicated by Wilbur Wiggins, USACE Savannah District, December 1999.

(2) Grouped Qty indicates that the dredge volume recorded included both inner harbor and entrance channel quantities.

Table 2-2b 2001 Dial Cordy percent sand estimates by channel mile

Location	Percent Sand Fraction	Average Grain Size Sand Fraction (mm)	Median Grain Size (mm)
20+000 to 27+000	88.9	0.46	0.44
10+000 to 19+000	83.4	0.69	0.66
0+000 to 9+000	84.1	0.70	0.70
-1+000 to -10+000	79.6	0.40	0.39
-11+000 to -20+000	78.1	0.69	0.69
-21+000 to -30+000	75.7	0.26	0.25
-31+000 to -40+000	69.2	0.12	0.11
-41+000 to -50+000	63.2	0.20	0.29
-51+000 to -60+000	91.4	0.58	0.59
TOTAL	79.1	0.45	0.46

Table 2-3 Inner harbor historical shoal volumes (USACE, 1996)

Time Period	Channel Depth (feet below MLW)	Annual shoaling rate (MCY/yr)
1923 to 1925	26	2.8
1931 to 1932	26	4.3
1939 to 1944	30	6.2
1953 to 1954	34	7.2
1953 to 1962	36	7.3
1972 thru 1981	38	7.2
FY91 thru FY93	38	6.5

3. PROJECT ENVIRONMENT

3.1. Geologic Setting

The study area of the Beach Erosion Study is composed of a complex barrier island, sea island, and near shore deltaic and ebb shoal featured system (Figures 1-1, 2-1 and 3-1). The Savannah River has had a major, but not singular influence on the development of the features found in the project area. The dynamics of sea level fluctuation, wind and wave processes, sediment characteristics and supply as well as tidal effects have each contributed to the modern geomorphology of this area of the Atlantic Coastal Plain.

The study area is classified as a mesotidal region (i.e., tidal ranges between 6 and 12 feet), with tidal fluctuations averaging 6.8 feet at the mouth of the harbor (USACE, 1996). Mesotidal regions are generally much more irregular than most microtidal shorelines, because higher tides support more tidal inlets, marshes and tidal flats. Since the study area has a large tidal range and is subjected to relatively low wave energy conditions, tidal current processes predominate over wave processes. The large sounds and extensive shoal systems extending perpendicular to the shoreline are features created by tidal dominated processes.

SAVANNAH HARBOR

Southeast Georgia is located in the Atlantic Coastal Plain Physiographic Province at the head of the Georgia Bight. The province is separated into three geomorphologic areas consisting of the Upper, Middle, and Lower Coastal Plain. Savannah Harbor lies in the Lower Coastal Plain where sediments generally consist of varying combinations of unconsolidated to partially consolidated sediments. Savannah Harbor also lies within an expansive Holocene fresh and salt marsh estuarine system that is typical of other estuaries found along the Southeastern Atlantic Coast.

The estuarine system of Savannah developed around a central feature, the Savannah River. Between 300 and 600 thousand metric tons of sediment annually has been transported down the Savannah River since 1950 (Meade, 1976). In the early part of the 20th century the Savannah River was probably transporting close to a million metric tons of sediment annually due to indiscriminant forestry and agriculture practices (Griffin et al., 1984). The reasons for the decrease in sediment probably came from initiation of soil conservation techniques and the construction of reservoirs in the Piedmont.

Sediment cores from previous deepening in the Savannah Harbor have indicated historical fluvial erosion by Savannah River into progressively older sediments. The sediments encountered in and around the Savannah Harbor appear to range from Holocene to late Miocene in age.

The younger (post-Miocene) sediments are made up of detrital alluvial and beach deposits and reworked Miocene sediments. The post-Miocene sediments are found at the surface throughout the area and are estimated to extend to a depth greater than 45 ft below land surface. Combinations of clays, silts, sands and gravels of Holocene age comprise the most recently deposited material. Holocene material was derived from the Piedmont and upper portions of the Coastal Plain then deposited during the formation of the flood plain of the Savannah River.

The Savannah Harbor area is further underlain with semi-consolidated to dense limestone and dolomite to depths on the order of 4000 to 5000 feet. They range in age from late Cretaceous (approximately 100 million years old) to Quaternary. The geology beneath the Cretaceous sediments consists of sedimentary and volcanic rocks of Triassic age to early Jurassic age (approximately 230 million years old to about 170 million years old, respectively). The crystalline basement rocks of the Paleozoic age make up the remaining underlying geologic structure (USACE, 1986).

HILTON HEAD ISLAND

Hilton Head Island is located 6 miles north of the Savannah River entrance. Hilton Head Island is a 13-mile long beach ridge-type barrier island formed of a series of prograding beach ridges (London et al., 1981). As typical of barrier islands in the mesotidal regime, a Holocene estuarine salt marsh developed landward of and between Hilton Head's ancient beach ridges.

Bruun (1977) states that the wide and shallow shelf seaward of Hilton Head Island, Gaskin Banks, was undoubtedly emerged above sea level at some period in the past. However, it did not face the ocean with a continuous shoreline but it was penetrated by tidal and river entrances. In a letter report to Olsen Associates, Inc., Vernon Henry hypothesized that Hilton Head Island was once fronted by both a relic barrier island formation and a back-barrier lagoon (Olsen Associates, Inc. 1986). Subsequent landward movement of the seaward barrier resulted in the entrapment of substantial lagoonal deposits by a relatively thin veneer of Holocene sediments in the area immediately offshore of Hilton Head.

Historic erosion trends since the late 1800's indicate a general erosion of the northernmost half of the island and the south end has experienced a small degree of accretion (London et al., 1981; Anders et al., 1990). Zarilllo et al. (1985) show that the southwest end of Hilton Head extended by approximately 1000 feet between 1898 and 1977. Sand supply for southwest growth is most likely due to erosion along the northern half of Hilton Head. Recent trends show erosion of the extreme southern end of the island as a result of Calibogue Sound's effects on the accretional spit at the extreme south end of the island (Olsen Associates, Inc., 1994).

CALIBOGUE SOUND

Calibogue Sound is the southernmost major inlet or embayment on the South Carolina coast. The drainage system of Calibogue Sound is completely tidal and serves the back-barrier areas of Daufuskie Island and a portion of Hilton Head Island. Calibogue Sound is connected to Port Royal Sound via two large tidal creeks at its headward end and to the Savannah River by the Cooper River, which enters the Sound on the west side.

The main body of Calibogue Sound consists of a single, deeply scoured tidal channel that exceeds 60 feet in depth in several places. It is likely that the deep tidal channel of the Sound is anchored in resistant Tertiary beds similar to Port Royal Sound, accounting for its long-term stability (Zarilllo et al., 1985).

DAUFUSKIE ISLAND

Daufuskie Island is defined as a sea island in a study by M. O. Hayes et al. (1980). Like Hilton Head, Daufuskie Island is also separated from the mainland by an expansive Holocene salt marsh and estuarine system. Until a recent beach fill project, the shoreline sediments consisted of a relatively thin veneer of fine sand underlain by semiconsolidated mud sediments. Erosion of the shoreline had resulted in a narrow, sandy high tide beach. Remains of old trees are numerous along the entire shoreline, and a substantial formation of fossilized peat is situated at the seaward point near the center of the island shoreline (Applied Technology and Management, Inc., 1996). The seaward side of the island is now covered by approximately 1.4 million cubic yards of sand from a privately sponsored beach nourishment project conducted in late 1998.

Historical erosion trends (i.e., 1860 to 1993) indicate general erosion along most of the seaward shoreline. The average shoreline movement for the entire island from the 1850s to 1983 is -7.4 feet per year (Hansen et al., 1987). Erosion rates following the 1998 beach fill project are expected to be much lower since the beach sand will serve as a protective barrier against erosion of the underlying mud sediments.

Turtle Island

Turtle Island is a landform with low relief bounded by the New River to the north and the Wright River to the south. Due to Turtle Island's low relief, it is commonly submerged in storm events evidenced by many washover fans and dominant high salt marsh vegetation. Turtle Island is most likely a Holocene deposit that was created with the development of the Savannah River deltaic complex. The average shoreline movement for the entire island from the 1850s to 1983 is -1.9 feet per year (Hansen et al., 1987).

Oyster Bed and Cockspur Islands

Oysterbed and Cockspur Islands are also part of the Savannah River deltaic complex. These landforms parallel the Savannah River near its mouth. Both of these islands likely developed as levee-flood plain deposits and were drowned to some degree as sea level rose to its present elevation. Early maps show Oysterbed as an intertidal shoal feature. Construction of training walls and placement of dredge spoil enlarged this island and eventually joined it to Jones Island.

Tybee and Little Tybee Islands

Tybee and Little Tybee Islands represent the northernmost barrier islands of the Georgia Coast. Although these islands are separated by Tybee Creek, together they comprise the cuspate foreland of the Holocene Savannah River delta. Tybee and Little Tybee Islands are products of the development of the deltaic barrier island complex in response to the influence of local coastal processes and sediment supplied primarily by the Savannah River and to a minor degree by littoral drift from the north.

As typical of the Georgia coast, these barrier islands are separated from the mainland by an extensive Holocene salt marsh and estuarine system. Seaward of this salt marsh, Tybee and Little Tybee Islands developed with well-defined beach and dune ridge complexes without obvious soil zones which further distinguish these islands as Holocene barrier island deposits. Tybee Island is composed of a series of these prograding beach and dune ridges which successively paralleled the shoreline immediately landward of the spring high tide level as the island developed in response to the outbuilding of the Savannah River deltaic complex. While the dune ridges are not readily apparent or as developed as Tybee Island, Little Tybee Island accumulated and established itself under the same scenario.

Tybee beach has been thoroughly studied over the last several decades for both academic and civic purposes. The majority of these studies have concluded that anthropogenic activities including beach renourishment/stabilization efforts and the adjacent navigation project have altered the islands significantly to the point that it is no longer possible to consider or assess the island complex as a natural system.

Oertel, Fowler, and Pope (1985) state that since 1854, the northeastern portion of Tybee Island had eroded at moderate to rapid rates, while the southeastern portion has experienced significant accretion. The north end of the island experienced two periods of high erosion rates from 1875 to 1900 and from 1918 to 1931. Oertel et al. (1985) attributes a majority of this rapid shoreline recession to several hurricanes that passed very close to the island. Posey and Seyle (1980) state that from 1920 to 1972 the average shoreline recession of the northeastern portion was 6.7 ft per year, and from 1854 to 1975 the middle portion experienced little net change, while the southern end accreted approximately 20 to 25 feet per year. More recently, the island has experienced increased shoreline erosion rates at the south end of the island.

WASSAW ISLAND

Like Tybee and Little Tybee Islands, Wassaw Island is separated from older landward islands by an expansive Holocene salt marsh. Wassaw Island is also a Holocene beach ridge island that developed from the Recent out-building of the Savannah River deltaic system. The well defined beach ridges without obvious soil zones, as found on Tybee and Little Tybee island, also further distinguish Wassaw island as a Holocene deposited barrier island complex. Griffin and Henry (1984) state that Wassaw Island had a history of deposition from 1858 to 1924 and dynamic stability from 1924 to 1974, resulting in net accretion from 1858 to 1974.

NEARSHORE AND INNER CONTINENTAL SHELF

The continental shelf that extends offshore from the Savannah River delta is conspicuously lacking in relief and extends 70 to 80 miles offshore. The continental shelf extends off the

Atlantic Coastal Plain with slopes averaging less than one degree. The shelf slope break occurs abruptly in the southern portion of the shelf near the 50-meter contour, but is not as well defined in the northern portion near the 80-meter contour.

The Savannah River has contributed significant amounts of sediment to the near shore and inner continental shelf, as suggested by the major lobate patterns seen offshore and south of the Savannah River delta (Kingery, 1973). The surficial inner continental shelf sediments are predominantly derived from coastal plain sediments and carried southward by longshore currents. The detailed nature of circulation patterns on the Georgia inner shelf is not well known. The shelf water circulation pattern is described as a complex system of tidal-current and wind generated eddies superimposed on a predominantly southward drift.

Sediment contributed by the Georgia rivers and estuaries is limited in its distribution on the continental shelf to a narrow nearshore band three to ten miles off the Georgia coast. In this area it had been established that a nearshore band of finer-grained sediments is flanked to the seaward side by coarser sediments. The nearshore band of finer-grained sediments has been considered to represent the full extent of modern or recent sedimentation on the shelf and the seaward coarser-grained sediments have been considered as "relict" sediments (Bigham, 1973). The coarser-grained sediments (~250 μm in mean size) are the repeatedly reworked remains of all the sediments that have been laid down on the continental shelf during the Pleistocene Epoch. As sea level has fallen and subsequently risen (at least seven times in the past two million years), the retreating shoreline has reworked these shelf deposits as the shoreface has migrated repeatedly landward. All finer-grained particles are removed by this process, leaving the coarser-grained material behind (Alexander, 2002).

3.2. Tides

The study area is located in a mesotidal region and experiences large semi-diurnal tides. The tidal currents in the Savannah River entrance are ebb dominant, whereas the currents in the Calibogue Sound are very slightly flood dominant (based on NOAA predicted tidal currents). The NOS tidal datums at the Fort Pulaski, Savannah River tidal bench mark, which are based on a 17 year time series from 1960 to 1978, are listed in Table 3-1.

3.3. Waves

Wind waves approaching Tybee Island are caused by two seasonally distinct climatic conditions (Nummendal, et al., 1977, Oertel and Howard, 1972). During spring and summer months, a high-pressure system (called a "Bermuda high") generally exists offshore between 65° W and 75° W longitude and 27° N and 37° N latitude. The clockwise rotation of air in these systems produces mild winds from the south that propagate low-energy waves into the study area. These waves transport littoral material from south to north during much of the period between early spring and mid-fall (Oertel, 1974). Since this is a relatively low-energy process, there is little evidence of erosional truncation (or scarping) of the upper part of the beach during these seasons.

Table 3-1 NOS tidal datums at Fort Pulaski

Datum	Elevation (ft MLLW)
Highest observed water level (10/14/1947)	11.15
Mean Higher High Water (MHHW)	7.47
Mean High Water (MHW)	7.10
Mean Tide Level (MTL)	3.66
National Geodetic Vertical Datum-1929 (NGVD)	3.38
Mean Low Water (MLW)	0.22
Mean Lower Low Water (MLLW)	0.00
Lowest observed water level (3/20/1936)	-1.35

The major climatic events of the fall and winter are related to strong low-pressure systems, including hurricanes. The counterclockwise flow of air in these systems produces strong winds from the north that drive high-energy waves onto the shore, causing north to south longshore transport over several days. These conditions often result in severe and rapid erosion of the shore. This rapid erosion often causes truncation or "scarping" of the upper foreshore and backshore. While these events are generally only a few days in duration, they produce significant shifts in shoreline position and beach elevation (Oertel et al., 1985).

The Wave Information Study (WIS) hindcast wave data produced by the USACE was used for the wave analyses in this study. Analysis of a 20-year hindcast data set for the station located closest to Savannah is presented in Section 7.2.2.

3.4. Storms

Griffin and Henry (1984) provide a discussion on the influence of hurricanes on the development of the study area shorelines and nearshore features:

> Hurricanes may well account for much of the sediment distribution patterns that occur along shoreline. Direct modification of barrier islands generally occurs by erosion of the seaward sides and deposition by overwash and inlet flow on landward sides, resulting in landward migration. Such results are reduced in Georgia by the broad nature of the islands.
>
> Major historical changes of the islands have likely resulted from erosion of dune areas. Foredunes are often completely removed by storm waves, which breach primary dune lines and initiate formation of washover fans. Dune erosion is intensified in devegetated foredune areas. Large quantities of sand may be carried seaward or freed from foredune complexes and washed, or later blown, inland to destroy additional vegetative cover along and behind dune lines (Nash, 1977).
>
> Because east coast hurricanes generally follow the path of warm air above the Gulf Stream, on an average only one "severe" hurricane strikes coastal Georgia every ten years (Carter, 1970). A considerably greater number of tropical cyclones have developed in the Atlantic, Caribbean Sea, and Gulf of Mexico and passed off the southeast coast without intensifying to hurricane status. An extremely severe hurricane passed through coastal Georgia in 1898. Several feet of water inundated the islands, and a hurricane surge of 35 feet was recorded from Sapelo Island to the north (Nash, 1977).
>
> Extratropical storms, or "Northeasters," are more frequent than hurricanes and generally occur during the autumn months. Although not as well documented as hurricanes, their erosion potential to the coastline is comparable, due to the longer duration and greater over-water fetch of winds associated with extratropical storms (Nash, 1977).

A list of notable hurricanes experienced at the study area from 1854 to 1983 is provided in Table 3-2.

3.5. River Flow

Freshwater discharges near Clyo, Georgia (RM 61) average 11,600 cubic feet per second with maximum and minimum annual mean discharges of 20,900 and 9,820 cfs, respectively, since 1962. The U.S. Geological Survey (USGS) station at Clyo, approximately 61 miles upstream of the mouth of the Savannah River, is the most downstream gage that records river discharges. Below this point, the Savannah River is tidal, with no major, constant fresh water inflows and USGS personnel have indicated that flow measurements would prove unreliable. Additionally, tidal current flow is significantly larger than freshwater flow for the lower Savannah River. Refer to the three-dimensional Hydrodynamic and Salinity Model Calibration Report (ATM, 2001b) for more detailed information on flows in the Savannah Estuary.

Table 3-2 Notable Hurricanes Experienced at Tybee Island, 1854 to 1983 (from Oertel et al., 1985)

Date	Classification**	Winds†
Sep 7-12, 1854	Hurricane	90
Aug 22-26, 1871+	Major	-
Sept 12, 1878+	Hurricane	-
Aug 21-29, 1881+	Hurricane	-
Aug 21-26, 1885	Extreme	125
Aug 15-Sep 2, 1883	Great	96
Sep 18-30, 1884	Hurricane	104
Sep 22-29, 1896	Hurricane	80
Sep 25-Oct 6, 1898	Extreme	-
Oct 9-23, 1910	Great	125
Aug 23-30, 1911	Major	106
Aug 5-15, 1940	Major	73
Oct 12-23, 1944	Great	120
Sep 11-20, 1945	Great	170
Oct 9-16, 1947	Hurricane	95
Sep 20-Oct 2, 1959	Major	150
Jun 4-14, 1966	Major	125

Notes:

* Compiled from data contained in Monthly Weather Review (Sugg, Pardue, and Carrodus 1977 and Ludlum 1963).

** Classification of hurricane intensity as determined by barometric pressure and windspeed:

Order of intensity:	Most intense-	Extreme
		Great
		Major
	Least intense-	Hurricane

† Maximum recorded windspeeds in mph at some point along the hurricane's path.

+ Not in original Oertel (1985) table; added data based on NOAA storm tracks.

3.6. Sediments

Numerous sediment data have been collected in the study area over the past 30 years. Sediment data was compiled from multiple sources, including data collected by: the Skidaway Institute of Oceanography, the National Oceanic and Atmospheric Administration (Bearden et al., 1999), the USACE Savannah District (1981), Olsen Associates, Inc. (1986), Law Engineering (1998), Dial Cordy and Associates, Inc. (2001) and Applied Technology and Management (1994 & 1996). The data sources included ocean bottom grab samples as well as vibracore samples. Since this study is interested in sediment transport along the ocean bottom, where the samples were collected via vibracore only the characteristics reported for the sediments at the surface of the core were included.

The sand fraction of the sediments data is plotted in Figures 3-2 and 3-4. The median grain size is presented in Figures 3-3 and 3-5. As shown by the sand fractions of the samples, the sediments in the study area are predominantly sandy material. However, in the navigation channel, a significant number of the grab samples were less than 50 percent sand. These samples are typically taken at slack tides and during this time, there is a thick layer of temporarily deposited mud that settles out. This is then collected by the sampler and later composited as part of the bottom sediments, although it is only there for a short period during each tidal cycle (Alexander, 2002).

The majority of the median grain sizes along the nearshore zone of Tybee Island are between 0.16 to 0.22 mm. In the bar channel, median grain size are relatively coarser along the stretch north of Tybee Island (0.13 to 1.2 mm, with an average of 0.49 mm), and much finer along the channel seaward of the first bend in the bar channel (0.13 to 0.67 mm, with an average of 0.29 mm).

3.7. Local Sea Level Rise

The rise in sea level may be responsible for a significant portion of the erosion of some shorelines. Bruun (1962) proposed a conservation of material method for calculating the amount of horizontal retreat on a beach due to sea level rise. Conceptually, as the shoreline moves landward, material is eroded from the foreshore and deposited offshore to maintain the same beach profile slope. The amount of retreat (x), as defined by Hands (1984) is

$$x = zX/Z$$

where

z = amount of sea level rise

X = horizontal distance from top of dune to point of closure depth, and

Z = vertical distance from top of dune to point of closure depth.

The closure depth Closure depth was approximated using Hallermeier's (1981) closure depth equation

$$d = 2.28\ H_e - 68.5\ (H_e)^2/(gT^2)$$

where

d = closure depth

H_e = extreme wave height

g = gravitational constant, and

T = wave period.

WIS 20-year hindcast wave data for Station 33 was used to calculate the closure depth. Extreme wave height is defined by

$$H_e = 5.6\ \sigma_s + H_s$$

where

H_e = extreme wave height

σ_s = mean standard deviation of significant wave height, and

H_s = mean significant wave height.

The calculated extreme wave height is 12.5 feet. Solving Hallermeier's equation yields a closure depth of 23.2 feet.

The estimate of shoreline retreat varies proportionally with the estimate of horizontal distance to the closure depth, which varies over the study area. The horizontal distance to the closure depth is approximately 7,800 feet at Tybee Island (assuming the closure depth is 23.2 feet at MWL). Given a sea level rise of 1.3 feet (ATM, Tidal Amplitude Study, 2001), and an average dune elevation of 12 feet MLW, the estimate of total shoreline retreat due to sea level rise for the period from 1859 to 1993 was calculated to be 290 feet, which is 2.2 feet per year. This is a relatively high rate of sea level rise induced erosion because of the flat beach profile slope off Tybee Island.

4. SHORELINE AND BATHYMETRY MORPHOLOGY

Shoreline and bathymetry data were compiled to analyze long-term trends in the study area. This section presents: a description of the data sources and the adjustment of the data to a common datum; comparison of historic shoreline positions; and comparison of bathymetric contours and calculation of bathymetric change.

4.1. Data Collection

Mean high water (MHW) shoreline data was collected from the sources listed in Table 4-1. With the exception of the 1993 shoreline, the data was digitized from the Cooperative Shoreline Movement Study conducted by the NOAA National Ocean Service (NOS), the USACE Coastal Engineering Research Center (CERC) and Statistical Services of South Carolina. The horizontal datum of the shorelines was converted from a Lambert projection of the 1927 North American Datum (NAD27) to a Georgia State Plane NAD83 projection. The Shoreline Movement Study shorelines cover all of South Carolina and also cover two-thirds of Tybee Island. The 1993 MHW shoreline was digitized from USGS rectified aerial photographs and extends from the north end of Hilton Head Island to the south end of Little Tybee Island. The horizontal datum of the 1993 MHW shorelines was converted from a Universal Transverse Mercator (UTM) Zone 17N NAD83 projection to a Georgia State Plane NAD83 projection.

Table 4-1 Shoreline data sources

Date	Description
1852	Field Survey
1859-63	Field Survey
1870-74	Field Survey
1900	Field Survey
1920	Field Survey
1964	Field Survey
1970-71	Field Survey
1982	NOS Aerial Photography
1993	USGS Aerial Photography

Shoreline and bathymetric data were compiled from the hydrographic charts listed in the Table 4-2. Mylar copies of the oldest three charts (dated 1854, 1897 and 1920) were obtained from the NOS Office of Coast Survey and digitized. The remaining data was obtained digitally from the GEODAS CD distributed by the National Geophysical Data Center (NGDC).

The horizontal datum for the charts is also included in Table 4-2. The Clarke spheroid of 1866 was selected as the national datum in 1880, replacing the Bessel spheroid of 1841. The datum was renamed the U.S. Standard datum in 1901 and the North American Datum (NAD) in 1913. An adjustment of the first-order triangulation was completed in 1931, resulting in the North American Datum of 1927 (NAD27). The charts were all adjusted to the Georgia State Plane NAD83 coordinate system using the AutoCAD Land Development software.

All of the data were relative to the Mean Low Water (MLW) datum, with the exception of the 1980 survey, which was relative to the Mean Lower Low Water (MLLW) datum. A vertical datum adjustment was applied to all of the hydrographic data to shift the data to the National Geodetic Vertical Datum of 1929 (NGVD29). The data were also compensated for the effects of sea level rise, which was estimated at 0.00948 feet per year based on the 1935 to 1994 water surface elevation record at Fort Pulaski. The total vertical datum adjustment applied to each data set is listed in Table 4-2.

Table 4-2 Hydrographic survey chart sources

Chart No.	Date	Source	Map Projection	Vertical datum adjustment (ft)
H-439	1854	U.S. Coast & Geodetic Survey	Bessel spheroid of 1841	-4.2
H-2296	1897	U.S. Coast & Geodetic Survey	Clarke spheroid of 1866	-3.8
H-4154	1920	U.S. Coast & Geodetic Survey	North American Datum	-3.5
H-5592	1934	National Ocean Service	NAD27	-3.4
H-5571	1934	National Ocean Service	NAD27	-3.4
H-9459	1974	National Ocean Service	NAD27	-3.2
H-9197	1971-73	National Ocean Service	NAD27	-3.2
H-9865	1980	National Ocean Service	NAD27	-3.4

4.2. Shoreline Morphology

The study area shorelines of Hilton Head Island, Daufuskie Island, Turtle Island, Oyster Bed Island, Cockspur Island and Tybee Island are plotted in Figures 4-1 and 4-2.

HILTON HEAD ISLAND

As shown in Figure 4-1, the historic shoreline trends show general erosion of the northern and central sections of Hilton Head Island and corresponding accretion of the southern half of the island. Bruun (1977) attributes the erosion of the northern half of the island to sea level rise and the tidal creek called "The Folly." Port Royal Sound is a littoral barrier to the predominant southward littoral drift, and Bruun states that the Folly behaves as a local littoral barrier on Hilton Head Island and has caused the creation of shoals storing approximately 100,000 cubic meters of sand.

Olsen Associates, Inc. (1987) states that little bypassing of littoral sediment to Hilton Head Island from the north occurs at Port Royal Sound. Consequently, most littoral transport that occurs along the shoreline results from erosion of the upland itself. The shorelines at the extreme ends of the island adjacent to the two tidal inlets have been most variable due to the impacts of nearshore shoal and marginal flood channel migration. Some stability had been afforded in recent decades as a result of the construction of relatively successful groin fields at these locations. Some of the accretion at the ends of the island is attributable to the placement of beach fills along the shoreline. Beach fill projects occurred along the front beach in 1969, 1990 (2,000,000 CY), 1997 (3,000,000 CY), and a nourishment of the south tip occurred in early 1999.

DAUFUSKIE ISLAND

Figure 4-2 shows the steady erosion of the portion of the Daufuskie Island shoreline that is exposed to the open ocean (i.e., the shoreline south of Hilton Head Island). Despite the sheltering of the island from wave energy by Barrett and Grenadier Shoals, waves and currents steadily eroded the consolidated mud sediments of this island until the completion of a beach fill project in 1998. The protective layer of sand provided by the beach fill has stabilized most of the shoreline, and future erosion rates are expected to be lower than historical rates. The obvious exception is the shoreline adjacent to the New River Inlet at the southern end of the island (known as Bloody Point). Historically, the New River Inlet has migrated in a northerly direction and it will likely continue that trend. Groins have been proposed to stabilize the inlet shoreline, but recent court rulings have forced the South Carolina Department of Health and Environmental Conservation (SCDHEC) Office of Ocean and Coastal Resource Management (OCRM) to deny all groin permits in South Carolina. Project monitoring of the beach fill project has not occurred, but the few beach profiles measured at the island by the OCRM indicate that only a small loss of fill has occurred over the two years following completion of the project.

Turtle Island

Turtle Island has experienced general erosion of the seaward shoreline. However, the south end of the island has accreted substantially. Much of this change occurred in the interval between the 1859-63 shoreline and the 1920 shoreline. In this time period, the jetties were constructed as well as training walls between Oyster Bed Island and Jones Island. This eliminated a majority of the flow between Oyster Bed Island and Turtle Island, which resulted in the accretion at the south end of Turtle Island. The south end of the island further accreted during the interval between the 1920 shoreline and the 1964 shoreline. During this time period dredge spoil was placed between Oyster Bed Island and Jones Island joining the two islands. This completely eliminated the flow of the Savannah River between the two islands and allowed the further accretion of the south end of Turtle Island. Following 1964, the south end of the island appears to have stabilized.

Oyster Bed and Cockspur Islands

The construction of training walls and jetties as well as the placement of dredge spoil at Oyster Bed Island has enlarged this from what was once an intertidal bank to an emergent island eventually connected to Jones Island. Most of the emergent sediments on Oyster Bed Island are attributable to dredge spoil, and it appears that only a small fillet of land on the north side of the jetty resulted from impoundment of littoral drift from the north.

Cockspur Island has shown continuous growth between the 1859-63 shoreline and the 1993-94 shoreline. Similar to Oyster Bed Island, dredge spoil was placed on Cockspur Island and accounts for much of the growth. Additionally, the construction of the south jetty resulted in the accretional fillet to the south of the jetty.

Tybee Island

For comparison, the shoreline positions of Tybee Island mapped in the studies by Oertel et al. (1985) and Griffin and Henry (1984) are shown in Figures 4-3 and 4-4. Based on analysis of beach ridge formations, Oertel et al. (1985) hypothesize that prior to 1854, the shore of Tybee Island was characterized by significant periods of accretion. The regularly spaced dune ridges on the southern end of the island suggest that accretion was continuous.

Development of the north end of the island was much more irregular, as indicated by the truncations between beach ridge sets.

The shoreline positions shown in Figure 4-2 illustrate two major trends in shoreline evolution: net accretion between 1859-63 and 1900, and net erosion between 1900 and the present. The period between 1859-63 and 1900 shows erosion of the northeastern end of the island as it shifted to the northwest, while the rest of the island accreted substantially. Oertel et al. (1985) attributes the erosion of the northeast end of the island to the passage of several hurricanes that passed very close to the island (see Table 3-2). The accretion along most of the island suggests that the Savannah River was contributing sediment generously to the island system during this period (Griffin and Henry, 1984). The harbor improvements may have contributed to this accretion by way of increasing the sediment discharge of the North Channel. During this period, the closure of the Cross Tides occurred (1881), and consequently, the cross-sectional areas and the mean depth in the Front River increased 20 percent (Black, 1893). Additionally, the numerous spur dikes and training walls along the Front River caused increased Front River velocities and bottom scour. The scoured sediments from the Front River were ultimately deposited outside the Savannah River entrance and may have contributed to the accretion on Tybee Island.

Harbor modifications between the 1850s and 1900 may have had other impacts to Tybee Island. The effort to reduce the flow in South Channel during the period prior to the construction of the jetties may have increased the growth of the north end of Tybee Island. Additionally, the construction of the jetties altered the ebb jet was directed away from the north end of Tybee Island.

Between 1900 and 1920, the shoreline shows rapid erosion. Griffin and Henry (1984) observed the erosion between the 1913 and 1925 shorelines and hypothesize that because this atypical erosion took place prior to dam construction, adoption of soil conservation practices, and occurred during a period in which the majority of other island systems in Georgia were rapidly accreting, the losses may be related to the dredging activities initiated in 1919. The shorelines in Figure 4-2 show that this erosion occurred by 1920, which is too soon to be caused by navigational dredging conducted in 1919 and later. However, substantial dredging in the harbor occurred between the 1890s and 1920 that likely contributed to the erosion of the Tybee Island shoreline. For example, the annual report of the Chief of Engineers in 1907 estimates that 17,000,000 cubic yards of material had been dredged from the harbor since 1873. Additional dredging of the harbor occurred between 1907 and 1920 in efforts to maintain the 26-foot MLW project.

Subsequent to the 1920 shoreline shown in Figure 4-2, erosion control structures (e.g., groins and seawalls) were built in response to the rapid erosion along Tybee Island began to influence the shoreline position. Oertel (1985) depicts the location of shoreline structures through 1985, as shown in Figure 4-5. These structures helped to stabilize erosion of upland areas and protect upland development, but the beaches continued to erode. By the late 1960's, the MHW shoreline had eroded to the seawall at the north end of the island, and use of the beach was restricted to periods of low water.

Beach nourishment efforts began in 1975 with the placement of 2.3 million cubic yards of sand along the Tybee Island shoreline. Following the project, large quantities of sand appeared at the north end of the island, outside of the fill project area. The northern third of the island was stable, the central part of the island was accretional, and the southern part of the project eroded substantially.

Recent structures were built to stabilize the south end of the island, including a south end terminal groin in 1986-87 and a south tip groin field in 1994. The chronology of recent beach renourishment and erosion control efforts at Tybee Island are summarized in Table 4-3.

4.3. Bathymetry Morphology

The digitized bathymetry data were interpolated onto a grid using the SURFER application. Color contour plots of the bathymetry data for 1854, 1897, 1920 and 1970-80 are shown in Figures 4-6 through 4-9, respectively.

The 1854 bathymetry in Figure 4-6 depicts the conditions that existed prior to significant dredging efforts in the Savannah harbor. This complex multi-inlet system does not lend itself to inlet processes categorization as easily as other single inlet systems. However, categorization of the inlet processes in the study area is simplified by considering the entire region between the south end of Hilton Head Island and the north end of Tybee Island as a single inlet. Then, this larger scale system is well-described by the ebb-tidal delta breaching discussed by Fitzgerald, Hubbard and Nummendal (1978), as shown in Figure 4-10B. The predominant southward littoral drift leaving Hilton Head Island moves offshore onto the

Barrett Shoals ebb-tidal delta feature. However, rather than having one main ebb channel, as shown in Figure 4-10B, Barrett Shoals is breached by three main ebb channels: two that conduct the ebb flow of Calibogue Sound, and a third that conducts the Savannah River ebb flow. The 1854 conceptual inlet model is shown in Figure 4-11, with the 18-foot contour used to define the shape of the ebb-tidal delta. In this system, wave energy causes the southerly migration of the channel features of the delta. Sediment bypassing occurs as ebb channels relocate in the updrift (northerly) direction, and the downdrift (southerly) most shoals “weld” onto the downdrift shoreline. In this case, it appears that the bypassed shoals would merge into the offshore platform seaward of Tybee and Little Tybee Islands.

Table 4-3 Chronology of recent beach renourishment and erosion control efforts at Tybee Island

Year	Action
1975	-- 800’ North End Terminal Groin constructed
1975-1976	-- First renourishment. -- 2,262,100 CY of sand placed on the beach between North End Terminal Groin and 18th Street.
1986-1987	-- 600’ South End Terminal Groin constructed. -- Rehabilitation of North End Terminal Groin. -- 1,200,000 CY of sand placed between groins. -- 157,000 CY of sand placed on 1,400’ of shoreline south of South End Groin.
1993	-- 1,500,000 CY of material placed on beach from Savannah Harbor deepening.
1994	-- South Tip Groin Field constructed by Georgia Ports Authority with State funds.
1995	-- 285,000 CY of material placed between South End Groin and 13th Street by Georgia Ports Authority. -- 50,000 CY of sand placed within South Tip Groin Field by Georgia Ports Authority.
2000	-- 1,200,000 CY of material placed on beach --- three short groins constructed along north beach

Source: U.S. Army Corps of Engineers, Savannah District

The overlay of the historic 18-foot contours of the study area illustrates the southward migration of the ebb channels on Barrett Shoals (Figure 4-12). The 1854 and 1920 contours show the channels moving southward, and the 1971-73 contours show the creation of a new ebb channel just north of the northern most ebb channel in 1854.

The 18-foot contours near the seaward most bend in the navigation channel show the migration of the Savannah River main ebb channel. The ebb channel width increases between 1854 and 1897 as shown by the increased distance between the contours. The width remains about the same from 1897 to 1920 and from 1920 to 1971-73/80. This is related to the fact that the southwestern migration of the 18-foot contour near the seaward most bend in the navigation channel is a natural migration of the ebb channel unrelated to the dredging of the navigation channel. At some point between 1920 and 1971-73, the shoal to the north of the navigation channel had reached the navigation channel near the seaward bend, precluding further migration in the southwest direction.

In the absence of the navigation channel maintenance, the Savannah River main ebb channel would continue to migrate in a southwesterly direction until the system becomes too hydraulically inefficient. As an ebb channel that conducted primarily Calibogue Sound ebb flow migrates southward, it would conduct increasingly greater ebb flow from the Savannah River, and eventually, it would become the new main ebb channel for the Savannah River. The shoal south of this new main ebb channel would migrate southwestward until "welding" onto the nearshore platform of Tybee and Little Tybee Islands.

Also of interest in Figure 4-12 is the northward migration of the 18-foot contour near the north end of Tybee Island. This should be viewed in concert with the 12-foot contours shown in Figure 4-13. The dramatic erosion of the shoal seaward of Tybee Island shown by the landward migration of the 12-foot contour may have contributed sediment to the accretion north of the island. Other factors that may have influenced the north end accretion may include: (a) the reduction in flow of South Channel that occurred with the development of the Savannah Harbor, and (b) the construction of the jetties.

The 12-foot contour plot also shows a smaller ebb-tidal delta breaching system within the larger scale Barrett Shoals system. The New River ebb-tidal shoal consists of sediments eroded from Daufuskie Island, and the main ebb-tide channel breached the shoal in the period between 1920 and 1971-73. This ebb-tidal delta feeds sediments toward the north end of Tybee Island, as shown in the conceptual inlet model in Figure 4-14. Whereas the Barrett Shoals tidal delta displays the characteristics of a mixed energy system, the New River inlet is tidally dominated, as indicated by the linear nature of the shoal system. The New River inlet is tidally dominated as a result of the sheltering of wave energy provided by Hilton Head Island, Barrett Shoals and Tybee Island.

The conceptual inlet bypassing model for the present conditions is shown in Figure 4-15 overlain on the 1970-80 bathymetry contours. In this figure, the New River ebb channel has relocated to the north, and the bypassed sediments should migrate in the southeastern direction. This figure also shows the interception of the Barrett Shoals sediments by the navigation channel near the seaward bend in the channel, and the transport of sediments from the north end of Tybee Island into the navigation channel. The rate at which these sediments are intercepted by the navigation channel is examined in the following section (Section 4.4).

The bathymetric data for each time period was compared for mutually inclusive areas, resulting in the elevation change plots shown in Figures 4-16 to 4-19. The 1854 to 1897 plot (Figure 4-16) shows the bathymetric changes that occurred during a period when many training dikes had been built in the inner harbor, and significant dredging of the inner harbor had begun. Additionally, the jetties had just been built during the 1890s. The rapid growth of a shoal to the north and northeast of Tybee Island is evident, which corresponds to the erosion of the northeast headland of the island. South of this area, the lowering of the platform east of Tybee Island has already started.

In the period from 1897 to 1920, dredging of the bar channel commenced. Figure 4-17 shows the negative change in elevation along the navigation channel location. The shoal at the north end of Tybee Island migrated in a northerly and easterly direction, with corresponding erosion south of the shoal. The southern ebb channel of the Calibogue Sound flow migrated southward, causing erosion on the north side of the submerged breakwater, and accretion along the south side of the breakwater.

The period from 1920 to 1970-80 (Figure 4-18) shows the significant deepening of the bar channel. A new ebb channel was created at the north end of Barrett Shoals. At the other end of Barrett Shoals, sediments migrated from the region near the submerged breakwater toward the seaward bend in the navigation channel. The shoal at the north end of Tybee migrated farther north and east, while the South Channel ebb channel switched to the south side of the shoal, resulting in erosion of the north and west side of the island. Also, the ebb channel of the New River switched to a new location in a more easterly orientation. The total change between 1854 and 1970-80 is shown in Figure 4-19.

4.4. Volumetric Bathymetry Change Calculations

The above studies and figures provide a good qualitative and quantitative history of bathymetric changes. However, a volumetric change analysis was performed to elucidate sediment transport trends. The study area was divided into subregions that approximately correspond or can be easily translated to the sediment budget calculations in section 8.0. These results can also be compared with nearshore sedimentation modeling simulations which are computed in section 8.0. Figures 4-20 through 4-22 and Table 4-4 present results from the volumetric calculations. The figures provide a color-coded map of the average rate of change in elevation for each cell, and the volumetric change rates (in 1,000s of CY/yr) are indicated in parentheses. Three different time periods were used: 1854 to 1920, 1920 to 1970-80 and 1854 to 1970-80. The consistent major trends apparent in the plots are: (1) the growth of the north Tybee shoal, (2) the deepening of the navigation channel, and (3) the erosion of the Tybee subaqueous platform (cells 15, 16, 19, 20, and 21). Barrett shoals and the Daufuskie/Turtle Island subaqueous platform loose material in the 1854 to 1920 period and gain material in the 1920 to 1978-80 period.

The cells can be grouped to represent morphological elements within the study area. Table 4-5 shows the cell groups and the volumetric change rates for each comparison period.

Table 4-4 Historic Volumetric Change Rates

Cell	Surface Area (ft^2)	Volume Change Rate (CY/yr)		
		1854 to 1920	1920 to 1970-80	1854 to 1970-80
1	7.0E+07	-8.1E+04	-5.0E+03	-4.8E+04
2	4.3E+07	-9.4E+04	2.0E+04	-4.5E+04
3	9.6E+07	4.8E+03	-5.2E+04	-2.0E+04
4	5.8E+07	3.3E+04	2.8E+04	3.1E+04
5	1.1E+08	-3.4E+04	5.7E+04	5.1E+03
6	5.3E+07	-5.5E+04	1.7E+04	-2.4E+04
7	1.2E+08	-1.0E+04	9.7E+04	3.6E+04
8	4.8E+07	-1.0E+04	1.6E+04	1.2E+03
9	4.7E+07	-2.1E+04	-2.7E+04	-2.4E+04
10	1.2E+08	-8.6E+04	-5.2E+03	-5.1E+04
11	3.2E+07	6.1E+04	-8.3E+03	3.1E+04
12	1.7E+07	9.4E+04	3.0E+03	5.5E+04
13	1.2E+07	1.9E+04	5.1E+04	3.3E+04
14	2.0E+07	-6.8E+04	-7.9E+04	-7.3E+04
15	2.8E+07	-5.3E+04	-4.5E+04	-5.4E+04
16	5.3E+07	-6.5E+04	-1.1E+04	-4.3E+04
17	1.5E+07	-3.5E+04	-4.0E+04	-3.7E+04
18	7.9E+07	1.5E+04	7.7E+04	4.2E+04
19	3.6E+07	-3.1E+04	1.6E+04	-9.2E+03
20	5.0E+07	-1.9E+04	-5.1E+04	-3.8E+04
21	4.7E+07	-1.3E+05	-4.6E+04	-1.0E+05
22	1.7E+07	-6.8E+04	-3.5E+04	-5.7E+04

Table 4-5 Volumetric Change Rates for Grouped Cells

Feature	Grouped Cells	Volume Change Rate (CY/yr)		
		1854 to 1920	1920 to 1970-80	1854 to 1970-80
Daufuskie/Turtle Island subaqueous platform	1, 4, 5, 8	-92,000	96,000	-11,000
Barrett shoals	3, 7, 9, 10, 18	-98,000	90,000	-17,000
Navigation channel	14, 17, 22	-170,000	-155,000	-167,000
Tybee north shoal	12, 13	113,000	54,000	87,000
Tybee Island subaqueous platform	15, 16, 19, 20, 21	-304,000	-138,000	-244,000

4.5. Recent Bar Channel Sedimentation Rates

Digital files of surveyed bathymetry in the bar channel was obtained from the USACE Savannah District which included surveys taken after maintenance dredging in 1999 and before dredging in 2000. The 1999 AD (after dredge) surveys were taken on January 22, February 4 and 9, and April 13. The 2000 BD (before dredge) surveys were completed on January 26 and February 4. Therefore, the change in bathymetry in the bar channel between these surveys represents approximate annual sedimentation rates.

The data were interpolated onto a grid in SURFER, and the change in elevation between the two data sets was calculated (Figure 4-23). The data show that the bar channel shoaled significantly at two locations: on the north side of the channel near the seaward bend in the channel, and in the section between the jetties and north of Tybee Island. The shoaling volume at the seaward bend in the navigation channel is approximately 700,000 cubic yards. The shoaling in navigation channel north of Tybee Island is approximately 209,000 cubic yards. The net volume change along the entire region surveyed in the navigation channel is approximately 1,152,000 cubic yards during 1999.

The shoaling at the offshore bend was limited to the north side of the channel, which supports the conclusion that bypassing of the channel does not occur at this location. If significant amounts of sediments were to bypass the channel, shoaling along the south side of the channel would have occurred as well.

During 1999 several hurricanes passed offshore of the study area, including: Hurricane Dennis in August, Floyd in September, and Irene in October. These storms caused increased wave action during this period and most likely resulted in higher shoaling rates in the bar channel as compared to average years (Section 2 showed average bar channel sedimentation volumes of approximately 700,000 CY/yr for years not immediately following deepening events).

5. CURRENT DATA COLLECTION

Current data was collected in the major tributaries and at offshore locations in the study area. The primary purpose of the current data collection was to provide data with which to calibrate the hydrodynamic model. The secondary objective was to also provide measured currents around an offshore "box" to assist a study conducted by the Skidaway Institute of Oceanography.

The current data was collected using an Acoustic Doppler Current Profiler (ADCP). Transects were performed by transporting the ADCP across transects by boat, or for the stationary measurements, the vessel-mounted ADCP was held stationary at a location for several minutes while measurements were taken. The ADCP is secured to the vessel with an aluminum boom system that recovers and deploys the ADCP over the side of the boat. When deployed over the side of the boat the ADCP transducers are orientated toward the ocean bottom. The transducers are submerged approximately 0.5 meters beneath the water surface. The boom system is adjusted each deployment such that the centerline of the ADCP cylindrical body is perpendicular to the river bottom. In this configuration, the ADCP can compute the velocity of the water column relative to the ADCP.

A laptop computer utilizes the RD Instruments software titled TRANSECT to communicate with the ADCP. The DOS-based software is user interactive and consists of several programs that communicate, configure, collect, replay, and process data. Communication is established through a cable from the ADCP to the series connection on the laptop.

The TRANSECT software enables the field technician to setup a configuration file for the expected water characteristics at a transect location. Parameters entered into the configuration file include maximum salinity, maximum depth, and acoustic signal processing options. Entry of the parameters enables the software to correctly calculate the speed of sound in water. Each configuration file was created specifically for the transect locations for that day. Expected maximum salinity and depths values were obtained from profile data recorded at stations proximal to the transect locations.

The current measurement locations are shown in Figure 5-1. Currents were measured at the inlet throats to capture peak ebb and flood discharge values at each location and collect sufficient discharge data to generate a sinusoidal discharge curve. Transects were performed approximately every hour to resolve the discharge curve at each inlet location.

The current data was collected between December 8th through 15th, 1999. The tidal conditions during this period are shown in Figure 5-2. Appendix C contains figures of measured data at different transects.

In post-processing, the stationary currents measurements at the offshore "box" (i.e., Stations HH16 through HH23) were depth averaged and averaged over 1-minute. The currents measured over the day are presented in Figures 5-3 through 5-8. The data at these stations are summarized in Table 5-1. The asterisks in Table 5-1 represent extrapolated values from a velocity ebb jet transect. Therefore, the transects were not performed at stationary points, as all other stations are presented in the table. The purpose for including these data was to represent the bar channel ebb jet characteristics.

The discharge values measured at the inlet transects were recorded from the reported discharge values of RDI's TRANSECT software. Discharge values obtained by using TRANSECT were checked in the field by repeating the transect in the opposite direction. Discharge values from the first and second transect runs were both recorded. The flow values at the transect locations for the major inlets are tabulated in Table 5-2. These flow values were utilized for the hydrodynamic model calibration (Section 6).

Table 5-1 Summary of point ADCP measurements

Location	Time (EST)	Depth Averaged		Bottom Bin	
		Velocity (cm/s)	Direction (deg)	Velocity (cm/s)	Direction (deg)
HH16	8:56	11.7	311.4	10.2	324.6
HH17	9:10	1.6	354.0	2.5	0.0
HH18	9:27	7.0	342.7	5.7	299.2
HH19	9:43	3.6	147.0	4.6	217.1
HH20	9:53	8.5	298.4	12.3	278.4
HH21	10:03	15.3	126.5	8.4	129.2
HH22	10:17	19.0	129.6	8.6	132.2
HH23	10:40	28.3	123.5	21.6	126.0
HH16	10:45	23.6	134.8	19.2	128.7
HH17	11:07	25.2	143.6	18.4	146.3
HH18	11:21	28.8	158.6	10.9	176.8
HH19	11:39	44.0	159.6	36.2	164.6
HH14*	12:11	101.2	112.6	62.0	116.4
HH21	12:29	38.0	144.3	25.4	154.5
HH22	12:41	42.4	136.2	17.4	132.7
HH23	12:58	45.4	128.7	39.8	128.4
HH15*	13:14	53.3	138.4	41.7	135.9
HH17	13:26	33.8	174.2	21.5	181.1
HH18	13:39	27.1	174.6	12.6	178.2
HH19	13:49	11.3	151.4	11.3	151.4
HH20	13:58	102.4	114.3	70.8	111.9
HH21	14:09	26.5	140.7	22.2	147.0
HH22	14:27	24.2	147.3	18.1	149.5
HH23	14:33	21.4	140.4	11.3	149.6
HH16	14:45	16.6	154.0	1.0	185.7
HH17	15:00	30.6	185.8	16.2	194.0
HH18	15:13	20.0	200.6	13.4	209.6
HH19	15:23	10.6	354.0	10.6	354.0
HH20	15:36	42.9	117.1	13.2	128.5
HH21	15:40	9.8	238.6	9.5	252.1
HH22	15:51	16.2	271.6	13.3	274.3
HH23	16:03	16.5	282.7	17.3	284.8
HH16	16:16	31.7	296.9	21.1	307.3
HH17	16:23	21.1	258.6	17.2	288.6
HH18	16:35	27.7	278.0	26.7	292.9
HH19	16:45	42.7	349.5	42.7	349.5
HH20	16:52	56.2	299.1	28.2	295.4
HH21	17:00	21.0	304.8	6.7	299.6

Table 5-2 Summary of ADCP measured flows

Calibogue Sound (HH01)		New River (HH02)		Wright River (HH05)		Savannah River (HH06)		South Channel (HH07)	
Time (EST)	Flow (m^3/s)	Time (EST)	Flow (m^3/s)	Time (EST)	Flow (m^3/s)	Time (EST)	Flow (m^3/s)	Time (EST)	Flow (m^3/s)
12/13/99 8:05	-7509	12/14/99 9:01	-1122	12/14/99 10:39	-154	12/15/99 8:13	-1507	12/15/99 8:55	-648
8:23	-7949.1	9:11	-1143	10:44	-160	8:27	-1941	8:59	-97
9:07	-8868.1	9:51	-1488	11:54	-46	9:27	-3243	9:03	-650
9:24	-8742	10:03	-1487	11:58	-44	9:36	-3416	9:56	-733
10:11	-8044.7	11:00	-1276	12:53	120	10:26	-4070	10:01	-741
10:28	-7851.9	11:09	-1266	12:56	129	10:35	-3962	11:05	-790
11:10	-5192.9	12:11	-482	13:56	413	11:33	-4052	11:10	-761
11:29	-3730.3	12:22	-365	14:00	399	11:43	-3889	12:05	-525
12:05	86.5	13:18	887	14:52	372	12:33	-2879	12:10	-492
12:21	1886.3	13:26	1089	14:56	347	12:47	-2531	13:09	6.5
13:12	7405.4	14:14	1587	15:46	244	13:37	-915	13:15	77
13:35	9005.1	14:21	1625	15:50	221	13:48	-434	14:11	805
14:12	10060.6	15:08	1525			14:36	1508	14:16	828
14:29	10170	15:17	1600			14:46	1821	15:08	1141
15:08	9415	16:01	1319			15:34	3338	15:13	1125
15:27	9034.3	16:05	1263			15:43	3466	16:03	1082
						16:37	-4065	16:09	1030
						16:47	-4124		
						17:11	-3988		

Note: Ebb flow is positive

6. HYDRODYNAMIC MODELING

A numerical model was used to simulate the hydrodynamic environment in the Savannah Estuary and offshore areas. The calibrated model was used to evaluate changes in the currents and flows for three different scenarios: the 1854 conditions, the existing conditions, and the post-deepening conditions. This section describes the model, the model inputs, the model calibration and the model results.

6.1. Model Description

In order to accurately simulate the geometric and bathymetric features of the study area, ATM chose a boundary-fitted coordinate, hydrodynamic model system. This system approach uses transformation functions such that all domain boundaries are coincident with coordinate lines. The transformation equations are applied to a user-defined grid of arbitrarily sized quadrilaterals, mapped to the coastal geometry of the water body in the study area. The hydrodynamic model and the mass transport model equations are written and solved on the boundary conforming, transformed grid, using a well-known finite difference solution technique (Spaulding, 1984; Thompson et al., 1977). The boundary-fitted hydrodynamic and transport models are contained within the model system called WQMAP (Water Quality Mapping and Analysis Program).

6.1.1. Grid Generator

The WQGRID component of WQMAP was used to generate a boundary fitted grid. The grid is specified by locating grid points along coastlines and bathymetric features such as channels and depth contours. Each point has assigned grid indices to keep track of how each grid point relates to its neighbors. The grid spacing in the domain is roughly determined by grid spacing at land boundaries. Finer grid resolution is specified for increased flow resolution. Once the boundary grid points along the shoreline have been specified, and any bathymetric feature grid point located, the gridding model generates all the remaining interior points. These points are constrained to obey a Poisson equation and their locations solved iteratively by a Poisson solver.

In general, the grid aspect ratio reflects *a priori* estimates of expected flows. This means that the longer grid dimension, if any, is oriented along the major axis of the flow. This approach is necessary because the hydrodynamic model has inherent time step restrictions based on the ratio of grid size to flow speed. Faster model simulations are possible when the grid is optimized in this manner.

A depth value must be assigned to each grid. Two methods are generally combined to create the array of grid depths. First, a database of bathymetric soundings with associated latitude and longitude for the area is accessed. Each grid is automatically assigned a depth value by interpolation from the database based on a distance-weighting algorithm. Once all grids have depths assigned, the results are shown graphically and may be edited in WQGRID. The second method is based on the experience of the modeler to more accurately specify depths. Tools are available to the user in WQGRID to select individual grids or groups of grids and specify depth values. This procedure becomes necessary when dredged channels or other bathymetric features are to be accurately represented.

6.1.2. Hydrodynamic Model

WQMAP includes a state-of-the-art hydrodynamic model. The WQMAP model has been applied successfully in many projects around the world, the model has been the subject of numerous technical papers, and it is a well-accepted model by engineering professionals and scientists within the hydrodynamic modeling community.

The state-of-the-art, boundary-fitted hydrodynamic model (Muin and Spaulding, 1997; Huang and Spaulding, 1995b; Swanson *et al*., 1989) was used to generate water surface elevations and current velocities. A detailed description of the hydrodynamic model is presented in Muin and Spaulding (1997). The boundary-fitted model matches the model coordinates with the shoreline boundaries of the water body, accurately representing the study area. This system also allows the user to adjust the model grid resolution as desired.

The boundary-fitted method uses a set of coupled quasi-linear elliptic transformation equations to map an arbitrary horizontal multi-connected region from physical space to a rectangular mesh structure in the transformed horizontal plane (Spaulding, 1984). The two-dimensional conservation of mass and momentum equations, with approximations suitable for lakes, rivers, and estuaries (Swanson, 1986; Muin, 1993) that form the basis of the model, are then solved in this transformed space. In addition, an algebraic transformation is used in the vertical to map the free surface and bottom onto coordinate surfaces.

The basic equations are written in spherical coordinates to allow for accurate representation of large model areas. The conservation equations for water mass, momentum (in two dimensions), and constituent mass, form the basis of the model. It is assumed that the flow is incompressible, that the fluid is in hydrostatic balance, the lateral friction is not significant, and the Boussinesq approximation applies. The boundary conditions are as follows:

- At land, the normal component of velocity is zero.
- At open boundaries, the free surface elevation must be specified.
- A bottom stress or a no-slip condition can be applied at the bottom.
- No water is assumed to transfer to or from the bottom.
- A wind stress can be applied at the surface.

The set of governing equations with dependent and independent variables transformed from spherical to curvilinear coordinates, in concert with the boundary conditions, is solved by a semi-implicit, split-mode finite difference procedure (Swanson, 1986). The equations of motion are vertically integrated and, through simple algebraic manipulation, are recast in terms of a single Helmholtz equation in surface elevation. This equation is solved using a sparse matrix solution technique to predict the spatial distribution of surface elevation for each grid. The vertically averaged velocity is then determined explicitly using the momentum equation.

6.2. Hydrodynamic Model Inputs

The model set-up requires the user to build a model grid based on the geometry of the study area (i.e., the shorelines and bathymetry), and develop the model inputs (i.e., the boundary forcings and model coefficients). The model inputs are described in the following sections.

6.2.1. Model Geometry

The model geometry is defined by the shorelines of the study area. For the existing conditions, a GIS shoreline basemap of the study area was created from NOAA GIS shorelines of South Carolina and Georgia. At Tybee Island, these shorelines were replaced by shorelines digitized from USGS 1994 aerial photographs.

The WQGRID program was used to develop the model grid. Figure 6-1 presents the computational grid used for the model study. The two-dimensional grid dimensions are I = 129 cells and J = 334 cells. The grid resolution varies from 60 meters wide in the smaller tributaries to up to 1.8 kilometers long in Port Royal Sound.

6.2.2. Bathymetry

The model bathymetry is based on an assembly of available bathymetry data. The data include GEODAS data from NOS and USC&GS surveys, and the data also include USACE survey bathymetry. Where overlaps occurred in data coverage, only the most recent data were used. The data were all converted to a common datum (i.e., NAD83 and NGVD29) and interpolated onto the model grid. The model bathymetry grid is shown in Figure 6-2.

6.2.3. Tidal Forcing

The tide height, or free surface elevation of the water, cycles from low to high tide approximately every 12.4 hours. This variation is caused by astronomical (i.e., the moon and sun) periodicity as well as regional scale basin harmonics. The NOS maintains a series of tide gauges along the coast including a station at Fort Pulaski. A time series of surface elevation recorded at this station was used to simulate the offshore water surface elevation variations. The original time series was phase shifted slightly to allow for the lag between the model offshore boundary and the actual location of the tide gage. The shift was performed incrementally until a match was obtained between the model elevation predictions at Fort Pulaski and the gage data.

The time series of tides at Fort Pulaski measured during December 1999 is shown in Figure 6-3. The tide appears very regular over the time period shown. The major variation is the spring/neap cycle that occurs roughly biweekly; this variation directly affects the strength of the tidal currents.

Other factors that cause variation in the free surface are the winds and atmospheric pressure. The frictional effects of winds tend to cause setup as water piles up along a coast, and setdown as winds push water offshore. Atmospheric pressure acts to depress the water level (i.e., due to high atmospheric pressure) or raise the water level (i.e., due to low atmospheric pressure). Both wind and pressure effects cause storm surges that can be large enough to overwhelm the tidal signal. Therefore, the best open boundary tide condition is to use is an actual time series measurement that includes all of these effects. With this approach, the model uses real data for the offshore forcing.

6.2.4. River Flow

The primary source of freshwater to the Lower Savannah River Estuary is from the Savannah River watershed. The upstream model boundary was placed at Clyo where the USGS maintains a stage height monitor. Detailed records, both past and present, are available for river flow past this site. Daily volume flow records were obtained from the USGS for the December 1999 period and used as model input. Figure 6-4 shows the discharge (volume flow rate) for Clyo during December 1999. This data was used as the upstream river boundary condition. This flow volume is small compared to the tidal flux through the Savannah River entrance, and hence, the freshwater flow has only a small influence on current velocities at the river entrance and offshore areas.

6.2.5. Marsh Storage

The hydrodynamic portion of the WQMAP modeling system incorporates the ability to simulate the inflow and outflow of water and mass to marsh areas that flood and dry over the tidal cycle. The following presents the formulation of the equations, assumptions and terms within this model update.

Marsh areas within WQMAP are treated as stand alone storage units connected to the model domain through small tributary feeder creeks. These creeks are representative of the geomorphology found within the coastal region of the southeastern United States. At the end of the creeks, model boundaries are defined similar to existing river inflow boundaries within the WQMAP modeling system. For each of these marsh boundaries the following physical parameters are specified:

- Marsh Surface Area
- Front Marsh Elevation
- Back Marsh Elevation
- Manning Coefficient

The front and back marsh elevations are used to allow the marsh to fill gradually; i.e. the surface area filled is a function of the elevation of the waters entering the creek. This is typical of coastal marshes that fill through feeder creeks passing through berms along the edge of the marshes. These berms, which separate the marshes from the main channels, are generally of a higher elevation than the areas behind them. Therefore, the marshes tend to fill through the creeks behind the berm areas. The inundated area gradually increases as the water surface elevation increases. The Manning coefficient is used to quantify the roughness due to vegetation and the bed surface. The length is defined as the ramp distance from the tributary feeder bank to the point at which the elevated marsh area is level. This term plays a role in the sensitivity of the marshes.

To calculate the flow into and out of the marshes two cases are defined. One is where the elevation in the main channel is above the front marsh elevation. In this case a one-dimensional simple open channel flow equation is solved of the form:

$$Q_m = 1/n\ A_{cs}\ R^{2/3}\ I^{1/2}$$

where: Q_m = Time rate of inflow/outflow to the marsh,

n = Manning's n value,

R = Hydraulic radius of flow area,

I = Surface slope between the open water and the boundary cell, and

A_{cs} = Cross-sectional area of flow through.

For the case where the water surface elevation in the main channel drops below the front elevation of the marsh, the flow is treated as a weir flow and the following equation is solved:

$$Q_m = C_d\ W_L\ (2g)^{1/2}\ (E_c - E_m)^{3/2}$$

where: C_d = Drag Coefficient (0.577),

W_L = Width of the flow,

g = Acceleration of gravity,

E_c = Water surface elevation in the channel, and

E_m = Water surface elevation in the marsh.

The flow into the marsh areas is then adjusted through manipulation of the friction factor within the flow equation. The surface slope term in each of the equations provides the feedback. For each time step the flow rate is calculated. The volume of water inside the marsh is then adjusted by multiplying the inflow rate by the time step. The surface elevation is then adjusted by the surface area of the inundated marsh.

6.3. Hydrodynamic Model Calibration

The calibration process included the iterative adjustment of the friction coefficient (Manning's n) until good agreement between measured and simulated flows at the major inlets was obtained. A global friction value of 0.03 was found to produce the best agreement between measured and simulated flow values. A time step of 15 minutes was used.

Figure 6-5 shows the agreement between the measured and simulated water surface elevations at Fort Pulaski. Figures 6-6 through 6-8 present the comparison of measured and simulated flow values. The results show good agreement of both amplitude and phase between the measured and simulated values.

Figures 6-9 through 6-13 present the comparison between measured and simulated current velocities. The measured data are averaged over short periods of time (1 minute), whereas the model simulations are averaged over a longer period of time (15 minutes). The measured data are influenced by short period fluctuations in direction and magnitude, and it is not expected that the simulated data will exactly match the measured data. Overall, the comparisons are in general agreement. Comparisons of direction are poor during low current conditions when the currents are changing directions. More importantly, during strong current conditions the direction and magnitudes match reasonably well.

6.4. Hydrodynamic Model Results

The hydrodynamic model was used to compare currents for three bathymetric conditions: the pre-deepening project conditions, the post-deepening project conditions, and the 1854 conditions. The post-deepening grid was created by increasing the model grid depths within the navigation channel to a depth of –50 feet MLW (Figure 6-9), which is the maximum depth alternative for the project.

The 1854 model grid required modification of the grid at the Savannah Entrance and adjustment of the grid alignment along the study area shorelines. The 1854 bathymetry data set is restricted to areas offshore, so data was digitized from an 1855 chart to provide the depths and shorelines within the Savannah Estuary. The combined 1854/55 bathymetry data, which had been adjusted to the NGVD datum, was interpolated onto the grid (Figure 6-

10). For the 1854 simulations, the same boundary forcing was used as in the other scenarios, except that the boundary water levels were lowered by 1.5 feet to reflect the local mean sea level elevation at the time.

The simulated ebb current field for an average tidal range is shown in Figure 6-11 for the existing conditions. The simulated flood current field is shown in Figure 6-12. The highest currents are generally within the navigation channel in the inner harbor and between the jetties, and also in Calibogue Sound near Braddock Point (at the south end of Hilton Head Island). The post-project currents fields are imperceptibly different from the existing condition current fields when plotted as a vector field. Therefore, additional plots of the post-project currents are not shown.

The 1854 simulated current fields are shown in Figures 6-13 and 6-14. These results show significant differences in comparison with the existing condition simulated currents. The Calibogue Sound currents are slightly higher in the areas south of Hilton Head Island for the 1854 scenario, which is a result of shallower depths in these areas. Similarly, the currents are slightly higher in the Savannah River entrance due to the shallower depths. The currents are significantly higher in the South Channel and Wright River entrances due to the increased flow through these areas. Most significantly, the ebb and flooding of the Savannah River entrance are not focused by the jetties and bar channel in the 1854 scenario, and therefore result in a wider pattern of high current velocities. Additionally, the ebb and flood currents were generally higher near the north end of Tybee Island in 1854.

The tidal prisms of the Savannah River were calculated using an M2 tidal forcing with a range of 6.9 feet (the average range at Fort Pulaski), and a river influx of 11,600 cfs (average measured flow at Clyo since 1962). The simulated tidal prisms are tabulated in Table 6-1. The simulations show a 33 percent increase in the North Channel ebb tidal prism and an 11 percent decrease in the South Channel ebb prism resulted from the changes in the Savannah Harbor between 1854 and the present (Table 6-2). The results also show that the deepening project will result in a 3 percent increase in the Savannah River ebb tidal prism and a 7 percent decrease in the South Channel ebb tidal prism. Additionally, the simulations predict that South Channel passed 20 percent of the tidal prism in 1854, presently passes 15 percent of the tidal prism, and will pass approximately 13 percent of the tidal flux post-deepening.

Black (1893) reported tidal prism values collected by Captain Carter for a survey submitted in 1890. Carter measured a total flood tidal prism of 1.85E+09 cubic feet, with 1.39E+09 cubic feet passing through the North Channel, and 4.63E+08 cubic feet passing through the South Channel (one-quarter of the volume passing through South Channel). Carter also measured a total ebb tidal prism of 2.57E+09, with 1.71E+09 cubic feet passing through North Channel and 8.57E+08 cubic feet passing through South Channel (one-third of the volume passing through South Channel). Black does not report the conditions (tidal range and river flow) that occurred during Captain Carter's tidal prism measurements. Therefore, the measurements reported by Black and the simulated values should not be directly compared.

Table 6-1 Simulated tidal prisms for Savannah River

	Existing tidal prism (ft^3)		Post-project tidal prism (ft^3)		1854 tidal prism (ft^3)	
	Ebb	Flood	Ebb	Flood	Ebb	Flood
North Channel	2.37E+09	2.03E+09	2.43E+09	2.11E+09	1.78E+09	1.54E+09
South Channel	4.08E+08	3.54E+08	3.79E+08	3.18E+08	4.57E+08	4.00E+08
Total	2.78E+09	2.39E+09	2.81E+09	2.42E+09	2.24E+09	1.94E+09

Table 6-2 Simulated percent change in tidal prisms for Savannah River

	Percent change from 1854 to existing conditions		Percent change from existing to post-project conditions	
	Ebb	Flood	Ebb	Flood
North Channel	33.10%	32.36%	2.69%	3.55%
South Channel	-10.67%	-11.46%	-7.11%	-10.12%
Total	24.16%	23.32%	1.25%	1.53%

6.5. Summary

The hydrodynamic model simulation results show that significant changes to the hydrodynamics of the Savannah River entrance have occurred since 1854. The current patterns show focusing resulting from the construction of the jetties and a general reduction in current speeds due to increased depths. The simulations also show that harbor changes since 1854 have also resulted in a 33 percent increase in the North Channel tidal prism and an 11 percent reduction of the South Channel tidal prism. Additionally, the deepening project will cause a small (3 percent) increase in the North Channel tidal prism, and 7 to 10 percent decrease in the South Channel tidal prism.

The results of the current simulations are used in Section 8 of this study to calculate sediment transport rates in the study area.

7. WAVE MODELING

A numerical wave model was used to investigate the wave environment in the Savannah area. The wave model simulates the refraction and diffraction of waves as they propagate from offshore areas over the model area bathymetric features and ultimately break upon the shorelines. The model was used to simulate wave propagation over the existing bathymetry, the historic 1854 bathymetry and the post-deepening bathymetry. This section provides a description of the wave model, the wave model inputs, and the model results.

7.1. Model Description

Numerical modeling is an invaluable tool in simulating wave propagation in that it can provide an assessment of changes in the wave field induced by proposed bathymetry before the changes are instituted. Additionally, wave transformation modeling has gained importance in the last 25 years because it has reached a level of sophistication where it is now used in place of physical models in many cases (Work and Kaihatu, 1997).

Many different numerical models exist for wave propagation, and most can be divided into two broad classes: phase averaged and phase resolving (Kaihatu, 1997). Kaihatu (1997) described phase averaged models as expressed in terms of the evolution of a conserved quantity such as wave energy and these models include WAM (Wave model), SWAN (simulating Waves nearshore) and STWAVE. Phase resolving models are formulated in terms of the free surface and are derived from the governing equations of fluid mechanics (Kaihatu, 1997). REFDIF1 and RCPWAVE are examples of phase resolving models. Kaihatu further explains that, on an operational basis, many of the energy resolving models are valuable for propagating waves over global distances, however, their formulations do not include the fine-scale propagation effects that strongly influence the wave field in the nearshore and coastal areas (1997).

The numerical wave propagation model REF/DIF1 (Kirby and Dalrymple, 1993) was used to predict potential project impacts on the nearshore wave environment for this study. REF/DIF1 is a weakly nonlinear parabolic refraction and diffraction model. It was chosen to use in this study because its formulations include the fine-scale propagation effects that strongly influence the wave field in the nearshore region (Kaihatu, 1997). Hsu *et al.* (1997) consider REF/DIF1 to be the most complete model for combined refraction and diffraction computation. Also, REF/DIF1 includes the effects of wave diffraction, unlike STWAVE and SWAN (Kelley et al., 1999).

Additionally, Kirby (1997) performed a study to assess the effectiveness of REF/DIF1 on the pre-project and post-project bathymetries. In the study, Kirby modeled the Savannah River Channel as a rectangular trench with a width of 500 feet, and two depths of 44 and 54 feet. Reflection and transmission coefficients for both trench depths and for a range of wave periods from 4 to 10 seconds, and angles of incidence from 0 to 90 degrees were computed. Additional information about this study can be found in Appendix B. In summary, Kirby (1997) found that REF/DIF1 predictions are consistent with more detailed theory and REF/DIF1 effectively simulates the wave conditions local to the Savannah River Channel area.

The weak nonlinearity of REF/DIF1 results from a Stokes perturbation expansion, in which wave heights are calculated to the second order while wave phase speed is corrected to the third order (Kirby and Dalrymple, 1986b). Stokes theory is not accurate for shallow water conditions, so a Hedges dispersion relationship is incorporated into REF/DIF1. Kirby and Dalrymple (1986a) combined the Stokes and Hedges theories to cover the range of deep to shallow water. The Stokes-Hedges hybrid solution was used in the REF/DIF1 calculations for this study.

REF/DIF1 is based on the parabolic mild slope equation that includes two major assumptions. The first is Berkhoff's mild slope equation, which is so named because it assumes a slowly varying bathymetry. However, Booij (1983) found that for bottom slopes up to 1:3 the mild slope model was accurate and for steeper slopes, it still predicted the trends of wave height changes and reflection coefficients correctly. The bathymetry utilized in this study can be characterized as consisting of mild slopes and therefore does not pose a problem.

The second assumption involves the parabolic approximation, or Radder's approximation, of the mild slope equation that reduces a boundary value problem to an initial value problem where only the initial condition and the two lateral conditions need to be specified (Kaihatu, 1997). One important consequence of the parabolic approximation is that waves can only propagate in the forward direction. Therefore, slightly reflected waves are included while waves reflected into the opposite direction (obtuse angle reflection) of wave propagation are neglected. Due to wave characteristics and channel depth, obtuse angle reflection from the channel will be minimal and is justified in being neglected.

The addition of a current field into REF/DIF1 is an important option for this study. REF/DIF1 has the ability to handle strong currents and transform them nonlinearly. The interaction of tidal currents and waves are important in the Savannah region because of the large tidal range and significant tidal currents. Other model preferences such as open boundary conditions and bottom boundary damping were implemented in this study.

7.2. Model Inputs

WaveMap2000 software was used in applying and executing the REF/DIF1 program. WaveMap2000 employs a geographic information system (GIS) interface where GIS bathymetries and REF/DIF1 grids are easily intermeshed. Currents were directly imported from the WQMAP hydrodynamic model simulations.

The necessary inputs to the model simulation include the study area bathymetry, the input wave characteristics and currents. These model inputs are described in the following three sections.

7.2.1. Bathymetry

The area studied extends approximately 42 statute miles alongshore, from Port Royal Sound to Wassaw Sound. The area also extends approximately 13 statute miles offshore of Tybee Island. The REF/DIF1 grid had a 70-meter resolution with 978 cells along shore and 426 cells across shore. The grid is large enough that any lateral boundary effects will not have an effect on the model solution in the area of interest (from Hilton Head Island to Little Tybee Island). Figure 7-1 illustrates the model grid area. Within this area, three bathymetric conditions were defined: existing conditions, post-project conditions characterizing the proposed deepening of the navigation channel, and 1854 conditions.

The existing bathymetry was defined by combining various survey data sets collected between 1934 and 1997. The data for the existing bar channel geometry were taken from a survey conducted by the USACE in May 1997. The 1997 USACE data were collected by fathometer and a tide staff located near Fort Pulaski provided tidal elevation. The remaining bathymetric data used in the model were taken from National Geophysical Data Center coastal relief model. This data is based on hydrographic surveys conducted by the National

Ocean Service between 1934 and 1980. However, the 1934 data is restricted to areas outside the study limits (Wassaw Sound) and areas sheltered from waves (Calibogue Sound behind Hilton Head Island). All other survey data do not date prior to 1971.

An inverse-distance technique was used to interpolate the random survey data onto the model grid. Figure 7-2 presents a plot of the pre-project data set used for the model study. The different colors represent a range of water depths, as indicated in the legend. Blue is the deepest water and red is above water, with all depths referenced to NGVD (National Geodetic Vertical Datum).

The post-project bathymetry was defined by the removal of all sediment above –50 ft MLW depth within the proposed bar channel boundaries. The 1854 conditions used the bathymetric data digitized from an 1854 hydrographic survey chart as described in Section 4. The data were combined with the existing bathymetry data set such that existing bathymetry data was used for areas outside of the 1854 survey limits. This includes areas north of the south end of Hilton Head Island, south of Tybee Island, inside the Savannah Harbor and far offshore. The bathymetry data set, which was referenced to NGVD, was offset by 1.5 feet to represent the lower mean sea level that existed in 1854. The data were interpolated onto the wave model grid to create a bathymetry grid representative of the 1854 conditions (Figure 7-3). The bathymetry landward of the 1854 shoreline location should be ignored (e.g., streams, rivers, and upland areas) as these are outside of the area of interest and are irrelevant to this wave study.

7.2.2. Wave Climate

Wave Data were obtained from a 20-year hindcast (1976-1995) performed by the US Army Corps of Engineers (USACE) Wave Information Study (WIS) for the U.S. Atlantic Coast at Station 33, located approximately 16 miles east of Savannah at 32.00 N 80.50 W, and in approximately 43 feet of water. The WIS data set used a time series that provides significant wave height, peak period and peak direction at three-hour intervals throughout the 20-year period from 1976 to 1995.

The WIS wave data was bin-sorted in 10-degree increments and an average offshore wave period, height, angle and frequency of occurrence were computed for each. Table 7-1a summarizes the five bins used for computations while Table 7-1b lists all bins from WIS data. These five bins comprise greater than 80 % of all onshore-directed waves. REF/DIF1 was executed for the five wave conditions using both pre- and post-project bathymetries as well as for 1854 conditions.

Storm conditions were also considered for this study. A Fisher-Typett type I extreme value distribution was applied to the 20-year WIS data set in order to obtain wave heights for storms with 10, 50 and 100 year return periods. The wave period associated with each extreme wave height determined by selecting the period that most commonly occurred in the WIS record during the extreme wave height conditions. As a general rule-of-thumb, wave heights can be extrapolated to return periods up to three times the length of record (Leenknecht et al., 1992). It should be noted here that the 100-year return period values are less reliable as a result of extrapolating beyond three times the data record (60 years).

Storm conditions were sorted into four bins comprising waves from the south, southeast, east, and east-northeast. Table 7-2 summarizes the five bins used for computations. Each storm direction was given an equal percent occurrence except the northeastern conditions because of the limited range which storms from the northeast occur, i.e., the bin size is half that of the other bins.

Table 7-1a Simulated average wave conditions

	Bin Direction Limits (degrees)	**Sig. Wave Height (meters)**	**Peak Period (seconds)**	**Average Wave Direction (degrees)**	**Occurrence (percent)**
Case 1	90-100	0.9	7.3	94.8	5.37
Case 2	100-110	0.9	8.2	104.7	13.3
Case 3	110-120	0.8	7.8	114.9	29.86
Case 4	120-130	0.8	6.9	123.6	8.03
Case 5	130-140	0.9	6.7	132.6	4.92

Table 7-2b WIS data in 10-degree increments

	Bin Direction Limits (degrees)	Sig. Wave Height (meters)	Peak Period (seconds)	Average Wave Direction (degrees)	Occurrence (percent)
	0- 10	0.9	3.7	4.4	0.78
	10-20	0.9	3.6	13.9	0.91
	20- 30	1	4	25.6	0.88
	30- 40	1.2	4.7	34	0.8
	40- 50	1.2	4.8	43.6	1.49
	50- 60	1.3	5	54.1	1.69
	60- 70	1.3	5.2	64.7	1.95
	70- 80	1.2	5.2	75.9	1.96
	80- 90	1	5.5	84.7	1.65
Case 1	**90-100**	**0.9**	**7.3**	**94.8**	**5.37**
Case 2	**100-110**	**0.9**	**8.2**	**104.7**	**13.3**
Case 3	**110-120**	**0.8**	**7.8**	**114.9**	**29.86**
Case 4	**120-130**	**0.8**	**6.9**	**123.6**	**8.03**
Case 5	**130-140**	**0.9**	**6.7**	**132.6**	**4.92**
	140-150	1	6.6	143.8	2.53
	150-160	1.2	5.7	154.2	1.55
	160-170	1.3	5.7	165.8	1.24
	170-180	1.3	5.7	174.7	0.96
	180-190	1.2	5.4	183.5	1.46
	190-200	1.2	5.2	194	1.09
	200-210	1.2	4.9	205.8	0.92
	210-220	1.2	4.9	213.7	0.75
	220-230	1.1	4.7	223.5	1.46
	230-240	1	4.5	234.2	1.28
	240-250	0.9	3.9	244.9	0.91
	250-360	1.05	4.0	304.6	12.24

Table 7-3 Simulated storm wave conditions

Average Wave Direction (degrees)	Sig. Wave Height (meters)	Peak Period (seconds)	Sig. Wave Height (meters)	Peak Period (seconds)	Sig. Wave Height (meters)	Peak Period (seconds)
	10-year Return Period		50-year Return Period		100-year Return Period	
67.5	3.42	8.0	3.89	9.5	4.09	11.0
90.0	5.17	12.0	6.33	15.0	6.83	17.0
135.0	5.94	15.0	7.60	19.0	8.30	21.0
180.0	3.82	10.0	4.61	12.0	4.94	13.0

7.2.3. Currents

Current Data were obtained from the hydrodynamic model simulations for the Savannah River and the nearshore coastal region about the mouth of the Savannah River. Imported current data were then interpolated onto the entire wave model grid. An example of imported current data can be viewed in Figure 7-4.

Typical spring tide current conditions were input into the REF/DIF1 model to test the model sensitivity to currents in the study area. These results did prove significant in altering the wave field and can be viewed in Figure 7-5, which shows waves simulations with and without peak ebb current effects. Note that the wave vector plot shows a vector for only 1 out of 5 grid cells in each direction for legibility.

Consequently, the current fields in the area were included in model simulations performed for sediment transport potential calculations. Average ebb, flood, high slack and low slack current conditions were run for each of the 5 different wave conditions for both pre- and post bathymetric conditions. Each tide condition was given equal weight in order to represent the semidiurnal tidal cycle.

7.3. Model Results

The wave refraction and diffraction patterns calculated by the REF/DIF1 model provide insight into the effects of the bathymetric features on the waves that reach the shoreline in the study area. Figures 7-6 through 7-10 present the wave model results for each average wave condition without an input current field. The figures show wave vectors superimposed on the modeled bathymetry. The wave vectors are graphically displayed as an arrow to quantify wave height and wave direction. The length of the wave vectors is proportional to the wave height, and the wave direction corresponds to the orientation of the arrow. A reference vector equivalent to a 1.0-meter wave height for mean conditions is shown in the lower left hand corner of each figure. Wave vectors are plotted for only one out of seven grid cells in each direction; the data would otherwise be too dense to provide a legible plot. It should be noted that these wave vectors have been smoothed to reduce the discretized effects of the model and to more easily view wave transformation trends.

To understand the effects of the deepening on the local wave environment, it is first important to understand the wave transformation processes that are affected. Reflection commonly occurs when waves encounter protruding barriers. In more general terms, wave reflection occurs when waves encounter abrupt depth transitions; therefore, the Savannah bar channel represents a reflective barrier where waves travel from shallower to deeper water over a short time period. Waves leaving the channel also encounter a reflective barrier in which they travel from deeper water to shallower water. It should be noted that waves that encounter a barrier perpendicularly will reflect more than waves that approach a barrier at an oblique angle.

Refraction is another aspect of wave transformation on which the bar channel will have an effect. Refraction is a result of the fact that waves in different depths travel at different speeds. Waves in the channel will travel faster than waves outside the channel where it is shallower. This results in the bending or refraction of wave crests. Convergence or divergence of wave energy can result from both reflection and refraction and wave energy is an important factor when considering sediment transport. It should also be noted here that waves that travel through a channel will refract more than waves that approach the channel perpendicularly.

Refractive and reflective processes occur simultaneously and it is often difficult to separate their effects. Although REFDIF1 does not explicitly model reflection of submerged trenches, the model often captures the reflective effects of submerged trenches through its refractive modeling. Kirby goes into this in greater detail in Appendix B.

Given these general descriptions of refraction and reflection, and considering the project deepening, one can draw conclusions about the expected project-induced changes to the wave environment:

- Waves originating from the east will primarily refract, resulting in wave height attenuation in the channel and an increase in wave height outside the channel;
- Waves originating from the northeast will refract and reflect and wave impacts to Tybee Island will be reduced;
- Waves originating from the southeast will refract and reflect and some increased wave energy may reach Tybee Island;
- Waves originating from the south will primarily reflect and should not affect Tybee Island because the proposed channel modifications will be upwave;
- The channel may protect Tybee Island from waves approaching from the northeast
- The channel may increase wave heights that encounter Tybee Island from eastern approaching waves.
- The channel will most likely have no effects on wave heights that encounter Tybee Island from southeastern and southern approaching waves.

Intuitively, it is impossible to determine if these effects are significant or not. The use of the numerical model enables the evaluation of the relative significance of these project related impacts.

7.3.1. Average Wave Conditions

The model results for the average wave conditions are shown in Figures 7-6 through 7-10. Figure 7-6 presents the Case 1 (95 degree) wave condition model results. Near the seaward bend in the navigation channel, the wave vectors reflect dramatically and wave heights exceed 1.0 meter. West of the channel is a "shadow zone" of reduced wave height caused by the reflection. This demonstrates the significant role the channel plays in affecting waves that break on Tybee Island.

It should be noted that REF/DIF1 does not calculate meaningful solutions for wave *direction* in areas where wave trains cross. This occurs only in Calibogue Sound, in the lee of Barrett Shoals, the simulated wave vectors show scattered wave directions, even after smoothing. For this reason, comparison plots of overlain vectors for two simulation scenarios (e.g., pre- and post-deepening scenarios, or with and without currents as shown in Figure 7-5) can appear confusing and difficult to interpret. REF/DIF1 does, however calculated reliable wave *heights* for these conditions. Therefore, plots of change in wave height were used to evaluate the simulated changes between the three bathymetry scenarios: existing, post-deepening project, and 1854 conditions. Also, the simulated wave height and direction are used in Section 8 for Tybee Island shoreline sediment transport calculations; however, because the simulated wave vectors do not cross along the Tybee Island shoreline, this issue is not a problem.

Figure 7-11 presents the change in Case 1 wave height resulting from the channel deepening. This plot shows the change in wave height as a percentage of the input offshore boundary wave height: $\Delta H = (H_{post\text{-}project} - H_{pre\text{-}project}) / H_{input} * 100$. This allows for the visualization of the simulated relative changes in the wave field. Figure 7-11 shows that the deepening results in greater refraction of wave energy by the navigation channel, as shown by the decrease in wave height propagating toward the south end of Tybee Island and the increased wave heights propagating toward Daufuskie Island.

Figure 7-7 represents Case 2 waves originating from 105 degrees for the existing conditions. Similar to Case 1 model results, the channel attenuates wave heights that reach north Tybee Island significantly. Figure 7-12 shows the change in wave height resulting from the proposed channel deepening. The results indicate an increase in refracted wave energy with only a small change in wave height along the Tybee Island shoreline, except at the north tip, which shows a 10 percent reduction in wave heights.

Figure 7-8 show the model results for Case 3 (115 degree) conditions for the existing bathymetry. Case 3 is the most important case in that it represents almost 30 percent frequency of occurrence. Figure 7-13 shows the simulated change in Case 3 wave heights resulting from the channel deepening. These results show only small changes in the wave heights along Tybee Island.

Figure 7-9 represents Case 4 waves originating from 125 degrees for the existing bathymetry. The results show the refraction of wave energy by the seaward length of the channel (Tybee Range) toward Tybee Island. Figure 7-14 shows that the deepening results in only small changes in wave height along Tybee Island.

Figure 7-10 presents the Case 5 (135 degree) wave condition model results for the existing bathymetry. Similar to Case 4, the results show a refraction of wave energy toward Tybee Island. Figure 7-15 shows that the deepening results in an increase in wave energy propagating towards the south end of Tybee Island.

In general, the model results for average wave conditions agree well with what one would expect based on simple wave transformation principles. Waves traveling in the channel experience significant wave height attenuation. Waves propagating from a more northerly direction are reflected by the navigation channel and result in wave attenuation near Tybee Island. Waves from a more southerly direction result in larger waves along Tybee Island than other directions. Wave refraction due to the channel is similar for both pre- and post-project conditions with some small changes (generally less than 10 percent). The net effect of these changes on coastal erosion is evaluated by examining the sediment transport potential, which is presented in Section 8.

7.3.2. Storm Wave Conditions

Storm waves were simulated for the existing and post-project model grids. The change in 10-year storm wave heights were calculated and plotted in Figures 7-16 through 7-19 for waves approaching from the northeast, east, and southeast, respectively. The 50 and 100-year storm plots show very similar patterns as those seen in the 10-year storm wave plots and are not presented herein.

For the northeast and east storm wave conditions (Figures 7-16 and 7-17), the channel deepening results in greater wave refraction and decreased wave energy to the west of the channel. The results generally show reduced storm wave heights along Tybee Island; however, only small changes in wave height (less than 5 percent) are evident along the study area shorelines.

The southeast wave condition model results (Figures 7-18) show an increase in the refraction of wave energy toward Tybee Island and decrease in wave energy to the north of the channel. However, for both this case and the south storm wave case (Figure 7-19), little change in wave height is evident along the study area shorelines.

7.3.3. 1854 Wave Conditions

Figures 7-20 through 7-24 present the REF/DIF1 simulation results for the five wave conditions for the 1854 bathymetry. These plots supply historical insights into wave transformation before the offshore channel was dredged. Note that the shoal locations on Barrett Shoals have moved, and the shoal feature seaward of Tybee Island is much larger in the 1854 bathymetry than present day conditions. Additionally, the Tybee Island seaward shoreline faces a more southeast orientation in 1854.

Figures 7-25 through 7-29 show the simulated change in wave heights between the 1854 and existing conditions. These plots show the resulting change in wave heights due to the movement of Barrett Shoals, the dredging of the bar channel, the increase in sea level and the "deflation" of the shoal seaward of Tybee Island.

Figure 7-20 presents the Case 1 (95 degree) wave condition model results. As compared to the existing conditions shown in Figure 7-6, the wave vector heights approaching Tybee Island are much more uniform. The shoal off Tybee Island creates a wave focusing on the shoreline due to refraction effects, and wave focusing on Barrett Shoals occurs at different locations due to the movement of the shoals. Figure 7-25 shows the simulated change in Case 1 wave height between the 1854 conditions to the existing conditions. For Case 1 wave conditions, the dredging of the navigation channel has resulted in a dramatic decrease in wave height near the middle of Tybee Island while resulting in a similar increase in wave height near the south end of the island.

Figure 7-21 represents Case 2 waves originating from 105 degrees. Similar to Case 1 model results, wave heights are more uniform along the Tybee Island shoreline than for the existing conditions. Also, similar to Case 1, Figure 7-26 shows a reduction in wave height to the west of the navigation channel. The plot shows a corresponding reduction in wave height along the northern portion of Tybee Island and an increase in wave heights near the south end of the island.

Figure 7-22 shows the model results for Case 3 (115 degree) conditions. Again, Case 3 is the most important case because it represents almost 30 percent frequency of occurrence (assuming that the wave climate is similar to that hindcast for the period 1976-1995). In comparison with the existing conditions (Figure 7-8), the 1854 waves generally approach the shoreline at a more northerly angle. As seen in Figure 7-27, the existing bathymetry simulations result in decreased waves at the north end of Tybee Island and increased wave heights at the south end of the island, as compared to the 1854 conditions. The increase in wave energy at the south end of the island appears to be the result of the change in the shape of the shoal seaward of Tybee Island. Whereas in the 1854 conditions some waves approaching the shoal continue propagating toward the north end of Tybee Island, the existing bathymetry causes these waves to refract westward toward the south end of Tybee Island. Figure 7-23 represents Case 4 waves originating from 125 degrees for the 1854 conditions, and Figure 7-28 presents the change in wave height for Case 4. Similar to Case 3, Figure 7-23 shows a more northerly direction along the Tybee Island shoreline in the 1854 conditions, as compared to the existing conditions (Figure 7-9). Figure 7-28 shows that changes since 1854 resulted in a reduction in wave height along the northern portion Tybee Island and an increase in wave heights along the southern two-thirds of the island.

Figure 7-24 presents the Case 5 (135 degree) wave condition model results. As with the other four wave cases, a more northerly wave direction along the Tybee Island shoreline is apparent. Additionally, this case shows wave focusing at the north end of the island. Figure 7-29 shows the change in wave height between the 1854 and existing conditions for the Case 5 waves. This plot shows a general increase in wave height along the Tybee Island shoreline.

7.3.4. Summary

The wave model results show that significant changes have occurred to the wave field between 1854 and the present. In particular, the changes in bathymetry have caused a dramatic change in the wave environment incident to the Tybee Island shoreline. The model results indicate that more northerly transport is expected along the Tybee Island shoreline for 1854 conditions as compared to the existing conditions (assuming the same wave conditions for both cases). The results show that wave refraction caused by the presence of the navigation channel has resulted in a general reduction in wave heights along the north to central shoreline. A reduction in wave shoaling because of deeper depths along the northern portion of the ebb shoal could also play a significant role in decreasing wave heights in this region. Along the central to southern shoreline, the change in the shape of the shoal seaward of Tybee Island has resulted in wave focusing and a general increase in wave heights.

The comparison of model results for the pre- and post-deepening conditions show that the deepening will result in changes to the wave transformation properties of the channel. These changes cause increases and decreases in average wave height along the island that are generally less than 10 percent. In comparison to the historic changes in wave energy along the island, these changes are small. The net effect of these changes on the shoreline will be evaluated in Section 8 by calculating the sediment transport potential. The simulated storm wave conditions show that the deepening project results in general reduction in the wave heights along the Tybee Island shoreline.

8. SEDIMENT TRANSPORT

In this section, the data and analyses from the previous sections of this report are used to evaluate sediment transport processes in the study area. The results of the wave model simulations are used to determine the longshore sediment transport potential along the Tybee Island shoreline. The wave and current simulations are used to determine sediment transport over the region between Hilton Head Island and Tybee Island. Lastly, a sediment budget is developed and the expected change to the sediment budget from the proposed deepening is determined.

8.1. Longshore Sediment Transport Potential

This analysis evaluates the project related impacts to the sediment transport processes along the Tybee Island shoreline. Based on changes to the sediment transport processes, conclusions can be made regarding the project induced impacts to shoreline stability in the area.

The approach used to evaluate the project related impacts to shoreline sediment transport processes was to use REF/DIF1 to transform each wave to breaking condition and then compute the longshore energy flux factor, P_{ls}, from the breaking wave heights and angles. The longshore energy flux factor is:

$$P_{ls} = \frac{\rho g}{16}(H^2 C_g)_b \sin(2\theta_{bs})$$

where H = wave height, C_g = wave group speed, b = subscript denoting wave breaking condition, ρ = density of water, g=acceleration due to gravity, Υ_{bs} = angle of breaking waves to the local shoreline. The potential longshore sediment transport rate is commonly correlated with the longshore energy flux factor. In fact, most longshore sediment transport rate formulations are directly proportional to the longshore energy flux. Therefore, the simulated trends in longshore energy flux is indicative of trends in sediment transport, and simulated relative changes in longshore energy flux between pre- and post-deepening scenarios are indicative of the expected relative changes in sediment transport rate.

The wave height, wave group speed, and angle of breaking waves were calculated from the REF/DIF1 output for all wave cases. The protocol used in finding the breaking wave height was to find the largest wave height cell within 200 meters of the shoreline for average waves (within 300 meters for storm waves). Shoreline angles for each cell along Tybee Island were also calculated. Using the above equation, a longshore energy flux was calculated for each cell along Tybee Island from the north terminal groin to the south terminal groin.

For the average wave conditions, the REF/DIF1 simulations that included current effects were used. These simulations included current fields of: peak ebb current, peak flood current, high slack tide and low slack tide. The net longshore energy flux was calculated for each wave direction by averaging the longshore energy flux for each of the four current conditions (each current condition was given equal weighting). These discretized values were then smoothed to more realistically simulate long-term average transport trends.

It should be noted that the longshore sediment transport potential analysis presented here only evaluates the transport due to breaking at the shoreline. Although the longshore current effects from wave refraction and wave-generated radiation stresses are included, the longshore current effects from wind and tides are not included. Inlet current effects are important to the longshore sediment transport rates near the ends of the island, but the hydrodynamic modeling has shown that the deepening project will not result in any significant changes to the current field. Therefore, the longshore sediment transport analysis focuses on the relative changes resulting from changes to the incident wave field along the island shorelines.

8.1.1. Average Wave Conditions

The longshore energy flux is presented in Figures 8-1 and 8-2 for pre- and post-project conditions. Figures 8-1 and 8-2 present each average wave condition and a composite of all average wave conditions while Figure 8-3 is the composite plotted next to plan view of Tybee Island in order to more clearly visualize sediment transport trends. The composite average was computed by multiplying each longshore energy flux by the percent of annual occurrence (listed in Table 7-1) and represents the net longshore energy flux.

The Case 1 and Case 2 results shown in Figure 8-1 represent waves approaching from 95 and 105 degrees, respectively. The longshore energy flux is relatively low in Case 1 and 2, as compared to Case 3. This is the result of the sheltering effect of Barrett Shoals and the navigation channel for these wave directions. The post-project results show negligible change for Case 1. The post-project results for Case 2 also show an increase in southerly transport at the south end of Tybee Island and a reduction in northerly transport at the north end of the island.

Case 3 (waves approaching from 115 degrees) is an important case because it not only has the highest percent occurrence, but it also exhibits the highest sediment transport rates. The comparison of pre- and post-project results indicate that the deepening will increase southerly transport at the south end of the island and decrease transport in the area from 800 to 1,800 meters from the southern terminal groin.

The Case 4 results in Figure 8-2 represents waves approaching from 125 degrees. This wave case results in northerly transport over most of the island, with the exception of close to zero transport near the center of the island. The post-project results show not change over the northern half of the island, an increase in southerly transport over the middle of the island, and a decrease in northerly transport over the southern portion of the island.

The Case 5 (135 degree wave direction) results show general northerly transport over most of the island. The results indicate that the channel deepening will cause a general reduction in sediment transport for these conditions.

The composite case in Figure 8-3 is indicative of the net sediment transport patterns due to wave breaking. This plot clearly shows a divergent nodal point located near the center of the island. North of this point the net sediment transport is northerly, and south of this point the net sediment transport is southerly. The location of this nodal point is about 2,100 meters north of the south terminal groin (about the location of 6^{th} Street), which agrees well with the location calculated by Olsen Associates, Inc. (1998). The post-project results indicate that the deepening project will result in no significant change over the northern half of the island. The results also show a reduction in southerly transport over the region from 1,000 to 1,800 meters, and an increase in southerly transport from 0 to 1,000 meters from the south terminal groin.

Interpretation of the changes in sediment transport rate caused by the deepening project requires an understanding of the relationship between shoreline changes and sediment transport rates. Accretion or erosion of the shoreline is caused by changes in the sediment transport rate along the shoreline. For the sign convention shown in Figure 8-3, with southerly transport shown as positive and northerly transport shown as negative, a positive *gradient* in the transport rate results in accretion, and a negative gradient in the transport rate results in erosion.

The gradient in sediment transport potential, dP_{ls}/dx, is included in Figure 8-3, and it referenced to the y-axis on the right-hand side of the plot. Positive values indicate accretion and negative values indicate erosion. Measured volume changes along Tybee Island between 1995 and 1998 presented by Olsen Associates, Inc. (1998) indicate that the shoreline is erosional along the entire northern half of the island. However, measured shoreline change from the same study does indicate an increase in shoreline from the north terminal groin to approximately 1,700 feet south of this groin. Although measured volumetric change shows erosion, measured shoreline change, which does not use cell averaging, agrees with simulated results that there is some accretion to the updrift (i.e., south) of the north terminal groin.

Another factor that affects erosion and accretion patterns predicted by the longshore transport potential analysis may be explained, in part, by the fact that some of the erosion potential near the center of the island is offset to the north by the seawall that lines the Tybee Island shoreline. South of the seawall, measured and simulated longshore transport are in agreement. Both show slight accretion for the first 1,000 meters south of the seawall, while further south erosion is exhibited. Taking these factors into account, the analysis performs well and is useful for evaluating relative changes resulting from changes to the incident wave field along the island shorelines.

The pre- and post-project results in Figure 8-3 show minor shifting in the patterns of accretion and erosion. The results show that the deepening will cause a small shift of the nodal point northward. However, this will not change the erosion potential of this area since the gradient in transport remains relatively unchanged. Erosion or accretion patterns in the region from 0 to 600 meters will not change since the gradient in transport potential does not change significantly in this area. The post-project results show a change in gradient in the transport potential in the region between 600 and 1,400 meters that will cause an increase in erosion or decrease in accretion in this area. For the area from 1,000 meters to 1,800 meters the pre-project shows a gradual decrease in southward transport, which would result in accretion over this area. The post-project results indicate that this accretion would occur within the area from 1,400 to 1,800 meters, and the area from 1,000 meters to 1,400 meters would be stable. Accretion and erosion patterns on the northern half of the island would be relatively unchanged by the deepening project.

The average longshore energy flux was calculated for the shoreline north and south of the predicted nodal point. The average longshore energy flux values and the relative change resulting from the proposed deepening project are shown in Table 8-1. The results indicate that the channel deepening will cause an average 3.7 percent reduction in net sediment transport along the northern half of the island and a 7.8 percent increase in net sediment transport along the south end of the island.

The change in sediment transport at the ends of the island are shown in Table 8-2. The results predict that the deepening will result in a 9.6 percent reduction in wave induced northward transport at the north terminal groin location and a 9.2 percent increase in southward sediment transport at the south terminal groin location.

The average longshore transport potential gradients were calculated along the island and are presented in Table 8-3. The analysis predicts that for the northern portion of the island (between 6th Street and the north terminal groin) the channel deepening will cause a 3.2 percent increase in accretion at accretional areas, and an 7.7 percent increase in erosion at erosional areas. Measured data between 1995 and 1998 show that this area has eroded at an annual rate of 154,500 cubic yards per year. Therefore, a 7.7 percent increase in the erosion of this part of the island is representative of the expected change caused by the channel deepening. A 7.7 percent increase in this rate is equivalent to an increase of 11,900 cubic yards per year. Changes along the island south of the nodal point (between 6th Street and the south terminal groin) show the effect of the deepening by reducing erosion by 8 percent and decreasing accretion by 21 percent, for a net effect of a 3 percent reduction of erosion. So the erosion rate is decreasing (less erosion) while the accretional areas are still gaining volume, however at a reduced rate (less accretion, but still accreting). Measured data between 1995 and 1998 indicate that the net change rate for the area from 6th street to the south terminal groin is -17,200 cubic yards per year. A 3.1 percent reduction of the rate is 533 cubic yards per year, which is a negligible amount of material in comparison to the typical annual sediment transport rates occurring along the Tybee Island shoreline. Averaged across the entire island, the results show a net one percent increase in erosion.

Figure 8-4 shows the difference in the calculated average longshore energy flux along Tybee Island for wave simulations that include current interaction and simulations that do not include current interaction. The results show significant changes between the two simulations; most notably, the "without currents" simulations would have predicted northerly transport along part of the shoreline near the south end of the island.

Table 8-1 Predicted change to average longshore transport potential along Tybee Island

Location	Average P_{ls}		% change
	pre-project	post-project	pre-to-post
North of nodal point	-303	-292	+3.7%
South of nodal point	159	171	+7.8%

Note: Positive values are southerly transport, negative values are northerly transport

Table 8-2 Predicted change to longshore transport potential at ends of Tybee Island

Location	P_{ls}		% change
	pre-project	post-project	pre-to-post
North terminal groin	-194	-175	+9.6%
South terminal groin	348	380	+9.2%

Note: Positive values are southerly transport, negative values are northerly transport

Table 8-3 Predicted change to longshore transport potential gradient along Tybee Island

Location	Average dP_{ls}/dx (J/m^2-s)		% change
	pre-project	post-project	pre-to-post
North of nodal point	-0.117	-0.132	-12.7%
South of nodal point	-0.210	-0.204	+3.1%
Accretion areas north of nodal point	0.185	0.191	+3.2%
Erosion areas north of nodal point	-0.873	-0.940	-7.7%
Accretion areas south of nodal point	0.144	0.113	-21.4%
Erosion areas south of nodal point	-0.641	-0.589	+8.1%
Total along island	-0.173	-0.175	-1.3%

Note: Positive dPls/dx values are indicative of accretion, negative values represent erosion

8.1.2. Storm Wave Conditions

The calculated longshore sediment transport potential was calculated for each of the simulated storm wave conditions listed in Table 7-2. The results are presented in Figures 8-5 through 8-10. Since the storm wave conditions have a longer period than the average wave conditions, the channel deepening will alter the storm waves to a greater degree than the shorter period average waves. This is because of that fact that longer waves react to the channel depth as a shallower *relative depth* (depth divided by wave length) than shorter waves. Therefore, the relative changes in sediment transport potential along the Tybee Island shoreline are expected to be greater for the storm wave conditions than for the average wave conditions.

10-Year Storm Conditions

Figures 8-5 and 8-6 presents transport comparison plots for the 10-year storm wave conditions. The northeast waves cause very little transport along the northern half of the island due to the sheltering effects of Barrett Shoals. The waves cause southerly transport along the southern half of the island, and the results from the northeast condition indicate that the deepening will cause only minor changes in sediment transport patterns. The east storm conditions show a significant decrease in southerly transport near the center of the island, and a general increase along the northern half of the island. The southeast storm conditions show a large reduction in transport along the center of the island, and a large increase in transport at the extreme north end of the island. The south storm conditions show relatively minor changes to the sediment transport patterns.

50-Year Storm Conditions

Figures 8-7 and 8-8 present transport comparison plots for the 50-year storm wave conditions. Similar to the 10-year storm wave conditions, the northeast storm conditions show little change in examining pre- and post-project effects. The east wave storm conditions indicate that the deepening project will significantly decrease southerly transport along the southern half of the island, and increase northerly transport over the northern portion of the island. The results also predict approximately twice as much northerly transport at the north terminal groin location.

For both the south and southeast plots, dominant northerly transport is apparent, as expected. Both cases show shifts in patterns of erosion and accretion, with more substantial changes occurring in the southeast case. It is interesting to note the switch in patterns of erosion and accretion between the southeast and south wave cases.

100-Year Storm Conditions

Figures 8-9 and 8-10 present transport comparison plots for the 100-year storm wave conditions. The northeast conditions show minimal transport for pre- and post-deepening. The east conditions exhibit a decrease in northerly and southerly transport after deepening. An increase in northerly transport is evident in the southeast conditions between 2,000 and 3,000 longshore meters, which would result in greater erosion near the center of the island. The transport curve caused by southerly waves shows minor changes.

With regard to shoreline changes, the principal concern associated with storm events is the threat of drastic erosion. Therefore, the project related impacts to storm wave events were summarized by evaluating predicted relative changes in erosion rates. This is a more conservative indicator of storm wave impacts than net change, since a large erosion of one segment of shoreline may balance with a large accretion of adjacent shoreline to result in a small net change. The relative change in erosion was calculated by summing the negative values of the longshore energy flux gradient (dP_{ls}/dx) for pre- and post-project scenarios and each storm wave case. The results are shown in Table 8-4. The results indicate that the channel deepening may increase erosion by up to 30 percent (10-year storm wave from the southeast) and may reduce erosion by up to 49 percent (100-year storm wave from the east).

Figures 8-5 to 8-10 show trends in increasing potential longshore transport with bigger wave conditions for each directional case. Subsequent erosion and accretion patterns also arise; however, small variations within each wave case can have large impacts on erosive episodes and the table below is a conservative analysis of these erosive episodes.

Table 8-4 Predicted relative change in wave-induced erosion during storm occurrences

Storm Condition	Change in Erosion by Wave Direction			
	ENE	E	SE	S
10-year storm	+16%	-12%	+30%	-1%
50-year storm	-10%	+8%	-3%	+6%
100-year storm	+27%	-49%	+4%	-17%

Note: Positive values represent increases in erosion, and negative values represent decreases in erosion

8.1.3. 1854 Wave Conditions

Potential longshore transport potential was also performed for the 1854 wave model simulations. The 1854 conditions were modeled for two reasons: to assess sediment transport for conditions without major anthropogenic changes to the system, and to gain insight into the relative magnitude of change when comparing the pre- and post-project simulations. These results can be found in the appendix Figures A-18 to A-19. The significantly different bathymetry and shoreline orientation provide much different sediment transport patterns along Tybee Island. In general, more northerly transport is predicted for 1854 bathymetry conditions.

8.2. Offshore Sediment Transport Calculations

As demonstrated in the wave simulation results, the navigation project has a significant effect on the transformation of waves as they propagate across the nearshore bathymetry. This can cause changes not only in the sediment transport along the project area shorelines, but it can also cause changes in the transport of sediments in the offshore regions of the study area. This section evaluates the potential changes to offshore sediment transport processes by calculating the sediment transport potential across the study area using simulated wave and currents information. This assists identification of the main sediment transport pathways in the offshore region and it will also evaluate the *relative* change caused by the navigation channel deepening.

8.2.1. Theory

The majority of sediment transport occurs within the surf zone, where breaking waves are the major mode of transport. Inner surf zone transport modeling is commonly performed with a wave model and empirical sediment transport equations. This has already been conducted in the previous wave modeling section. Other programs such as GENESIS can also be run to estimate sediment transport in the surf zone. This was done for Tybee Island in a 1998 Olsen Associates, Inc. study. In these models, longshore currents (which are created by breaking waves) are the mode of sediment transport. Tidal currents can also be applied to the certain wave models such as REF/DIF1 in order to more accurately estimate transport within the surf zone, which was done for this study.

It has been estimated that 10 to 30 percent of total sediment transport occurs seaward of the breaker line (Walton, 1998). This transport of sediment by a steady, uniform flow outside of the surf zone has not yet been addressed. In an area where tidal currents are dominant factors, current dependent sediment transport should be studied further.

Many different sediment transport models that use currents as the primary mode of sediment transport and incorporate a large region are available. Some examples include Ackers-White, Einstein, Meyer-Peter y Muller, Van-Rijn, and Frijlink formulations. Additionally, transport equations by wave and current action over a large region are available from Bagnold, Bijker, Van-Rijn, and Englunde-Fredsoe.

The Ackers-White formulation modified by the Bijker formulation was initially chosen for this study in order to include both current and wave action over the entire wave model region. The Ackers-White equations predict sediment transport as a primary function of grain-size, depth, and depth-averaged velocity. The equations are valid for uniformly graded, non-cohesive sediment with a grain diameter ranging from 0.04 mm to 4.0 mm (White, 1972). The Bijker equations were included in order to reflect an increase in transport rate when ambient currents are accompanied by surface waves. Additional information can be found in the Scheffner (1996) paper that applies these equations to the USACE LTFATE program used to predict long term movement of dredged material placed in open water. In his research, Scheffner (1996) found that this modeling approach is capable of producing realistic simulations of sediment transport and bathymetric change.

The Ackers-White formulations include three dimensionless quantities. The first is a nondimensional grain size parameter. The second parameter represents particle mobility. This parameter includes depth-averaged velocity inputs as well as shear velocity calculations. The third parameter defines a sediment transport ratio. This parameter involves the calculation of a nondimensional sediment transport function in the form of mass flux per unit mass flow rate.

A modification of the transport equations was proposed by Bijker (1967) to increase the transport rate if ambient currents are accompanied by surface waves. This modification includes calculations of wave orbital velocity and amplitude at the bed.

The Bagnold formulation was the second sediment transport model chosen for this study. The Bagnold formulation is an energetics-based sediment transport model based on the idea that a portion of fluid energy is expended in maintaining a sediment transport load (Bagnold 1963, 1966). It assumes that the sediment is transported in two distinct modes: suspended and bedload transport modes. This model can predict cross-shore sediment movement under the combination of mean flow and asymmetric orbital velocities. For a domain with a variety of sediment sizes, it can reflect the change of grain size through the fall velocity. The Bagnold model also includes vertical beach gradients and bottom current velocity in its formulations.

8.2.2. Methodology

The wave grid used for the REF/DIF1 modeling was utilized for this study (see Figure 7-1). The grid was truncated to include only the region of interest between the south end of Hilton Head Island and the south end of Tybee Island due to the large computer files associated with using the entire wave model grid. The five average wave conditions that included four different current stages were used for the sediment transport calculations. Therefore twenty different wave cases were used.

Currents for the existing conditions, post-project conditions and 1854 conditions were output at 15 minute intervals over a typical tidal cycle from the hydrodynamic model and interpolated onto the sediment transport grid. Comparisons of the interpolated currents are shown in Figures 8-11 through 8-14. The vector plots show only one out of 10 grid cells in each direction for legibility. The 1854 currents (Figures 8-11 and 8-12) show significantly different orientation from the present conditions, as the currents were not constrained by the jetties. The pre- and post-project condition vectors are so similar that subtracting pre-project from post-project vectors was implemented. As seen in Figures 8-13 and 8-14 where peak ebb and peak flood vector differences are all less than 2 cm/s. Pre-project conditions exhibit larger slightly currents in both peak ebb and peak flood conditions. Some of the smaller vectors seen in the figures can be ascribed to numerical error.

Two different sedimentation models were used: the Ackers-White model and the Bagnold model. The Ackers-White formulations include three dimensionless quantities. The first is a nondimensional grain size parameter. The grain size data presented in Section 3 were interpolated onto the grid and used in preliminary sediment transport calculations. However, the equation showed too much sensitivity to this variation. Therefore, a regional average grain size of 0.26 mm was used for the entire grid. The Bagnold model did use a varying grain size. Grain sizes were regionally averaged over the grid and can be seen in Figure 8-15.

The sediment transport calculations over the model grid were performed at 15-minute intervals over a typical 12.42 hour tidal cycle. A different current condition was run with each of the five average wave conditions and for each bathymetry grid. The cases were averaged for the five wave cases, with each wave cases weighted by the percent occurrence. Therefore 50 current conditions were run with 5 wave conditions for 3 different bathymetries for a total of 450 runs.

8.2.3. Results

The sediment transport calculations are not calibrated to measured values. Therefore, the results of this analysis are useful only for predicting the *relative* changes in transport for the different scenarios. Prediction of the absolute magnitudes of the transport rates are not the objective of this analysis. The absolute sediment transport quantities are evaluated in the sediment budget analysis later in this section.

Figure 8-16 is a comparison of both models under peak ebb conditions. Both models exhibit similar trends however the Bagnold model predicts much larger transport in the wave sheltered regions such as north of Tybee Island, Savannah Channel, and Calibogue Sound. This agrees with observations that currents in these areas do move sediment.

Figures 8-17 through 8-19 present Bagnold sediment transport plots of the study area for typical ebb and flood conditions as well as the tidally averaged case. For the figures, the color convention remains unchanged in that black vectors represent pre-project while purple vectors represent post-project. Vector length is proportional to sediment transport. Transport vectors were smoothed to reduce discretization effects and to more clearly see transport trends.

The sediment transport patterns shown in Figures 8-17 and 8-18 illustrate that sediment transport is orders of magnitude larger in shallow areas that experience significant wave action than deeper areas or areas that do not have significant wave action. The major pathways of sediment transport occur along Barrett Shoals, the Daufuskie Island shoals (Grenadier Shoal and the New Inlet ebb spit), and the Tybee Island subaqueous platform to the southeast of the island.

The tidally averaged sediment transport plot in Figure 8-19 shows, as expected, much smaller sediment transport magnitudes than with the peak ebb and flood cases. A nodal area off Tybee Island is evidenced in this plot that agrees with measured shoreline data where net transport is to the north above this nodal area and south below it. The Daufuskie and Turtle Islands observations and measured data also agree with the basic trends shown in Figure 8-19.

The change in transport rates between the existing and post-project scenarios is presented in Figures 8-20 through 8-22. These plots show color contours of the change in transport along with vectors of the simulated existing sediment transport rates. The vectors are plotted only to show general direction of sediment. The results show that the changes in sediment transport rates are expected to be within plus or minus five percent of the existing maximum rates. Inspection of the plots indicates that no increases in transport rate will occur along the channel that will increase sediment trapping by the bar channel. In general, increased transport is evidenced in the channel, on the Tybee subaqueous platform, and on Barrett Shoals. Decrease transport is evidenced to the southern portion of the Tybee subaqueous platform as well as south of Hilton Head Island. Almost no change is evidenced off Turtle and Daufuskie Islands.

Figures 8-23 through 8-26 show similar plots for the comparison of 1854 and existing conditions (present condition bathymetry is contour plotted). The areas of greatest transport along Barrett Shoals are shifted in the two scenarios as a result of the southward shifting of the shoals over time. The orientation of the ebb tide sediment transport between New River Inlet and Barrett Shoals is more northerly in the 1854 conditions as a direct result of the current direction associated with the unjettied Savannah River entrance (Figure 8-23). The contours of change in transport display the reduction of transport offshore from the midpoint of Tybee Island. The increase in depth of the subaqueous platform in this area has reduced the transport rates by up to 100 percent. South of this area, the transport rates have increased dramatically due changes in wave refraction over the bathymetry and reflection by the navigation channel.

Comparison of the three scenarios illustrate that the changes induced by the channel deepening are an order of magnitude less than the changes in sediment transport that have occurred since 1854.

8.3. Sediment Budget

A sediment budget describes the movement and deposition of sand within a specified area, or control volume. The basic underlying principle is that a conservation of mass exists for sand in coastal regions (i.e., sand losses and gains can be accounted for).

Delineation of a control volume is the first step in establishing a sediment budget. Once this is performed, shoals, beaches, and inlets can be further discretized into workable units. Erosional and accretional areas can more easily be distinguished once these cells are established.

The control volume northern boundary is the south end of Hilton Head Island while its southern boundary is the south end of Tybee Island. The shoreline constitutes the western boundary while the eastern boundary includes all shoals and depths within the depth of closure. The depth of closure is located where no significant depth changes occur over times of engineering significance (typically, 10 to 50 years). This control volume was then broken down into 6 different cells, which include the major beaches, shoals, and inlets in the region. Figure 8-27 depicts the arrangement used for this study.

A number of assumptions were necessary to simplify this complex littoral system:

- The Savannah River source of littoral material is zero. Following the dam construction on the river and the regular maintenance dredging of the inner harbor, this is a valid assumption. The load carried by the river to the ocean is fine material that does not contribute to the littoral system.

- Impoundment rate by the jetties is zero. Small fillets have accreted on the north and south of the jetties. Significant quantities of littoral material are not likely to accumulate at the jetties since they do not obstruct any major sediment transport pathways, at present.

- There is no upland source of material that is transported south from the bay of Calibogue Sound.

- Littoral drift into Calibogue Sound from Daufuskie Island and Hilton Head Island is transported to the ebb shoal complex, thereby eliminating the necessity of including the Calibogue Sound flood shoal in the sediment budget.

- New River and Wright River are small enough sediment sources to be considered negligible as compared to the other sediment sources in the system.

- Offshore losses are typically calculated using the below equations:

$$\Delta y_{sl} = \frac{L_c}{B + D_c}$$

$$Q_{sl} = \Delta y_{sl} \Delta x D_A$$

where y_{sl} represents shoreline change due to sea level change, L_c is the cross shore distance from datum to depth of closure, B is berm height, D_c is depth of closure, and Q_{sl} is the resulting sea level induced sediment transport rate. For the Savannah area these numbers are comparatively small (i.e., less than 10,000 CY/yr) and are considered negligible.

8.3.1. Previous Sediment Transport Studies and Data

An important aspect in developing a sediment budget is an examination of previous studies conducted in the area. It should be noted that quantifying sediment transport over a region is difficult and that the following studies are estimates, and some estimates conflict with others.

HILTON HEAD ISLAND

Olsen Associates, Inc. (1992) estimated annual historical sediment losses at 164,000 cubic yards per year. A net southerly movement of sand is apparent on the southern half of the island and its estimated change in volume is a loss of 42,000 cubic yards per year to the south. A net northerly movement is apparent in the north and its estimated change in volume is 75,000 cubic yards per year lost to the north. Hilton Head Island has borrowed sediment from Joiner Bank, Barrett Shoals and upland sources to periodically renourish its beaches.

Olsen Associates, Inc. (1987) found that gross sediment transport across the headland in the center of the island was approximately 750,000 cubic meters per year.

Daufuskie Island

Applied Technology and Management, Inc. design notes from 1997 for a Daufuskie Island beach nourishment project measured historic change in volume at 50,000 cubic yards per year with approximately 40,000 cubic yards per year directed to the south.

Turtle Island

The WES analysis of South Carolina shorelines from 1853 to 1983 estimated Turtle shorelines as eroding 1.9 feet per year (Anders et al., 1990). No other useful historical studies were found for this island.

Tybee Island

Olsen Associates, Inc. (1998) estimates an annual loss of 172,000 cubic yards per year. This estimate was established from post-nourishment monitoring surveys conducted from 1995 to 1998 between the north and south terminal groins. Additionally, approximately 158,500 cubic yards per year were lost to the north end of the island and 20,500 cubic yards per year were lost to the south end of the island.

In earlier studies, the USACE (1970) estimated that transport to the north was 69,000 cubic yards per year while transport to the south was 181,000 cubic yards per year. Another study of Tybee Island by the Savannah USACE (1973) estimated normal southerly movement of littoral material at 498,000 cubic yards per year.

Shoals

Barrett Shoals, which is between Hilton Head and Daufuskie, is an important aspect of the littoral system because of its size, volume, and placement. Two borrow site excavations have occurred here in 1999. One excavation was for Daufuskie Island and took 1.4 million cubic yards of sand. The other excavation was for Hilton Head Island renourishment and took 200,000 cubic yards of sand.

8.3.2. Existing Inlet Sediment Budget

Table 8-5 depicts the known sediment budget values used for this study. The table includes navigation channel sedimentation rate, volumetric change rates based on historic bathymetry changes, and volumetric erosion rates based on recent shoreline erosion rates.

The navigation channel sedimentation rate is based on: (1) the sedimentation rate of the bar channel (Stations 0+000 to -60+000) estimated in Section 2.2 at 710,000 CY/yr, and (2) the estimate of shoaling in the navigation channel near Jones Island (Stations 24+000 to 0+000) of 100,000 CY/yr provided by the USACE (June, 2000).

For a sediment budget, only littoral (sand) material is of concern. Therefore, it is important to distinguish the mud fraction of the maintenance dredge material from the sand fraction. The sand fraction of the calculated bar channel sedimentation volume was determined using the average of sediment grab samples collected by Dial Cordy and Associates (2001) for the samples that were located in the two areas of significant shoaling. For the shoaling in the bar channel, the sand fraction averaged 76 percent. For the shoaling near Jones Island, the sand fraction averaged 85 percent.

The estimates of shoaling of littoral material in the navigation channel were calculated as the sand fraction times the total shoaling rate. Therefore, the bar channel rate is 540,000 CY/yr (0.76 times 710,000 CY/yr) and the rate in the channel near Jones Island is 85,000 CY/yr (0.85 times 100,000 CY/yr).

The historic bathymetry changes in Table 8-5 are based on the results in Section 4.5. The shoreline erosion rates for Daufuskie Island and Tybee Island are based on previous sediment transport studies (Section 8.3.1). An estimate of the volumetric loss rate of Turtle Island was calculated as the average shoreline erosion rate of 1.9 feet/yr (Anders et al., 1990) times the shoreline length of 13,000, times the vertical distance from top of berm elevation (+10 MLW) to offshore platform depth at edge of bathymetry coverage (- 2 MLW).

The bathymetric and shoreline erosion data in Table 8-5 can be grouped together similar to the cells represented in the sediment budget. The sum of the Daufuskie and Turtle Island shoreline erosion rates and the subaqueous platform equals +30,000 CY/yr. The sum of the Tybee Island shoreline erosion rates and the subaqueous platform equal -317,000 CY/yr.

These data are inserted into the sediment budget layout are shown in Figure 8-28. The remaining values in the sediment budget were completed based on the following:

(1) It is assumed that the sandy material that shoals in the channel between Stations +24+000 and 0+000 is drawn into the entrance during flood tides. It is assumed that the flood currents draw sediments equally from the north and south side of the entrance. Inspection of

the shoaling patterns in the entrance (Figure 4-23) supports the conclusion that some of the sediments shoal from the north side of the entrance. Therefore, the Daufuskie and Turtle Island cell is the source of approximately 45,000 CY/yr (i.e., roughly half of the 85,000 CY/yr of sand dredged from the channel near Jones Island). Because the subaqueous platform near Daufuskie Island and Turtle Island gained material in excess of the shoreline erosion losses, there must be some source of sediments to the cell that are unaccounted (possibly from Wright River, New River or Calibogue Sound).

(2) The bar channel was assumed to be a sink: no bypassing to the north or south is expected. Therefore, the littoral sedimentation rate of the bar channel should provide an estimate of total gross littoral transport (625,000 CY/yr).

(3) The sedimentation in the outer portion of the bar channel is assumed to be indicative of the directional gross transport. For the channel between Station -25+000 and -60+000, the ratio of the sedimentation on the northern side of the bar channel to the sedimentation on the southern side of the bar channel is 80:20 (based on the AD99 to BD00 survey presented in Figure 4-23). It is assumed that this is representative of the ratio of gross south transport to gross north transport. This ratio is applied to the northern boundary of the sediment budget. Admittedly, this is based on only a one-year period, and it is likely to vary year-to-year.

(4) Given that the channel shoals littoral material at a rate of approximately 625,000 CY/yr, of which 155,000 CY/yr is accounted for between the Tybee north shoal and the Daufuskie Island/Turtle Island cell, 470,000 CY/yr must come from southward transport off Barrett Shoals and northward transport off the Tybee Island subaqueous platform. Using the ratio of southerly to northerly transport of 80:20, then the contribution from Barrett Shoals is 380,000 CY/yr and the contribution from the Tybee Island subaqueous platform is 90,000 CY/yr.

(5) If Barrett Shoals is assumed to be stable (i.e., it is not gaining 90,000 CY/yr as estimated in Section 4) then the net input from the northern boundary is 380,000 CY/yr.

(6) Given the different shoreline and bathymetric contour orientations at the boundaries, the net transport at the north boundary (near Hilton Head Island) is not equal to the net transport at the southern boundary (near Tybee Island). Given that the Tybee Island and subaqueous platform cell is losing 320,000 CY/yr, and 250,000 CY/yr is accounted for in losses to the north shoal and the navigation channel, then the net transport out of the southern boundary is 70,000 CY/yr.

The completed sediment budget is shown in Figure 8-29.

8.3.3. Project Impacts to Sediment Budget

Channel deepening impacts to the sediment budget are of paramount importance to this study. As shown in the previous sections, the channel deepening will have almost no effect on the current environment, but it will result in some changes to the coastal wave environment. Based on incident wave energy to the shoreline, calculations of relative changes to erosion and accretion changes indicate that the channel deepening will result in a small (12,000 CY/yr) increase in sediment transport to the north end of Tybee Island. The calculation of relative change in sediment transport in the offshore areas showed that the project will not cause significant changes to littoral interception by the navigation channel.

Table 8-5 Volume change rates input to sediment budget

Area	Rate (1000 CY/yr)
Navigation channel sedimentation rates:	
Bar Channel (0+000 to -60+000)	+540
Jones Island (24+000 to 0+000)	+85
Total	+625
Historic bathymetry changes:	
Tybee north shoal	+50
Tybee subaqueous platform	-137
Daufuskie & Turtle Island subaqueous platform	+96
Barrett Shoals	+90
Shoreline erosion rates:	
Tybee Island to south	-21
Tybee Island to north	-159
Turtle Island	-11
Daufuskie Island	-50

9. CONCLUSIONS

This study compiled relevant historical, and environmental data, and conducted numerical modeling simulations to describe the existing Savannah River/Calibogue Sound inlet system. The study then used this information to determine the potential effects that the proposed Savannah Harbor Expansion Project may have on the erosion of adjacent shorelines.

Section 2 documents the anthropogenic impacts to the Savannah River entrance dating back to the early 1700's. Maintenance dredging of the Bar Channel, construction of the jetties, and construction of the submerged breakwater are three elements of the navigation project that significantly altered the coastal processes in the area. An analysis of the maintenance dredging in the bar channel shows that the maintenance dredging rate increased from approximately 500,000 CY/yr to 700,000 CY/yr with the channel deepening from a 26 foot depth to a 36 foot depth. Subsequent deepenings to 40 and 44 foot depths did not significantly increase maintenance dredging rates (the maintenance dredging rate has been approximately 710,000 CY/yr for the present 44 foot depth). Because deepening subsequent to the 26 foot depth has not increased sedimentation rates in the channel, it is concluded that the bar channel has been a total littoral sink since the 26 foot depth.

Section 3 reviewed the project environment. The study area is composed of a complex barrier island, sea island, and near shore deltaic and ebb shoal featured system. Section 3 provided a description of the geologic setting, tides, waves, storms, river flow, sediments and local sea level rise.

Section 4 analyzed the shoreline and bathymetric morphology of the study area from 1854 to the present. The historic shoreline changes document the changes in the configuration of Hilton Head Island, Daufuskie Island, Turtle Island, and Tybee Island. Shoreline changes resulted from many factors, including: sea level rise, changes in sediment sources (i.e., upland erosion prior to sediment conservation practices, dam construction on the Savannah River, and dredging of the Savannah River), wave and storm environment. It was noted that construction of the entrance jetties corresponded to immediate changes along the Tybee Island shoreline.

Section 4 presents a conceptual inlet sediment bypassing model for the Savannah River Entrance. The complex inlet system including Calibogue Sound, New River, Wright River and the Savannah River is characterized as a large ebb-tidal delta breaching system. Prior to anthropogenic changes, sediments moved southward along Barrett Shoals through the southerly migration of shoals as ebb channels are elongated and abandoned. Shoals eventually bypass the inlet and “weld” onto the Tybee/Little Tybee Island subaqueous platform. Some sediments migrated northward toward north Tybee Island and the Savannah River entrance, some sediment migrated west toward Tybee Island and some of the sediments migrated southward along the coastline.

The bathymetric analysis in Section 4 shows the evolution of the nearshore morphology between 1854 and 1970/80. The analysis shows: (1) the growth of the shoal at the north end of Tybee Island, (2) the deflation of the subaqueous platform seaward of Tybee Island, (3) the relative stability of the Daufuskie Island and Turtle Island subaqueous platforms, (4) the establishment of a deep channel on the north side of the submerged breakwater, and (5) the interception of the toe of Barrett Shoals at the seaward bend of the bar channel. Analysis of the 1999 AD and 2000 BD bar channel surveys shows the major sedimentation areas of bar channel are: (1) the north side of the seaward bend of the channel where it intercepts sediments from Barrett Shoals, and (2) near the north end of Tybee Island.

Section 5 documents the field data collection of currents in the study area for calibration of the hydrodynamic model. The currents were measured at velocities in excess of 1 m/s in the study area.

Section 6 describes the setup and calibration of a hydrodynamic model. The model calibration shows good agreement with measured flows at Calibogue Sound, New River, Wright River, North Channel and South Channel, as well as measured current velocities at offshore locations. The model was used to simulate pre- and post-channel deepening currents, as well as 1854 currents. The simulations show that the deepening project will not significantly alter the current velocities in the study area (i.e., pre- to post-project changes are less than 2 cm/s). The 1854 scenario is modeled to evaluate the study area environment prior to major dredging operations and construction of the jetties and submerged breakwater. The changes between the present conditions and the 1854 conditions provide a comparison to assess the relative magnitude of the pre- to post-project induced changes.

Section 7 presents the application of a wave model to simulate wave refraction/diffraction for the pre- and post-channel deepening bathymetries as well as the 1854 bathymetry. The intent of the wave model simulations is to determine the potential *relative* change resulting from the proposed deepening project. Wave simulations included five average wave directions and four current fields for each bathymetry. Storm wave simulations were included also. The wave model results indicate that the present navigational channel has a significant effect on wave transformation processes (refraction, diffraction, reflection and shoaling). The channel redistributes wave energy resulting in an apparent “sheltering effect” in the lee of the channel. This commonly causes the sheltering of the north end of Tybee Island from waves incident from the east. The wave model results show that significant changes have occurred to the wave field between 1854 and the present. In particular, the changes in bathymetry have caused a dramatic change in the wave environment incident to the Tybee Island shoreline. The comparison of model results for the pre- and post-deepening conditions show that the deepening will result in changes to the wave transformation properties of the channel. These changes cause increases and decreases in average wave height along the island that are generally less than 10 percent. In comparison to the historic changes in wave energy along the island, these changes are small.

In Section 8, the data and analyses from the previous sections of this report are used to evaluate sediment transport processes in the study area. The results of the wave model simulations are used to determine the longshore sediment transport potential along the Tybee Island shoreline. The wave and current simulations are used to determine sediment transport over the region between Hilton Head Island and Tybee Island as a method to evaluate the potential *relative* change in sediment transport rates (not predict actual transport rates). Lastly, a sediment budget is developed and the expected change to the sediment budget from the proposed deepening is determined.

The longshore sediment transport potential analysis indicates that the proposed deepening will result in a small increase (i.e., 12,000 CY/yr) in sediment transport to the north on Tybee Island. The change in transport to the south on Tybee Island is negligible.

The offshore sediment transport calculations indicate that the change in sediment transport rates between 1854 and the present are large (up to 100 percent change in transport magnitude). By comparison, the predicted change in sediment transport rates resulting from the channel deepening is small (i.e., typically less than 5 percent).

A sediment budget was developed for the study area based on measured shoreline and bathymetry changes and a number of simplifying assumptions. The budget includes 625,000

CY/yr of littoral material (i.e., sand) removed from the navigation channel. The only cell in the budget showing significant losses is the cell representing Tybee Island and the subaqueous platform. This cell is losing an estimated 320,000 CY/yr through transport to the north Tybee shoal, transport directly into the navigation channel and transport to the south. The predicted increase in northerly transport at the north end of Tybee Island of 12,000 CY/yr (based on the longshore sediment transport potential calculations) is less than 4% of this total loss rate, and is within the uncertainty of the sediment budget analysis.

When considering the large changes to the coastal environment caused by the navigation project (as shown by the comparison of 1854 simulations to existing condition simulations), the changes induced by the proposed 6-foot deepening of the 44-foot channel are small. The navigation channel is already a littoral sink and totally interrupts the natural sediment bypassing system. This study indicates that the project will result in small changes to the wave field, and thereby causing small changes to the sediment transport environment. The end result is that the deepening project will cause a small change to a system in which the navigation channel is already exerting its maximum potential as a littoral barrier.

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