Development of a Long-Term Coastal Zone Management Plan for the Delaware Atlantic Coast

by

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ABSTRACT

Because tourism is the largest source of income for coastal towns, the presence of a recreational beach has become vital to their continuation and existence. There are many options in response to a threatening coastline. Presently, beach nourishment is the response of choice for many states because the natural environment is maintained and the shoreline is still protected. However, beach fills do not change the erosive conditions that caused the need for nourishment. As a result, the renourishment of the beaches becomes a continuous, expensive cycle.

The intent of this study is to define a beach renourishment strategy that will be necessary over the next fifty years in order to maintain a suitable beach. First, various factors affecting the shoreline are studied. Based on available and collected site data, erosional effects and sediment transport along the coast are quantified. Secondly, the behavior and lifetime of beach fills and subsequent fills are analyzed. This includes analyzing the behavior of nourishment projects on the Delaware coast. These investigations are then combined to calibrate a numerical model that is adapted to the Delaware coast and surrounding conditions.

The model predicts that the need for renourishment will occur less frequently over time. However, because of a shortage of sediment transport within the model boundaries, the numerical model overestimates the lifetimes of the fills. Based on previous fill requirements, it is known that fills will be necessary at a higher renourishment rate than that predicted by the model. The nourished shorelines in the presence of sea level rise are also presented as well as shorelines in a retreat scenario.

Chapter 1

INTRODUCTION

1.1 Background

Beaches naturally respond to their surrounding conditions. It is when this response threatens to interfere with man-made structures that the manipulation of the shoreline becomes essential. With the constant threat of erosion, storms, sea level rise, and increased construction and habitation, the maintenance of the beach, the protection of structures, and the assurance of continued local, state, and federal revenue becomes a major public issue.

Because tourism is such a large source of income to these coastal towns, the presence of a recreational beach has become vital to their continuation and existence. As the width of the beach decreases and overcrowding occurs, fewer tourists return the following year. If fewer people are returning, the amount of money coming into the area falls. As a result of less money being spent, the general upkeep of structures is forgone and tourism continues to decline. Ultimately, a slow demise of the coastal town is experienced.

There are many options in response to a threatening coastline. The first decision that must be made in coastal management involves whether or not to maintain the existing beach. A choice that is rarely considered is beach retreat which involves allowing the beach to naturally respond to the conditions to which it is exposed. It is important to understand that this does not imply allowing the beach to narrow until it disappears. The active beach will maintain a particular width, thus no losses in revenue. Instead, the landward boundary of the beach will also recede with the loss of land occurring inland. There are many supporters of this "non-reaction." It allows the coastline to return to its original more natural state and is believed will lead to a more responsible pattern of decreased development near the beach. On the other hand, it involves the relocation or removal of a large number of structures as the beach moves inland.

The alternative to beach retreat is shoreline protection. There are many options in shoreline protection. They are classified as either "hard" or "soft" structures. "Hard" structures involve the construction of large structures such as groins, jetties, breakwaters, or seawalls. There has been a general trend to steer away from these bulky designs. They are difficult to remove, alter the existing coastal processes, and deprive neighboring beaches of their sand source from littoral drift.

A more natural form of shoreline protection is beach nourishment or "soft" structures. Beach fills do not change the erosive conditions that caused the need for nourishment. It simply provides temporary relief. In addition, a beach fill is not a long-term solution. Renourishment is necessary which can become problematic when a sufficient sand source is not nearby or large enough to sustain a long-term strategy. Regardless, this has become the response of choice for many states because the natural environment is maintained and the shoreline is still protected. Not only is beach nourishment the only option that actually introduces sand to the system, but the fill can trap sand updrift while still serving as a source of sand to downdrift beaches.

In any case, the preservation of the beaches is a continuous, expensive cycle. From the maintenance or removal of structures to the constant renourishment of the beach, local, state, and federal governments must plan for the future maintenance of the coastline. This does not simply involve deciding how to maintain the beach. It includes construction guidelines, reconstruction limitations, the education of the property owners, and the designation of financial responsibility.

1.2 Objective and Scope

This study is being performed for Delaware's Department of Natural Resources (DNREC) in conjunction with the University of Delaware's Marine Policy Department. The two principle objectives are to estimate the long-run cost of beach retreat and the long-run cost of beach nourishment for the state of Delaware for a 50-year time frame. The two estimates will be evaluated and compared to determine a practical and economically feasible long-term beach management policy for the state.

The scope of coastal engineering work required by this study involves determining a beach nourishment strategy that will be necessary over the next fifty years to maintain a suitable beach. First, various factors affecting the shoreline are studied.

Based on available and collected site data, existing conditions along the coast are quantified. Secondly, the behavior and lifetime of beach fills and subsequent fills are investigated. This includes analyzing the behavior of nourishment projects on the Delaware coast. These investigations are then combined to calibrate a numerical model that is adapted to the Delaware coast and surrounding conditions. This allows for the estimate of future needs for beach maintenance.

1.3 Summary of Activity on the Delaware Ocean Coast

The Delaware coastline has been a region of high coastal activity. Groin fields have been capturing sand within the littoral drift in the Rehoboth, Dewey, and Bethany beaches for several decades. Dewey and Rehoboth beaches have a total of fourteen groins along the coast, and Bethany and South Bethany have nine. Relative to the shoreline, the groins are not significantly long, and depending on the season, some are no longer visible due to sand coverage. Some of the groins consist of piles, but a majority are rubble structures. Regardless, all have been impacted by wave activity and storms. This has reduced the effectiveness of the groins, as well as increased permeability. Therefore, it has been the general opinion of several state officials that the groins are generally ineffective and have little influence on the coastline.

In addition to the groins, the only other "hard" structures on the Delaware coast are two jetties. These jetties were placed on either side of Indian River Inlet in order to stabilize the inlet for vessel access. Because the jetties significantly impact sand traveling to the northern beaches, a bypassing system has been set up by the US Army

Corps of Engineers. The bypassing operations are in effect throughout the tourist off-season. Fortunately, since the system was established in February 1990, DNREC has maintained extensive records of production volumes and costs. Tables 1.1 and 1.2 provide annual bypassing volumes per month and by pumping season in cubic yards, respectively. The volumes that are bold are high for the period of measurement. On average, over 90,000 cubic yards of sand are pumped each year. Through 1999, more than 908,000 cubic yards of sand have been pumped over Indian River Inlet. As indicated by the tables, production was increased significantly during the 1996 season and continued at a higher rate in subsequent years. While this level of bypassing is expected to continue due to needs north of the inlet, it is now coming at a sacrifice of beaches south of the inlet.

In addition to volumes, the state also maintains records of costs associated with the operations. The average annual cost of this operation is less than $160,000. This is approximately $1.80 per cubic yard. An annual summary of the costs is provided in Table 1.3. It is important to note that bypassing began in February 1990. Therefore, the values for that year are for eleven months. In addition, the data provided terminated in October 1999 so that year is for a ten-month period. One more feature of this data worth noting is the field for fuel. Beginning in 1998, values were not provided for fuel associated with trucks. However, this cost is minimal with respect to other costs and is not expected to have a significant influence on the results.

Table 1.1: Bypass Production by Month in Cubic Yards

	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999
JAN	407	4,105	11,092	6,334	3,818	7,470	1,329	12,419	**17,338**	14,264
FEB	**22,177**	8,066	5,353	8,485	13,841	8,762	6,223	14,588	1,157	14,839
MAR	5,167	12,888	8,152	12,986	6,492	9,439	10,368	12,730	**17,874**	10,410
APR	15,601	11,348	2,187	8,560	13,926	6,072	9,561	18,078	**19,454**	5,790
MAY	10,188	6,656	7,559	**11,074**	8,156	9,496	4,073	8,616	7,230	4,697
JUN	**7,347**	0	0	87	0	0	0	0	0	0
JUL	**8,392**	220	0	0	0	0	0	0	0	0
AUG	**7,760**	74	66	0	0	0	0	0	0	0
SEP	12,001	6,538	6,747	0	8,671	0	13,901	15,189	**20,384**	8,264
OCT	15,706	17,912	16,688	0	12,799	6,243	**24,212**	11,845	16,004	8,517
NOV	3,726	11,510	5,984	13,086	12,292	10,561	12,407	9,808	8,101	**15,897**
DEC	4,529	3,018	3,848	7,196	4,580	10,708	13,572	11,656	**13,617**	10,229
Totals	113,001	82,335	67,676	67,808	84,575	68,751	95,646	114,929	**121,159**	92,907

Table 1.2: Bypass Production by Pumping Season (September-August) in Cubic Yards

Season	Volume
1989-1990	77,039
1990-1991	79,319
1991-1992	73,387
1992-1993	80,793
1993-1994	66,515
1994-1995	79,581
1995-1996	59,066
1996-1997	**130,523**
1997-1998	111,551
1998-1999	108,106
Total	865,880

Table 1.3: Indian River Inlet Sand Bypass Plant Operating Expenses

YEAR	SALARIES + OEC	ELECTRIC	FUEL	GENERAL (CS, S&M)	TOTAL COST	CY PUMPED	COST/CY
1990*	$92,123	$1,159.64	$11,823	$7,284	$112,391	112,731	$1.00
1991	107,895	723.88	9,216	21,643	139,477	82,335	1.69
1992	103,322	803.99	6,901	14,783	125,811	67,676	1.86
1993	104,720	824.45	6,609	56,100	167,680	67,808	2.47
1994	108,133	821.83	8,866	21,863	139,684	84,575	1.65
1995	108,291	833.27	8,926	40,154	158,204	68,751	2.30
1996	131,326	717.73	11,959	58,418	202,421	95,648	2.12
1997	132,250	951.90	13,090	40,825	187,117	114,929	1.63
1998	136,438	841.12	9,662*	49,292	196,233	121,159	1.62
1999*	112,166	849.45	6,650*	55,324	174,990	66,781	2.62

*Data limited or missing

The most significant impact on the Delaware beaches has been beach nourishment. The first significant beach nourishment project occurred in Fenwick in October 1988. This was soon followed by projects in and around Bethany Beach. Beach nourishment did not begin in Dewey and Rehoboth Beach until 1993. Since then renourishment has occurred every two to four years. On average, the cost per cubic yard of sand for a nourishment project is approximately $4.60.

In addition to the nourishment projects financed by the state, additional nourishment has occurred at Fenwick. The town of Fenwick is located at the southern border of Delaware. Bordering the town to the south is Ocean City, Maryland which is a large tourist spot. As a result, this area also experiences significant beach nourishment projects. Not only does Fenwick benefit from the migration of the sand placed by the projects, the state of Maryland extends the projects into the Fenwick city limits. Table

1.4 summarizes the beach nourishment projects that have taken place on the Delaware ocean coast per DNREC. The projects that have been performed by Maryland and extended into Fenwick are designated as unincorporated (unincorp.).

Table 1.4: Summary of Beach Nourishment Projects in Delaware

LOCATION	DATE	VOLUME(CY)	COST ($)	COST/CY
Fenwick Island	October 1988	333,500	$1,572,993	$4.72
Bethany Beach	June 1989	284,500	1,630,241	5.73
Sea Colony	July 1989	132,600	770,058	5.81
South Bethany	August 1989	231,600	1,307,849	5.65
Middlesex Beach	October 1989	63,700	357,905	5.62
Fenwick (unincorp.)	August 1991	126,800	443,603	3.50
Fenwick (unincorp.)	August 1992	37,000	269,234	7.28
South Bethany	September 1992	192,749	905,786	4.70
Fenwick	September 1992	144,900	716,916	4.95
Bethany Beach	October 1992	219,735	1,037,303	4.72
Dewey Beach	July 1993	5,755	30,210	5.25
Dewey Beach	July 1994	592,878	2,402,230	4.05
North Indian Beach	July 1994	20,992	61,400	2.92
Indian Beach	July 1994	4,778	20,435	4.28
Bethany Beach	August 1994	184,452	838,953	4.55
South Bethany	August 1994	98,419	452,165	4.59
Fenwick	September 1994	8,236	32,396	3.93
Fenwick (unincorp.)	September 1994	60,000	336,873	5.61
Dewey Beach	July 1998	453,500	1,948,000	4.30
North Shores	August 1998	188,000	721,630	3.84
Rehoboth Beach	August 1998	274,300	1,087,750	3.97
Bethany Beach	September 1998	321,700	1,321,572	4.11
Sea Colony	September 1998	128,000	419,479	3.28
South Bethany	September 1998	168,900	707,635	4.19
Fenwick	October 1998	56,100	457,000	8.15
Fenwick (unincorp.)	October 1998	85,000	469,390	5.52

Extreme events such as northeasters and hurricanes are also extremely influential on beach nourishment performance and shoreline position. On average, Delaware experiences a hurricane every seven years. While that has not been the case in the past decade, there have been an unusually high number of northeasters. Table 1.5 is a summary of the worst storms to occur on the Delaware coast through 1998 as provided by DNREC. The ranking is based on the water level that occurred due to waves, wind, and storm surge. It is important to note that more than 50% of the thirty worst storms to impact Delaware's ocean coast took place in the 1990s. Those storms, including another that took place in 1989, are shaded in the table. These seventeen storms are of importance as they occurred during the monitoring program performed by DNREC.

Table 1.5: Storm History for the Delaware Coast

RANK	DATE	TIME (EST)	STORM TYPE	STORM NAME
1	03/06/62	2100	Northeaster	
2	01/04/92	748	Northeaster	
3	01/28/98	930	Northeaster	
4	02/05/98	300	Northeaster	
5	09/27/85	700	Hurricane	Gloria
6	03/03/94	500	Northeaster	
7	10/25/80	1000	Northeaster	
8	03/29/84	700	Northeaster	
9	03/08/96	330	Northeaster	
10	12/12/92	1018	Northeaster	
11	10/22/61	800	Hurricane	Esther
12	10/14/77	900	Hurricane	Evelyn
13	10/31/91	1500	Northeaster	
14	09/18/36		Hurricane	No. 13
14	10/22/72	1000	Northeaster	
14	11/14/97	1300	Northeaster	
15	01/02/87	1100	Northeaster	
16	10/08/96	2230	Northeaster	
17	10/14/53		Northeaster	
17	12/09/73	700	Northeaster	
18	01/13/64	800	Northeaster	
19	09/25/92	1942	Tropical Storm	Danielle
20	11/10/69		Northeaster	
20	02/24/98	642	Northeaster	
21	11/15/81		Subtropical	Storm 3
22	01/10/56		Northeaster	
22	10/19/89	1200	Northeaster	
23	12/02/86		Northeaster	
24	02/26/79		Northeaster	
25	12/12/60		Northeaster	
25	05/24/67		?	
26	12/13/96	1024	Northeaster	
27	12/14/93	900	Northeaster	
28	12/20/95	724	Northeaster	
29	01/03/93	948	Northeaster	
30	11/10/91	1118	Northeaster	

Chapter 2

DELAWARE'S HISTORICAL SHORELINE AND VOLUMETRIC CHANGES

In order to effectively predict the performance of a beach fill, it is essential to understand the natural reaction of the shoreline to its present conditions. The ideal approach is to investigate the historical behavior of the beach. Delaware's Department of Natural Resources (DNREC) has maintained a database of thirty-six surveys spaced along the entire Delaware ocean coast. Figure 2.1 illustrates the location of the surveys along the coast. In addition, general information for each transect is available in Table 2.1. The transects are comprised of cross-shore surveys that generally extend from the dune system to beyond the depth of closure. Since 1990, surveys have been performed twice a year and even more frequently following a beach fill. On average, these surveys take place in the summer (March-June) and winter (August-November). Table 2.2 provides the dates of the recent and more regular surveys performed on the ocean coast by DNREC and used in this study. In addition, DNREC began performing a more extensive coverage of surveys in the Rehoboth and Dewey, Bethany, and Fenwick areas.

As illustrated in Table 2.2, while the survey program has occurred for 17 years, data collection has only really occurred regularly for approximately 10 years for the

Delaware coast. A critical aspect of any monitoring program is the duration and frequency with which data is collected. Ideally, information is gathered for a transect several times a year over many decades. However in most situations, data collection is prompted by an extreme event or nourishment project. If transects are surveyed consistently over a long period of time, extreme events are taken into account in the analysis but are less likely to bias the results.

Nevertheless, the data was analyzed using a program called the Interactive Survey Reduction Program (ISRP) developed by the US Army Corps of Engineers (USACE). The program was originally developed in order to process beach and nearshore survey data collected at the Waterways Experiment Station's Coastal Engineering Research Center's Field Research Facility in Duck, NC. It is a general program used for the entry and limited processing of survey data collected using common survey techniques. For this study, the program's use was limited to extracting shoreline positions, as well as volume differences between two successive surveys. The program was also used to visually approximate the berm height and depth of closure of each profile.

It is important to note that while historical records are a convenient way of quantifying the behavior of a beach, it must be understood that many factors are accounted for in the analysis. That is, the figures are not solely representative of the beach response in typical wave conditions. Instead, they include extreme events, human intervention, and beach nourishment projects themselves. While this may be an effective way to take into account factors that are not easily determined, the conditions under which the data was collected must also be fully understood. Otherwise, extreme or infrequent events are averaged into the analysis, thus influencing the perception of what may actually occur in the future. This especially becomes an issue when the data collection has occurred for only a short amount of time, as in our case. For example, the 1990s were a period of high storm frequency. While it has been predicted that this unusually high occurrence of storms is likely to continue, this factor may overestimate sediment loss in the beach profile and underestimate the lifetimes of beach nourishment projects. As a result, the shoreline change and volumetric investigations are performed in conjunction with the beach nourishment and storm timelines.

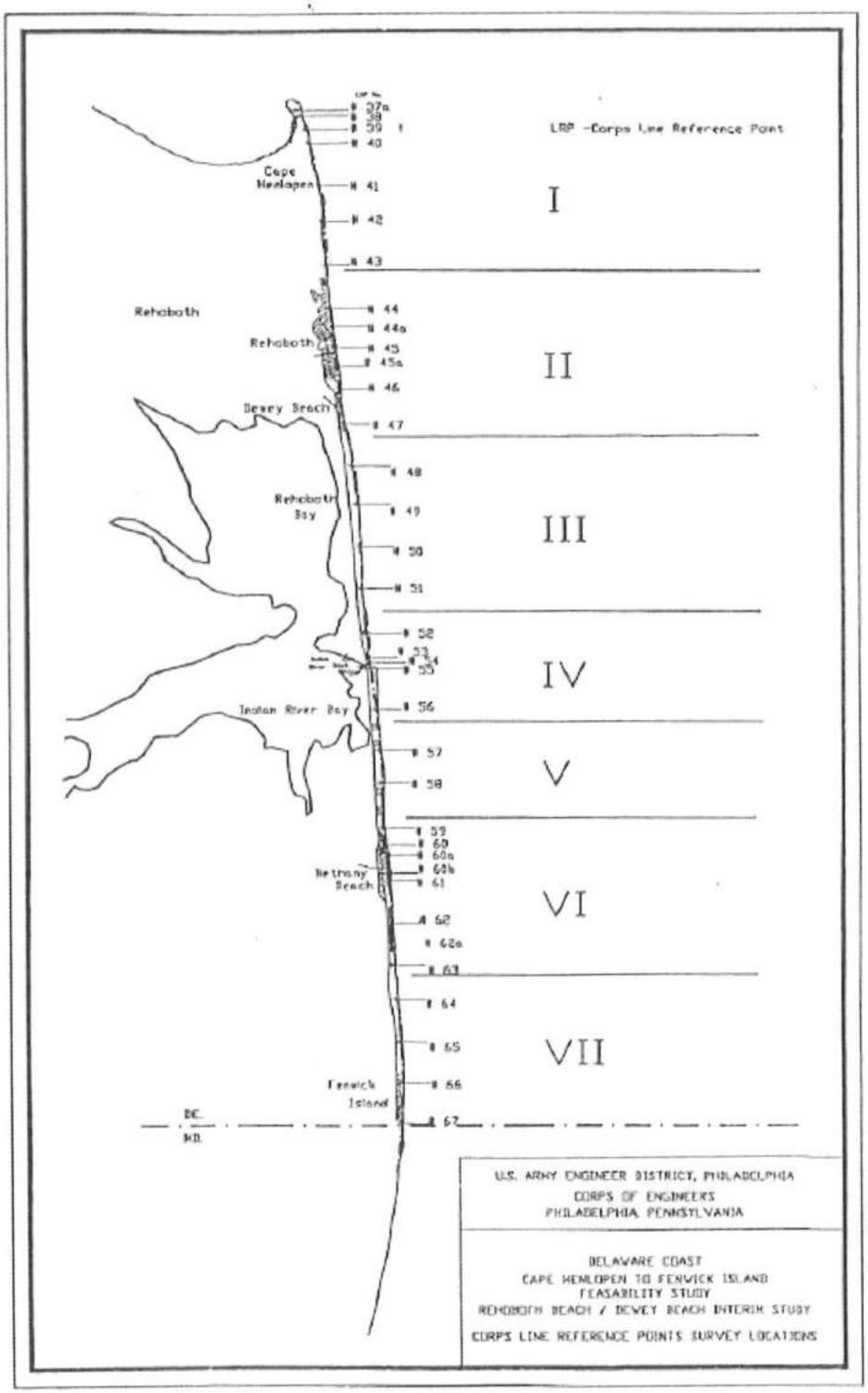


Figure 2.1: Profile locations for the Delaware ocean coast.

Table 2.1: Profile Summary for the Ocean Coast of Delaware

	Starting Location		Orientation (from North)	
Profile	**Northing**	**Easting**	**Degrees**	**Area**
38	290960	748878	71	Cape Henlopen
39	289210	749428	77.5	Cape Henlopen
40	287517	749954	85.5	Cape Henlopen
41	282408	751176	67.5	Cape Henlopen
42	277619	751610	86.5	Cape Henlopen
43	272203	751966	86	Cape Henlopen
44B	269802	751996	86	North Shores
44	266708	752521	85.60	North Shores
44A	264485	752827	83.5	Rehoboth
45	261830	753302	82.5	Rehoboth
45A	259405	753503	82.5	Rehoboth
46	256661	753765	79	Dewey
47	252075	754561	82.5	Dewey
48	246705	755230	80	State Park
49	241870	755787	86.5	State Park
50	236483	756228	87	State Park
51	231334	756356	93	State Park
52	225795	756835	88	State Park
53	222716	757245	70	Indian River Inlet
54	222108	757381	90	Indian River Inlet
55	221370	757807	90	Indian River Inlet
56	216147	758008	84.5	State Park
57	211003	758608	84.5	State Park
58	206813	758769	89	State Park
59	201233	758949	81.5	State Park
60	199144	759358	74	Bethany
60A	197917	759658	84	Bethany
60B	195626	759883	88.5	Bethany
61	194656	759899	91	Bethany
62	189393	760162	86	South Bethany
62A	187708	760088	89.65	South Bethany
63	184013	760270	87	State Park
64	179695	760609	76	State Park
65	174361	760961	96.95	State Park
66	169267	761245	87.5	Fenwick
67	164165	761155	88.54	Fenwick

Table 2.2: Dates of Surveys Performed by DNREC

Survey Number	Survey Date
112	October 1982
113	August 1984
114	April 1985
115	June 1986
116	August 1987
117	April 1988
118	April 1990
119	November 1990
120	May 1991
121	October 1991
122	April 1992
123	January 1993
124	August 1993
125	April 1994
126	September 1994
127	May 1995
128	October 1995
129	March 1996
130	August 1996
131	March 1997
135	May 1999

2.1 Historical Shoreline Positions

In this section, the historical shoreline positions are investigated to determine a representative recession rate for the Delaware coast. As stated earlier, these recession rates will be functions of the storms and beach fills that have impacted the coast. Therefore, the beach response before and after these events will also be presented here. The positions include features such as the adjustment of the beach to an equilibrium position after a fill, the seasonal variability in beach profiles, as well as the response of a beach to build an offshore bar during a storm and the return of the bar to the visible beach afterward.

For each profile, the recession rates were determined for the entire data set as well as the summer and winter seasons. The shoreline change rates were determined by performing a linear regression analysis on the shoreline positions extracted from each profile. Figure 2.2 shows an example of the analysis for profile 47, located in Dewey Beach. For each profile, the seasonal positions are differentiated and trendlines are provided for the annual, summer, and winter surveys. The shoreline positions of each survey relative to its reference point are provided for each profile in Appendix A. Table 2.3 provides the annual and seasonal shoreline change rates from the regression analysis for each profile.

While there are certainly fluctuations in the shoreline rates along the coast, virtually the entire coastline north of Indian River Inlet is experiencing erosion. In contrast, other than the state park between South Bethany Beach and Fenwick Island, most of the southern Delaware coast is accreting. This would lead one to believe that the amount of sand bypassed at Indian River Inlet should be increased. However, according to DNREC, it is unlikely that the volumes will be significantly increased any further as the massive pumping effort is now at a detriment to adjacent commerce that depends on the existing beach for their livelihood. Instead, this difference is most likely attributed to the higher number of nourishment projects that have taken place in the Bethany and Fenwick areas. This substantial increase in beach nourishment efforts has managed to stabilize the region. This is particularly the case in Fenwick which is also benefiting from nourishment projects in Ocean City, Maryland.

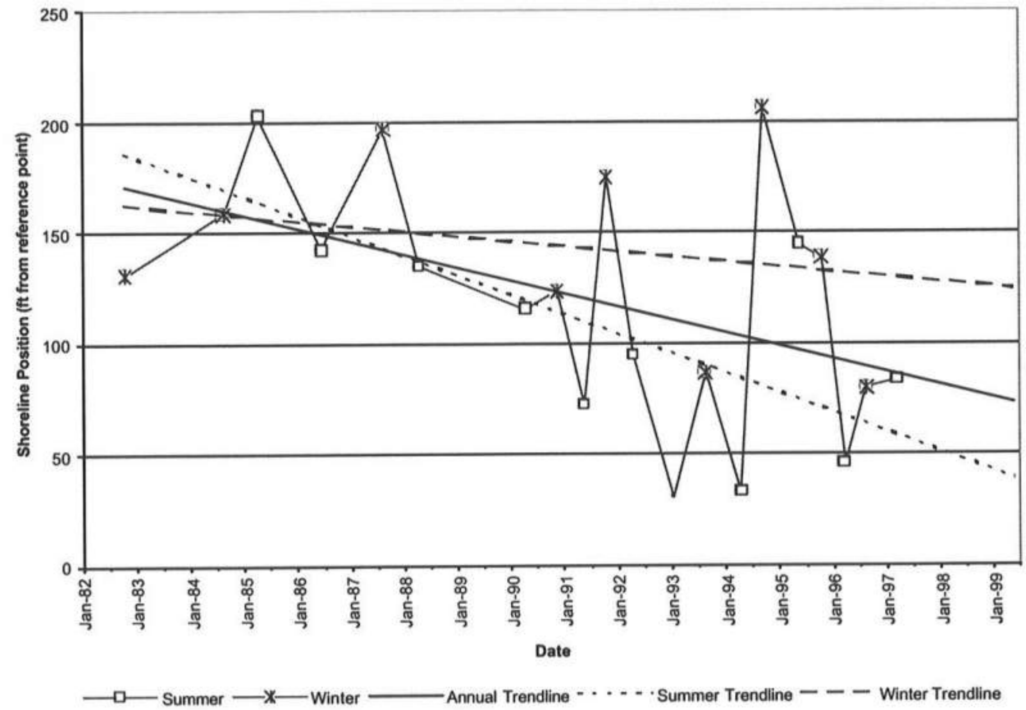


Figure 2.2: Example of historical shoreline positions and trendlines for profile 47 in Dewey Beach.

Table 2.3: Delaware Ocean Coast Historical Shoreline Change Rates

Profile	Approximate Location	Annual Shoreline Change Rate (ft/yr)	Summer Shoreline Change Rate (ft/yr)	Winter Shoreline Change Rate (ft/yr)
38	Cape Henlopen	-0.55	-7.30	-0.79
39	Cape Henlopen	0.56	0.44	0.50
40	Cape Henlopen	-13.37	-1.88	-14.11
41	Cape Henlopen	-1.54	-3.75	-0.02
42	Cape Henlopen	-3.40	-3.94	-3.08
43	Cape Henlopen	-4.41	-1.89	-4.84
44B	North Shores	-2.35	-2.49	-2.19
44	North Shores	2.37	1.20	3.66
44A	Rehoboth	2.18	0.47	3.49
45	Rehoboth	-0.94	-2.13	0.28
45A	Rehoboth	-2.04	-3.98	0.36
46	Dewey	1.13	-1.79	3.96
47	Dewey	-5.89	-8.84	-2.27
48	State Park	-5.02	-4.37	-4.66
49	State Park	-4.84	-24.81	-2.71
50	State Park	-1.36	-2.22	0.48
51	State Park	-0.06	-1.21	1.81
52	State Park	-0.55	-2.13	2.66
53	Indian River Inlet	-3.13	-1.31	-6.13
54	Indian River Inlet	1.37	12.42	-1.73
55	Indian River Inlet	-3.01	-9.05	4.01
56	State Park	3.00	3.63	2.70
57	State Park	2.71	4.95	1.23
58	State Park	2.12	1.35	2.47
59	State Park	7.78	8.21	7.58
60	Bethany	13.11	16.87	11.79
60A	Bethany	3.89	-5.51	5.82
60B	Bethany	6.20	4.09	7.09
61	Bethany	-11.60	-14.11	-2.78
62	South Bethany	0.10	-0.88	1.02
62A	South Bethany	9.69	1.55	16.51
63	State Park	-1.41	-2.65	1.36
64	State Park	-1.18	-10.42	2.07
65	State Park	-1.29	-5.64	3.21
66	Fenwick	4.93	1.56	8.16
67	Fenwick	7.71	5.29	11.05

Note: Negative values indicate erosion.

2.1.1 Beach Response to Storms and Beach Fills

The shorelines for Dewey and Rehoboth, Bethany, and Fenwick beaches before and after extreme events and human interventions are presented in this section. They are presented together due to the extreme changes they have on the shoreline and because of the interaction that occurred between the two during the 1990s. In several circumstances, storms occurred following a beach fill. While some may consider the beach fill a waste of money due to significant sediment losses, the additional sand provided immeasurable protection for the coastal structures and environment. In addition, while the shoreline appears further inland, the sediment loss is less than believed. Rather than being permanently lost, the majority of sand is located in an offshore bar and will return during the calm season. In contrast, there are occurrences of an individual storm on the beach prompting a nourishment project due to the severe impact the storm had on the coastline and the inability of the beach to rebuild itself.

As mentioned earlier, northeasters and tropical storms occurred in high frequency during the 1990s. Fortunately, this coincides with the survey program performed along the Delaware coast by DNREC. In 1992, the second most severe storm to impact the Delaware coast hit on January 4, and the tenth worst storm battered the coast on December 12. In addition, nourishment projects took place in Bethany and Fenwick beaches in September and October of the same year. Therefore, the response of the shoreline for this dynamic period is presented here. Figures 2.3 and 2.4 show how the shoreline behaved relative to an initial shoreline for the Bethany and Fenwick beaches, respectively. While Dewey and Rehoboth beaches were also impacted by the January and December storms of 1992, no fills were planned for the beach, and thus not presented here. However, the shorelines for the rest of the storms and beach fills that affected Delaware are provided in Appendix B.

In both figures, the significant retreat following the January storm is illustrated relative to the initial shoreline position. The next shoreline position indicates where the beach nourishment efforts were concentrated. The surveys in Bethany were performed one month after the sand placement. Therefore, the shoreline position for November 1992 also reveals the shoreline adjustment that occurred following the fill. Finally, the fourth shoreline shows a second retreat of the shoreline following the December storm. The severity of the storm in this case may be underestimated as the surveys took place five to six months after the storm. On the other hand, the shoreline position may exemplify the resiliency of the beach to naturally recover after an energetic storm or harsh winter season.

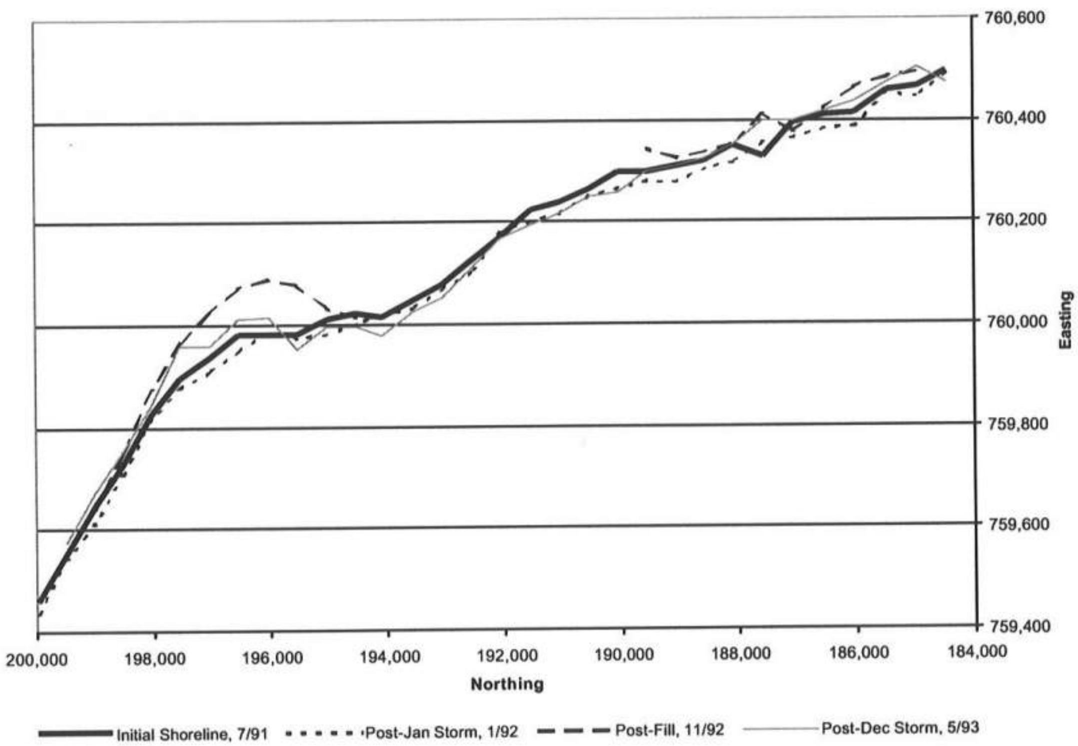


Figure 2.3: Bethany and South Bethany Beach shorelines in 1992.

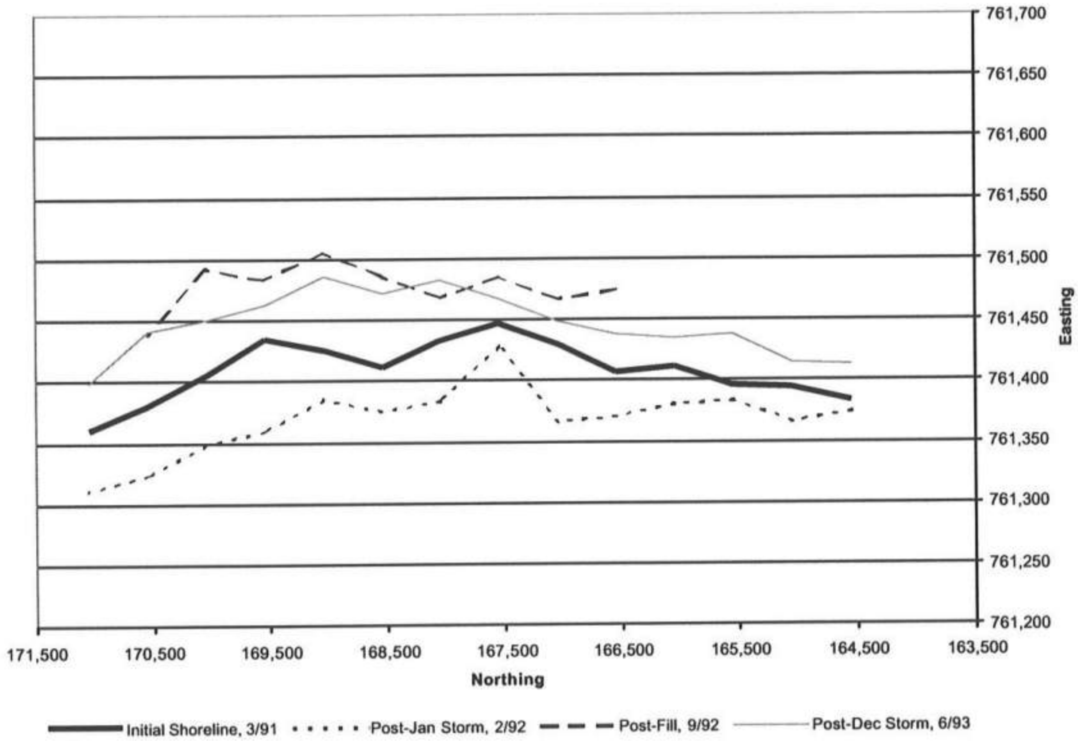


Figure 2.4: Fenwick Island shorelines in 1992.

2.2 Volumetric Changes

A second approach used to quantify shoreline change rates is through a volumetric analysis. This method is a more accurate estimate of shoreline change as the entire profile is taken into account rather than one position in time. Profile features such as offshore bars and berm shape are incorporated in the results.

As mentioned earlier, ISRP determines the cut and fill volumes per unit width between consecutive surveys. The Bruun Rule is then applied which states that the rate of shoreline retreat, $\partial R/\partial t$, is expressed as

$$\frac{\partial R}{\partial t} = -\frac{1}{(h_* + B)}\frac{\partial V}{\partial t} \tag{2.1}$$

in which $\partial V/\partial t$ is the volumetric rate of change per unit length of beach and h_* and B are the depth of closure and berm height, respectively. This theory is based on the assumptions that the profile shape is constant and that sand volume in the profile must be conserved. In addition, the depth of closure is defined as the offshore depth beyond which beach profiles taken over time at a given location converge. Theoretically, little or no sediment transport occurs seaward of this point.

By plotting the surveys performed for each profile on a graph, the berm height and depth of closure can be estimated as the general position at which the profile features converge. The volumetric change rates are then determined by summing the total change in volume over the profile and dividing by the time over which the changes took place. By dividing the rate by the sum of the berm height and depth of closure, the long-term shoreline change rate can be ascertained.

Table 2.4 presents the rates resulting from this method. As before, the results indicate accretion south of Indian River Inlet. In contrast, this approach reveals a tendency of accretion north of the inlet. Figure 2.5 provides a comparison between the recession rates determined by the historical shoreline positions and the volumetric change analyses. In general, both approaches have fairly similar results.

Table 2.4: Shoreline Change Rates Based on Volumetric Analysis

Profile	Approximate Location	Shoreline Change Rate (ft/yr)
38	Cape Henlopen	0.93
39	Cape Henlopen	0.90
40	Cape Henlopen	-9.22
41	Cape Henlopen	1.81
42	Cape Henlopen	-2.29
43	Cape Henlopen	-2.96
44B	North Shores	-1.22
44	North Shores	-0.32
44A	Rehoboth	0.23
45	Rehoboth	0.97
45A	Rehoboth	0.22
46	Dewey	0.55
47	Dewey	-1.67
48	State Park	-1.44
49	State Park	1.81
50	State Park	-0.84
51	State Park	-0.05
52	State Park	1.08
53	Indian River Inlet	1.85
54	Indian River Inlet	6.28
55	Indian River Inlet	-6.44
56	State Park	5.07
57	State Park	4.39
58	State Park	2.56
59	State Park	6.77
60	Bethany	4.41
60A	Bethany	2.90
60B	Bethany	1.01
61	Bethany	-19.10
62	South Bethany	-1.29
62A	South Bethany	8.53
63	State Park	-2.58
64	State Park	-0.44
65	State Park	-0.33
66	Fenwick	2.72
67	Fenwick	7.04

Note: Negative values indicate erosion.

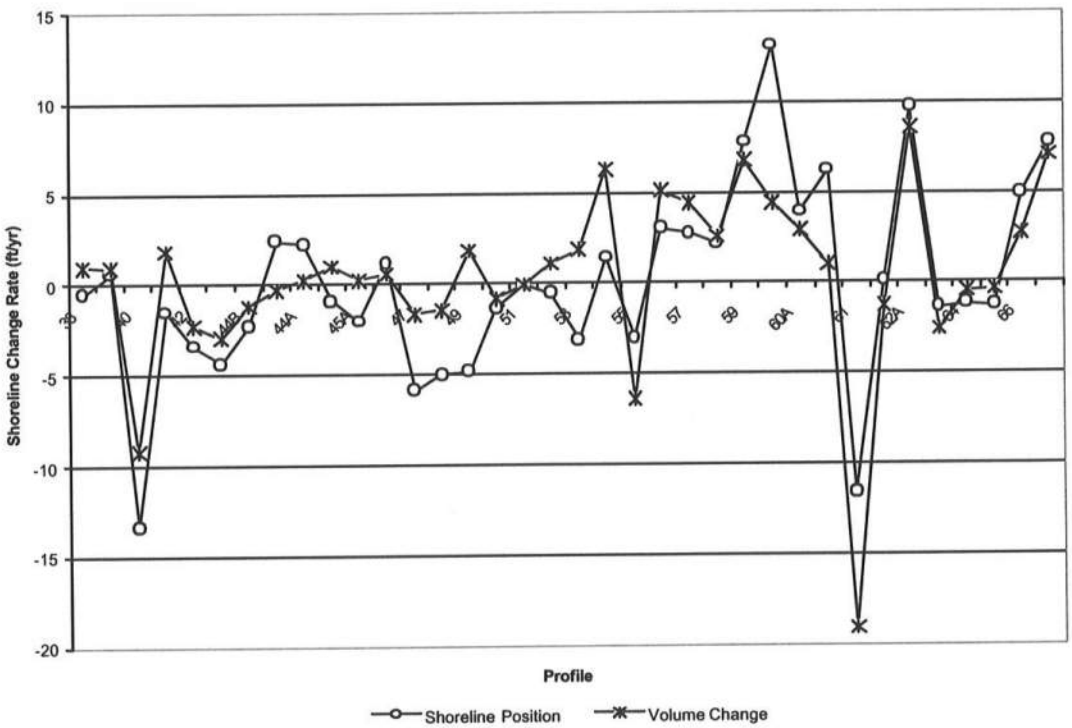


Figure 2.5: Comparison of shoreline change rates based on historical shoreline positions and volumetric changes. (Note: Negative rates indicate erosion)

Chapter 3

LITTORAL DRIFT

Sediment is transported along a coastline through the action of waves and currents. As waves approach a shoreline, they begin to shoal, refract, and ultimately break as a result of the interaction with the seabed. In doing so, energy is transmitted to the sediment causing it to move along the bottom and up into the water column. If waves approach the shoreline obliquely, a longshore component is introduced into the system, and sediment is carried parallel to the beach. It is this mechanism that is the motivation for the shore stabilizing structures that extend across the surf zone. These cross-shore structures are designed to trap the sediment as it travels along the shore. In some circumstances, it performs so well that it does so at the expense of downdrift beaches. The jetties at Indian River Inlet are an ideal example of the impact shore normal structures can have on a coastline. The southern jetty was so effective at maintaining a navigable depth in the inlet that a bypassing system was developed by the US Army Corps of Engineers to transfer sediment to the highly depleted coast north of the inlet. However, groins and jetties are not the only structures responsible for impeding longshore transport. Beach nourishment projects also trap sand at the updrift side.

Fortunately, they are also a source of sand at the downdrift side. It is because of the dependency of these shore stabilization projects on the supply of sand from longshore transport that littoral drift is investigated here.

3.1 Longshore Transport

There are only a few methods by which littoral drift can be determined. This is due in part to the complexity associated with the breaking waves and turbulence in the nearshore region. Several methods have been utilized in the field, some of which involve surveying accumulations and monitoring bypassing at groins and jetties. A short-term approach employs particle tracking technology. By injecting the beach with fluorescent dye or other distinguishable solutions, the sediment can be tracked over a short period of time for a specific wave environment. In addition, physical experiments have been performed in laboratories but tend to underestimate transport rates due to scale effects.

Numerically, transport rates have been based on the flux of energy of the breaking waves approaching the beach. The alongshore energy flux per unit length of beach, P_L, is represented as

$$P_L = (EC_g)_b \sin\theta_b \cos\theta_b = \frac{1}{16}\rho g H_b^2 C_{gb} \sin 2\theta_b \tag{3.1}$$

in which

E is the total average wave energy per unit surface area,

C_g is the group velocity,

θ is the wave angle relative to the shoreline,

ρ is the density of water,

g is acceleration of gravity,

H is the wave height, and

b is a subscript denoting breaking wave conditions.

In 1963, Inman and Bagnold introduced the following dimensionally-correct relationship correlating the rate of longshore sediment transport, Q_ℓ, with the energy flux,

$$Q_\ell = \frac{KP_L}{(\rho_s - \rho)g(1-p)} = \frac{K(EC_g \sin\theta\cos\theta)_b}{(\rho_s - \rho)g(1-p)} \tag{3.2}$$

where

ρ_s is the density of the sediment,

p is the porosity, and

K is a constant found by Komar and Inman (1970) to be 0.77.

Unfortunately, this equation is dependent on breaking wave conditions which in most cases, is not available for a site. However, if the bathymetry is considered straight and parallel and the energy losses from deep water are neglected, it is possible to express the transport almost solely in terms of deepwater wave conditions. Multiplying both sides of Equation 3.2 by the shallow water speed, C_b, and using the specific gravity of the sediment, $s = \rho_s - \rho$, gives

$$Q_\ell = \frac{K(EC_g \cos\theta)_b C_b}{\rho g(s-1)(1-p)} \frac{\sin\theta_b}{C_b} \tag{3.3}$$

The conservation of energy and Snell's Law can be applied to relate the breaking wave conditions to the deepwater conditions (subscript "o")

$$\left(EC_g \cos\theta\right)_b = \left(EC_g \cos\theta\right)_o \quad (3.4)$$

$$\frac{\sin \theta_b}{C_b} = \frac{\sin \theta_o}{C_o} \quad (3.5)$$

The remaining C_b can be replaced using Equation 3.4 and applying the following the shallow water relationships and asymptotes

$$E_b = \frac{1}{8}\rho g H_b^2 \quad (3.6)$$

$$H_b = \kappa h_b \quad (3.7)$$

$$C_b = \sqrt{g h_b} \Rightarrow h_b = \frac{C_b^2}{g} \quad (3.8)$$

where κ is the breaker index taken to be 0.78 and h_b is the water depth at which waves break. Therefore,

$$H_b = \frac{\kappa C_b^2}{g} \quad (3.9)$$

The deepwater equations are

$$E_o = \frac{1}{8}\rho g H_o^2 \quad (3.10)$$

$$L_o = \frac{gT^2}{2\pi} \quad (3.11)$$

$$C_o = \frac{L_o}{T} = \frac{gT}{2\pi} \quad (3.12)$$

in which L is the wavelength and T is the wave period. Substituting these values and rearranging gives

$$Q_\ell = \frac{KH_o^{2.4} g^{0.6} T^{0.2} \sin 2\theta_o}{16(s-1)(1-p)2^{1.4}\pi^{0.2}\kappa^{0.4}} \frac{\cos^{0.2}\theta_o}{\cos^{0.2}\theta_b} \tag{3.13}$$

Approximating the cosine ratio by one and expressing the deepwater wave angle, $\theta_o = \beta - \alpha$, as a function of shoreline orientation, β, and direction of wave propagation, α, result in the net littoral drift being represented as

$$Q_N = \hat{Q} \sin 2(\beta - \alpha) \tag{3.14}$$

where

$$\hat{Q} = \frac{KH_o^{2.4} g^{0.6} T^{0.2}}{16(s-1)(1-p)2^{1.4}\pi^{0.2}\kappa^{0.4}} \tag{3.15}$$

Expanding the trigonometric relationship reveals the positive and negative drift rates

$$Q_N = \hat{Q} \sin 2\beta \cos 2\alpha - \hat{Q} \cos 2\beta \sin 2\alpha \tag{3.16}$$

where the first term on the right hand side is the positive drift and the second term is the negative drift. The positive drift is denoted as transport to the right looking offshore. Therefore, for Delaware positive drift is to the south and negative drift is to the north.

Two other values that can be deduced from this procedure are the total sediment transport, Q_T,

$$Q_T = \sqrt{\left(\hat{Q} \cos 2\alpha\right)^2 + \left(\hat{Q} \sin 2\alpha\right)^2} = \hat{Q} \tag{3.17}$$

and the effective deep water wave direction, α_T,

$$\alpha_T = 0.5 \tan^{-1}\left(\frac{\hat{Q} \sin 2\alpha}{\hat{Q} \cos 2\alpha}\right) \tag{3.18}$$

The effective deepwater wave direction is the wave angle that would result in zero transport for a shoreline normal oriented in the same direction. These two terms are related to the net drift, Q_N, by the following equation

$$Q_N = Q_T \sin 2(\beta - \alpha_T) \tag{3.19}$$

Assumptions made in the above calculations are as follows

(i) Linear theory is valid for the wave transformation process,

(ii) Bottom topography is composed of straight and parallel bottom contours (necessary for application of Snell's Law of Refraction),

(iii) No drastic changes in the bottom are encountered in the shallow areas seaward of the breaker line up to the beach,

(iv) Littoral drift is dependent on wave action (rather than on tidal currents or wind-driven transport), and

(v) There is an adequate supply of sand available for transport.

3.2 Data Sources

Because littoral drift rates can be expressed in deepwater conditions, potential longshore sediment transport rates can be estimated using CERC's Wave Information Study (WIS) hindcast wave estimates. This study provides data sets of wave conditions generated from monthly wind information and verified using wave buoy data. The result is a time series of wave and wind data at three-hour intervals for twenty-year time periods.

Two hindcast data sets are available for each station. The first covers the time period from 1956 to 1975 and the second from 1976 to 1995. The difference between these two databases is that the latter includes storms and extreme events while the former is purely operational conditions. Because these energetic events are responsible for significant sand transport, the transport rates are determined using the data from 1976 to 1995.

There are three stations off the coast of Delaware. For this study, station 2066 located in a water depth of 18 m was used to represent the offshore climate. The wave climate for this station during 1976 to 1995 is illustrated in a wave rose in Figure 3.1. The mean wave height is less than 2 m 92% of the time. In addition, the peak wave period is only greater than 11 seconds 15% of the time with almost 50% of the waves shorter than 7 s. As for the waves that are traveling towards the Delaware coastline, 73% are within 90 to 135 degrees relative to the north. The summary of the wave environment provided by the Coastal Engineering Data Retrieval System (CEDRS) is located in Appendix C.

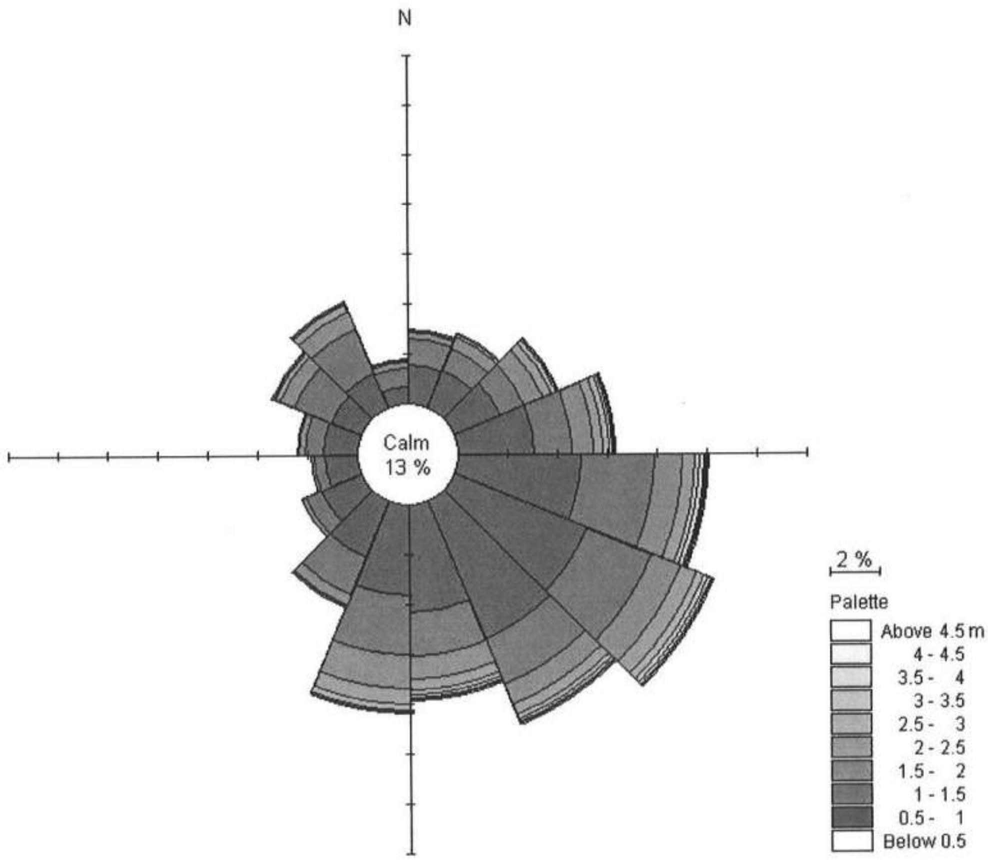


Figure 3.1: Wave rose for WIS Station 2066.

3.3 Littoral Drift Roses

In 1973, Walton and Dean developed a tool to graphically represent the longshore sediment transport in a functional "rose" format using offshore wave data. These littoral drift roses are generated by plotting the littoral drift on a polar plot using the energy flux methodology described previously. A valuable feature of the roses is that they are independent of the shoreline configuration. Therefore, the longshore transport can be determined for any shoreline orientation. For the Delaware Atlantic coast, the Dewey and Rehoboth, Bethany, and Fenwick coastlines are oriented at 82, 87, and 94 degrees, respectively, measured clockwise from the north.

Utilizing a computer program written using Equation 3.14, the longshore transport rates for the Delaware coast were determined. The resulting littoral drift roses are provided in the following figures. Figure 3.2 presents the total positive and negative drift, as well as the total drift in cubic yards per year. In addition, Figure 3.3 provides the potential net positive and negative littoral drift rates which, as mentioned earlier, is the sum of the total positive and negative drift.

As indicated by the roses, the coast experiences a net littoral drift to the north for the majority of the time. This is consistent with previous findings and observations. However, these values are extremely larger than anticipated and significantly overestimate the actual transport experienced along the Delaware coast. For example, Figure 3.3 estimates the potential net littoral drift along the coast to be approximately 2 million cubic yards per year. If the average wave conditions of 1.0 m, 7.0 s, and 110 deg are used in Equation 3.14, the result is a net littoral drift rate ranging between 2 to 3 million cubic yards per year.

The reason for this error is unclear. Regardless, it is clear that the littoral drift estimations are incorrect, especially when compared to bypassing volumes at Indian River Inlet which are on the order of 100,000 cubic yards per year. Therefore, this information is not considered in further investigations.

The littoral drift roses are also useful in identifying special features in the shoreline behavior. For example, occasionally nodal points occur for a coast with varying shoreline orientations. A nodal point is a location for which the average positive and negative transports have the same magnitudes, thus resulting in no net drift. On a littoral drift rose, this point is represented as the intersection between the total positive and negative littoral drifts, or where the net littoral drift is zero.

A nodal point has been identified along the southern Delaware coast in the vicinity of Bethany and Fenwick beaches. In 1986, Mann and Dalrymple performed an analysis of the nodal point in Delaware. It was estimated to be in the vicinity of York Beach located approximately seven miles south of Indian River Inlet. However, it is mentioned that its position varies annually and can be located as far north as Indian River Inlet and as far south as Ocean City, Maryland.

The nodal point as shown in Figures 3.2 and 3.3 is expected to occur for a shoreline oriented 120 deg relative to the north. Unfortunately, this shoreline orientation is not present along the Delaware coast for which the wave conditions are representative. It is surprising that the data is inconclusive in detecting a nodal point as previous studies have at a minimum located a general vicinity for this point. Therefore, these results are also not considered further.

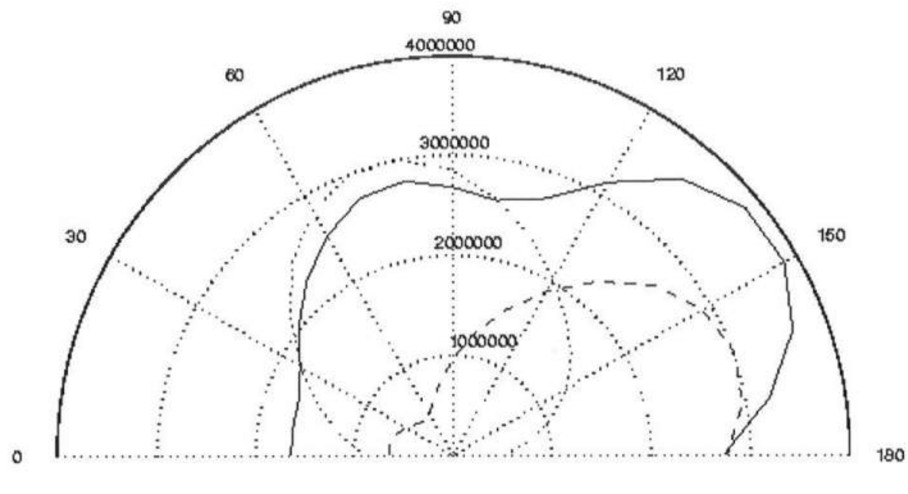


Figure 3.2: Annually averaged total littoral drift in cubic yards per year for Delaware Atlantic coast. (——— total drift; – – – – – total drift in (+) direction; ………… total drift in (-) direction)

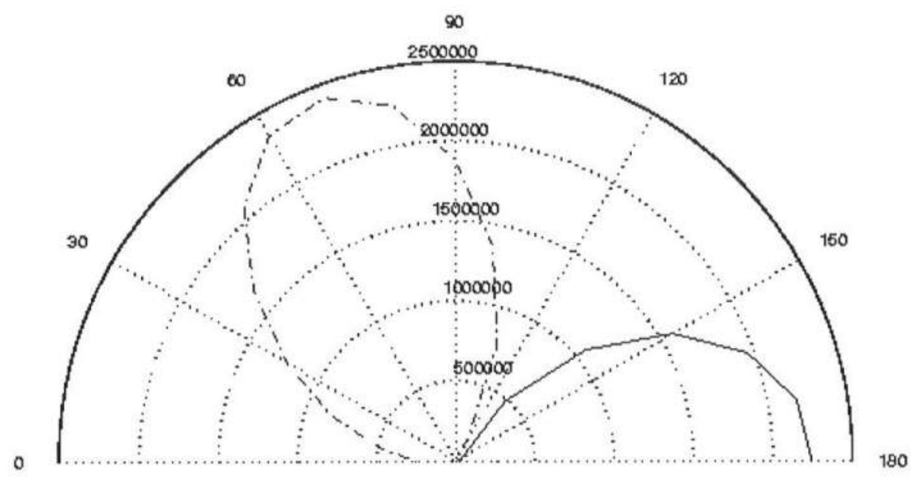


Figure 3.3: Annually averaged potential net littoral drift in cubic yards per year for Delaware Atlantic coast. (——— net drift in (+) direction; - · - · - net drift in (-) direction)

Chapter 4

BEACH NOURISHMENT

With more than 50% of the population of the United States living within 50 miles of the coastline, and more than 85% of the shorelines eroding, it is no surprise that towns and governments have turned to shore protection to restore and maintain the beaches. As mentioned earlier, there are various methods that have been employed to rebuild beaches and protect the structures lining them. While hard structures such as groins, breakwaters, and seawalls are effective in trapping sediment, they starve downdrift beaches of sand, limit the recreational value of the beach, and are difficult to remove. Therefore, there has been a recent tendency to stay away from these intrusive structures for relief from erosion. In contrast, because beach nourishment maintains the natural environment while still protecting the coastline, it has become the most accepted approach for beach restoration and will therefore be the focus of this section.

There are many factors to which a beach is exposed. Each of which has its own impact on the behavior of the shoreline. While it is valuable to understand these influences, it is important to realize that beach nourishment does little to alter the erosive factors affecting a beach. Therefore, this section analyzes placement options as well as the behavior of the fill after placement in order to optimize the lifetime and performance of the fill. Because beach fills are a temporary relief for a receding shoreline rather than a solution, renourishment is often necessary once the sand from the initial fill is depleted. For that reason, renourishment over a long period of time is also a subject of investigation here.

4.1 Equilibrium Profiles

Before the alteration of the existing beach is investigated, it is important to explore the natural behavior of the beach. The coastal environment is a highly dynamic system of actions and reactions between the land and the sea, each working to protect itself and ensure its continuance. While the complexity of their relationship will never be fully understood, a basic assumption of equilibrium is presented here in order to represent a stable condition between the land and sea that will be useful for further investigations.

This concept is based on the idea that a beach exposed to a continuous wave environment will eventually establish a stable profile. While it is understood that the forces acting on a beach are constantly changing, this concept provides a platform with which to investigate how various forces alter the beach. An empirical relationship between the sediment diameter and profile shape was developed by Dean (1983) to describe this equilibrium profile and is given as

$$h(y) = Ay^n \cong By^n D^{\frac{1}{3}} \tag{4.1}$$

where

$h(y)$ is the water depth,

y is the distance offshore,

A is the sediment scale parameter approximately proportional to $D^{\frac{1}{3}}$,

n is an exponent usually taken to be ? ,

B is an unknown variable which is case dependent, and

D is the median sand grain diameter.

This equation results in a concave upward shape that is more mildly sloped for finer sediments. Figure 4.1 shows a relationship between the sediment diameter and the sediment scale parameter. As D increases, A increases, and the equilibrium profile becomes steeper. Therefore, a coarse material will result in a steeper sloped profile than a fine material. Nevertheless, the sediment tends to be sorted along the profile with the coarser sediments located in shallow water and the finer material in deeper water. These will become valuable concepts in determining how a beach fill will behave after it is placed.

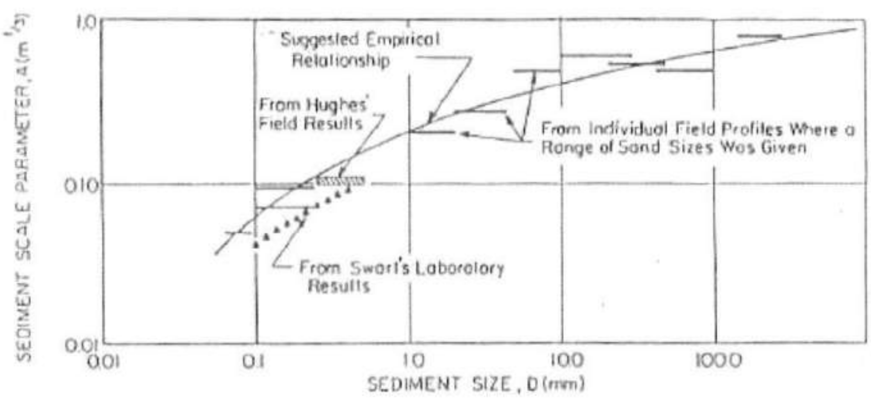


Figure 4.1: Relationship between the sediment diameter and the sediment scale parameter. (Dean, 1987; modified from Moore, 1982)

4.2 Quality of the Borrow Material

When choosing a sand source, it is important to know the characteristics of the sediment. The compatibility of the fill material with the native material is a primary indicator of the success of the project. Ideally, the fill and native sediment will have the same characteristics. However, in most cases, this is not feasible. From there the question arises; is sand coarser or finer than the native more suitable as beach protection? Because coarser sediment is more stable, it is more likely to be less affected by the wave and current forces. On the other hand, sediment coarser than sand limits the recreational function of the beach. Finer material is more susceptible to erosional forces and will most often be carried offshore and out of the project area. Therefore, sediment finer than the native is less effective and should not be used for nourishment. The optimal choice is to use sediment as coarse or slightly coarser than the native. This will ensure the livelihood of the beach fill while maintaining its recreational value.

Based on the equilibrium profile concept, it can be assumed that when sand is added to a beach, it will adjust to a profile characteristic of the fill material. Therefore, if sediment the same size as the native material is placed on a beach, it will equilibrate to a profile identical to the original profile. If the sediment is coarser, there will be an adjustment to the profile associated with the coarse sediment and so on. Figure 4.2 illustrates three types of nourished profiles most likely to occur based on the volume added and the size of the fill material relative to the native sediment. The sediment scale parameter rather than the mean sediment diameter is used for the comparison between the fill material, A_F, and the native material, A_N. The profiles are described as intersecting, non-intersecting, and submerged depending on the fill material being coarser than, equal to, or finer than the existing sediment, respectively.

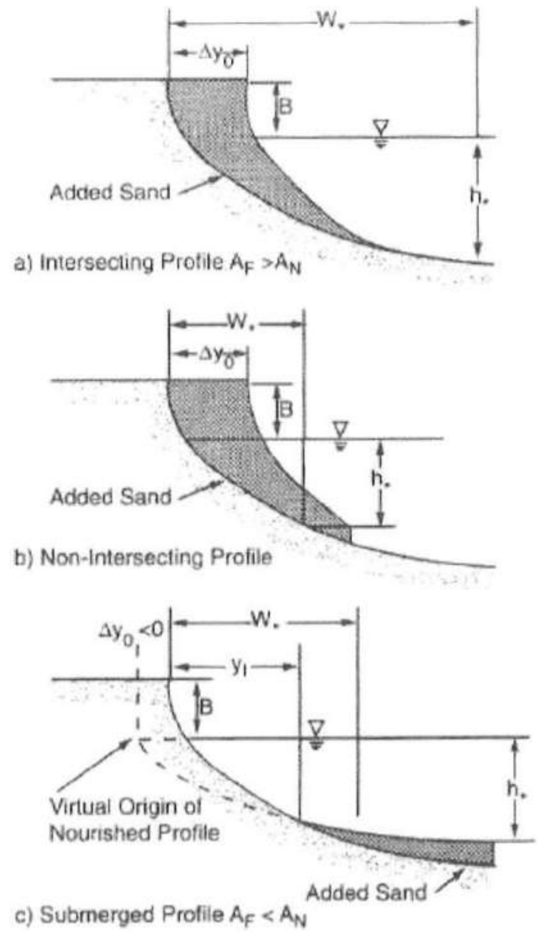


Figure 4.2: Three types of nourished profiles. (Dean, 1991)

Now, if the fill material that is equal to the native is placed in a profile shape the same as the original, it is safe to assume that there will be little or no adjustment period necessary for the fill. However, placing fill in such a precise manner and shaping it offshore is a difficult and costly operation. Since waves and sediment interact to achieve equilibrium, it is more reasonable to place the material onshore with a steep slope and allow the waves to shape the beach accordingly.

As an aside, it is not always necessary for the fill to be placed onshore. Some experiments have investigated placing the fill offshore of the eroding beach and allowing wave action to move the material to the beach. This too would be more reasonable than shaping the beach manually, however, the material is not always moved to the beach by natural processes. In contrast, the sediment could also be placed within the dune system, thus allowing the wind to be a contributing factor in the distribution of the material.

Regardless of where the fill is placed, there is the initial redistribution of the beach fill to its equilibrium position, and there is the initial loss of the fines present in the fill. When the sediment is placed onshore and adjustment occurs, less beach is visible. This does not indicate that the sediment is lost. Rather, the sediment is now positioned offshore thus creating a more stable profile. In addition, the movement of this sediment offshore results in shallower water which will cause the waves to break further offshore, thus providing more protection.

Regardless, due to the demand from visitors, it is necessary that a wide dry beach be available during the tourist season. Therefore, it is important to make certain that a beach will be present after readjustment occurs. This resultant dry beach width is also a function of the sediment size of the fill material. Due to the steeper profile that results from using coarser material, a wider beach is associated with coarser beach sand. Figure 4.3 illustrates the beach width and profile shape resulting from placing the same volume of various sized sediment. In Figure 4.3a, the sediment is coarser than the native, and a steep intersecting profile with a wide beach results. Figure 4.3b shows a non-intersecting profile with a slightly more narrow beach resulting when the fill and native material are the same. In Figure 4.3c, the fill material is finer than the existing sediment and a narrow beach with a mildly sloped, non-intersecting profile will occur. Figure 4.3d illustrates the limiting case in which no dry beach occurs and all the sand is used to create a mild offshore slope.

In summary, in order to achieve a wide beach with finer material, a larger volume of material is required which becomes more expensive when coarser sediment is available as a source. While finer material remains less resistant to erosive forces than coarser material, there is more sediment to erode and a mild offshore slope is still present. As a result, there becomes a balance that must be achieved between maximizing the dry beach width while still maintaining a mild underwater slope. In reality, economics rather than engineering become the decisive aspect in optimizing beach fill performance. Factors such as availability of material, proximity of the source to the project site, mobilization, demobilization, necessary maintenance, and available funds are most likely to limit the project options than sediment properties.

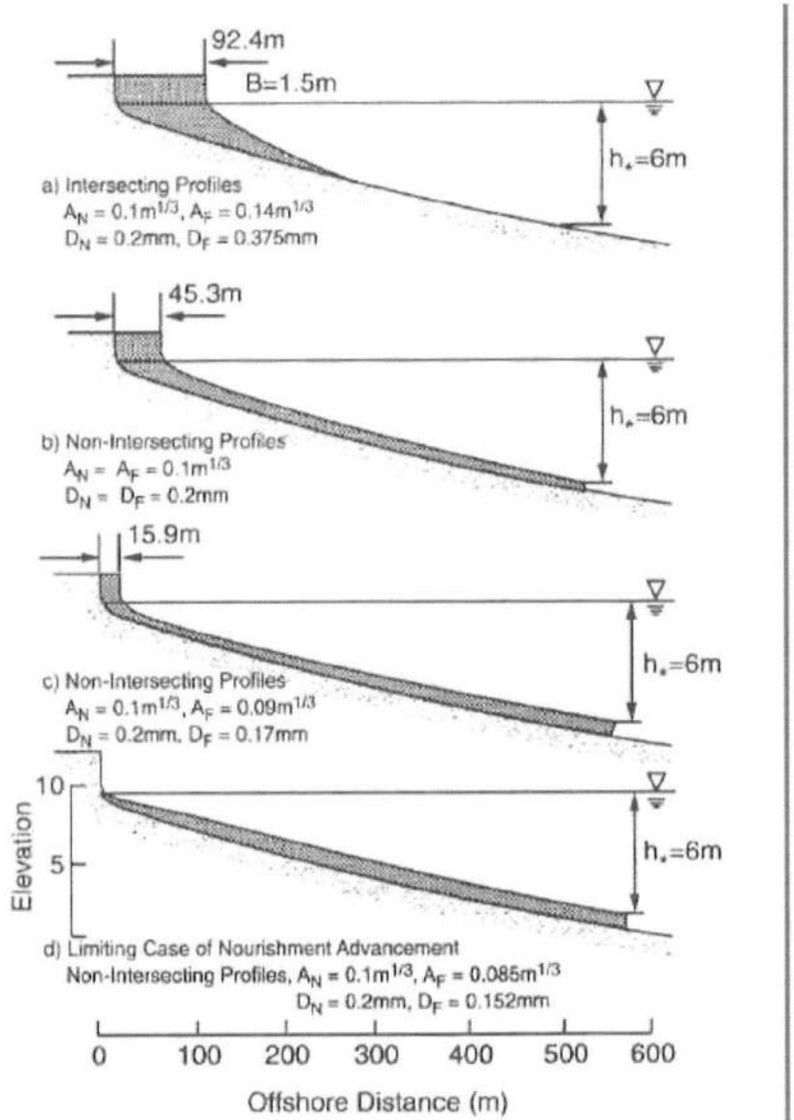


Figure 4.3: Effect of nourishment scale parameter, A_F, on width of resulting dry beach. Four examples of decreasing A_F with same volume per unit beach length added. (Dean, 1991)

4.3 Beach Nourishment Behavior

Until now the behavior of beach fills has been explored through profile cross-sections which are representative of the cross-shore sediment transport. This section will study the longshore transport of the beach fill by observing the planform response of the nourishment with the goal of maximizing the fill lifetime. In addition, the dimensions of the fill will be considered to determine the optimal shoreline configuration.

In 1956 Pelnard-Consideré developed a model based on the representation of a single contour which is, in most cases, taken as the shoreline. This model is essentially a diffusion equation that represents the "spreading out" action of a beach fill. By assuming equilibrium beach profiles, any location on the profile can be determined if one position is known. This type of model is called a one-line model and will also be discussed in Chapter 5. It is important to note one of the limitations of this model. It assumes that the variation of the shoreline orientation is small. Fortunately, the Delaware coastline is generally regular and does not experience any significant changes in its orientation.

The one-line model is based on the combination of two primary concepts: the sediment transport equation mentioned earlier in Chapter 3 and expressed in the form

$$Q = \frac{K H_b^{5/2} \sqrt{\frac{g}{\kappa}} \cos 2(\beta - \alpha_b)}{16(s-1)(1-p)} \tag{4.2}$$

and the conservation of sand represented as

$$\frac{\partial y}{\partial t} + \frac{1}{(h_* + B)} \frac{\partial Q}{\partial x} = 0 \tag{4.3}$$

By combining these two equations and assuming $\partial y/\partial x << 1$, the Pelnard-Consideré equation becomes

$$\frac{\partial y}{\partial t} = G\frac{\partial^2 y}{\partial x^2} \tag{4.4}$$

The variable G is called the shoreline diffusivity and is defined as

$$G = \frac{KH_b^{5/2}\sqrt{\frac{g}{\kappa}}\cos 2(\beta - \alpha_b)}{8(s-1)(1-p)(h_* + B)} \tag{4.5}$$

Note that G has the dimensions of length²/time.

For a rectangular fill with alongshore length, ℓ, and offshore width, the evolution of the fill can be expressed as

$$y(x,t) = \frac{\Delta y_o}{2}\left\{erf\left[\frac{\ell}{4\sqrt{Gt}}\left(\frac{2x}{\ell}+1\right)\right] - erf\left[\frac{\ell}{4\sqrt{Gt}}\left(\frac{2x}{\ell}-1\right)\right]\right\} \tag{4.6}$$

where the error function is defined as

$$erf(z) = \frac{2}{\sqrt{\pi}}\int_0^z e^{-u^2}du \tag{4.7}$$

The coordinate system for this equation is defined in a manner in which x is the alongshore axis, and y is the distance offshore.

Equation 4.6 is expressed graphically in Figure 4.4. It can be seen that the ends of the fill diffuse outward as the ocean works to restore equilibrium. Notice that only half of the evolution is shown. This is because the fill progression is symmetrical regardless of the wave direction. This is explained through the effects of the longshore sediment transport. The updrift side acts like a shore normal structure trapping sediment as it

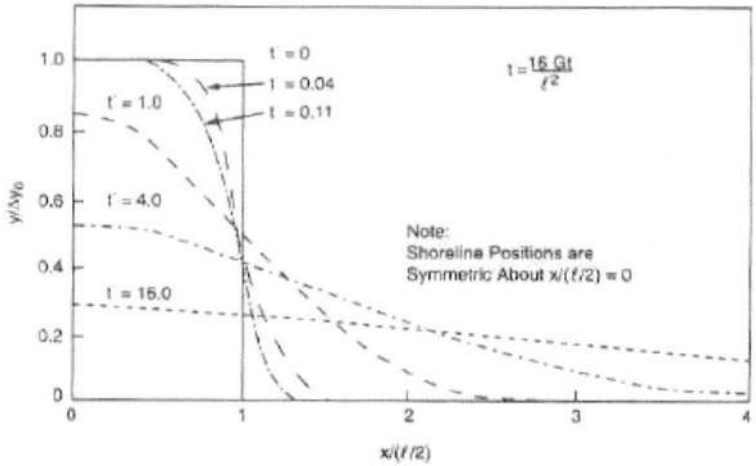


Figure 4.4: Non-dimensional shoreline evolution for rectangular fill. (Dean,1983)

travels alongshore. The downdrift portion serves as a sand source and is spread out along the coast by way of the littoral drift. This is why beach fills are sometimes placed slightly updrift of the depleted beach. Therefore, while beach nourishment traps sand, it also introduces sand into the system. In fact, beach nourishment is the only method of beach restoration that addresses the lack of sediment within the system.

Because the ocean works to return the irregular shoreline back to a more natural state after a fill, it would seem reasonable to shape the fill with tapered ends in order to slow the rate at which the reconfiguration occurs. The tapered ends have the effect of reducing the evolution of the fill in its initial stages and ultimately increasing its lifetime. This is the type of fill that is placed in Ocean City, Maryland with the taper extending into Fenwick Island. A comparison of this type of fill with a rectangular fill is shown in Figure 4.5 (Dean, 1983). Table 4.1 quantifies these changes in terms of the cumulative losses from the area over the first five years. Notice that the tapered fill experiences a loss that is 33% less than the rectangular fill.

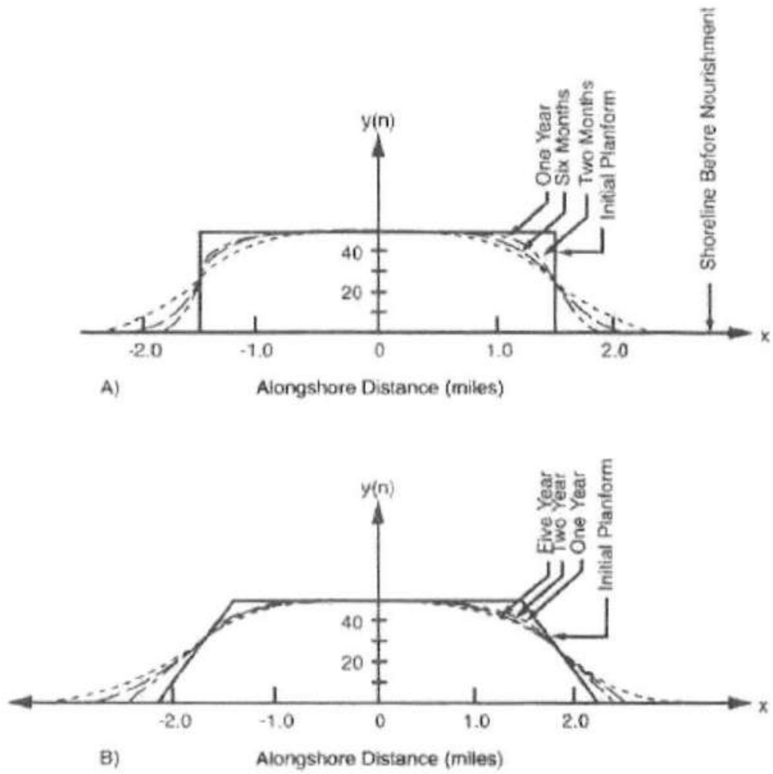


Figure 4.5: Comparison of diffusive losses between rectangular fill and fill of the same volume with tapered ends. (Dean, 1993)

Table 4.1: Comparison of Cumulative Percent Losses for Rectangular Fills and Fill with Tapered Ends. (G=0.2 ft^2/sec; ℓ=3 miles; Y=55 ft) (Dean, 1983)

Years After Placement	Cumulative Percent Loss	
	Rectangular Planform	Rectangular Planform with Tapered Ends
1	5.7	2.4
2	9.5	4.6
3	11.8	6.6
4	13.8	8.3
5	15.5	9.8

4.4 Periodic Renourishment

Because this study entails the 50-year maintenance of the Delaware coast and because it is highly unlikely that one beach nourishment project will survive for 50 years, the concept of periodic renourishment must be examined fully. Determining an appropriate renourishment period will enable the state of Delaware to budget for the on-going presence of a recreational beach, in turn guaranteeing a continuous revenue from coastal tourism. Therefore, the lifetime and evolution of various renourishment scenarios will be investigated in this section.

There are many assumptions that are made in estimating the longevity and renourishment rate of a particular beach fill. Primarily, it must be understood that the impact of extreme events such as northeasters or tropical storms is not included in the analysis. The life of each project will vary significantly depending on the climatic conditions. In addition, the annual loss of fill material from the site may be higher than the average erosion rate. This is due to the heightened objective of the wave environment to reduce the perturbation in the shoreline. Lastly, this approach assumes an unlimited supply of sediment. That is, the constant renourishment will always be possible due to a readily available sand source. It is because of these unanticipated and unpredictable factors that a contingency plan must also be included in the initial formulation of the project.

In determining the renourishment rate for a beach it is necessary to understand how a fill behaves when another is introduced to the system. More specifically, how the planform will react to a periodic addition of sediment to the system. Equation 4.6 can be used to express the planform evolution of renourished beaches as well as those that receive a single fill. However, it must be determined at what point during a fill's progression is it most ideal to renourish. The optimum fill is decided to be the renourishment scheme that maximizes its longevity while minimizing the required volume of material.

For this study, fills were allowed to experience volume losses of 25%, 50%, and 75% before another was placed. The 50% case is a special situation. The time it takes for 50% of the fill material to leave the project area is described as the half-life and is a useful parameter in characterizing beach fill behavior. In addition, notice that the scenario for a 100% loss of the beach fill is not illustrated here. According to Equation 4.6, the beach fill never fully leaves the project site. This has a tendency to overestimate the lifetime of the fill. Nevertheless, this analysis will allow for some speculations to be made concerning this case.

Figures 4.6, 4.7, and 4.8 show the planform behavior of the multiple nourishments for losses of 25%, 50%, and 75%, respectively. Note that time is expressed in its dimensionless form, $t' = 16Gt/\ell^2$. Each graph illustrates the initial fill and the final planform configuration before renourishment for three nourishment projects. Notice that each time another fill is placed, the juncture between fill and the existing shoreline is lessened.

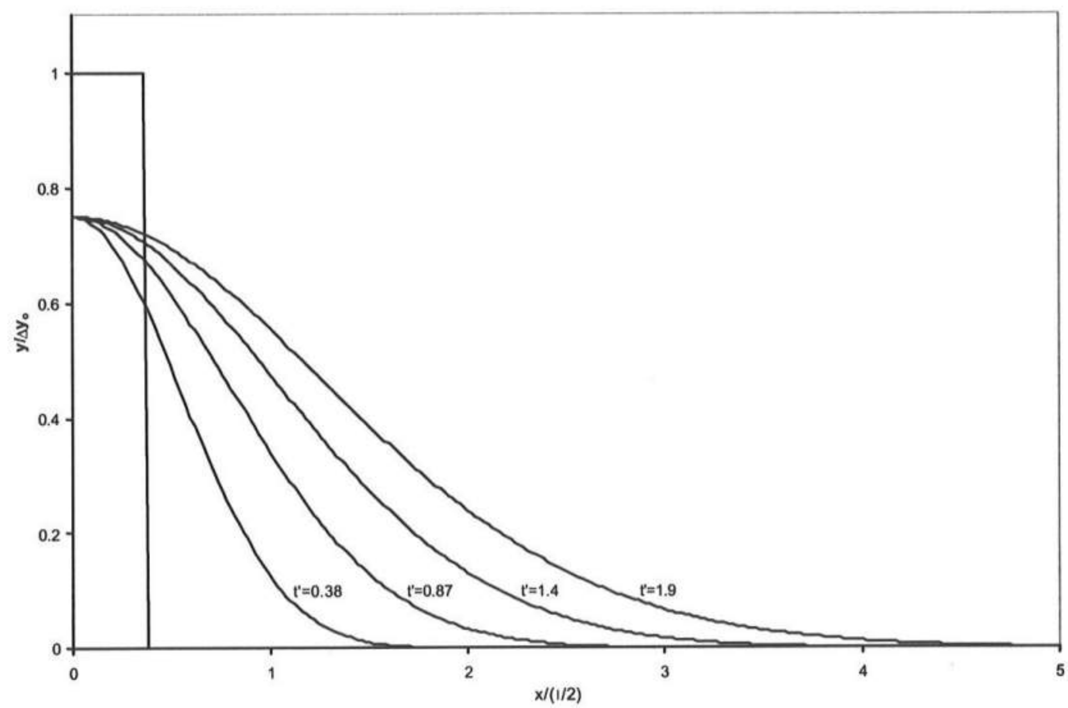


Figure 4.6: Dimensionless planform evolution for project allowed to experience a 25% loss before renourishment.

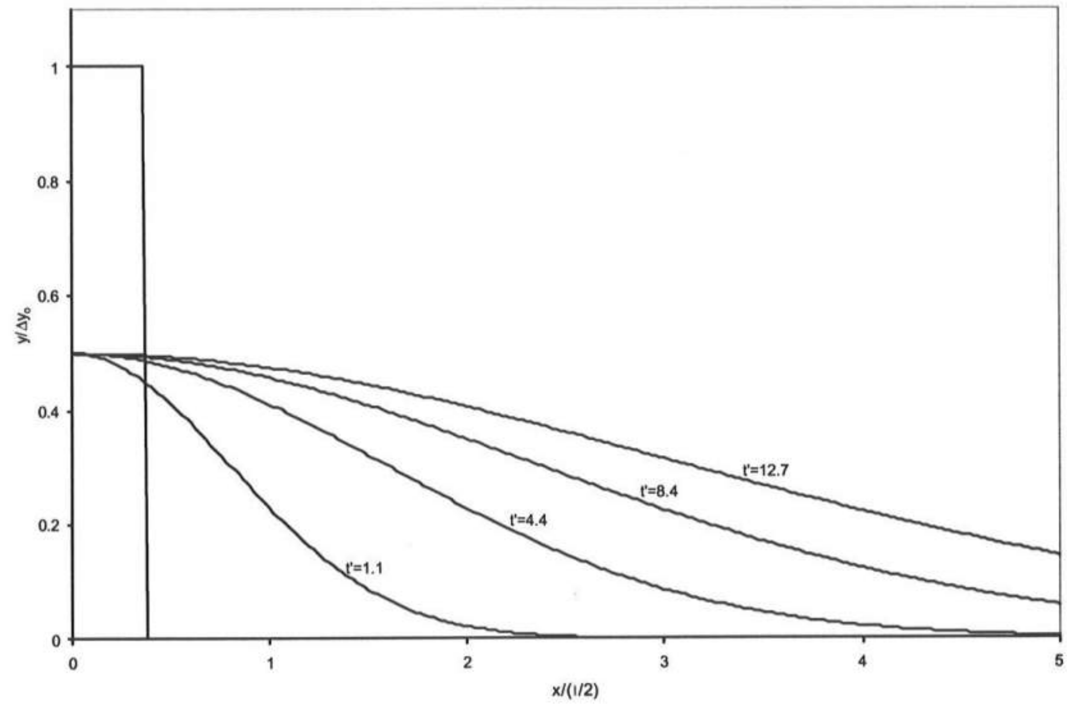


Figure 4.7: Dimensionless planform evolution for project allowed to experience a 50% loss before renourishment.

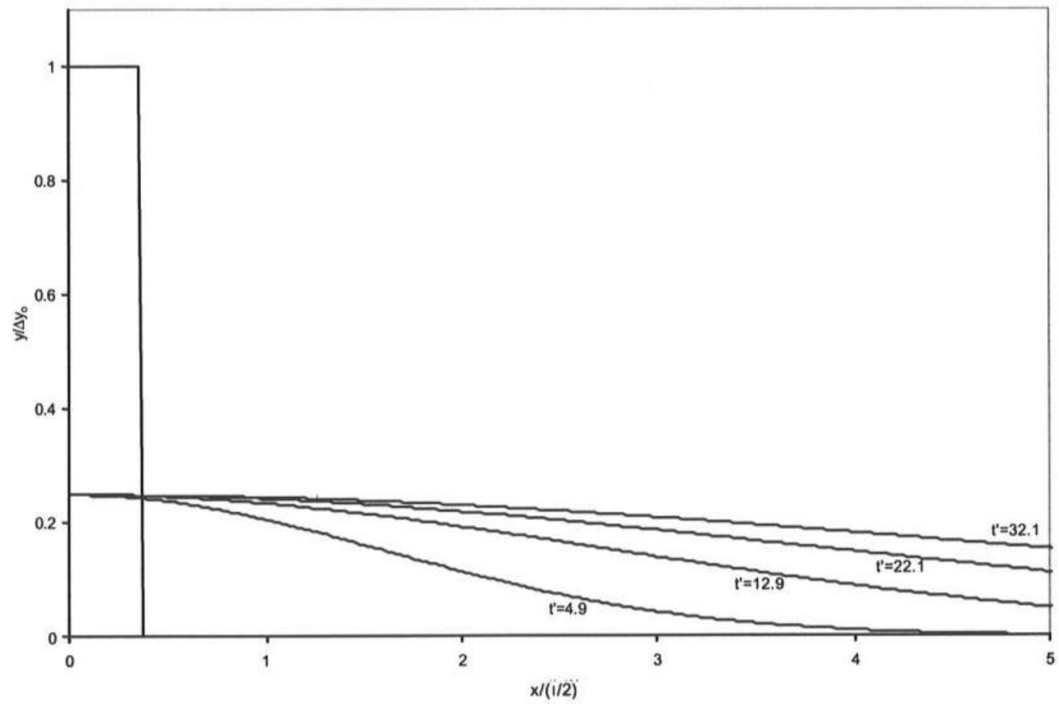


Figure 4.8: Dimensionless planform evolution for project allowed to experience a 75% loss before renourishment.

Next, it must be determined which of these scenarios provides the longest protection. The life expectancy of a nourishment project due to diffusive losses can be determined using the beach planform response discussed earlier. By defining the proportion of sand, $M(t)$, remaining at the project site as

$$M(t) = \frac{1}{\Delta y_o \ell} \int_{-\ell/2}^{\ell/2} y(x,t)\,dy \tag{4.8}$$

and substituting in Equation 4.6 for $y(x,t)$, the proportion of remaining sand is described as

$$M(t) = \frac{\sqrt{4Gt}}{\ell\sqrt{\pi}}\left(e^{-\left(\ell/\sqrt{4Gt}\right)^2} - 1\right) + erf\left(\ell/\sqrt{4Gt}\right) \tag{4.9}$$

This expression is valid for

$$\frac{\sqrt{Gt}}{\ell} < \frac{1}{2}$$

For values larger than one-half,

$$M(t) \cong \frac{1}{2\sqrt{\pi}}\frac{\ell}{\sqrt{Gt}} \tag{4.10}$$

Figures 4.9, 4.10, and 4.11 illustrate the proportion of material remaining after the initial fill for projects that are allowed to experience losses of 25%, 50%, and 75% of the original volume, respectively, before it is refilled. For each scenario, eight renourishment projects are performed after the initial fill. Note that in all cases, the need for a renourishment occurs less frequently over time. This is due to the overall increase of sediment within the system.

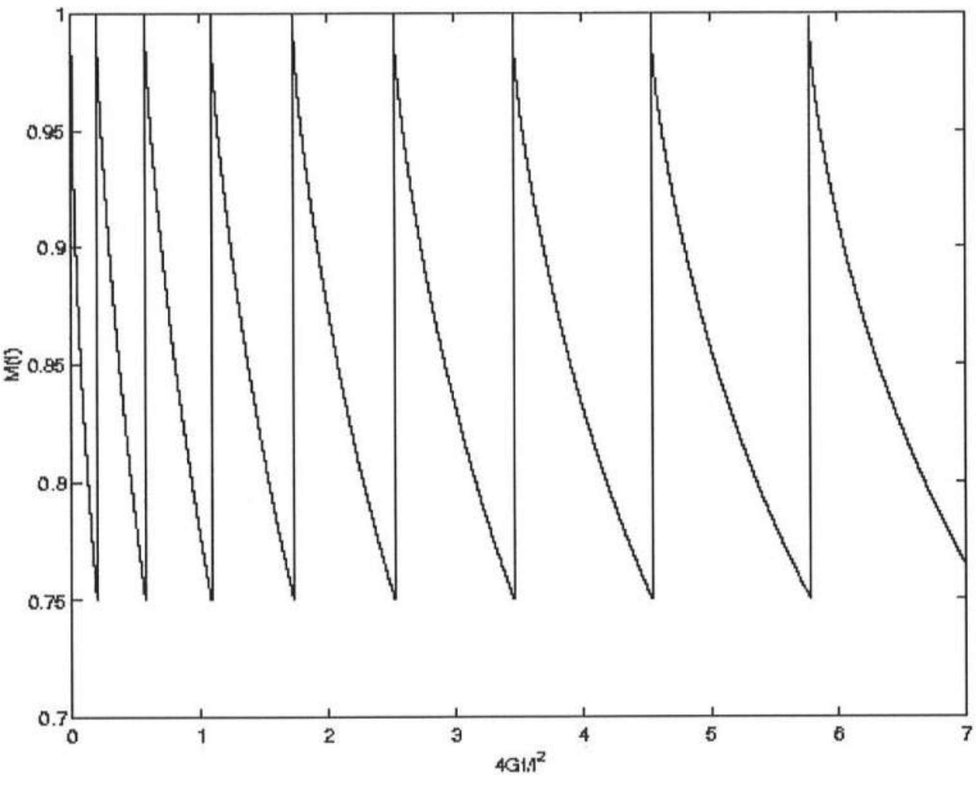


Figure 4.9: Percentage of material remaining in project area versus dimensionless time for fills allowed to experience a 25% loss before renourishment.

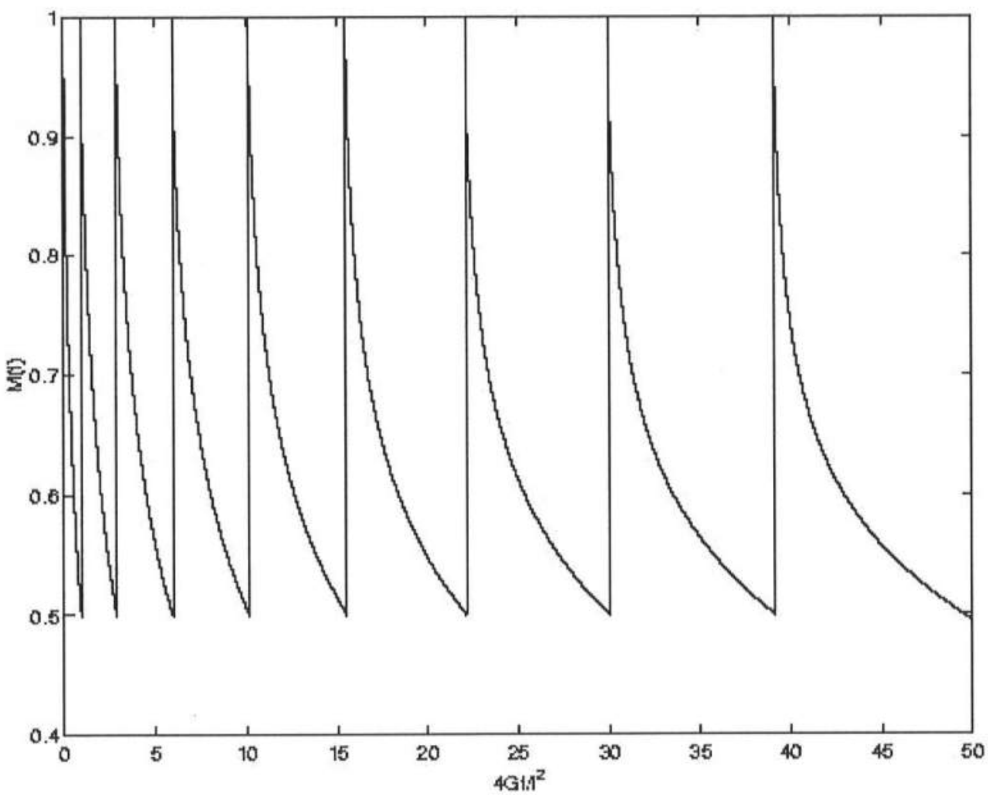


Figure 4.10: Percentage of material remaining in project area versus dimensionless time for fills allowed to experience a 50% loss before renourishment.

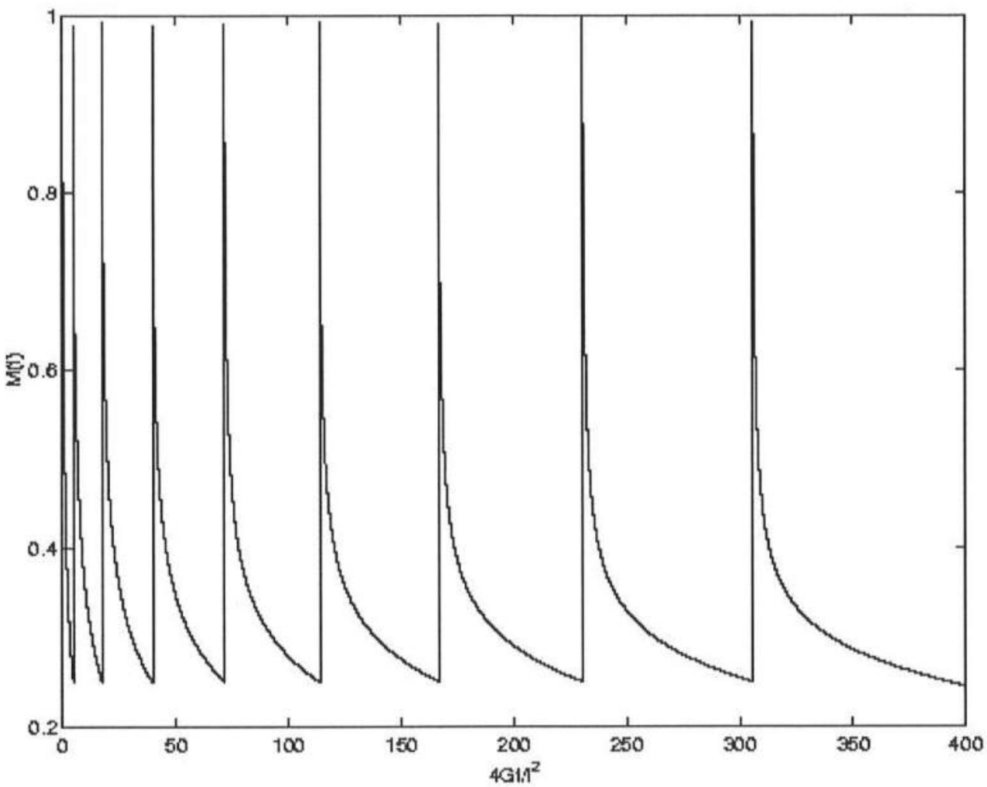


Figure 4.11: Percentage of material remaining in project area versus dimensionless time for fills allowed to experience a 75% loss before renourishment.

By allowing all three scenarios to occur for the same amount of time, it can be determined which scenario experiences the highest amount of sediment loss. Figure 4.12 illustrates the total percentage of material lost for each case over the same period of time. Notice that the longer the fill is allowed to spread out, the less amount of material required for each renourishment over time. This would lead to the conclusion that it is more feasible to allow the fill to diffuse as long as possible before refilling. This assumption is further supported by the equilibrium profile theory. Each time a fill is placed, an adjustment occurs. Therefore, it is more reasonable to lessen the number of fills in order to reduce the total adjustment losses that occur with each renourishment. In addition, each time a nourishment project is performed, there are high initial and closing costs that are incurred such as feasibility studies, sediment source investigations, pre- and post-fill surveys of the shoreline, and mobilization and demobilization. Therefore, it is more practical and economically feasible to limit the number of times these costs must be incurred.

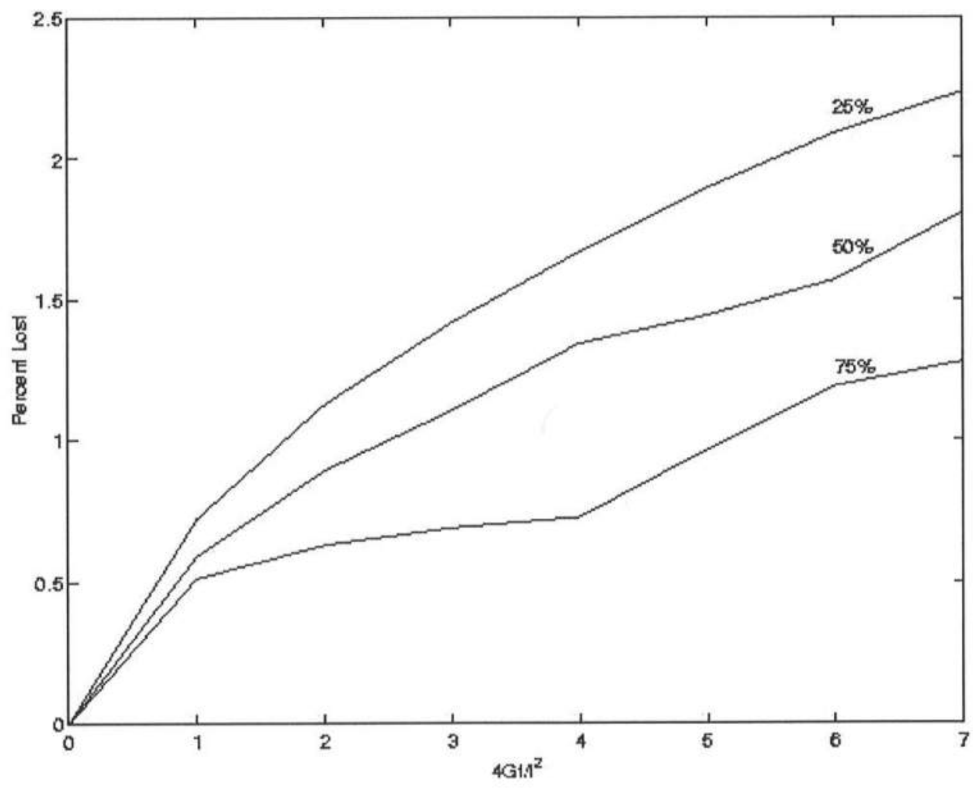


Figure 4.12: Percentage of material lost in project area versus dimensionless time for various renourishment scenarios.

Chapter 5

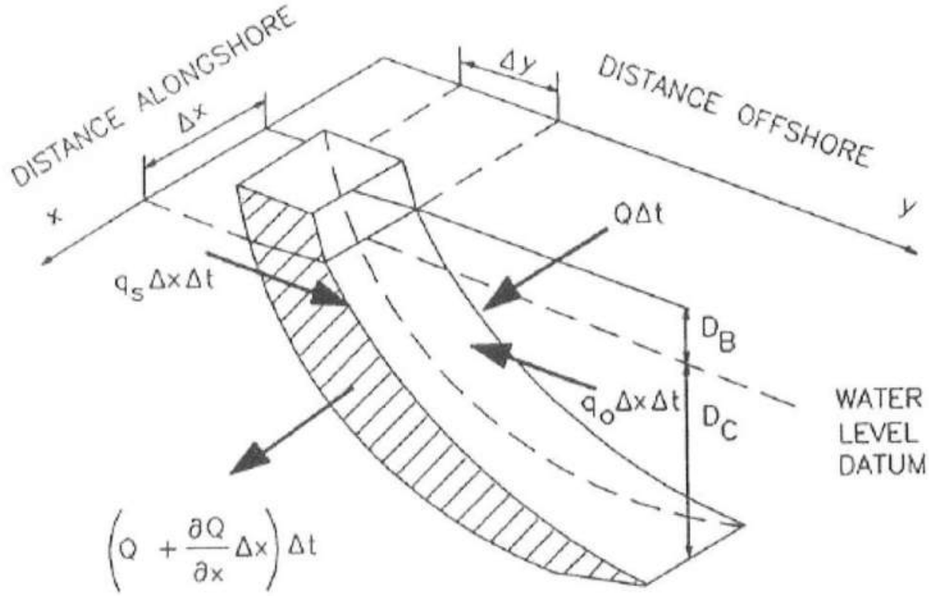
BEACH FILL EVOLTION USING THE GENESIS MODEL

Up to this point, various factors influencing beach morphology have been investigated on an individual basis. Each of these factors, while having their own impact on a coastline, are highly interrelated and work in conjunction with one another to create an extremely dynamic and complex system. However, actually combining them and allowing them to interact over a time period is a difficult computation numerically. A numerical model of long-term shoreline evolution known as the **GENE**ralized Model for **SI**mulating **S**horeline Change (GENESIS) has been developed by Hans Hanson and Nicholas Kraus at the US Army Corps of Engineers' Waterways Experiment Station (WES). This model is a simplified one-line model that allows for the representation of real life scenarios such as coastal structures and beach fills, and accounts for their interactions. This type of model is ideal because it provides a tool with which to investigate a variety of design alternatives. This ultimately allows for the optimization of a project's performance using realistic conditions. However, because it is generalized, it is recommended that the model not be the only method used in project design.

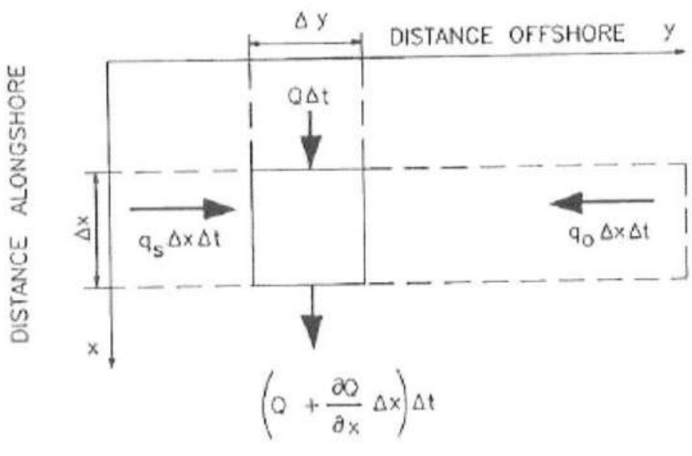
In this section, GENESIS will be used to predict the evolution of the Delaware coastline for a 50-year project life based on various nourishment scenarios. In addition, the theory and design of the model will be provided in order to understand its capabilities and limitations. Ultimately, a multiple renourishment cycle will be developed for each region of the Delaware coastline. It is important to note that while this chapter does provide a general introduction to the model, there are aspects of GENESIS that are not provided or discussed in detail. The model is discussed in much greater depth in Technical Report CERC-89-19: *GENESIS: Generalized Model for Simulating Shoreline Change; Reports 1 and 2 (1989).*

5.1 Introduction to GENESIS

GENESIS is a one-line numerical modeling system that simulates shoreline change produced by spatial and temporal changes in the longshore sediment transport. Like most models, GENESIS predicts the position of one contour which in most cases is taken to be the shoreline. As described earlier in section 4.3 for a one-line model, it is assumed that the beach profile translates seaward or landward while maintaining its characteristic shape. Using the right-handed Cartesian coordinate system illustrated in Figure 5.1, the position of the shoreline at an alongshore point, x, is denoted as y^*. The change in the shoreline position and the length of the shoreline segments are defined as Δy and Δx, respectively. In addition, the berm elevation, D_B, and the depth of closure, D_C, limit the profile vertically.



a. Cross-section view



b. Plan view

Figure 5.1: Coordinate system and profile definition sketch for the GENESIS modeling system.

The governing equation for the model is based on the conservation of sand volume. If the change in the volume of sand in a section is described by the net amount of sand that entered or exited the section from any of its four sides, then it is possible to express that volume change as $\Delta V = \Delta x \Delta y (D_B + D_C)$. Factors that contribute to changes in the volume include variations in the longshore transport rate, Q, and line sources and sinks, q. The net volumetric change for the longshore transport rate can be expressed as $\Delta Q \Delta t = (\partial Q/\partial x)\Delta x \Delta t$ and the source or sink as $q \Delta x \Delta t$. By adding these terms and taking the limit as $\Delta t \rightarrow 0$, the governing equation for the rate of change of the shoreline position is given by

$$\frac{\partial y}{\partial t} + \frac{1}{(D_B + D_C)}\left[\frac{\partial Q}{\partial x} - q\right] = 0 \tag{5.1}$$

5.1.1 Longshore Sediment Transport

The longshore sediment transport rate is expressed empirically in GENESIS as

$$Q = \left(H^2 C_g\right)_b \left[a_1 \sin 2\theta_{bs} - a_2 \cos\theta_{bs} \frac{\partial H}{\partial x}\right]_b \tag{5.2}$$

where

H is the wave height,

C_g is the wave group speed,

b is a subscript denoting breaking wave conditions, and

θ_{bs} is the breaking wave angle relative to the shoreline.

The coefficients a_1 and a_2 are non-dimensional and given by

$$a_1 = \frac{K_1}{16(\rho_s/\rho - 1)(1-p)(1.416)^{5/2}}$$

and

$$a_2 = \frac{K_2}{8(\rho_s/\rho - 1)(1-p)\tan\beta(1.416)^{7/2}}$$

where

K_1, K_2 are transport parameters, treated as a calibration constants,

ρ_s is the density of sand (2.65 10^3 kg/m3 for quartz sand),

ρ is the density of water (1.03 10^3 kg/m3 for seawater),

p is the porosity of sand on the bed (taken to be 0.4), and

$\tan\beta$ is the average bottom slope from the shoreline to the depth of active longshore sand transport.

The first term in Equation 5.2 corresponds to the "CERC formula" described in the SPM and described earlier in section 3.1. It accounts for the transport produced by obliquely incident waves. The second term in Equation 5.2 describes the longshore transport that is induced by the longshore gradient in the breaking wave height $\partial H_b/\partial x$. While the contribution of this term is usually much smaller than that from obliquely incident waves, it provides an improvement in the modeling results where diffraction due to structures produces a significant difference in the breaking wave height.

While the coefficients K_1 and K_2 have been empirically estimated, they are used in GENESIS as model calibration parameters and are referred to as transport parameters. Parameter K_1, originally determined by Komar and Inman (1970), controls the magnitude of the longshore sediment transport rate as well as the time scale of the shoreline change simulations. According to the GENESIS Technical Reference, K_2 is typically 0.5 to 1.0 times the value of K_1. It is not recommended to vary K_2 much beyond $1.0K_1$ as it may result in an exaggerated shoreline change in the vicinity of structures and numerical instability may occur.

5.1.2 Sources and Sinks

The change in the volume of sand in a section is also a function of any sources or sinks in the system. Examples of sources include rivers and cliffs, whereas sinks can be inlets and entrance channels. Wind blown sediment can be either a source or a sink. While the current version of GENESIS does not allow for the representation of sources and sinks, beach fills and bypassing volumes are possible and can be used to implement sources or sinks or direct changes in the shoreline position.

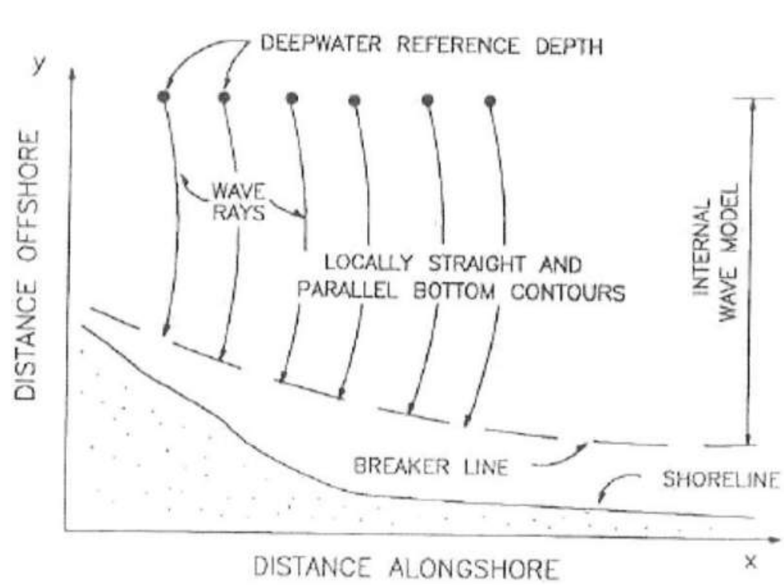
5.1.3 Wave Transformation

The GENESIS system is composed of two major submodels. The first model which has already been discussed, determines the longshore transport rate and shoreline change. The second is a wave model that calculates breaking wave height and angle based on wave information provided at a reference point offshore. Offshore wave information can be obtained from hindcast calculations or from a wave gage. However, GENESIS is designed in a way that is highly compatible with the WIS hindcast database. Therefore, WIS station 2066 offshore of the Delaware coast was use as wave input.

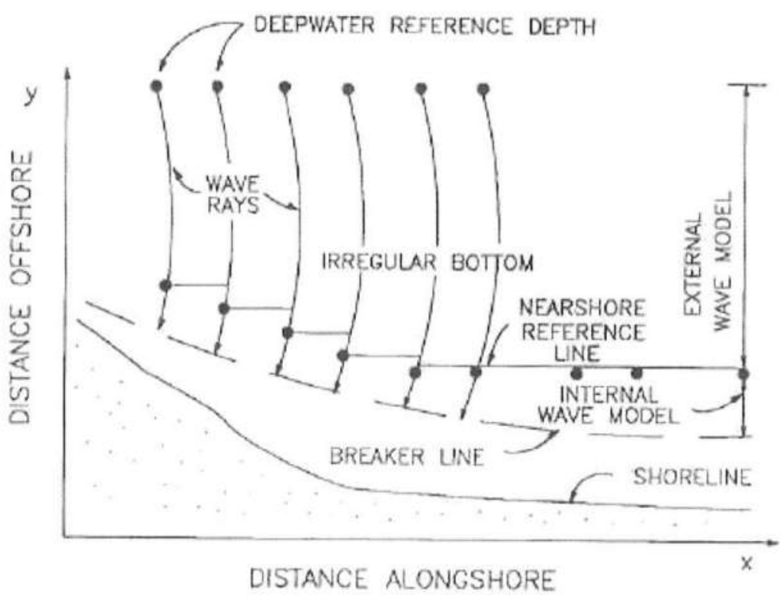
The GENESIS wave submodel is called the internal wave transformation model. GENESIS is able to accommodate an independent external wave transformation model which transforms deepwater waves from an offshore reference point to a nearshore reference line. The internal model alone is sufficient if the sea bottom consists of approximately straight and parallel contours. An external wave model incorporates the actual irregular bathymetry and transforms the waves to specified depths in the nearshore region for which breaking has yet to occur. The output is then used as input to the internal model. Figure 5.2 illustrates the applicability of these wave models.

The GENESIS internal wave model was designed to be compatible with the external model RCPWAVE (**R**egional **C**oastal **P**rocesses **WAVE** Model) developed by the US Army Corps of Engineers' Waterways Experiment Station (WES). RCPWAVE is a stable linear wave transformation model based on the mild slope equation and applicable for open coasts. The program incorporates diffractive effects produced by an irregular bathymetry as it solves for wave height and angle values directly on a grid. It is ideal in that it allows for the simulation of large areas due to its efficiency. More detailed information about the RCPWAVE model can be found in the WES Technical Report CERC-86-4.

The GENESIS internal model transforms the waves from the deepwater reference point or the nearshore reference line (depending on whether an external model is used) initially without accounting for diffraction due to structures. These results are then modified by incorporating the changes in the wave field due to each diffraction source. Before diffraction is taken into account, there are three unknowns in the breaking wave



a. Transformation by internal wave model only



b. Transformation by external and internal wave models

Figure 5.2: Applicability of the internal and external wave transformation models.

calculations: the wave height, wave angle, and breaking depth. There are three equations that are used to determine these unknowns.

The first equation is used to calculate the breaking wave height of waves that have been altered by refraction and shoaling and is described as

$$H_b = K_R K_S H_{ref} \tag{5.3}$$

in which

H_b is the breaking wave height at an arbitrary point alongshore,

K_R is the refraction coefficient,

K_S is the shoaling coefficient, and

H_{ref} is the wave height at the offshore reference point or nearshore reference line depending on which wave model is used.

The coefficients K_R and K_S are determined from linear wave theory.

The equation for depth-limited wave breaking is given as

$$H_b = \gamma D_b \tag{5.4}$$

where D_b is the depth at breaking, and γ is the breaker index given as a function of the deepwater wave steepness, H_o/L_o, and average beach slope, $\tan\beta$,

$$\gamma = b - a\frac{H_o}{L_o} \tag{5.5}$$

where

$a = 5.00\left[1 - e^{(-43\tan\beta)}\right]$,

$b = 1/\left[1 + e^{(-60\tan\beta)}\right]$,

H_o is the deepwater wave height, and

L_o is the deepwater wavelength.

The wave angle at breaking is given by Snell's Law which assumes locally straight and parallel contours,

$$\frac{\sin \theta_b}{L_b} = \frac{\sin \theta}{L} \tag{5.6}$$

where

θ is the wave angle,

L is the wavelength determined from the dispersion relation, and

b is a subscript denoting breaking conditions.

For information regarding the transformation of the waves due to interaction with various structures refer to the GENESIS Reports 1 and 2.

5.1.4 Numerical Solution Scheme

In order to determine the response of the shoreline to wave action, Equations 5.1, 5.2, and 5.4 must be solved simultaneously. However, in order to model a realistic scenario over time with real structures, Equation 5.1 must be solved using a numerical solution procedure. Figure 5.3 demonstrates the staggered grid used by GENESIS. The grid system defines shoreline positions, y, at the center of the grid cells and the transport rates, Q, at the cell walls. Grid cell 1 specifies the left boundary and grid cell N is at the right requiring N shoreline positions to be specified. There are $N+1$ values of the longshore transport rate since there are $N+1$ walls enclosing the N cells. The Q_1 and

Q_{N+1} values are required to be defined by the user while the remaining y and Q values are calculated by the model. Since the transport rate is a function of the wave conditions, all wave quantities are calculated at cell walls. Structure tips are also located at Q-points. Sand sources and sinks are located at y-points.

GENESIS then solves the system by using the Crank-Nicholson implicit scheme in which the new shoreline position depends equally on values calculated on the old and new time step. For more information regarding the solution scheme or the numerical accuracy and stability, refer to GENESIS Reports 1 and 2.

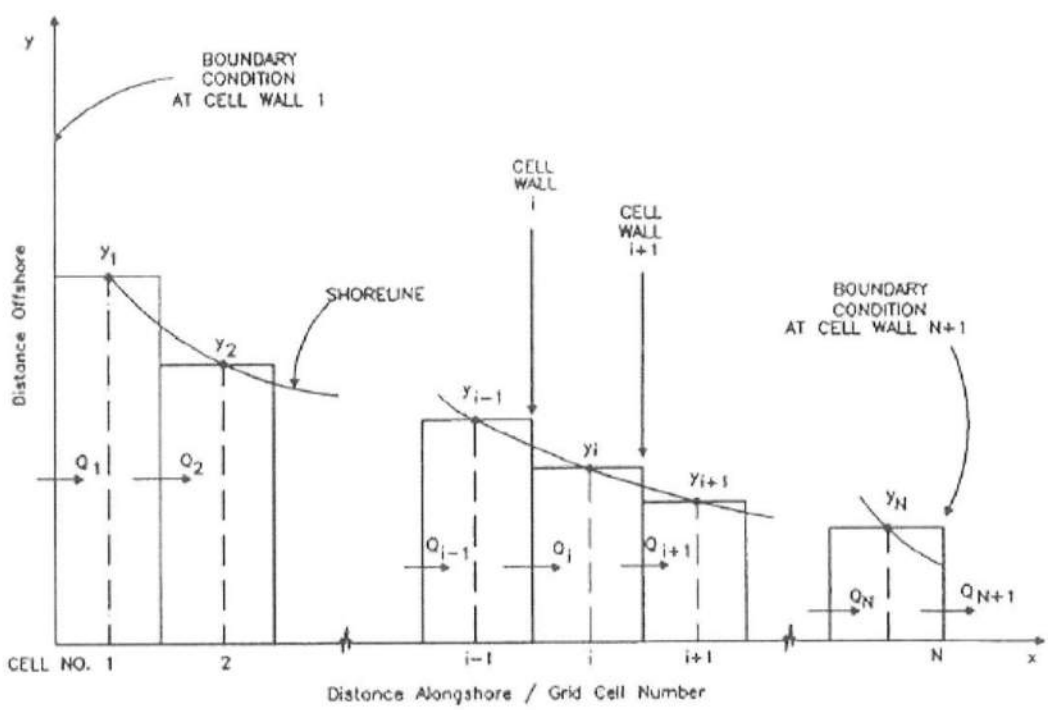


Figure 5.3: Finite difference staggered grid definition sketch.

5.1.5 Boundary Conditions

GENESIS requires that Q be specified in some form at both of the project boundaries. The lateral boundary conditions have a significant impact on the system as the interior grid positions directly depend on them. Ideally, points on the shoreline in which the littoral drift is terminated such as jetties or inlets are available to use as boundaries. However, this is not always the case. GENESIS provides three options with which to define the project boundaries: pinned-beach, gated, and moving shoreline position.

The pinned-beach condition is useful for a boundary that is not expected to change significantly over time. However, it is important that the boundary be located a considerable distance from the project site in order to assure that the condition does not influence changes that may occur at the site. The gated boundary condition is useful when shore-perpendicular structures are present at either end of the grid system. The effect of this type of boundary is formulated in terms of the amount of sand that can pass the structure. The moving shoreline position boundary condition allows the user to specify a boundary that moves at a constant rate. It is an expansion of the pinned-beach condition in which the movement is specified as zero.

5.1.6 Basic Assumptions

It is essential for any modeling procedure that the assumptions used in the model are fully understood. This ensures that the model is not used in a manner for which it is not capable. The most basic assumption for one-line shoreline change modeling is that the beach profile moves seaward and landward while maintaining its equilibrium shape. It is based on the notion that only one point is sufficient to specify any point along the profile. In addition, only one contour line is necessary to describe the change in the beach plan shape and volume as the beach accretes and erodes.

The second assumption is that sand is transported between two limiting elevations on the profile. The seaward limit is the depth of closure described earlier, and the landward limit is the berm elevation.

The model also requires a predictive expression for the net longshore sediment transport rate. For open coast beaches, the transport rate is taken to be a function of the breaking wave height and alongshore direction. The horizontal circulation in the nearshore, which actually moves the sand, is not directly considered.

Lastly, the model must be applied to a shoreline that exhibits a long-term trend in shoreline behavior. It assumes that breaking waves and boundary conditions are the major factors controlling long-term beach change rather than cyclical or random events.

5.1.7 Capabilities and Limitations

GENESIS is a shoreline change model designed to calculate the movement of the shoreline from one state of equilibrium to another in response to various natural and engineered perturbations. It is not intended to model shorelines in scenarios that involve beach change unrelated to coastal structures, boundary conditions, or spatial differences in wave-induced longshore sediment transport. Examples include beach change inside inlets or in areas dominated by tidal flow, beach change produced by wind-generated currents, storm-induced beach erosion in which cross shore sediment transport processes are dominant, and scour at structures. Therefore, it is important to understand fully the capacity to which the model can manage a particular scenario. Table 5.1 from the GENESIS Technical Reference summarizes the major capabilities and limitations of Version 2.

Table 5.1: Major Capabilities and Limitations of GENESIS Version 2

CAPABILITIES

Almost arbitrary numbers and combinations of groins, jetties, seawalls, detached breakwaters, and beach fills

Compound structures such as T-shaped, Y-shaped, and spur groins

Bypassing of sand around and transmission through groins and jetties

Diffraction at detached breakwaters, jetties, and groins

Coverage of wide spatial extent

Offshore input waves of arbitrary height, period, and direction

Multiple wave trains (as from independent wave generation sources)

Sand transport due to oblique wave incidence and longshore gradient in height

Wave transmission at detached breakwaters

LIMITATIONS

No wave reflection from structures

No tombolo development (shoreline cannot touch a detached breakwater)

Minor restrictions on placement, shape, and orientation of structures

No direct provision for changing tide level

Basic limitations of shoreline change modeling theory

5.2 Application of the GENESIS Model to the Delaware Coast

In this section the GENESIS model is applied to the three primary beaches of the Delaware coast to estimate advance fill and renourishment requirements for a 50-year project life. In addition, the model will be used to predict the evolution of the shoreline in a retreat scenario. This process examines the current renourishment cycle, evaluates its performance, and investigates the outcome of a termination of this cycle all within a reasonably realistic and representative context.

5.2.1 Preparation of the Model

In order to run GENESIS, the preparation of an input data file called the "Start" file is necessary. General information required for each simulation consists of the designation of the grid system and simulation time scales, beach characteristics, environmental parameters, behavioral responses of the model, boundary conditions and calibration parameters. Additional information not necessarily required for each run involves structural descriptions, beach fills, and bypassing operations.

Table 5.2 provides some of the required input for each beach. Notice that each beach has 199 grid cells. The version of GENESIS used in the simulations had a limit of 200 cell walls, thus 199 cells. Therefore, Δx was defined in a manner that allowed for the maximum number of grid cells possible for the project reach. The time step chosen is same time step used for station 2066 of the WIS wave hindcast study. Because detailed sediment characteristics for each beach were not readily available and GENESIS assumes uniform sediment, an average grain size diameter of 0.285 mm was used for each beach.

Table 5.2: Several "Start" File Input Values for Each Delaware Beach

	Dewey Beach	Bethany Beach	Fenwick Island
Starting Point (Northing)	242,220	180,118	164,346
Starting Point (Easting)	756,094	760,755	761,388
Ending Point (Northing)	277,597	210,955	179,710
Ending Point (Easting)	751,725	758,939	760,816
Shore-Normal Angle (° from N)	84	86	86
Number of Grid Cells	199	199	199
Δx (ft)	180	156	78
Δt (hrs)	3	3	3
Grain Size Diameter (mm)	0.285	0.285	0.285
Average Berm Height (MWL) (ft)	7	7	7
Depth of Closure (ft)	24	24	24
Depth at Offshore Wave Input (ft)	32.8	32.8	32.8
K_1	0.1	0.04	0.05
K_2	0.1	0.05	0.05

In addition, a typical berm height and depth of closure of 7 ft and 24 ft, respectively, were also used for each beach.

For the simulations performed in this study, the external wave model RCPWAVE was used to transform the waves from a depth of 59 ft (18 m) to a depth of 32.8 ft (10 m) just outside the depth of closure. From that depth, the internal wave model brought the waves into the nearshore region to determine sediment transport rates. Because RCPWAVE takes into account the influence of an irregular seabed, bathymetric information was required as input. This information was provided by the Philadelphia District of the US Army Corps of Engineers. An illustration of the bathymetry is provided in Figure 5.4.

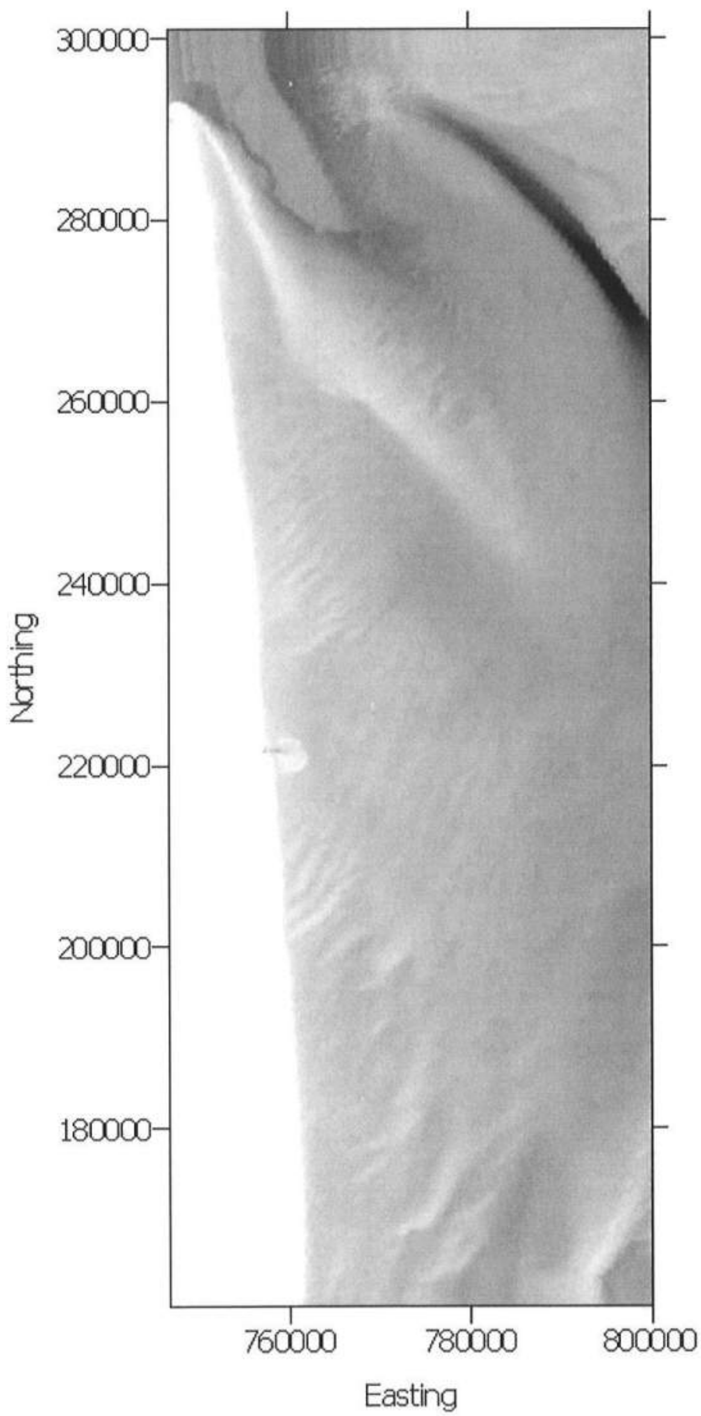


Figure 5.4: Image of bathymetry provided by US Army Corps of Engineers.

The shorelines used in this model were those provided by DNREC. The most recent shoreline for each beach was chosen assuming it extended the entire length of the desired model reach and was comprehensive enough to accurately portray the features of the beach. The shoreline positions for each cell of the model were then determined using a supplementary program provided as part of GENESIS.

As recommended by the GENESIS models, several calibration and verification runs were performed in order to determine characteristic K_1 and K_2 values. As mentioned earlier, Komar and Inman (1970) recommend a K_1 value approximate to 0.77. However, as made evident in Table 5.2, extremely low values were selected. Based on the preliminary runs performed using shorelines and beach fills scenarios provided by DNREC, small transport parameters were required in order to recreate a beach subject to the same magnitude of sediment loss experienced by the Delaware coastline.

Because the position of the shoreline at the model boundaries is unknown in 50 years, the suitable boundary condition for each of the models is a complicated but important task. However, a method was implemented that requires GENESIS to determine the shoreline change at the boundary rather than the user. As mentioned earlier, it is possible to define a shore-connected structure at the project boundaries by specifying the length of the structure from the x-axis and the permeability factor of the structure. The permeability factor denotes transmission of sand over, through, and landward of a structure. A structure that does not allow any sediment to be transported has a permeability of 0. A completely "transparent" structure is assigned a permeability of 1. In addition, GENESIS allows the shoreline to extend beyond the length of the groin resulting in a buried structure. Therefore, the boundary conditions defined for each beach consisted of a "transparent" non-diffracting groin buried landward of the shoreline. This approach "forces" GENESIS to define the behavior of the boundaries rather than the user.

There are several groins present along the Delaware coastline in Dewey and Rehoboth Beaches and Bethany and South Bethany Beaches. The position, lengths, and condition of these groins were provided by DNREC. Most of these groins are older structures that have performed their function in that they are low relative to the mean low water and are either slightly protruding from the shoreline or are completely buried. Nevertheless, they were included in the simulations. One problem that was encountered deals with a limitation on the specification of structures in GENESIS. It is required that structures have at least two cells between them. In some cases, groins were located next to each other at a distance less than the required two-cell length. For each case, the groins were positioned as closely and precisely as possible in order to represent an accurate shoreline within the realms of the model.

As for the introduction of beach fills into the model reach, GENESIS allows fills to be place incrementally over a period of time. This results in a beach fill that is still exposed to the wave environment as it is placed. For this study, fills were placed in the project area over a month. When specifying fills, the sediment properties are not entered. This is because GENESIS assumes the fill sand has the same characteristics as the native sand. Therefore, the behavior of a coarser or finer material in the system cannot be modeled. Instead, the start and end grid points and berm width of the fill are all that are required to represent a beach fill. Table 5.3 presents the representative beach fills used for each beach. Note that the width of the added beach entered in the start file is the added berm width after adjustment. Therefore, the readjustment of the beach fill to an equilibrium position must be taken into account when specifying the project width. For the simulations performed for this project, it was assumed that 50% of the initial berm width was lost during readjustment.

Table 5.3: Representative Beach Fill Input for Each Beach

	Dewey Beach	**Bethany Beach**	**Fenwick Island**
Starting Point (Northing)	250,960	186,190	164,580
Ending Point (Northing)	256,300	199,120	169,590
Starting Grid Point	120	77	131
Ending Grid Point	150	160	195
Beach Fill Length (ft)	5,400	13,000	5,500
Berm Width After Adjustment (ft)	60	40	40

5.2.2 Results of the Simulations

Once the "Start" file is ready, the simulations can be executed. For this study, runs were performed in 10-year spans for 50 years. At the end of the simulation, the output was investigated to determine when a fill is needed. When determining when a beach fill was appropriate, several factors were taken into consideration. The first reason of course was whether or not a fill was necessary. There were several occurrences of a fill being placed when the previous had not experienced a significant amount of distribution. This was done because a more than reasonable amount of time had passed since the previous fill. Secondly, fills were organized in a consecutive manner among the beaches. This approach was taken because economically it is more feasible for a state to create a renourishment cycle for its beaches that coincide with one another. This reduces the high mobilization and demobilization costs associated with nourishment projects. Table 5.4 provides the dates that fills were placed for each beach.

Once it was determined when fills should be placed, the runs were repeated with the beach fills included. The resulting shoreline was then used as the initial shoreline for the next 10-year simulation. Figures 5.5, 5.6, and 5.7 present the shoreline locations at the end of each 10-year period for Dewey and Rehoboth Beach, Bethany and South Bethany Beach, and Fenwick Island, respectively.

Notice that Dewey and Rehoboth Beach maintains a fairly regular shoreline with nearly all erosion occurring downdrift of the nourishment project location. Bethany and South Bethany experience substantial accumulation in the nourishment area and toward the northern end of the project area. Fenwick Island behaves similarly to Dewey Beach in that there is significant depletion of sediment downdrift of the fills and Bethany Beach in that there is an excessive buildup of sediment at the location of the fills. In all three cases there is an insignificant quantity of sediment transport evident from the collection of sand in the vicinity of the renourishment site and the low number of renourishment projects required over the course of the project life.

Table 5.4: Start Dates of Renourishment Projects for Each Beach

Dewey and Rehoboth Beach	Bethany and South Bethany	Fenwick Island
9/15/2000	10/15/2000	11/15/2000
2/15/2005	3/15/2005	4/15/2005
4/15/2012	5/15/2012	6/15/2012
1/15/2021	2/15/2021	3/15/2021
8/15/2034	9/15/2034	10/15/2034
1/15/2046	10/15/2045	---

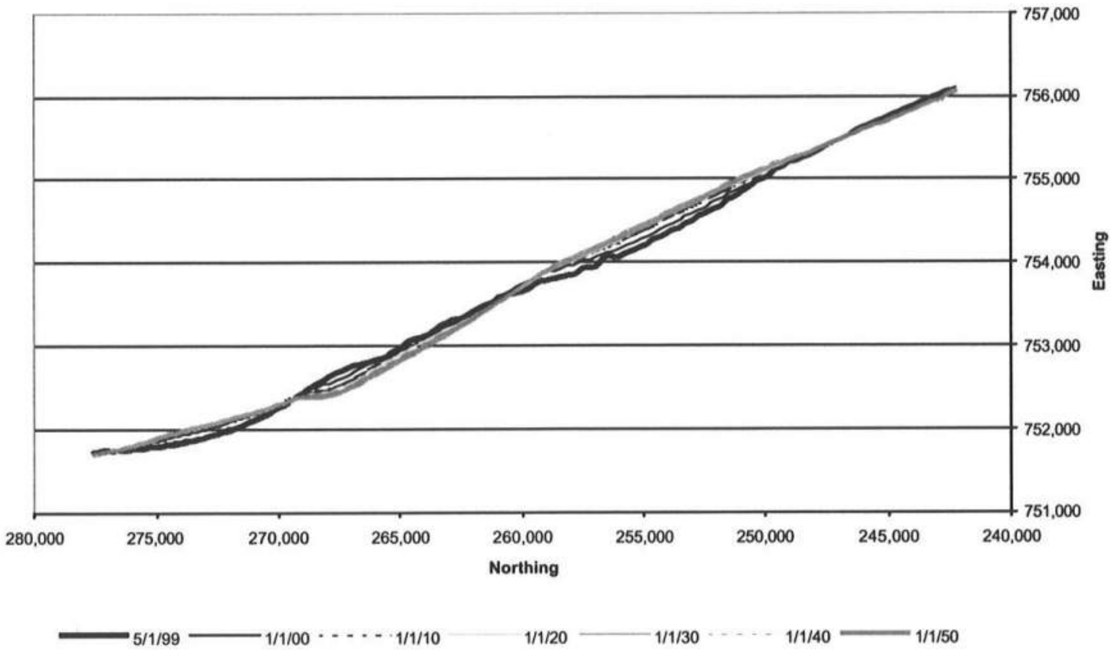


Figure 5.5: Shoreline positions at 10-year increments for a 50-year renourishment plan for Dewey and Rehoboth Beach.

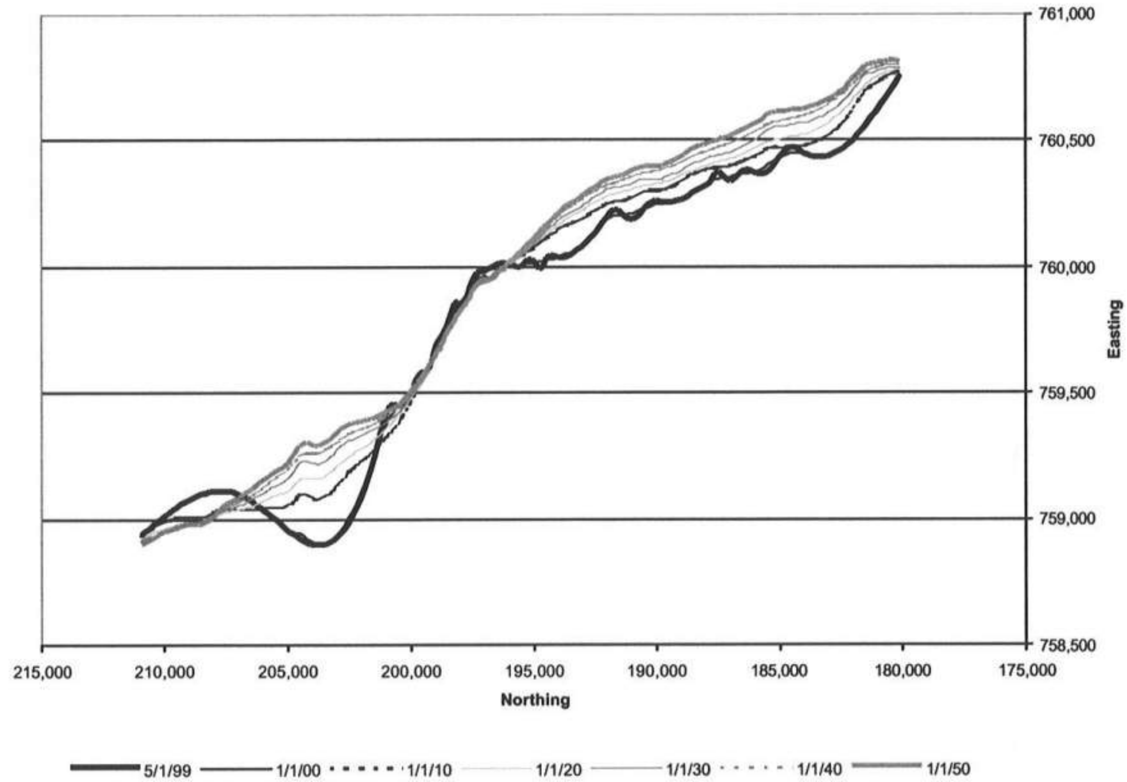


Figure 5.6: Shoreline positions at 10-year increments for a 50-year renourishment plan for Bethany and South Bethany Beach.

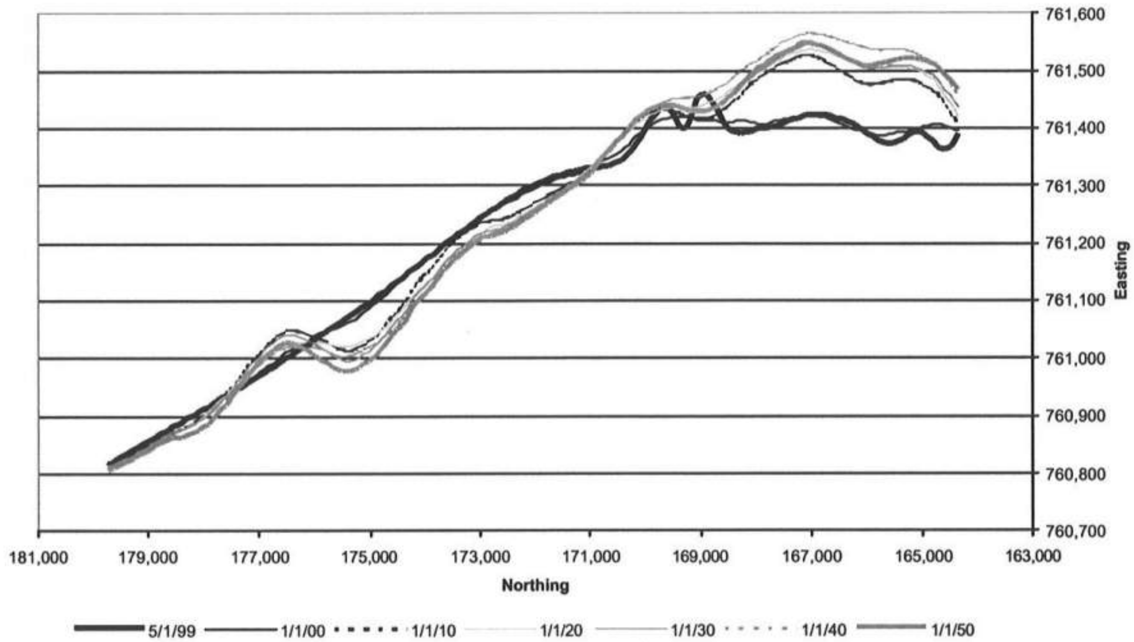


Figure 5.7: Shoreline positions at 10-year increments for a 50-year renourishment plan for Fenwick Island.

As predicted by the Pelnard-Consideré model, more time is permitted between fills as the project advances. However, based on the fill requirements during the 1990s, it is known that fills will be necessary at a higher renourishment rate than that predicted by GENESIS. Notice that in Fenwick Island, a fill is not necessary during the last 15 years of the 50-year project life. This is not expected to be the case. Rather, due to sea level rise, it is expected that beach nourishment rates will be constant if not higher. Therefore, a sea level rise was manually applied to the GENESIS output. On average, the Mid-Atlantic states are experiencing a 1 ft per century sea level rise rate. Assuming an average beach slope of 1:20, a 2-ft retreat of the shoreline can be expected every 10 years. Figures 5.8, 5.9, and 5.10 illustrate the influence a constant sea level rise would have on the shoreline of each of the three Delaware Beaches. Unfortunately, the influence of sea level rise on the shoreline position is not significant enough to explain the lack of sediment distribution at the project site.

In addition to predicting the position of the shoreline in the presence of fills, an investigation was performed in order to estimate the shoreline position in a retreat scenario. This assumes that 1) all renourishment cycles cease, 2) the beach maintains a constant, operable width by moving the landward limit of the beach with the shoreline, and 3) structures are removed as the beach moves landward and occupies the property. This guarantees the presence of a beach and ultimately, a continuous income from tourism. Figures 5.11, 5.12, and 5.13 provide the resultant shoreline in a retreat scenario for each beach. While more erosion occurs along the coastlines, there are sections of the beaches that still experience an unusual amount of sediment accumulation.

Regardless of the scenario, the longshore sediment transport occurring along the Delaware coast is inaccurate. There are two possibilities that may provide explanations for this poor representation. One rationale is that GENESIS underestimates the sediment transport due to oblique waves. Recall that the transport parameters are entered by the user as calibration coefficients. As mentioned earlier, extremely small values were required in order to reproduce the changes that occurred in the measured shoreline. This resulted in an unusually small quantity of sediment transport. The user is then faced with the dilemma of reproducing a measured shoreline or increasing the transport of sediment along the shoreline.

A second explanation is that storms and random events, not the average wave climate, is responsible for the majority of the shoreline change on the Delaware coast. GENESIS assumes that breaking waves and boundary conditions are the major factors controlling long-term beach change rather than cyclical or random events in the beach system. Even if storms are not the main source of shoreline change along the coast, perhaps they are more responsible for the distribution of beach fills than initially anticipated.

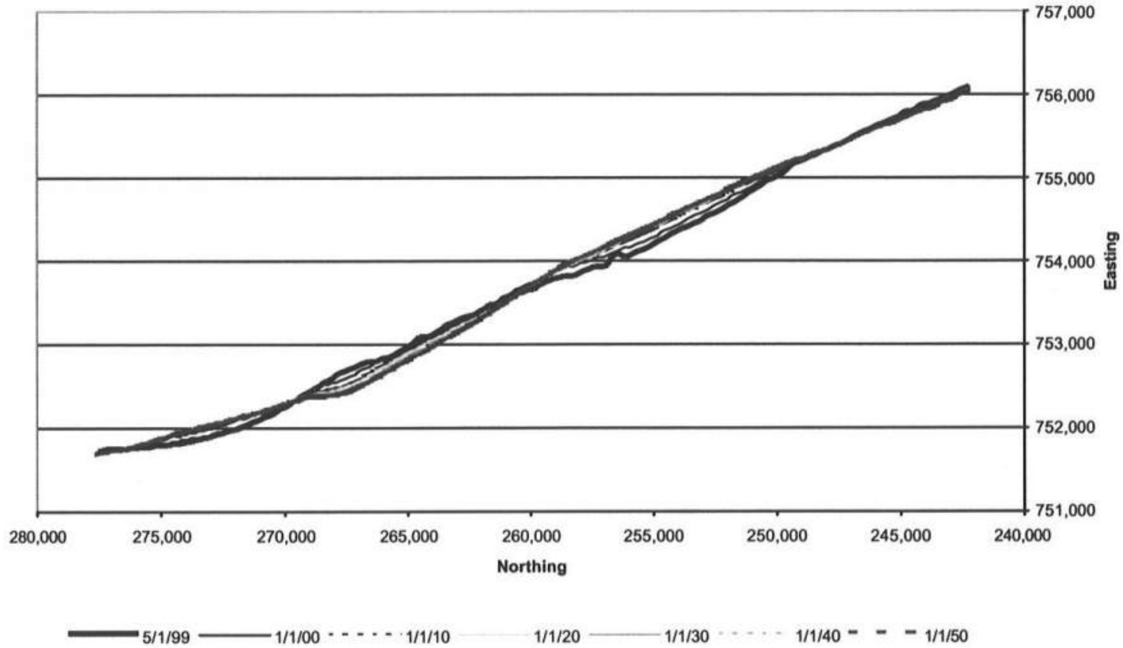


Figure 5.8: Shoreline positions at 10-year increments for a 50-year renourishment plan with sea level rise for Dewey and Rehoboth Beach.

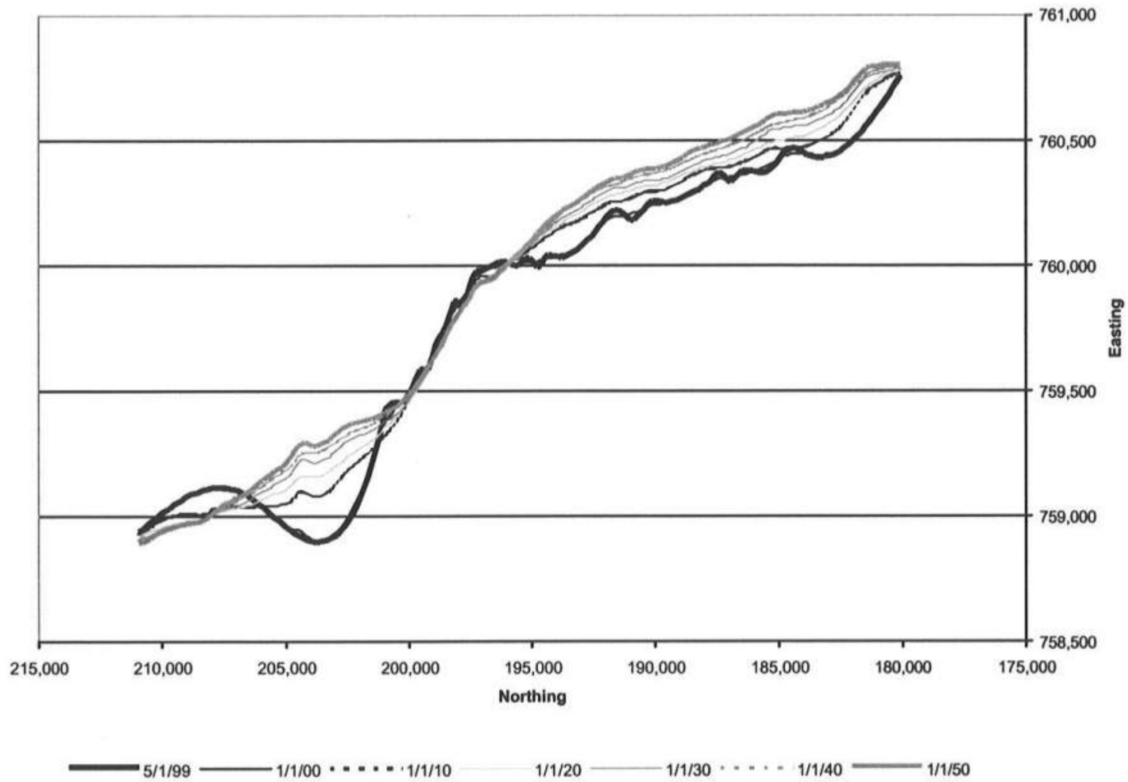


Figure 5.9: Shoreline positions at 10-year increments for a 50-year renourishment plan with sea level rise for Bethany and South Bethany Beach.

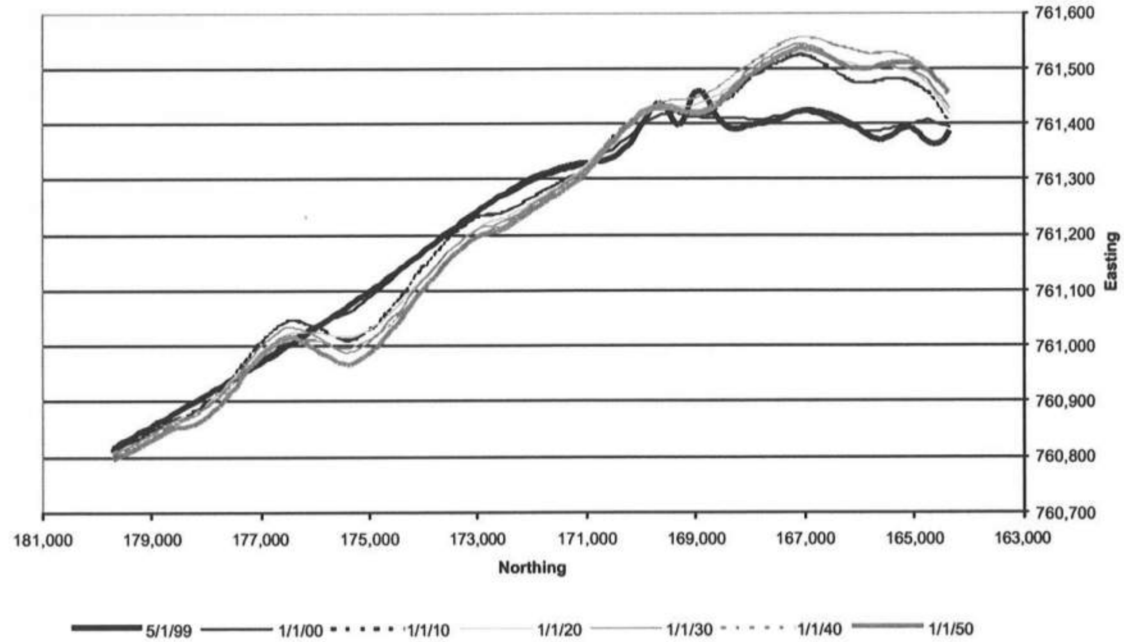


Figure 5.10: Shoreline positions at 10-year increments for a 50-year renourishment plan with sea level rise for Fenwick Island.

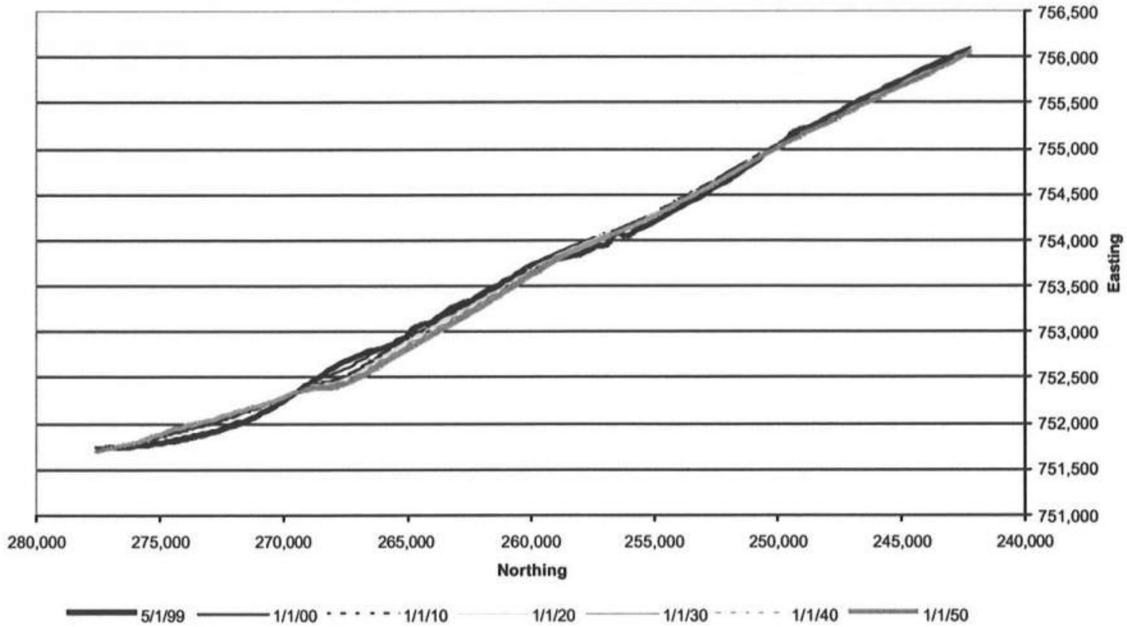


Figure 5.11: Shoreline positions in a retreat scenario at 10-year increments for Dewey and Rehoboth Beach.

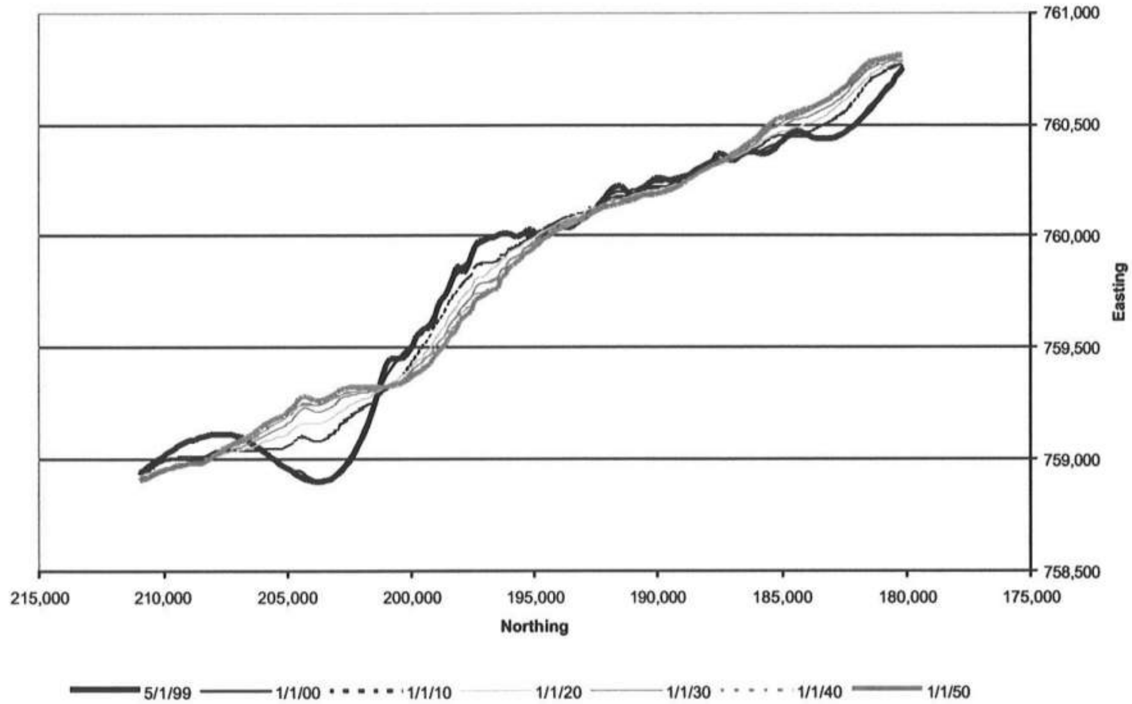


Figure 5.12: Shoreline positions in a retreat scenario at 10-year increments for Bethany and South Bethany Beach.

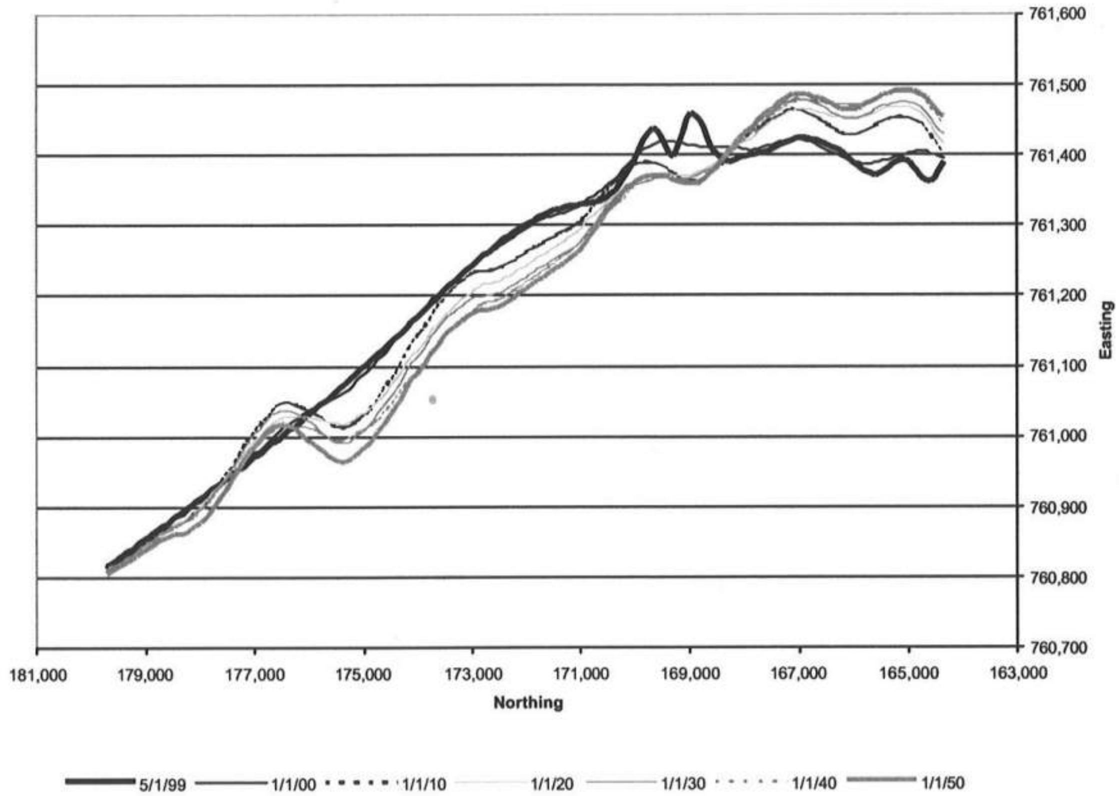


Figure 5.13: Shoreline positions in a retreat scenario at 10-year increments for Fenwick Island.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

The scope of coastal engineering work required by this study involved determining a beach nourishment strategy that will be necessary over the next fifty years to maintain a suitable beach. First, various factors affecting the shoreline were studied. Based on available and collected site data, existing conditions along the coast were quantified. Secondly, the behavior and lifetime of beach fills and subsequent fills were investigated. This included analyzing the behavior of nourishment projects on the Delaware coast. These investigations were then combined to calibrate a one-line numerical model that was adapted to the Delaware coast and surrounding conditions with the intent of developing a long-term estimation of beach fill requirements. Unfortunately, the model was inconclusive as it predicted unusually long lifetimes for the beach fills, ultimately underestimating the sediment requirements that will be necessary to maintain the beach width.

Because beach fills are not permanent solutions to the erosion epidemic, it is necessary for legislative bodies to develop a flexible long-term management plan for their coastal communities. While beach nourishment is currently the preferred choice of action for shoreline maintenance, governments at all levels are exploring other options. Due to expected increases in the demand for sediment and recreational pressures and the limited supply of suitable beach material, beach renourishment is an increasingly expensive alternative. This is exacerbated by the increase in irresponsible coastal habitation. Even with coastal protection, post-storm disaster relief costs have sky rocketed in the last decade. As a result, many states are considering retreat as a more viable option for coastal management. It ensures the presence of a beach and thus continual revenue from tourism through the relocation or demolition of structures rendered inadequate by a receding shoreline. While retreat appears to be a reasonable solution, there are several drawbacks worth mentioning.

The first being property rights. It is unreasonably optimistic to assume that property owners will willingly abandon or demolish their homes and businesses. Rather, an increase in disputes and lawsuits will arise from property owners declaring that their right to do as they wish on their property has been violated, and the courts have repeatedly protected these rights regardless of the legislation in place. Therefore, it will be necessary to incorporate judicial costs into this plan.

In addition, part of the revenue experienced by tourism includes the money spent at local restaurants, shops, and hotels. The retreat option includes the loss of these structures as well, the very sources of profit the policy makers intend to protect. Therefore, the retreat option will also require some sort of contingency to ensure that these sources of revenue are ever present within the community.

The last drawback considers the reality of a continuously receding beach. If t is allowed to erode, eventually the substructure of the beach town will become exposed rendering the beach unusable and undermining the original intent of the retreat option. As a result, this option will require the removal and redesign of the substructure system of the town.

While additional alternatives are not investigated here, there are several additions that may be incorporated into a long-term coastal management plan which will eventually reduce the costs required for shoreline maintenance. First and foremost is the establishment and continuation of an extensive coastal monitoring program. The best way to predict beach behavior is to understand how it responds to various factors. This will require at a minimum regular summer and winter monitoring. Ideally, the program will be extended to include pre- and post-fill conditions and as much as possible pre- and post-storm configurations. In addition, monitoring will continue after these events in order to record the long-term behavior. The means through which the monitoring can be performed includes profile surveying (which DNREC is currently executing), aerial photography, and satellite imagery. In addition to the topographic and bathymetric measurements, continued recording of the meteorological conditions as well as the water levels and wave climate is critical in understanding the environment to which the beach is exposed. While it is important that the monitoring program be developed, it is crucial that the system be maintained for a long period of time. More important than understanding the response of a shoreline to a particular event is understanding how a beach has behaved after a duration of storms, fills, sea level rise, erosion, etc.

A second addition to the coastal management plan involves the education of the existing and potential home and business owners of the community. This will include information about designing and building a more "storm-friendly" home or business, evacuation plans, legislation, future management plans for the beaches, general coastal behavior, protecting structures during storms and rebuilding after them, etc. In addition, this program will include forums that would allow property owners to voice their question and concerns. Informing the residents of issues affecting their homes, families, and businesses and teaching them ways to protect them will reduce the costs incurred by storms as well as open the lines of communication between policy makers and property owners.

Lastly, a coastal management program must include legislative and regulatory guidelines. Most importantly, this will include guidelines concerning construction of new structures as well as repairs on existing ones. Examples of legislation already in place for other states include building requirements such as elevated first floors and break-away panels, setback limits, and rebuilding restrictions. While exceptions and disagreements will occur, this is the most effective way to reduce the excessive storm damage mitigation costs experienced after a major event.

Appendix A

HISTORICAL SHORELINE POSITIONS AND TRENDLINES FOR EACH PROFILE LOCATED ALONG THE DELAWARE COAST

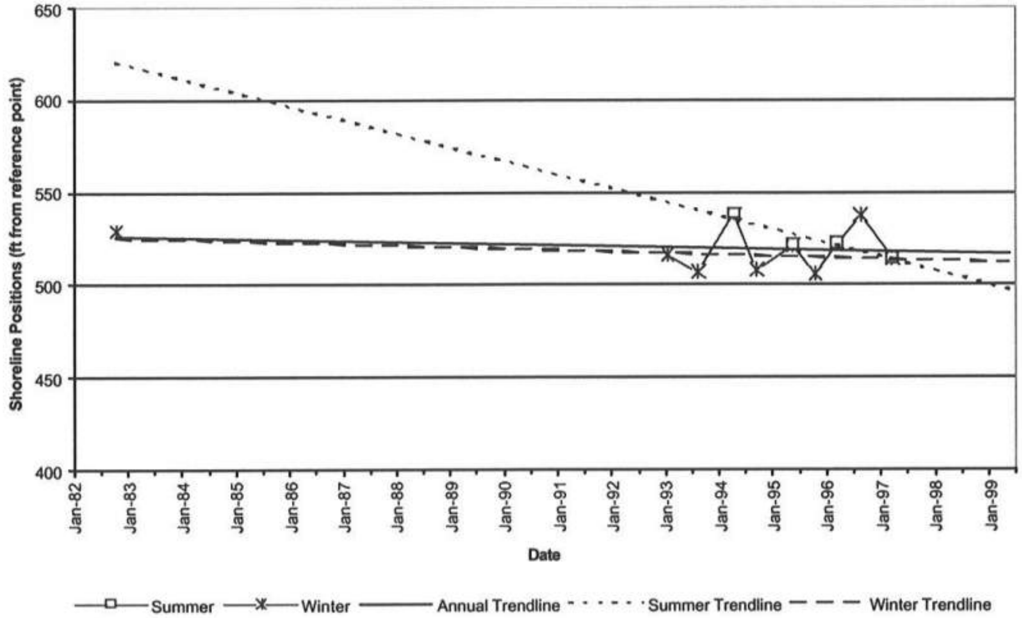


Figure A.1: Historical shoreline positions and trendlines for Profile 38.

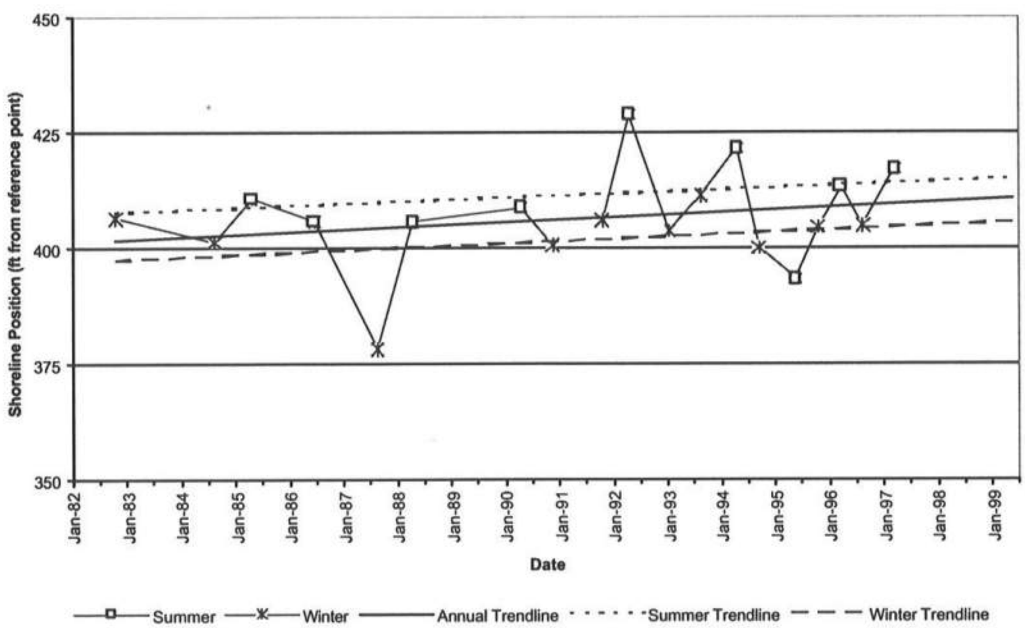


Figure A.2: Historical shoreline positions and trendlines for Profile 39.

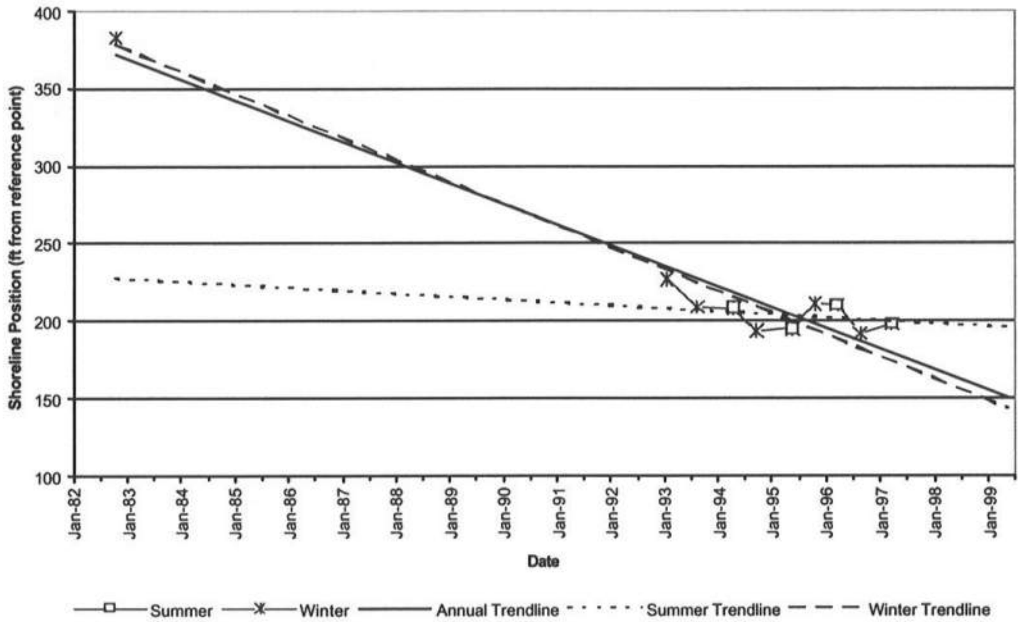


Figure A.3: Historical shoreline positions and trendlines for Profile 40.

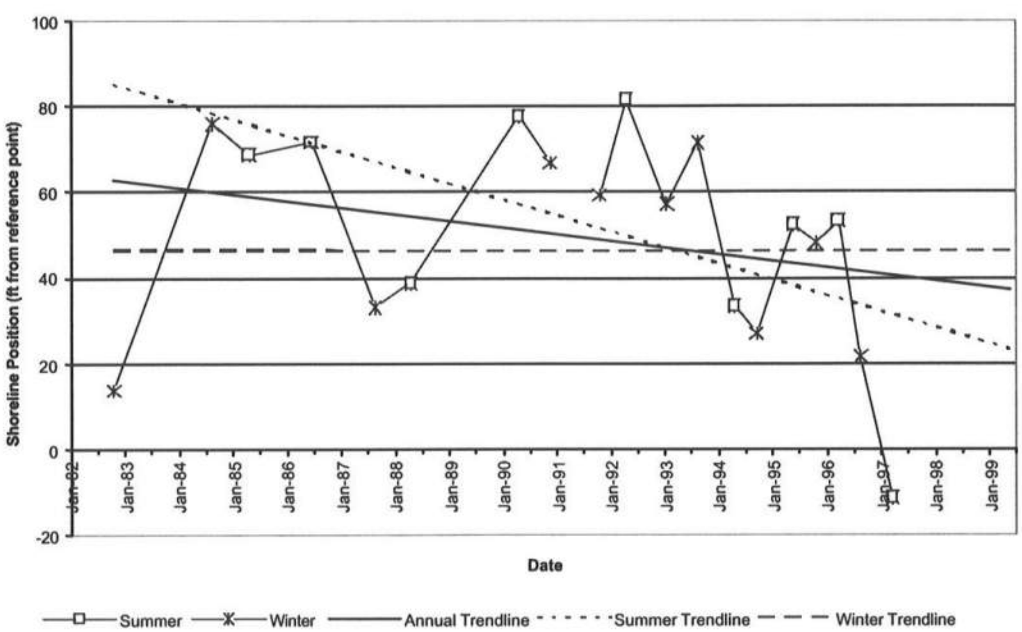


Figure A.4: Historical shoreline positions and trendlines for Profile 41.

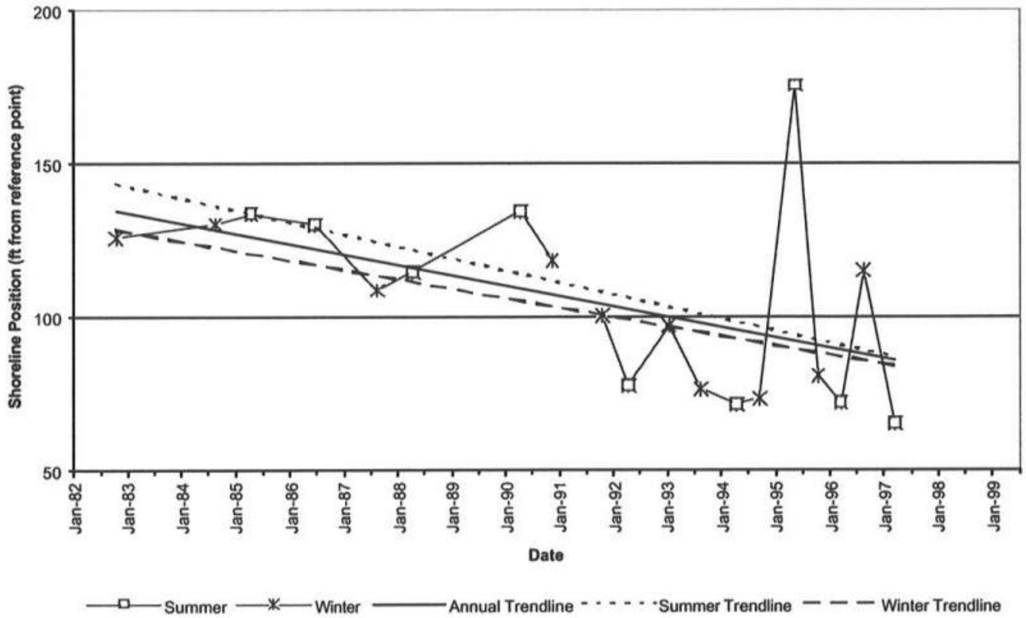


Figure A.5: Historical shoreline positions and trendlines for Profile 42.

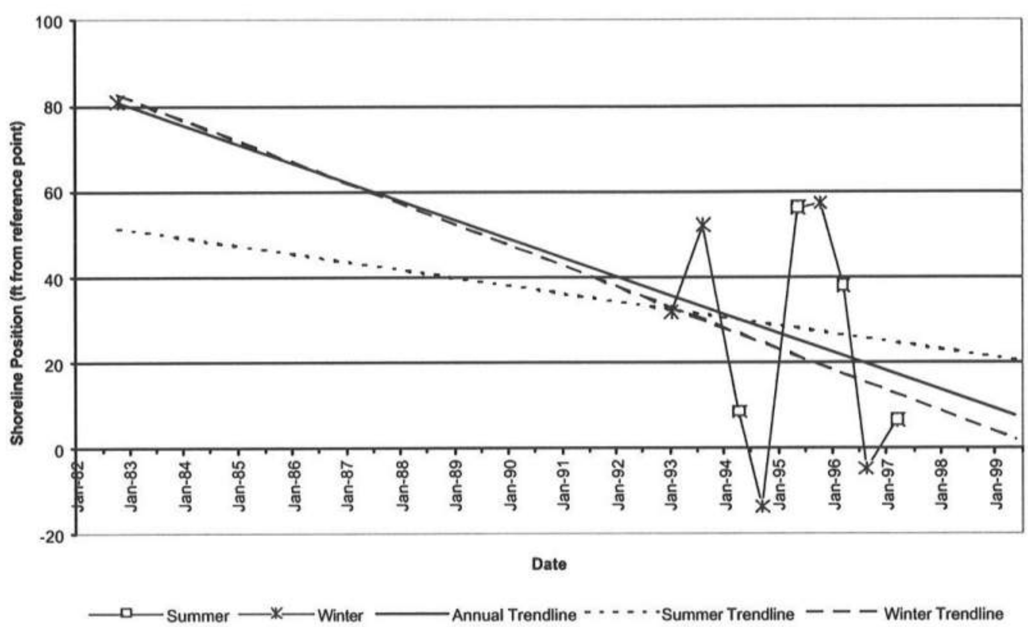


Figure A.6: Historical shoreline positions and trendlines for Profile 43.

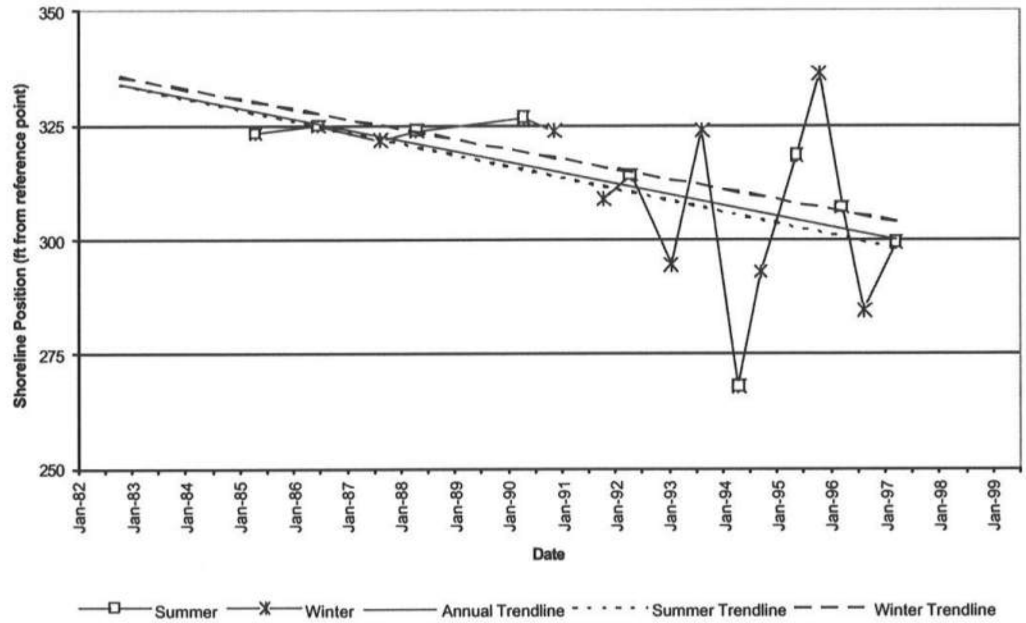


Figure A.7: Historical shoreline positions and trendlines for Profile 44B.

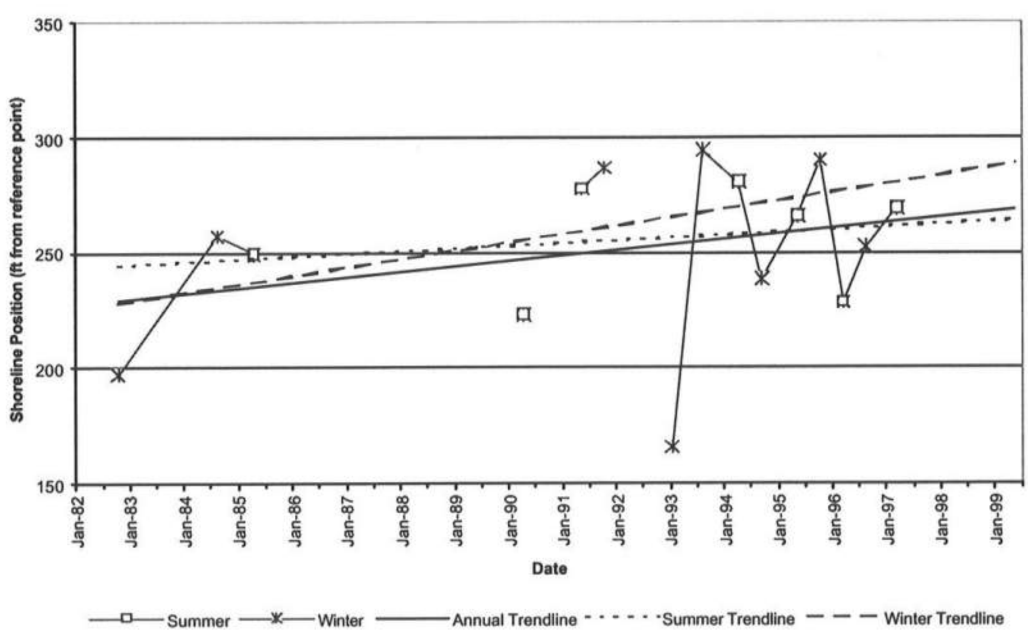


Figure A.8: Historical shoreline positions and trendlines for Profile 44.

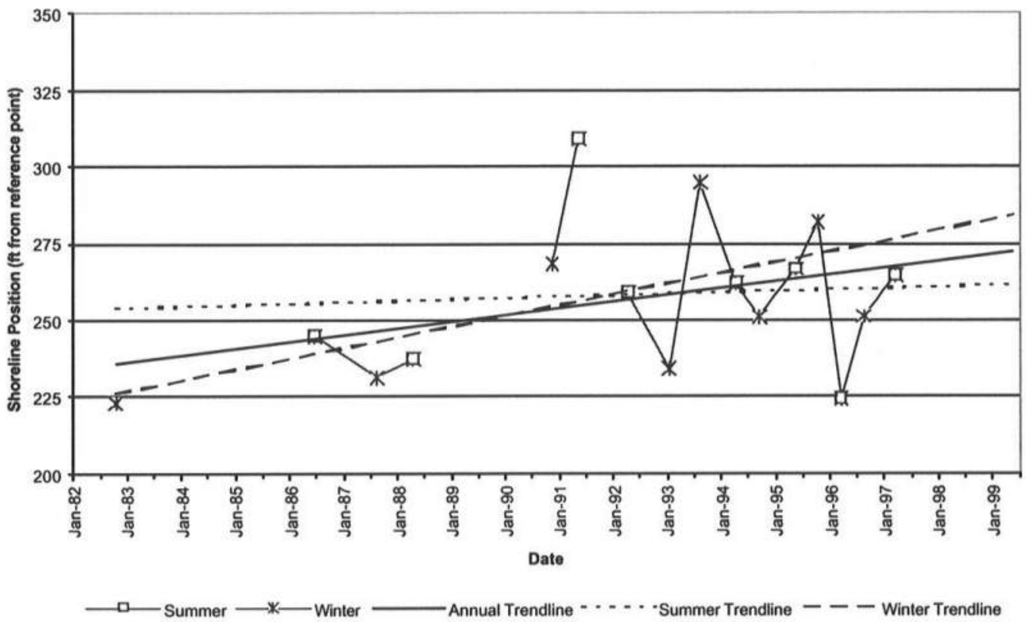


Figure A.9: Historical shoreline positions and trendlines for Profile 44A.

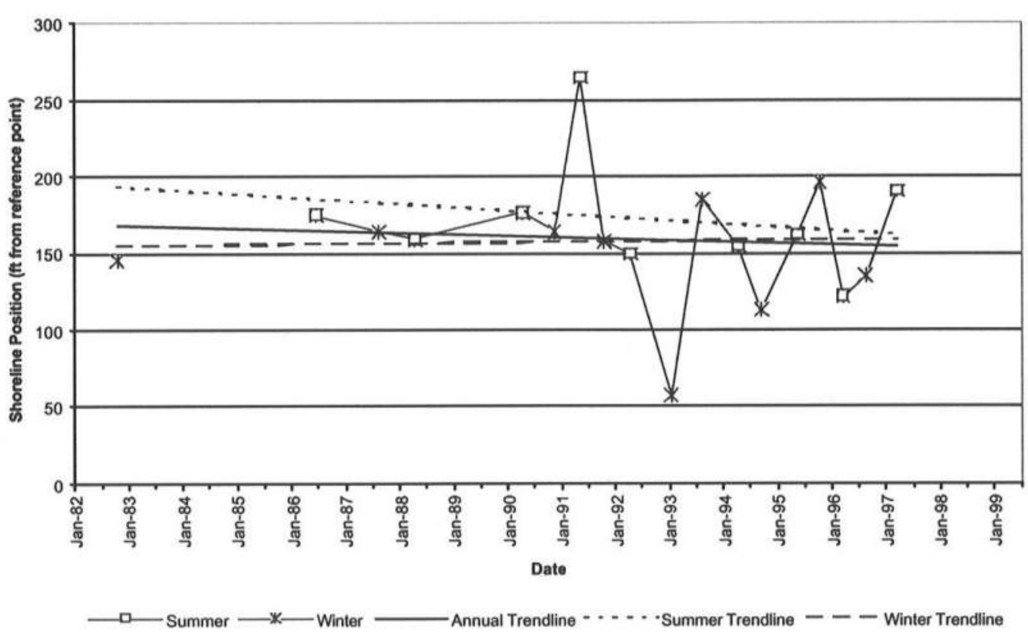


Figure A.10: Historical shoreline positions and trendlines for Profile 45.

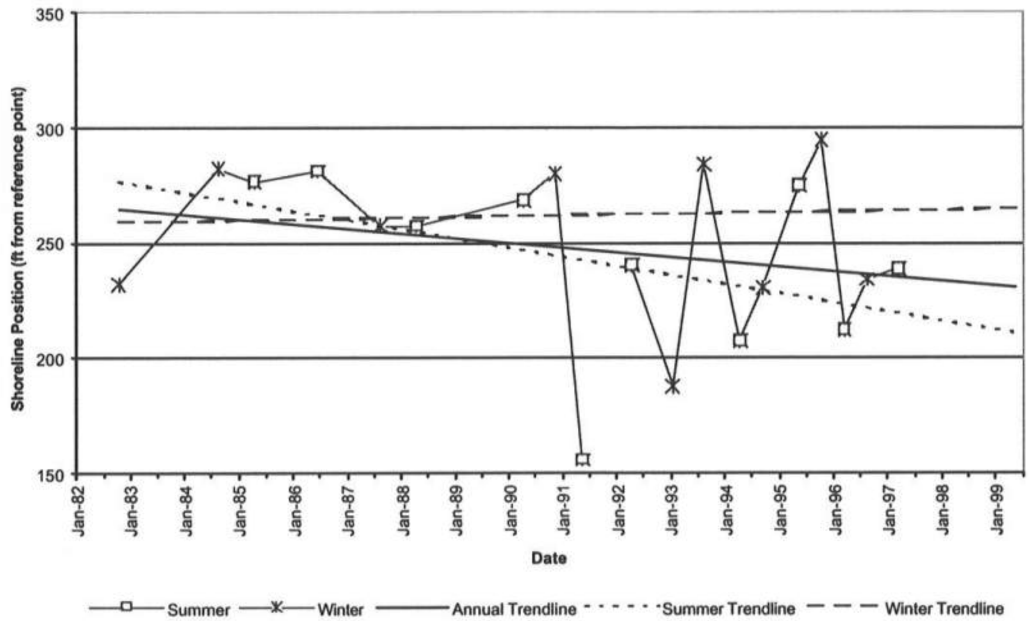


Figure A.11: Historical shoreline positions and trendlines for Profile 45A.

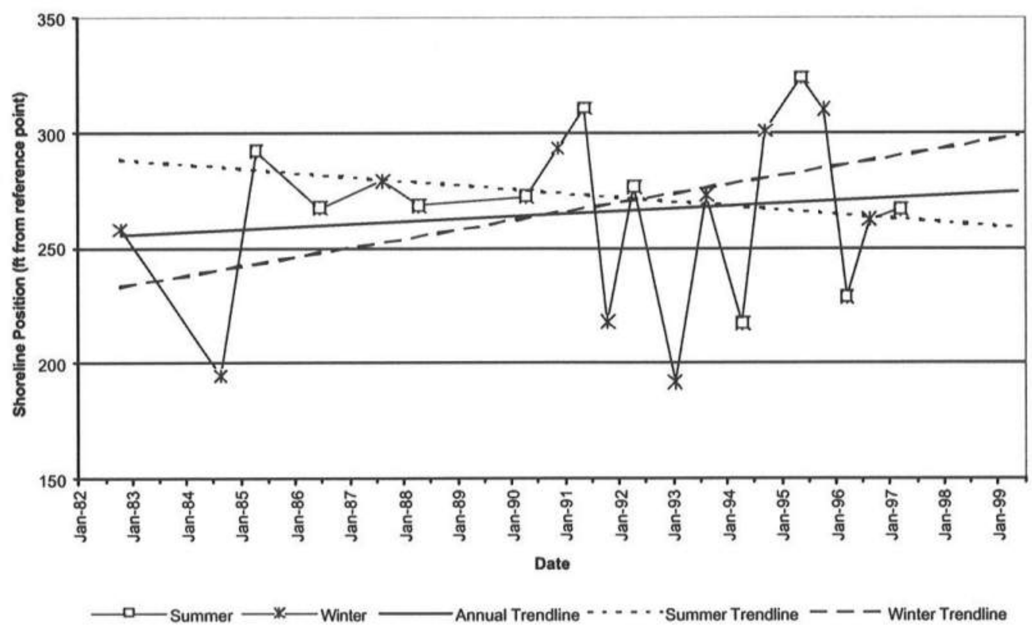


Figure A.12: Historical shoreline positions and trendlines for Profile 46.

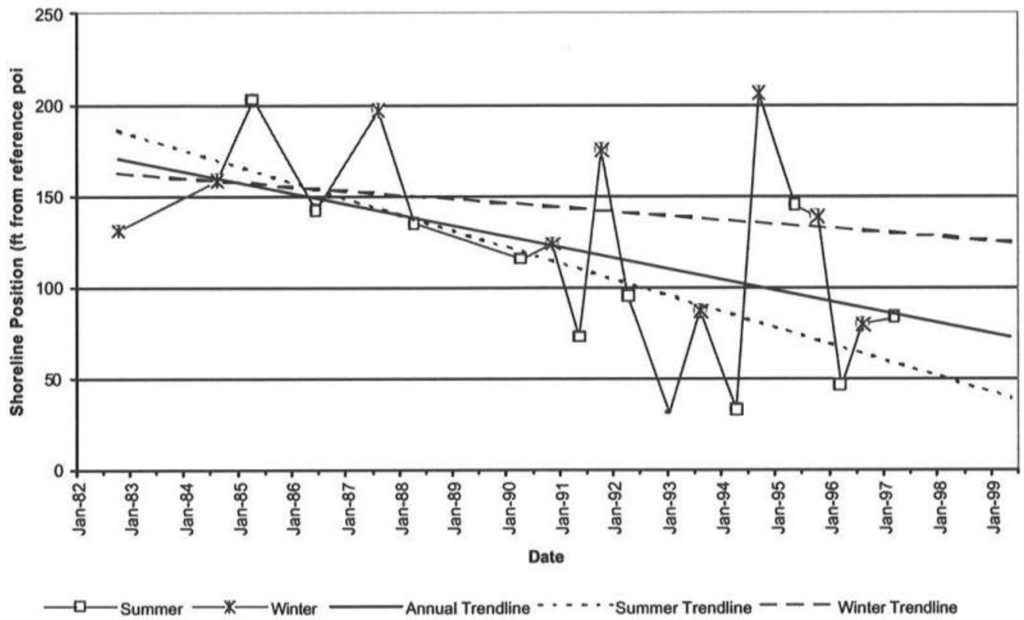


Figure A.13: Historical shoreline positions and trendlines for Profile 47.

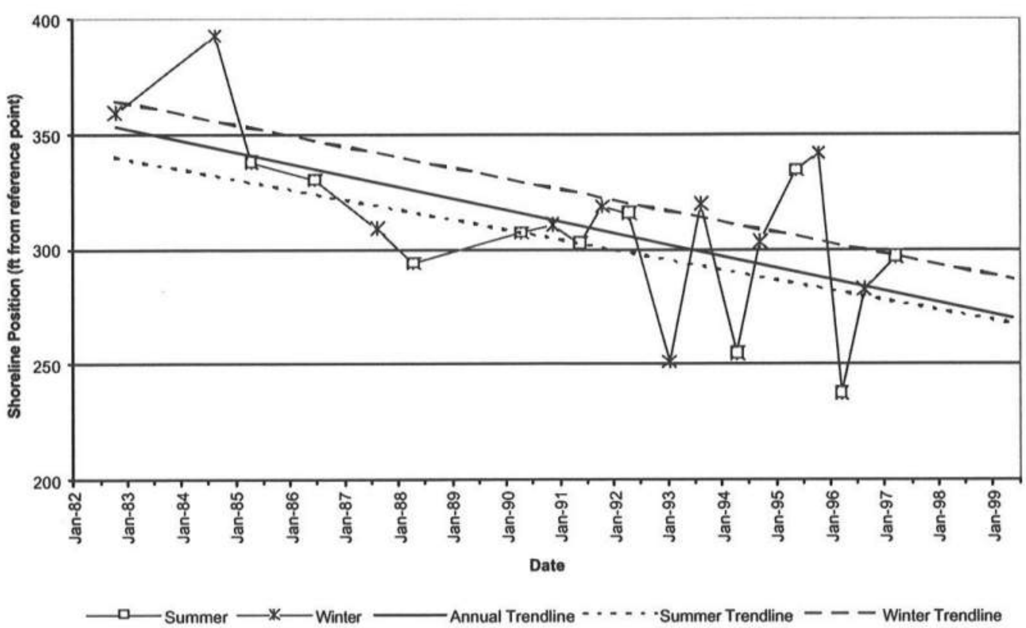


Figure A.14: Historical shoreline positions and trendlines for Profile 48.

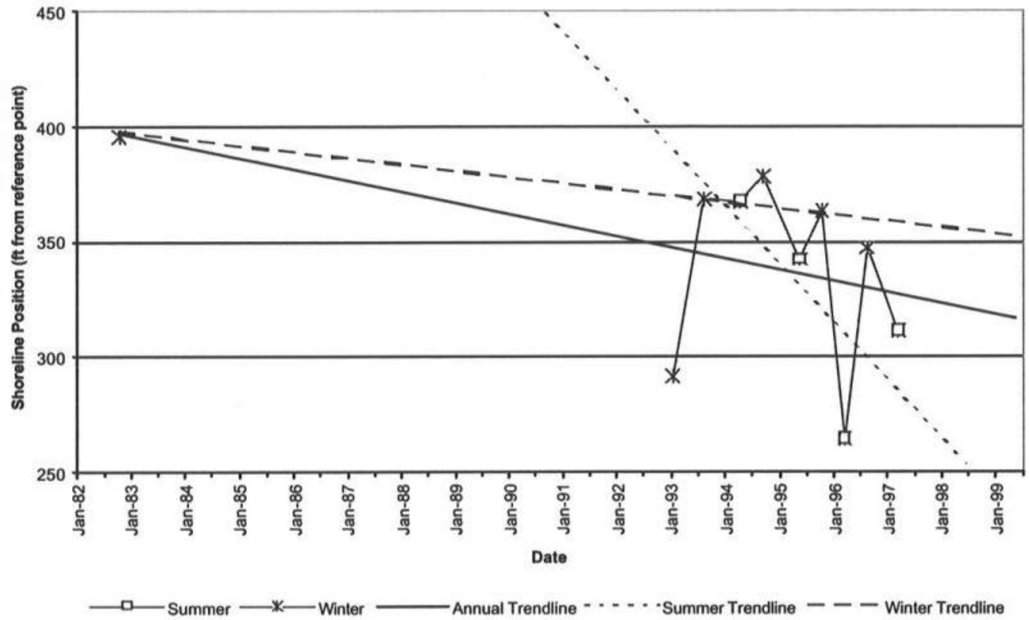


Figure A.15: Historical shoreline positions and trendlines for Profile 49.

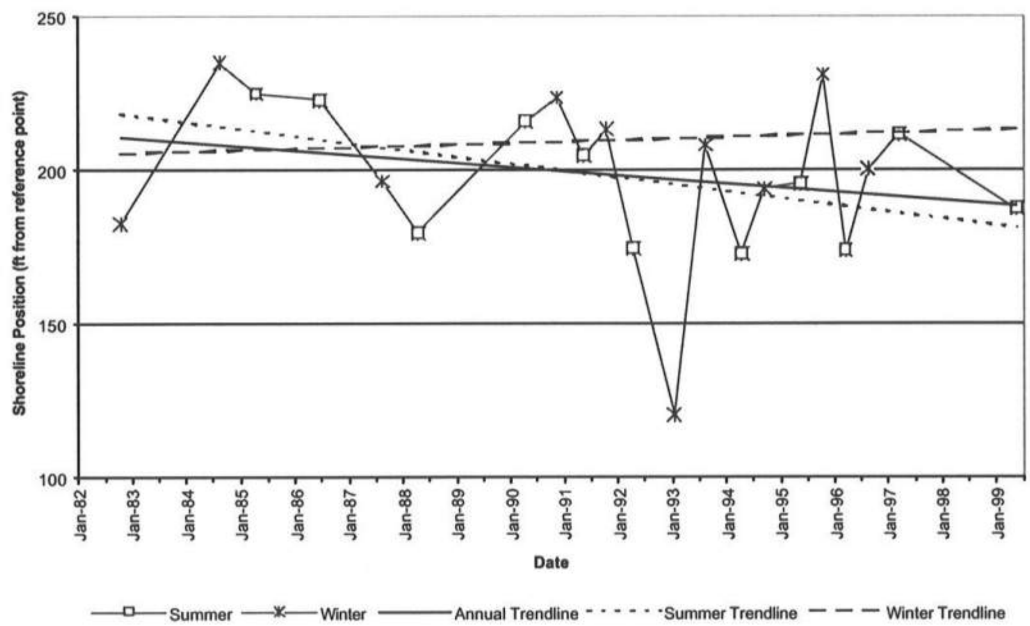


Figure A.16: Historical shoreline positions and trendlines for Profile 50.

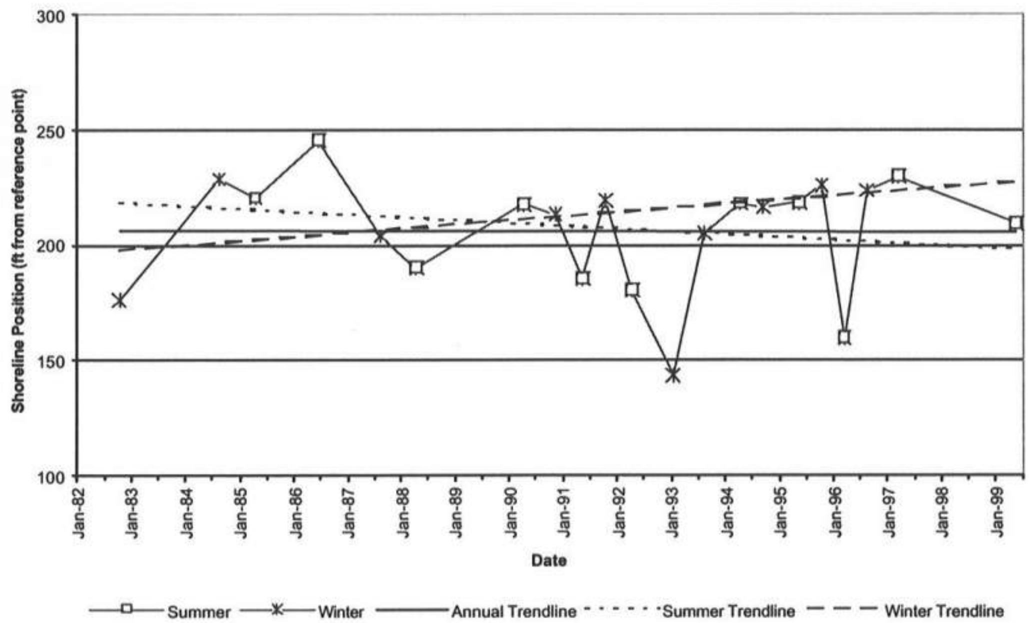


Figure A.17: Historical shoreline positions and trendlines for Profile 51.

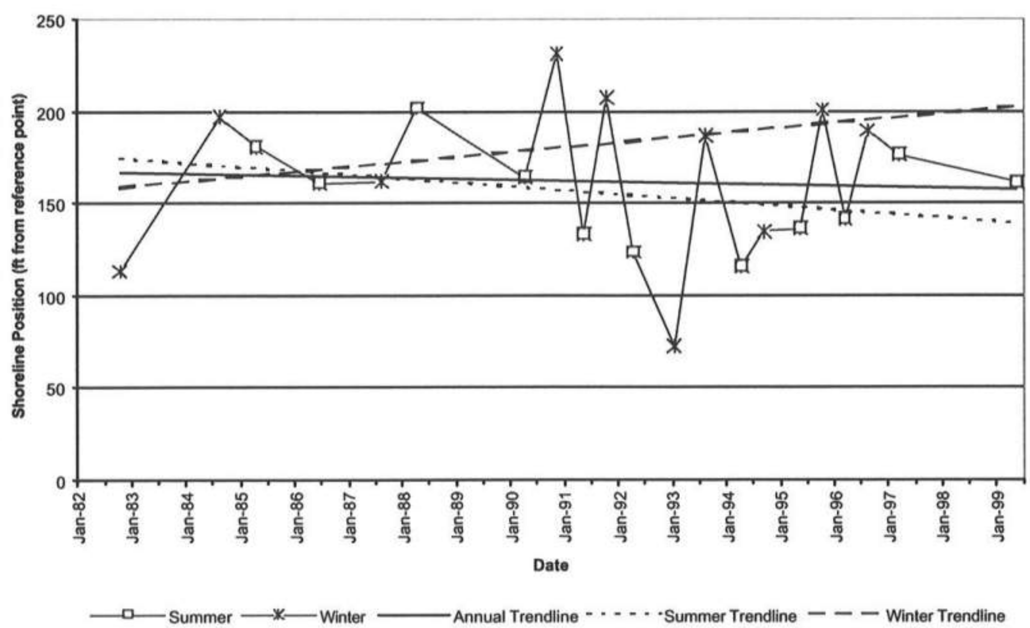


Figure A.18: Historical shoreline positions and trendlines for Profile 52.

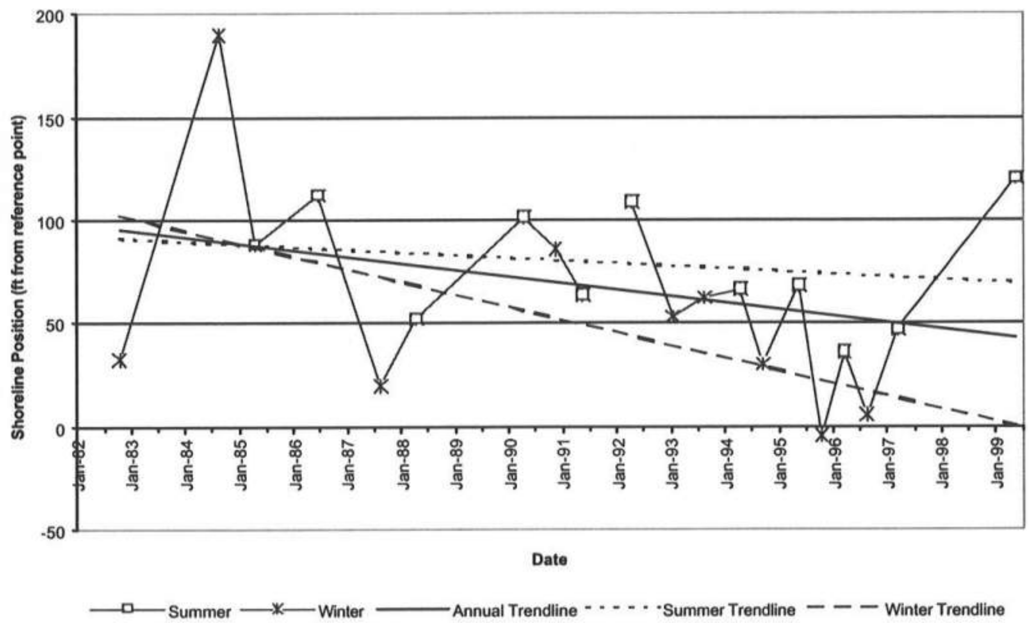


Figure A.19: Historical shoreline positions and trendlines for Profile 53.

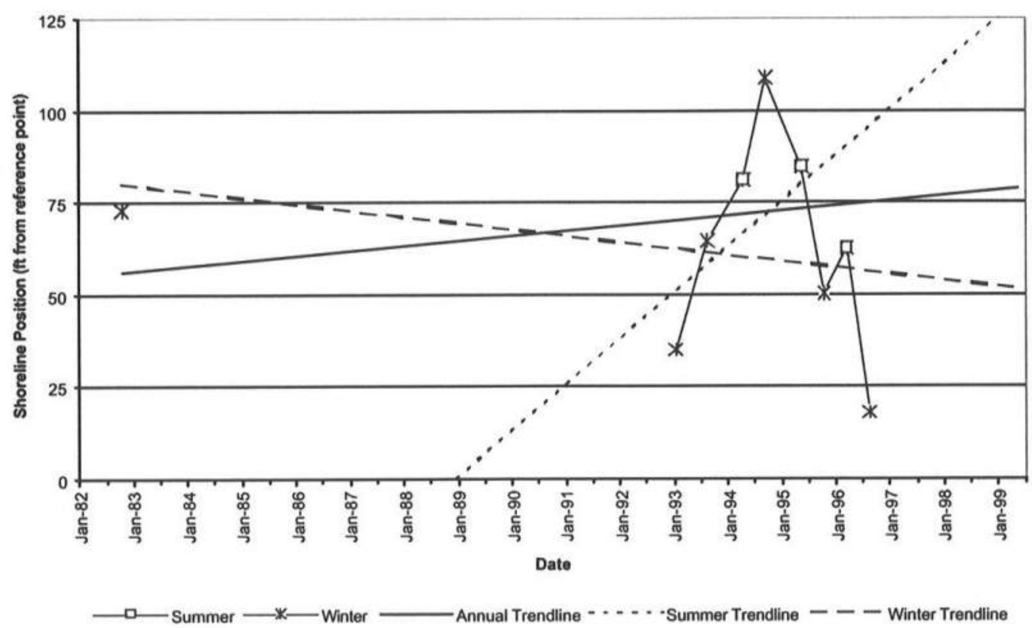


Figure A.20: Historical shoreline positions and trendlines for Profile 54.

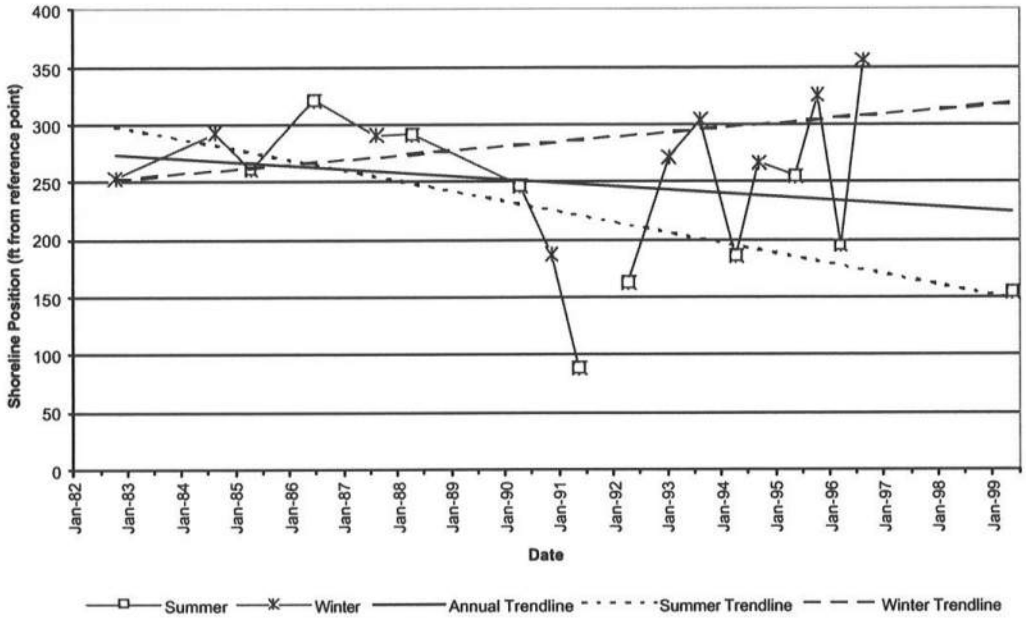


Figure A.21: Historical shoreline positions and trendlines for Profile 55.

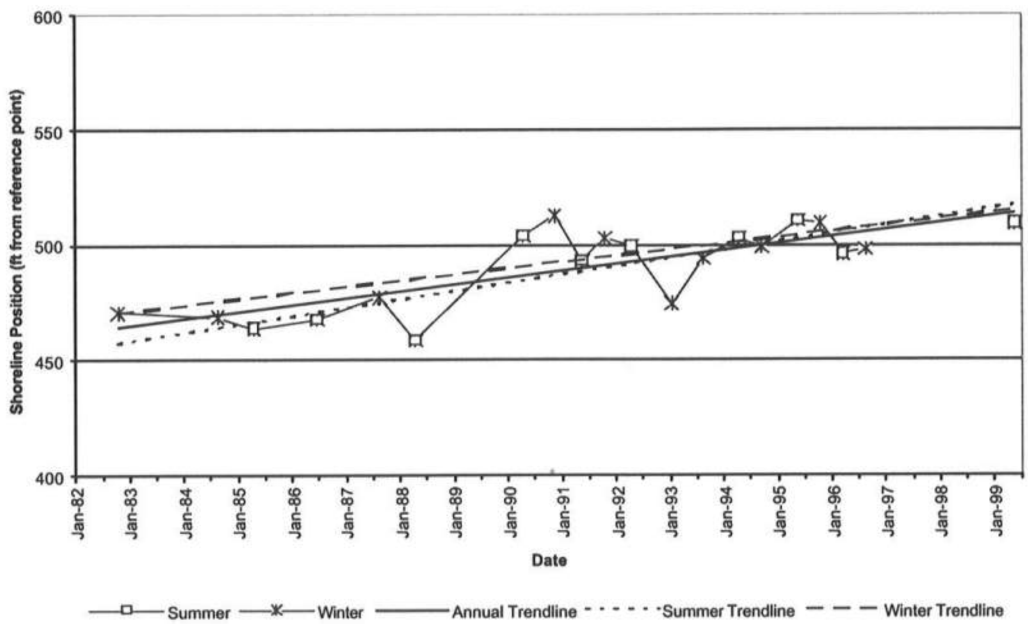


Figure A.22: Historical shoreline positions and trendlines for Profile 56.

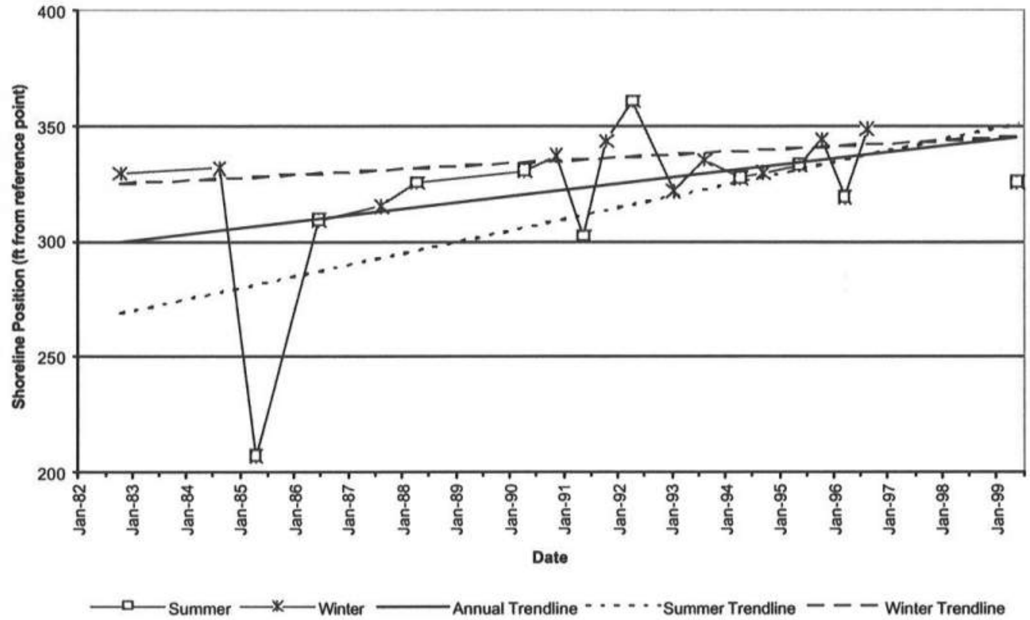


Figure A.23: Historical shoreline positions and trendlines for Profile 57.

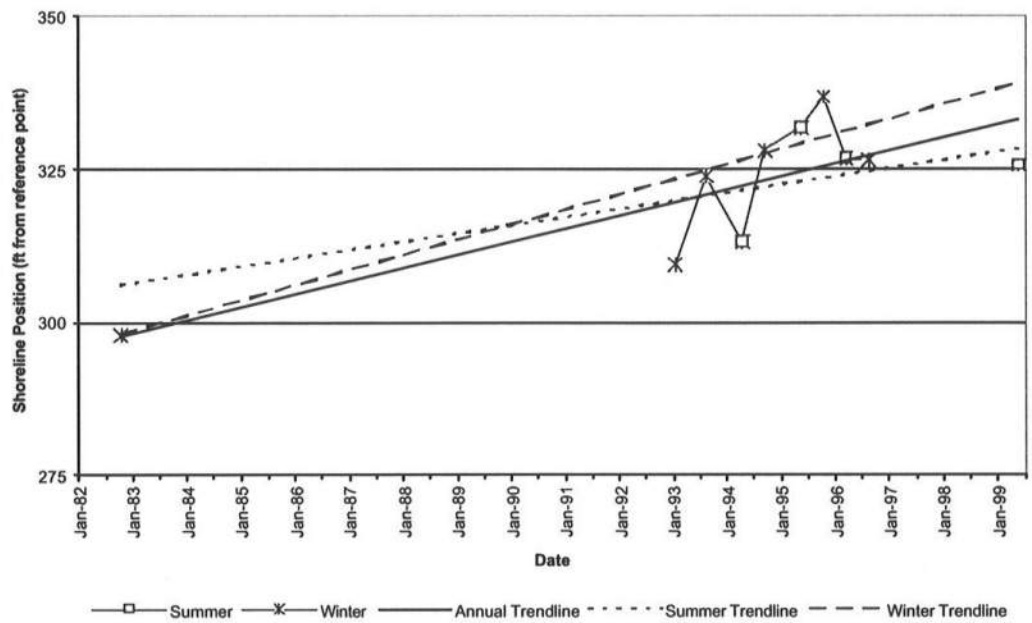


Figure A.24: Historical shoreline positions and trendlines for Profile 58.

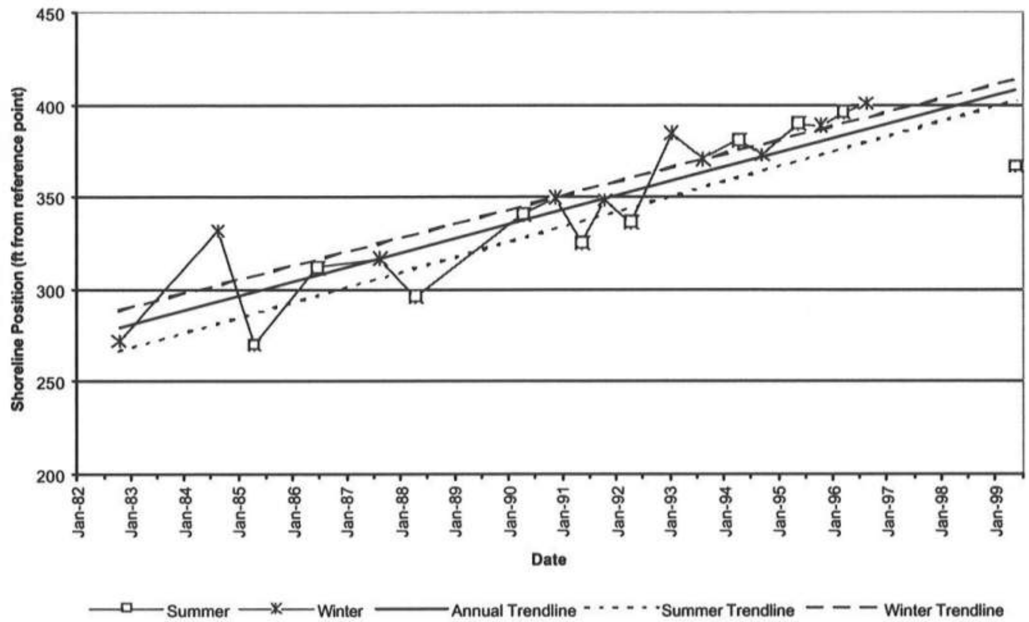


Figure A.25: Historical shoreline positions and trendlines for Profile 59.

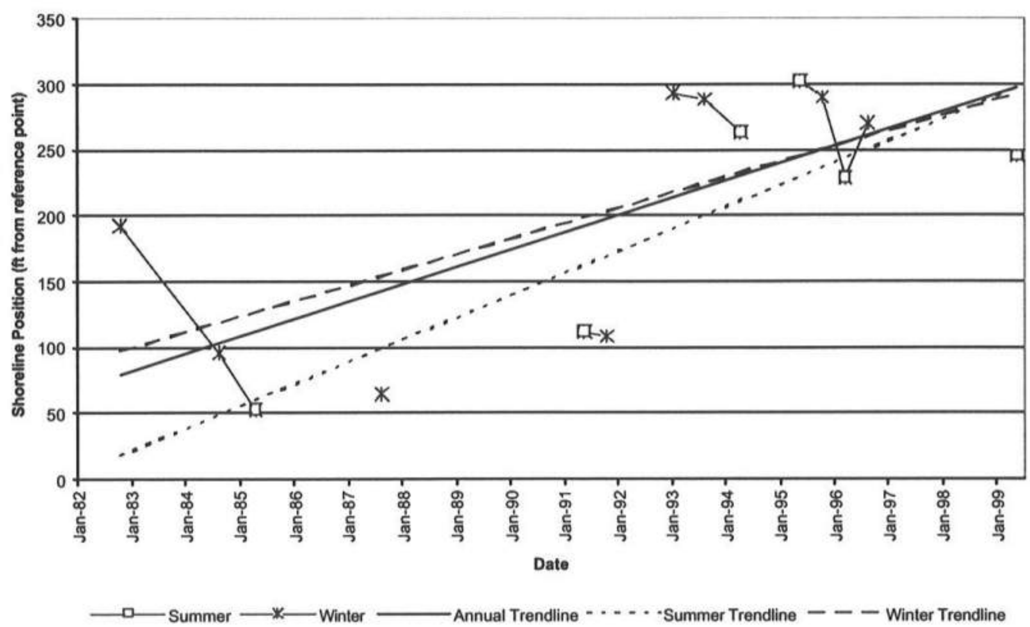


Figure A.26: Historical shoreline positions and trendlines for Profile 60.

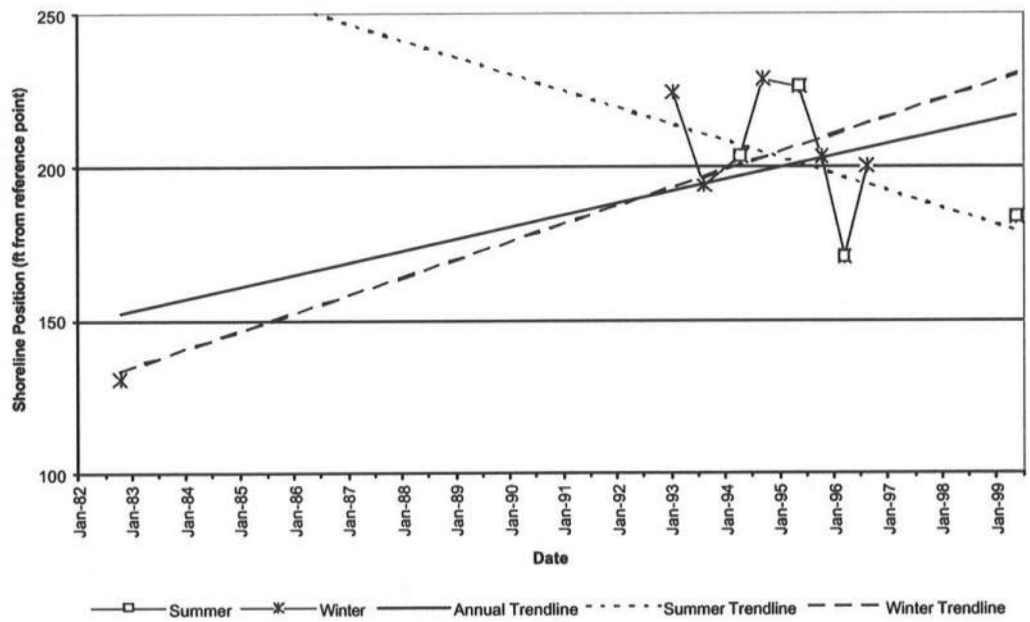


Figure A.27: Historical shoreline positions and trendlines for Profile 60A.

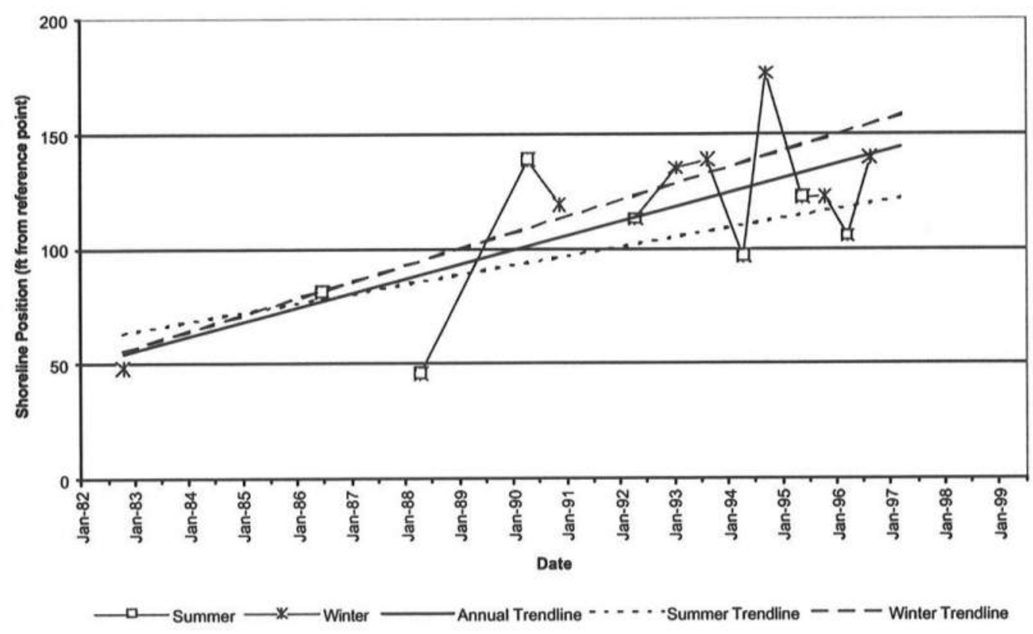


Figure A.28: Historical shoreline positions and trendlines for Profile 60B.

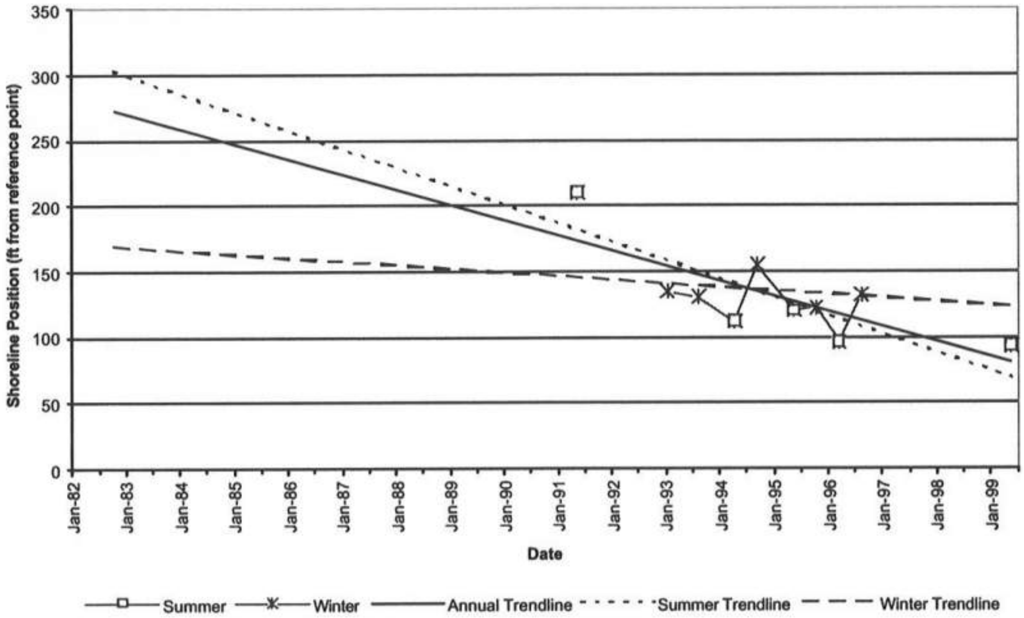


Figure A.29: Historical shoreline positions and trendlines for Profile 61.

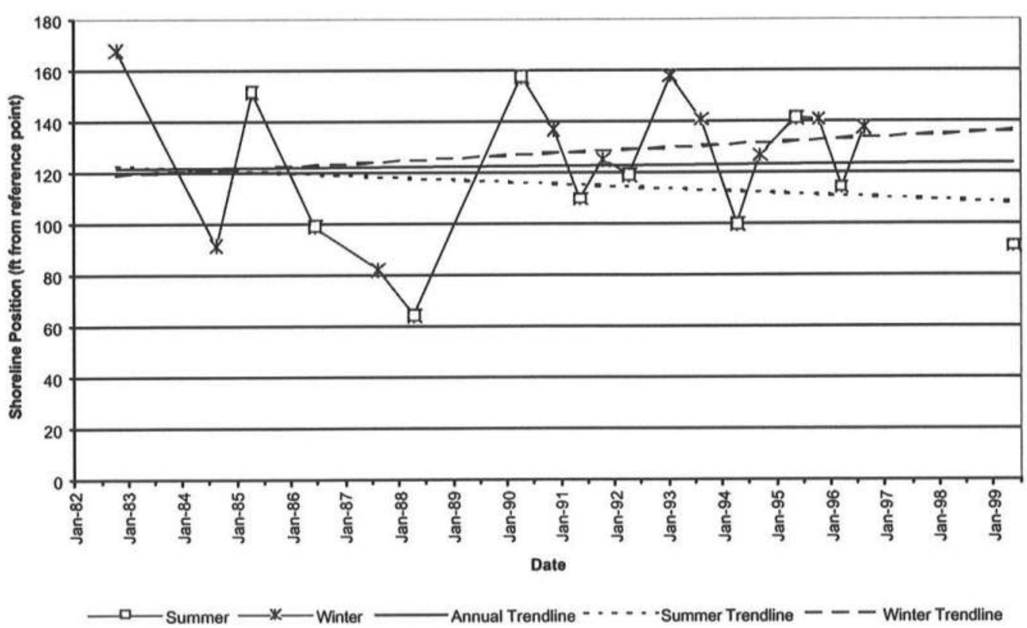


Figure A.30: Historical shoreline positions and trendlines for Profile 62.

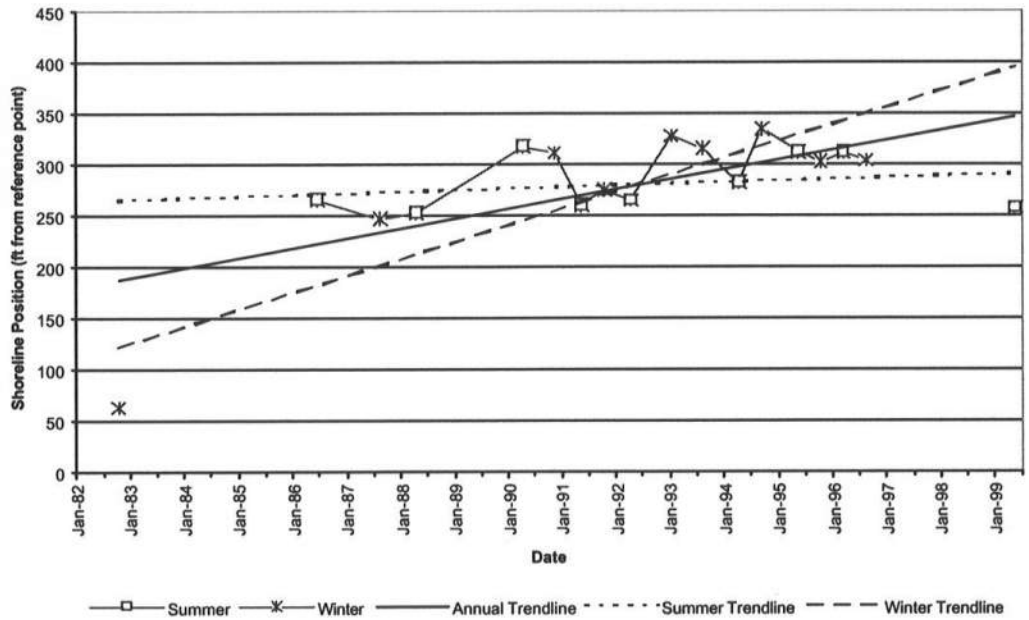


Figure A.31: Historical shoreline positions and trendlines for Profile 62A.

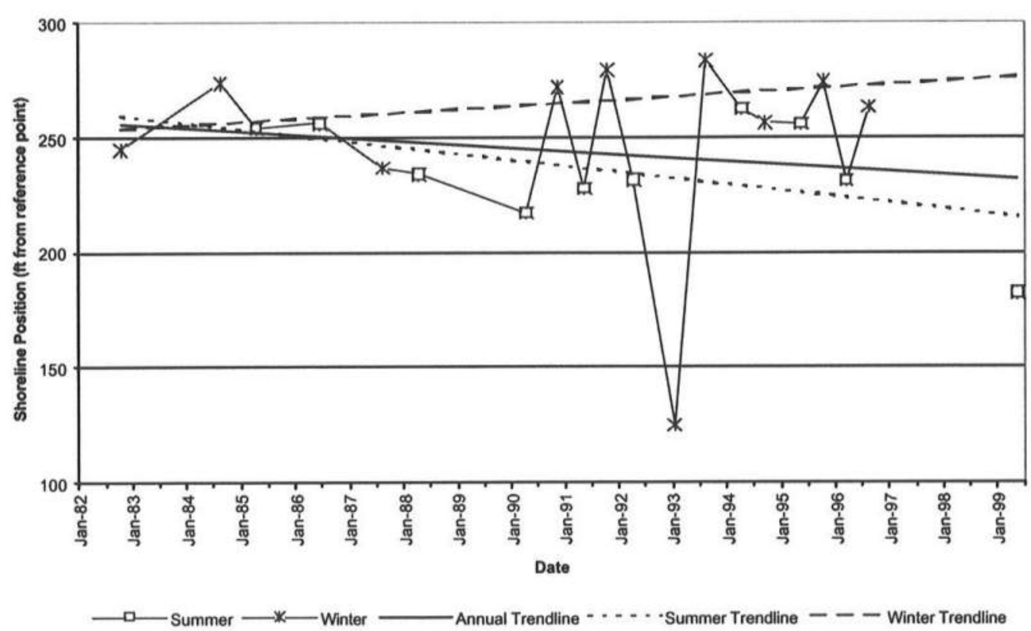


Figure A.32: Historical shoreline positions and trendlines for Profile 63.

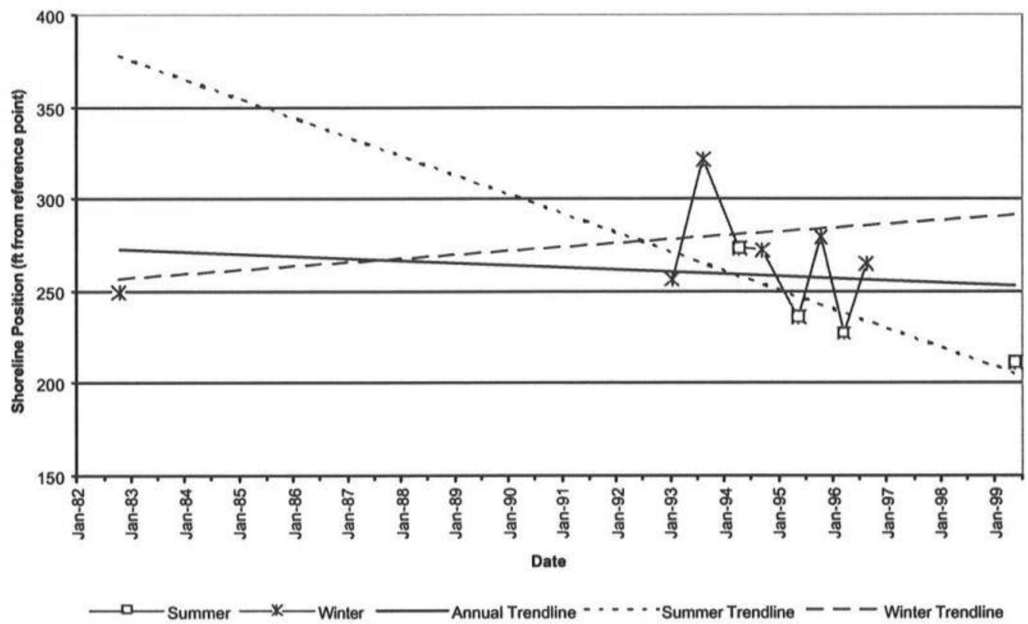


Figure A.33: Historical shoreline positions and trendlines for Profile 64.

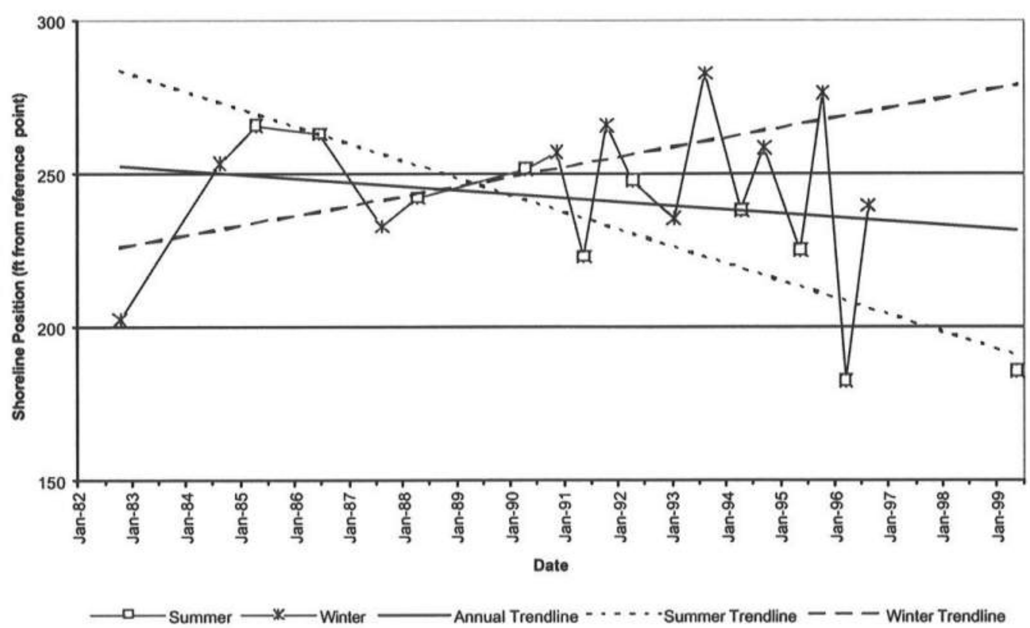


Figure A.34: Historical shoreline positions and trendlines for Profile 65.

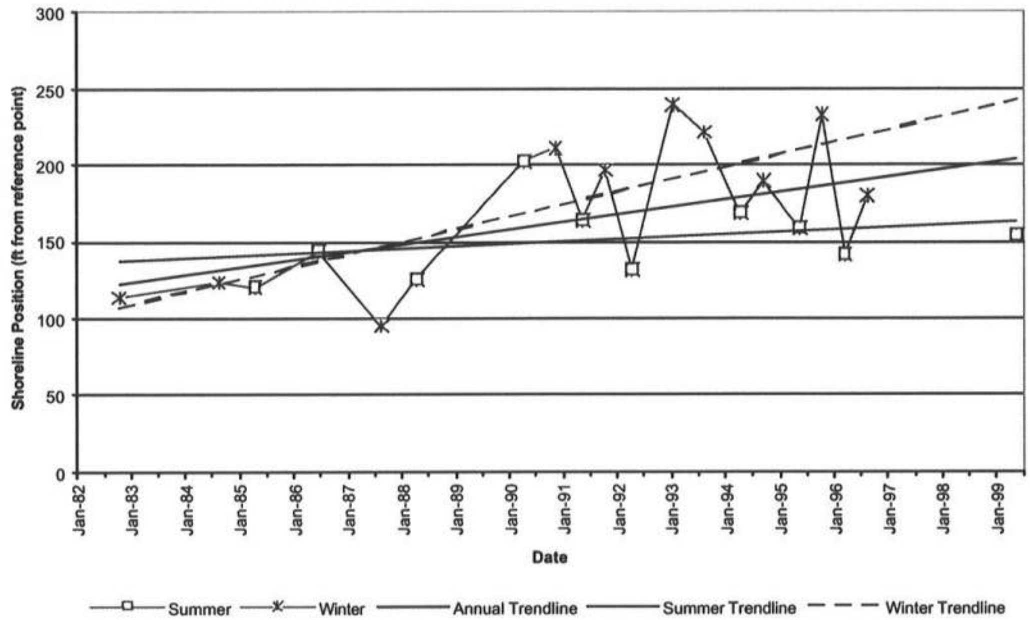


Figure A.35: Historical shoreline positions and trendlines for Profile 66.

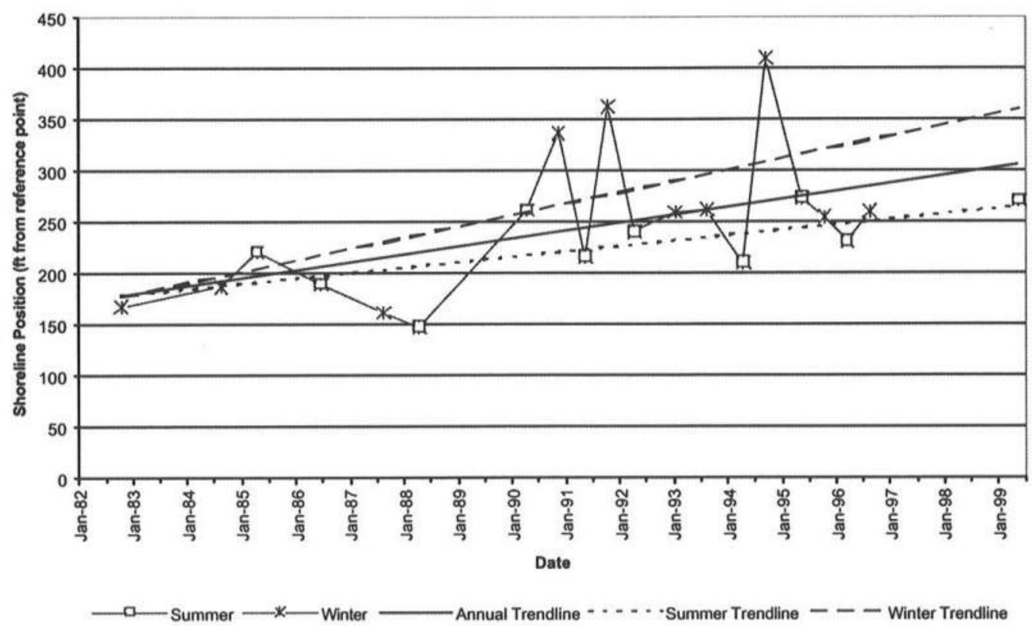


Figure A.36: Historical shoreline positions and trendlines for Profile 67.

Appendix B

SHORELINE POSITIONS BEFORE AND AFTER SIGNIFICANT BEACH FILLS AND STORMS ALONG THE DELAWARE COASTLINE

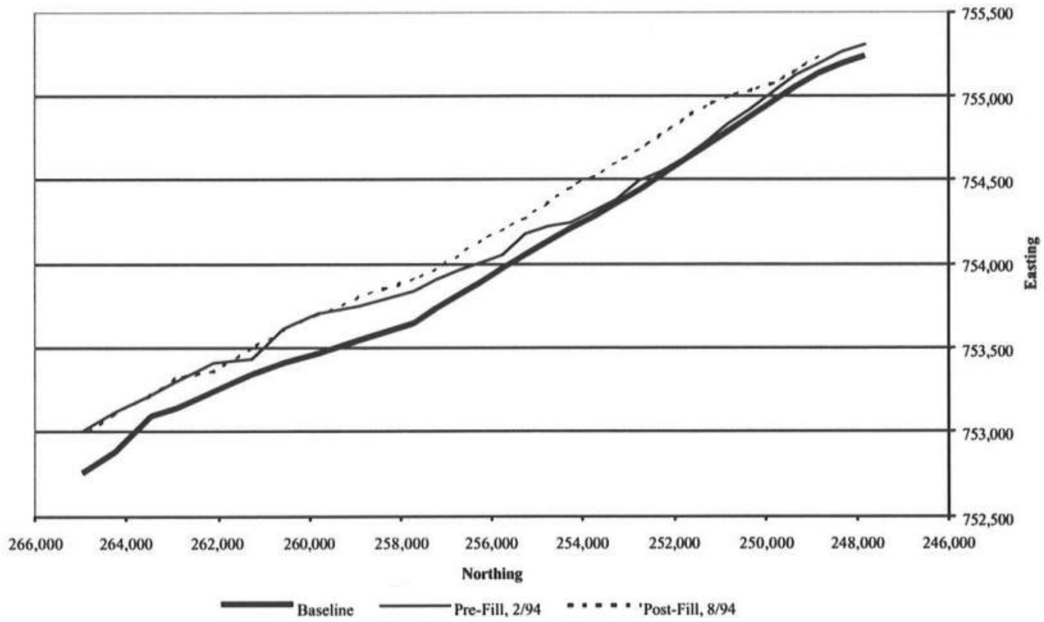


Figure B.1: Pre- and post-fill shoreline positions for the July 1994 beach fill in Dewey and Rehoboth Beaches.

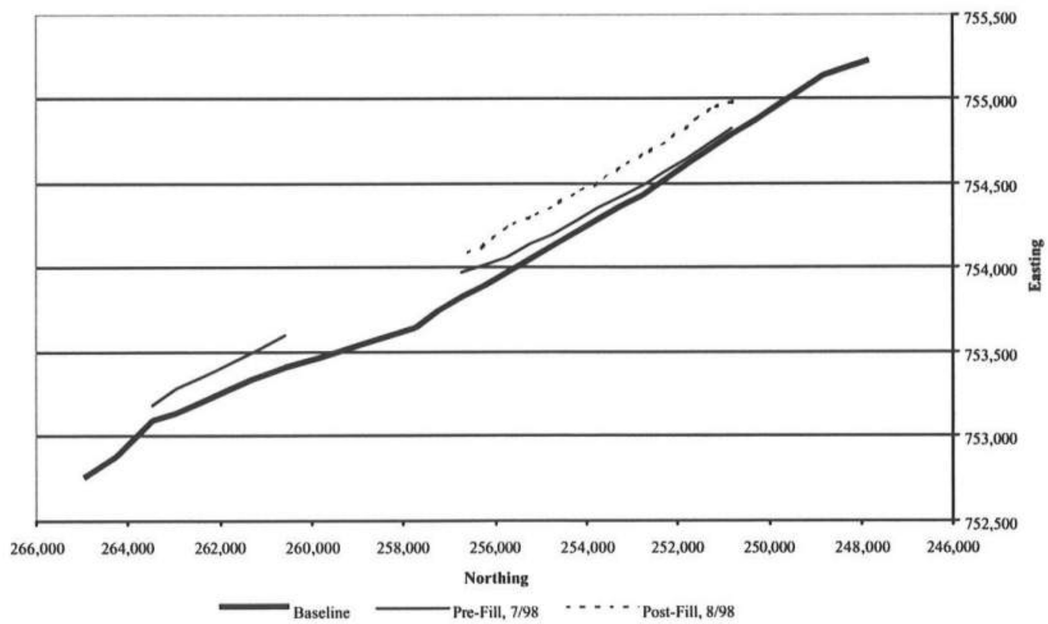


Figure B.2: Pre- and post-fill shoreline positions for the July 1998 beach fill in Dewey and Rehoboth Beaches.

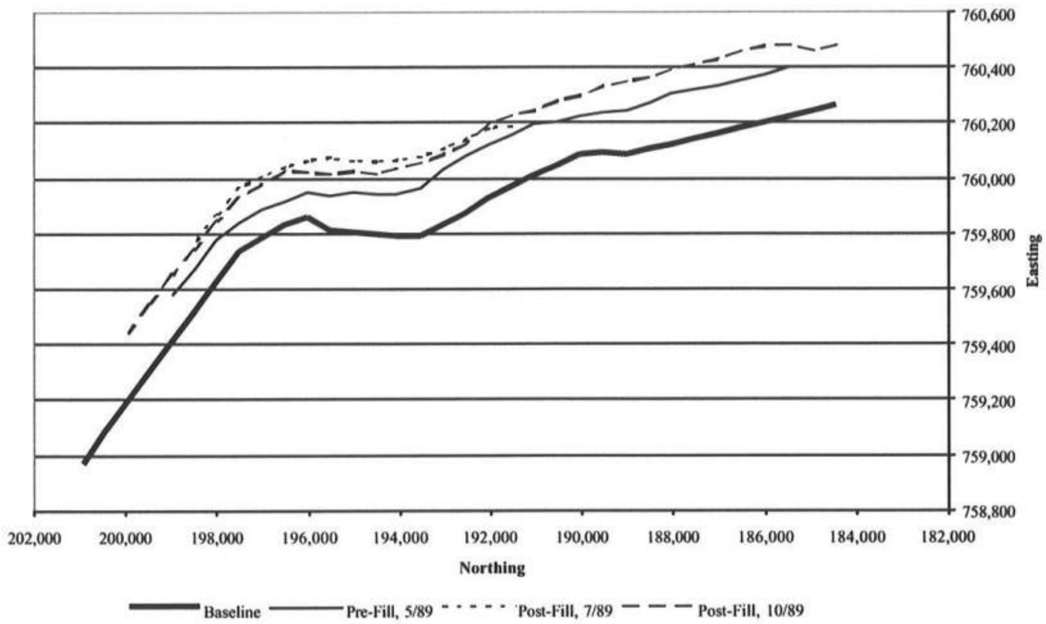


Figure B.3: Pre- and post-fill shoreline positions for the July 1989 beach fill in Bethany and South Bethany Beaches.

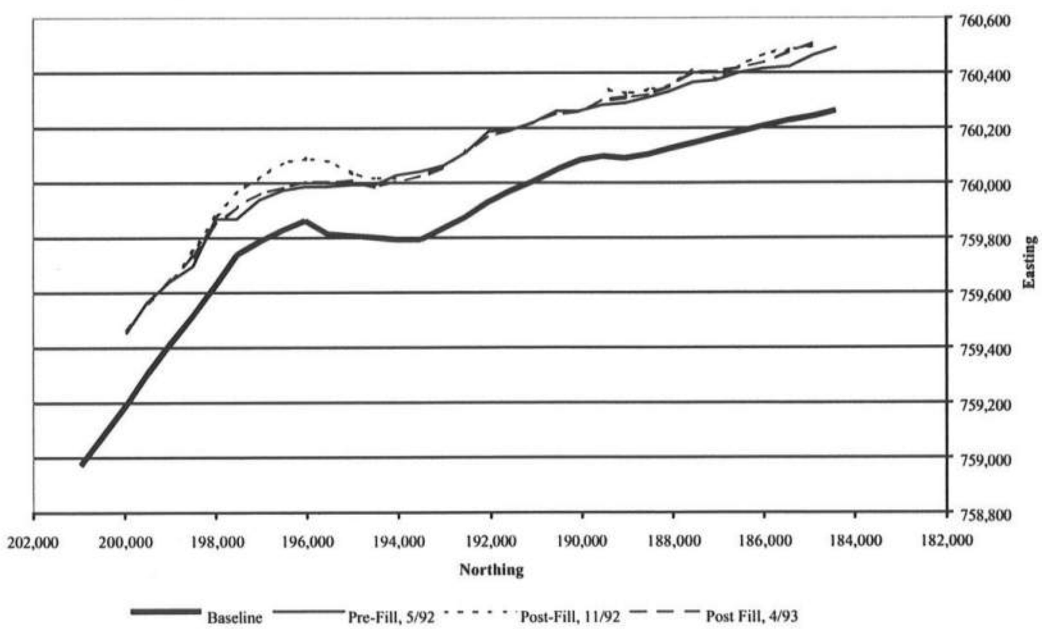


Figure B.4: Pre- and post-fill shoreline positions for the September 1992 beach fill in Bethany and South Bethany Beaches.

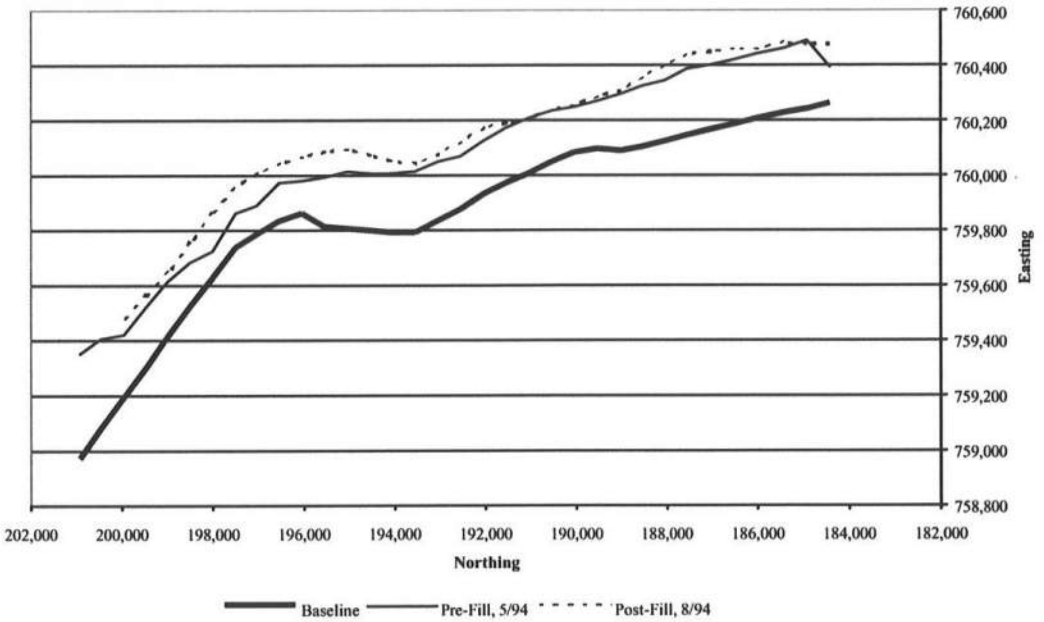


Figure B.5: Pre- and post-fill shoreline positions for the September 1994 beach fill in Bethany and South Bethany Beaches.

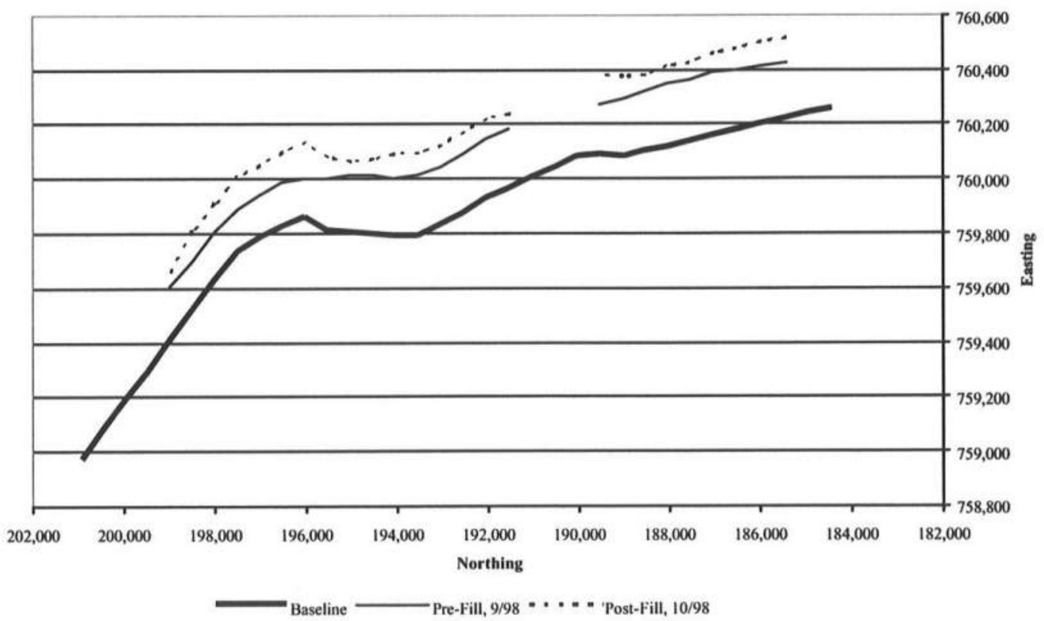


Figure B.6: Pre- and post-fill shoreline positions for the August 1998 beach fill in Bethany and South Bethany Beaches.

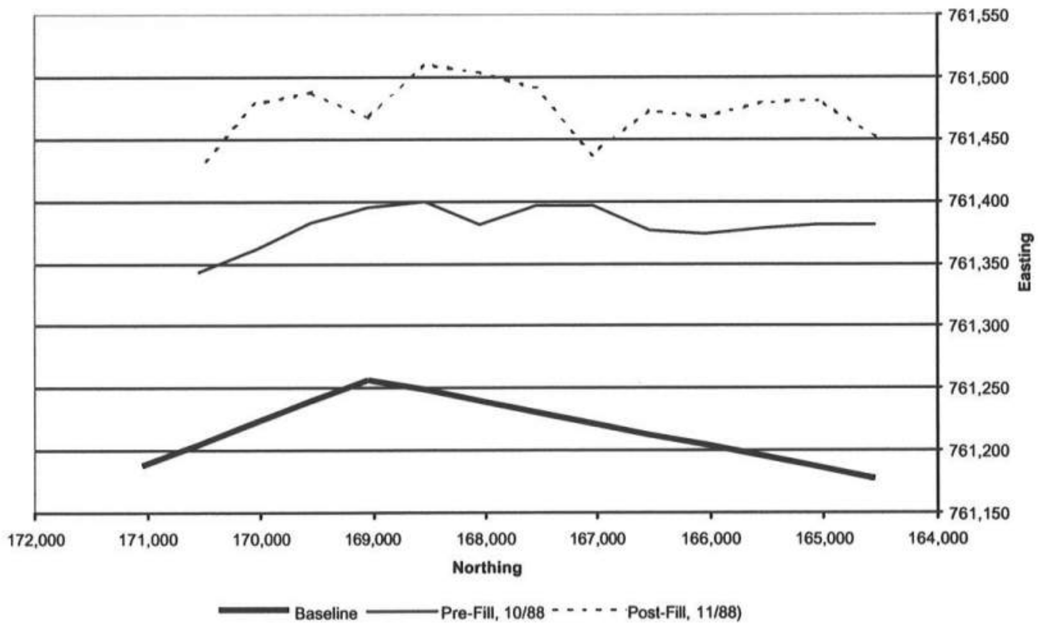


Figure B.7: Pre- and post-fill shoreline positions for the October 1988 beach fill in Fenwick Island.

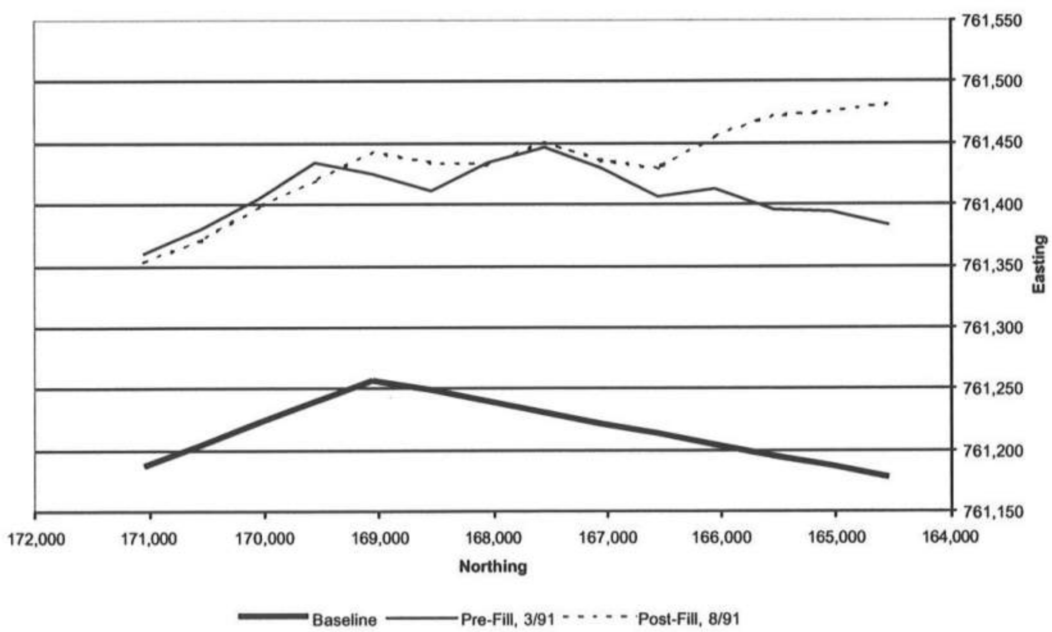


Figure B.8: Pre- and post-fill shoreline positions for the August 1991 beach fill in Fenwick Island.

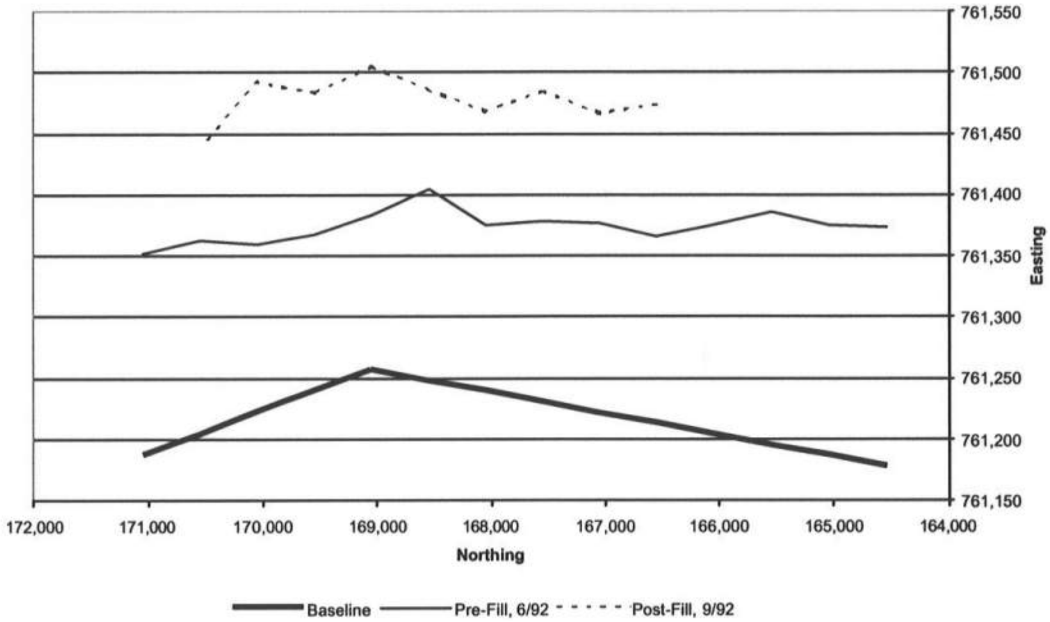


Figure B.9: Pre- and post-fill shoreline positions for the September 1992 beach fill in Fenwick Island.

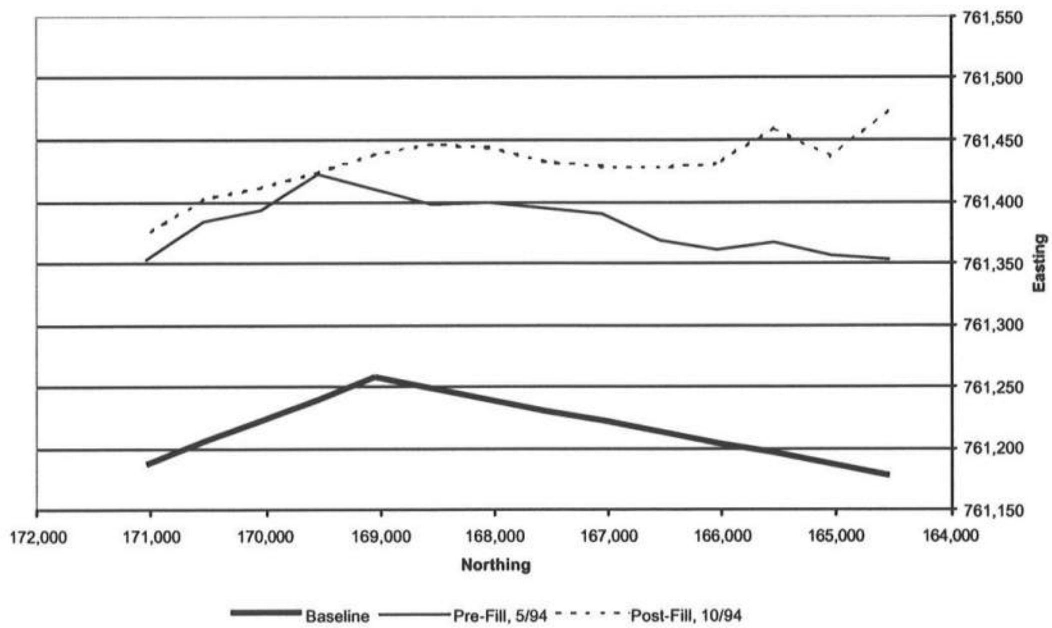


Figure B.10: Pre- and post-fill shoreline positions for the September 1994 beach fill in Fenwick Island.

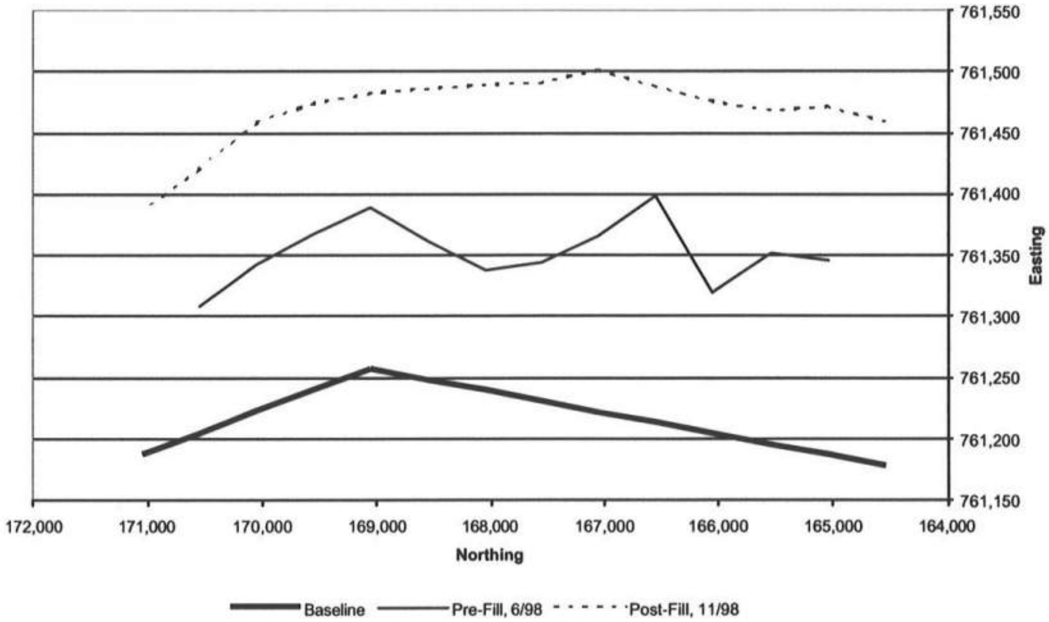


Figure B.11: Pre- and post-fill shoreline positions for the October 1998 beach fill in Fenwick Island.

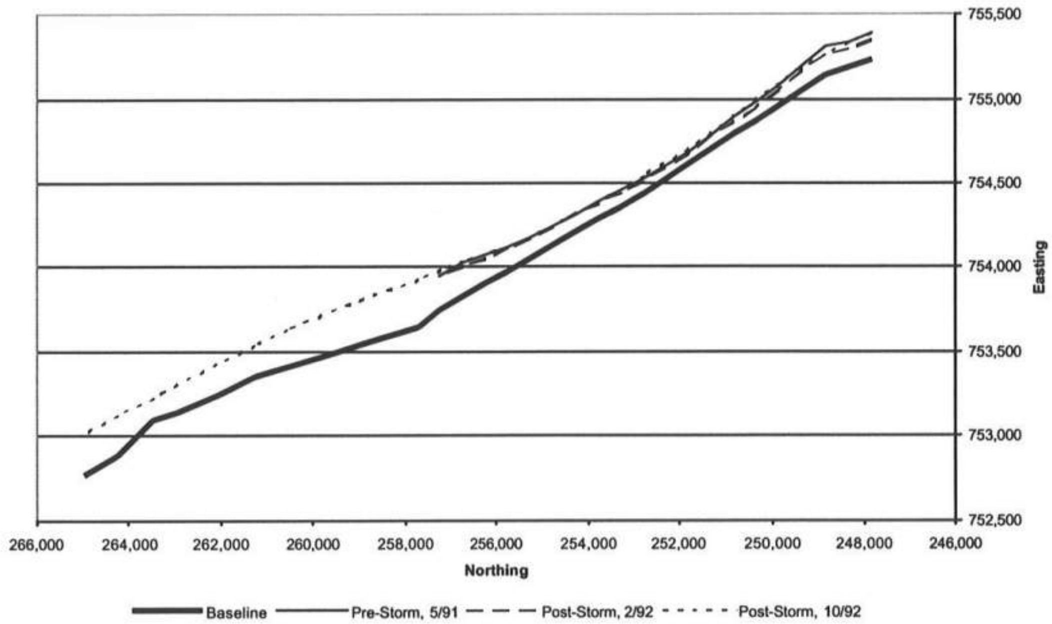


Figure B.12: Pre- and post-storm shoreline positions for the January 1992 northeaster at Dewey and Rehoboth Beaches.

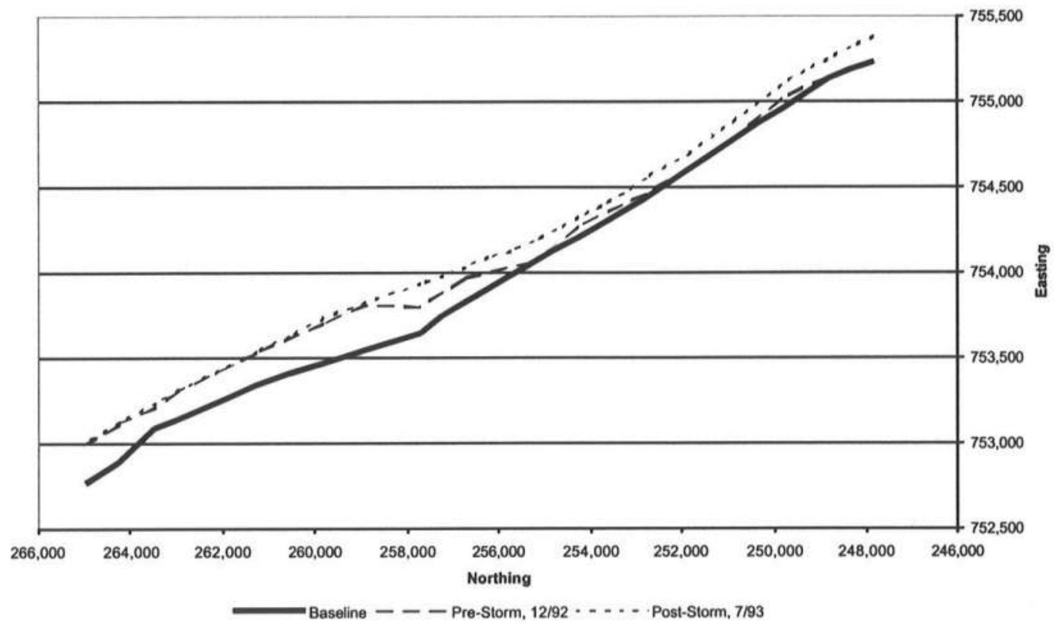


Figure B.13: Pre- and post-storm shoreline positions for the December 1992 northeaster at Dewey and Rehoboth Beaches.

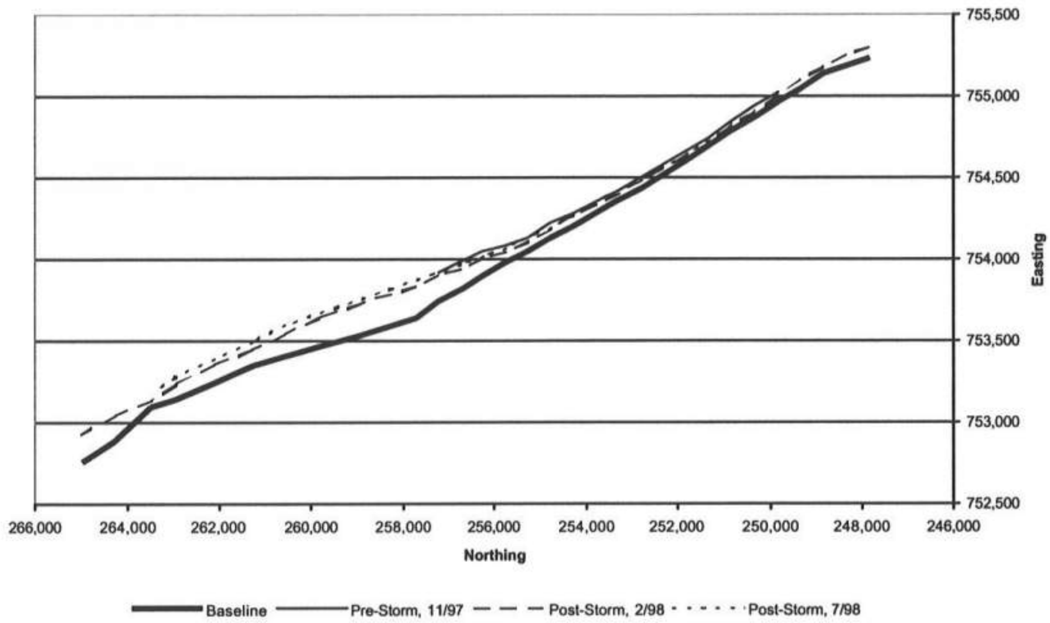


Figure B.14: Pre- and post-storm shoreline positions for the January 1998 northeaster at Dewey and Rehoboth Beaches.

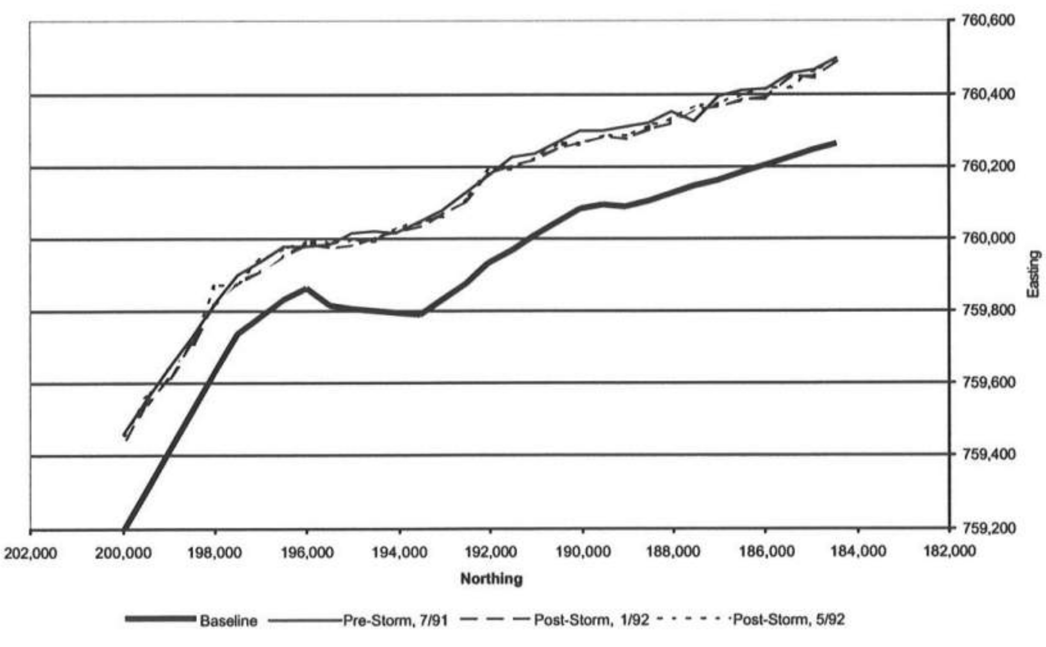


Figure B.15: Pre- and post-storm shoreline positions for the January 1992 northeaster at Bethany and South Bethany Beaches.

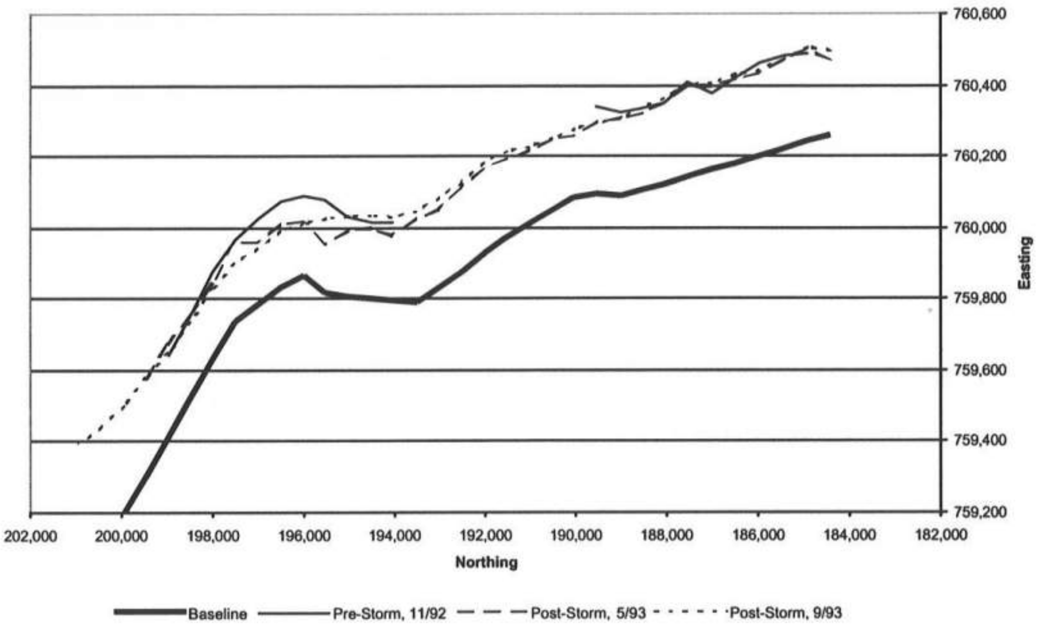


Figure B.16: Pre- and post-storm shoreline positions for the December 1992 northeaster at Bethany and South Bethany Beaches.

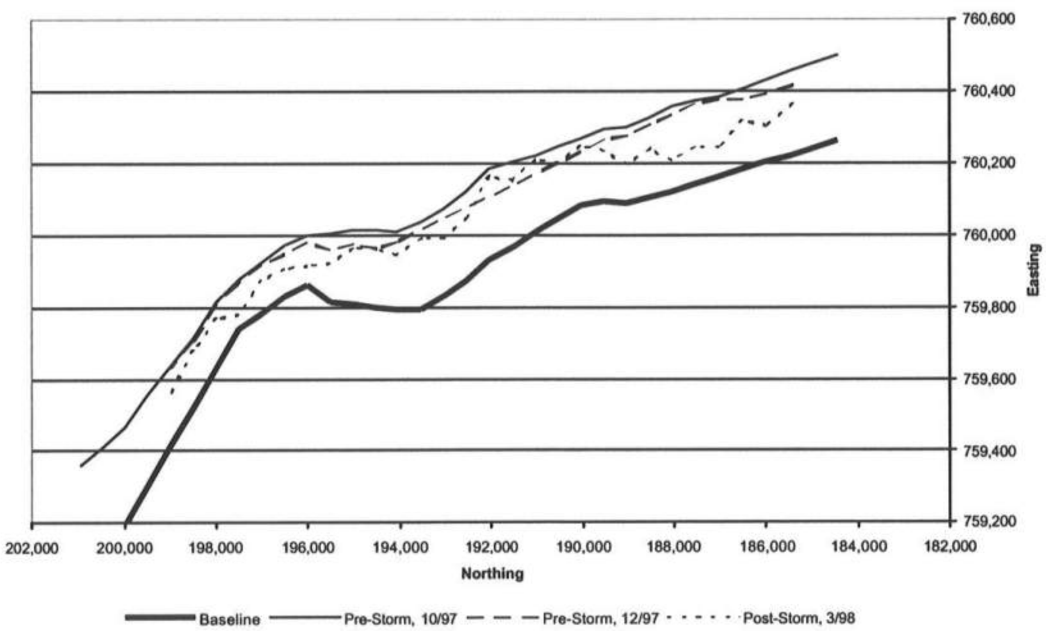


Figure B.17: Pre- and post-storm shoreline positions for the January 1998 northeaster at Bethany and South Bethany Beaches.

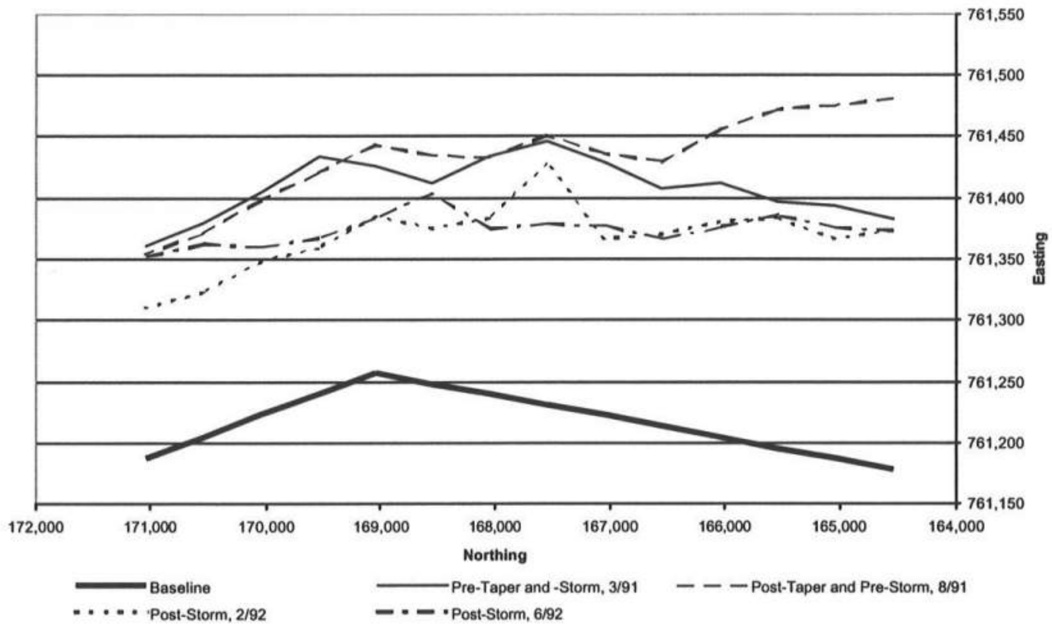


Figure B.18: Pre- and post-taper and storm shoreline positions for the January 1992 northeaster at Fenwick Island.

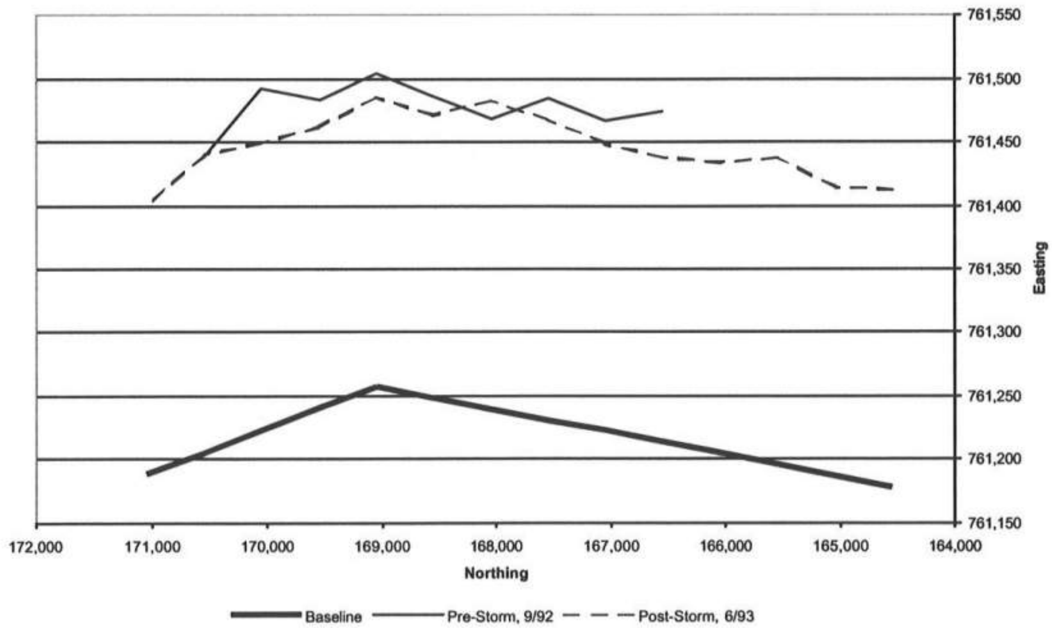


Figure B.19: Pre- and post-storm shoreline positions for the December 1992 northeaster at Fenwick Island.

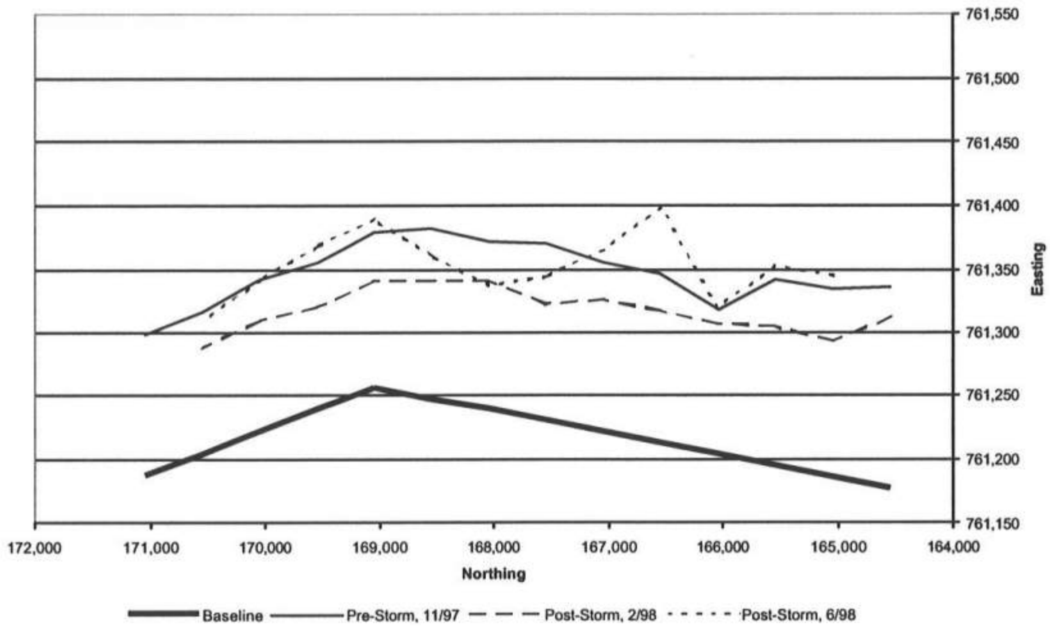


Figure B.20: Pre- and post-storm shoreline positions for the January 1998 northeaster in Fenwick Island.

Appendix C

SUMMARY OF THE 1976-1995 WAVE ENVIRONMENT FOR STATION 2066 AS PROVIDED BY THE COASTAL ENGINEERING DATA RETREIVAL SYSTEM (CEDRS)

WIS ATLANTIC UPDATE -- WITH HURRICANES 1976 - 1995
LAT: 38.75 N, LONG: 75.00 W, DEPTH: 18 M
SUMMARY OF WAVE INFORMATION BY MONTH

STATION: 66

OCCURRENCES OF WAVE HEIGHT BY MONTH FOR ALL YEARS

Hmo(m)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
0.00 - 0.49	350	390	388	505	511	822	1468	1206	629	547	381	517	7714
0.50 - 0.99	1687	1545	1589	1774	2576	2882	2862	2861	2307	2124	1703	1638	25548
1.00 - 1.49	1393	1416	1435	1370	1188	876	495	645	1133	1378	1422	1477	14228
1.50 - 1.99	762	611	743	619	470	176	118	152	457	536	715	777	6136
2.00 - 2.49	445	292	404	307	149	36	13	56	159	205	303	327	2696
2.50 - 2.99	177	124	173	117	41	8	2	14	45	97	141	112	1051
3.00 - 3.49	82	63	124	66	16	.	.	11	23	40	74	42	541
3.50 - 3.99	40	35	50	28	7	.	2	9	15	16	39	33	274
4.00 - 4.49	16	26	37	7	2	.	.	4	15	10	13	13	143
4.50 - 4.99	6	10	12	6	.	.	.	2	10	4	4	12	66
5.00 - 5.49	1	5	4	1	.	.	.	.	5	3	4	6	29
5.50 - 5.99	1	3	1	.	.	.	.	.	.	.	1	6	12
6.00 - 6.49	.	.	.	.	.	.	.	.	.	.	.	.	0
6.50 - 6.99	.	.	.	.	.	.	.	.	2	.	.	.	2
7.00 - 7.49	.	.	.	.	.	.	.	.	.	.	.	.	0
7.50 - 7.99	.	.	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.49	.	.	.	.	.	.	.	.	.	.	.	.	0
8.50 - 8.99	.	.	.	.	.	.	.	.	.	.	.	.	0
9.00 - 9.49	.	.	.	.	.	.	.	.	.	.	.	.	0
9.50 - 9.99	.	.	.	.	.	.	.	.	.	.	.	.	0
10.00 - GREATER	.	.	.	.	.	.	.	.	.	.	.	.	0
TOTAL	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440

STATION: 66

OCCURRENCES OF PEAK PERIOD BY MONTH FOR ALL YEARS

Tp(sec)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
3.0 - 3.9	377	382	258	317	333	579	788	560	401	291	382	458	5126
4.0 - 4.9	983	1008	862	748	800	906	930	679	608	773	936	1176	10409
5.0 - 5.9	677	648	568	489	419	546	576	515	349	393	582	710	6472
6.0 - 6.9	294	304	375	379	541	617	498	651	439	323	319	292	5032
7.0 - 7.9	236	250	319	425	719	627	633	878	651	490	345	182	5755
8.0 - 8.9	273	308	356	417	897	730	870	773	636	682	450	219	6611
9.0 - 9.9	372	414	443	576	619	495	457	390	490	637	474	363	5730
10.0 - 10.9	550	408	486	524	285	186	142	142	305	537	549	404	4518
11.0 - 11.9	459	306	451	370	156	78	51	76	240	324	326	356	3193
12.0 - 12.9	303	216	380	313	75	19	4	93	154	243	185	250	2235
13.0 - 13.9	208	120	237	150	67	6	7	77	137	151	94	244	1498
14.0 - 14.9	118	69	95	36	23	5	3	48	112	58	68	124	759
15.0 - 15.9	54	18	55	40	21	6	1	16	100	25	30	118	484
16.0 - 16.9	18	20	33	12	4	.	.	21	68	14	27	55	272
17.0 - 17.9	17	20	12	4	1	.	.	21	42	13	4	5	139
18.0 - 18.9	9	8	3	.	.	.	.	9	21	2	8	2	62
19.0 - 19.9	5	9	4	.	.	.	.	2	20	3	11	.	54
20.0 - LONGER	7	12	23	.	.	.	.	9	27	1	10	2	91
TOTAL	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440

STATION: 66

OCCURRENCES OF PEAK DIRECTION BY MONTH FOR ALL YEARS

Dp(deg) DIRECTION BAND & CENTER	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
348.75 - 11.24 (0.0)	127	215	159	109	127	129	109	122	132	120	162	175	1686
11.25 - 33.74 (22.5)	54	85	60	49	54	61	50	83	90	75	72	70	803
33.75 - 56.24 (45.0)	53	63	41	32	61	50	23	57	94	75	51	63	663
56.25 - 78.74 (67.5)	201	181	195	163	258	177	35	197	272	299	171	136	2285
78.75 - 101.24 (90.0)	604	557	861	844	692	406	255	596	960	1085	657	788	8305
101.25 - 123.74 (112.5)	974	770	953	900	977	793	843	1147	1251	1231	1085	882	11806
123.75 - 146.24 (135.0)	912	796	913	964	1091	1095	1260	1193	1064	845	745	619	11497
146.25 - 168.74 (157.5)	189	158	317	350	431	498	550	452	161	181	230	133	3650
168.75 - 191.24 (180.0)	262	283	378	441	468	548	654	428	225	224	292	254	4457
191.25 - 213.74 (202.5)	145	159	157	175	204	277	316	229	154	121	173	238	2348
213.75 - 236.24 (225.0)	101	148	110	83	119	174	269	116	74	108	120	194	1616
236.25 - 258.74 (247.5)	120	101	72	55	61	130	126	98	44	76	102	152	1137
258.75 - 281.24 (270.0)	152	107	73	98	73	120	110	57	54	78	167	174	1263
281.25 - 303.74 (292.5)	337	176	149	140	83	115	127	51	63	108	228	309	1886
303.75 - 326.24 (315.0)	416	360	267	204	117	113	136	51	82	188	282	421	2637
326.25 - 348.74 (337.5)	313	361	255	193	144	114	97	83	80	146	263	352	2401
TOTAL	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440

AUL2-391

WIS ATLANTIC UPDATE -- WITH HURRICANES 1976 - 1995
LAT: 38.75 N, LONG: 75.00 W, DEPTH: 18 M
OCCURRENCES OF WAVE HEIGHT AND PEAK PERIOD FOR 22.5-DEG DIRECTION BANDS

STATION: 66 (348.75 - 11.24) 0.0 DEG

Hmo (m) \ Tp (sec)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	963	.	.	.	.	.	.	.	.	.	963
1.00 - 1.99	552	133	.	.	.	.	.	.	.	.	685
2.00 - 2.99	.	35	.	.	.	.	.	.	.	.	35
3.00 - 3.99	.	3	.	.	.	.	.	.	.	.	3
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1515	171	0	0	0	0	0	0	0	0	1686

STATION: 66 (11.25 - 33.74) 22.5 DEG

Hmo (m) \ Tp (sec)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	469	1	.	.	.	.	.	.	.	.	470
1.00 - 1.99	278	36	.	.	.	.	.	.	.	.	314
2.00 - 2.99	.	19	.	.	.	.	.	.	.	.	19
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	747	56	0	0	0	0	0	0	0	0	803

STATION: 66 (33.75 - 56.24) 45.0 DEG

Hmo (m) \ Tp (sec)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	294	56	5	.	.	.	.	.	.	.	355
1.00 - 1.99	98	135	43	1	.	.	.	.	.	.	277
2.00 - 2.99	.	7	14	3	.	.	.	.	.	.	24
3.00 - 3.99	.	2	2	3	.	.	.	.	.	.	7
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	392	200	64	7	0	0	0	0	0	0	663

STATION: 66 (56.25 - 78.74) 67.5 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	245	340	340	167	76	17	.	.	.	.	1185
1.00 - 1.99	59	377	252	37	11	3	.	.	.	.	739
2.00 - 2.99	.	18	169	87	7	.	.	.	.	.	281
3.00 - 3.99	.	.	11	47	7	1	.	.	.	.	66
4.00 - 4.99	.	.	1	3	9	.	.	.	.	.	13
5.00 - 5.99	.	.	.	1	.	.	.	.	.	.	1
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	304	735	773	342	110	21	0	0	0	0	2285

STATION: 66 (78.75 - 101.24) 90.0 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	292	542	920	1154	1032	592	181	23	2	.	4738
1.00 - 1.99	34	503	755	446	552	412	167	26	.	.	2895
2.00 - 2.99	.	46	168	83	92	51	50	1	.	.	491
3.00 - 3.99	.	.	42	83	23	7	.	.	.	.	155
4.00 - 4.99	.	.	2	16	4	3	.	.	.	.	25
5.00 - 5.99	.	.	.	.	1	.	.	.	.	.	1
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	326	1091	1887	1782	1704	1065	398	50	2	0	8305

WIS ATLANTIC UPDATE -- WITH HURRICANES 1976 - 1995
LAT: 38.75 N, LONG: 75.00 W, DEPTH: 18 M
OCCURRENCES OF WAVE HEIGHT AND PEAK PERIOD FOR 22.5-DEG DIRECTION BANDS

STATION: 66 (101.25 - 123.74) 112.5 DEG

Hmo(m)	Tp(sec) 3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	247	569	2448	2518	1007	208	67	49	62	4	7179
1.00 - 1.99	39	430	703	1105	789	391	97	31	24	.	3609
2.00 - 2.99	.	34	145	192	216	72	42	5	.	.	706
3.00 - 3.99	.	.	37	57	70	32	9	3	.	.	208
4.00 - 4.99	.	.	1	16	26	28	9	1	.	.	81
5.00 - 5.99	.	.	.	.	8	10	3	.	.	.	21
6.00 - 6.99	.	.	.	.	1	1	.	.	.	.	2
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	286	1033	3334	3888	2117	742	227	89	86	4	11806

STATION: 66 (123.75 - 146.24) 135.0 DEG

Hmo(m)	Tp(sec) 3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	301	716	3187	2470	577	107	30	15	12	6	7421
1.00 - 1.99	39	424	727	1020	638	185	57	24	11	1	3126
2.00 - 2.99	.	37	164	174	190	101	42	11	5	.	724
3.00 - 3.99	.	.	31	56	36	22	2	7	4	.	158
4.00 - 4.99	.	.	1	16	12	10	.	5	10	.	54
5.00 - 5.99	.	.	.	1	8	1	.	.	4	.	14
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	340	1177	4110	3737	1461	426	131	62	46	7	11497

STATION: 66 (146.25 - 168.74) 157.5 DEG

Hmo(m)	Tp(sec) 3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	430	781	927	218	2	.	.	.	.	.	2358
1.00 - 1.99	59	513	229	75	7	.	.	.	.	.	883
2.00 - 2.99	.	62	148	47	8	.	.	.	.	.	265
3.00 - 3.99	.	.	41	63	10	.	.	.	.	.	114
4.00 - 4.99	.	.	1	19	5	1	.	.	.	.	26
5.00 - 5.99	.	.	.	.	2	2	.	.	.	.	4
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	489	1356	1346	422	34	3	0	0	0	0	3650

STATION: 66 (168.75 - 191.24) 180.0 DEG

Tp(sec)

Hmo(m)	3.0- 4.9	5.0- 6.9	7.0- 8.9	9.0- 10.9	11.0- 12.9	13.0- 14.9	15.0- 16.9	17.0- 18.9	19.0- 20.9	21.0- LONGER	TOTAL
0.00 - 0.99	986	986	61	.	.	.	.	.	.	.	2033
1.00 - 1.99	139	1376	267	2	.	.	.	.	.	.	1784
2.00 - 2.99	.	142	390	24	.	.	.	.	.	.	556
3.00 - 3.99	.	.	40	34	.	.	.	.	.	.	74
4.00 - 4.99	.	.	.	8	2	.	.	.	.	.	10
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1125	2504	758	68	2	0	0	0	0	0	4457

STATION: 66 (191.25 - 213.74) 202.5 DEG

Tp(sec)

Hmo(m)	3.0- 4.9	5.0- 6.9	7.0- 8.9	9.0- 10.9	11.0- 12.9	13.0- 14.9	15.0- 16.9	17.0- 18.9	19.0- 20.9	21.0- LONGER	TOTAL
0.00 - 0.99	1025	119	6	.	.	.	.	.	.	.	1150
1.00 - 1.99	499	565	33	.	.	.	.	.	.	.	1097
2.00 - 2.99	.	51	45	1	.	.	.	.	.	.	97
3.00 - 3.99	.	1	3	.	.	.	.	.	.	.	4
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1524	736	87	1	0	0	0	0	0	0	2348

AUL2-393

WIS ATLANTIC UPDATE -- WITH HURRICANES 1976 - 1995
LAT: 38.75 N, LONG: 75.00 W, DEPTH: 18 M
OCCURRENCES OF WAVE HEIGHT AND PEAK PERIOD FOR 22.5-DEG DIRECTION BANDS

STATION: 66 (213.75 - 236.24) 225.0 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	921	1	.	.	.	.	.	.	.	.	922
1.00 - 1.99	490	172	1	.	.	.	.	.	.	.	663
2.00 - 2.99	.	30	.	1	.	.	.	.	.	.	31
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1411	203	1	1	0	0	0	0	0	0	1616

STATION: 66 (236.25 - 258.74) 247.5 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	781	.	.	.	.	.	.	.	.	.	781
1.00 - 1.99	303	40	.	.	.	.	.	.	.	.	343
2.00 - 2.99	.	13	.	.	.	.	.	.	.	.	13
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1084	53	0	0	0	0	0	0	0	0	1137

STATION: 66 (258.75 - 281.24) 270.0 DEG

Tp(sec)

Hmo(m)	3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	967	.	.	.	.	.	.	.	.	.	967
1.00 - 1.99	251	39	.	.	.	.	.	.	.	.	290
2.00 - 2.99	.	5	1	.	.	.	.	.	.	.	6
3.00 - 3.99	.	.	.	.	.	.	.	.	.	.	0
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1218	44	1	0	0	0	0	0	0	0	1263

STATION: 66 (281.25 - 303.74) 292.5 DEG

Tp(sec)

Hmo(m)	3.0- 4.9	5.0- 6.9	7.0- 8.9	9.0- 10.9	11.0- 12.9	13.0- 14.9	15.0- 16.9	17.0- 18.9	19.0- 20.9	21.0- LONGER	TOTAL
0.00 - 0.99	930	.	.	.	.	.	.	.	.	.	930
1.00 - 1.99	480	379	.	.	.	.	.	.	.	.	859
2.00 - 2.99	.	94	.	.	.	.	.	.	.	.	94
3.00 - 3.99	.	3	.	.	.	.	.	.	.	.	3
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1410	476	0	0	0	0	0	0	0	0	1886

STATION: 66 (303.75 - 326.24) 315.0 DEG

Tp(sec)

Hmo(m)	3.0- 4.9	5.0- 6.9	7.0- 8.9	9.0- 10.9	11.0- 12.9	13.0- 14.9	15.0- 16.9	17.0- 18.9	19.0- 20.9	21.0- LONGER	TOTAL
0.00 - 0.99	917	.	.	.	.	.	.	.	.	.	917
1.00 - 1.99	796	693	.	.	.	.	.	.	.	.	1489
2.00 - 2.99	.	221	.	.	.	.	.	.	.	.	221
3.00 - 3.99	.	7	3	.	.	.	.	.	.	.	10
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1713	921	3	0	0	0	0	0	0	0	2637

WIS ATLANTIC UPDATE -- WITH HURRICANES 1976 - 1995
LAT: 38.75 N, LONG: 75.00 W, DEPTH: 18 M
OCCURRENCES OF WAVE HEIGHT AND PEAK PERIOD FOR 22.5-DEG DIRECTION BANDS

STATION: 66 (326.25 - 348.74) 337.5 DEG

Hmo (m)	Tp (sec) 3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	893	.	.	.	.	.	.	.	.	.	893
1.00 - 1.99	758	553	.	.	.	.	.	.	.	.	1311
2.00 - 2.99	.	184	.	.	.	.	.	.	.	.	184
3.00 - 3.99	.	11	2	.	.	.	.	.	.	.	13
4.00 - 4.99	.	.	.	.	.	.	.	.	.	.	0
5.00 - 5.99	.	.	.	.	.	.	.	.	.	.	0
6.00 - 6.99	.	.	.	.	.	.	.	.	.	.	0
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	1651	748	2	0	0	0	0	0	0	0	2401

STATION: 66

OCCURRENCES OF WAVE HEIGHT AND PEAK PERIOD FOR ALL DIRECTIONS

Hmo (m)	Tp (sec) 3.0-4.9	5.0-6.9	7.0-8.9	9.0-10.9	11.0-12.9	13.0-14.9	15.0-16.9	17.0-18.9	19.0-20.9	21.0-LONGER	TOTAL
0.00 - 0.99	10661	4111	7894	6527	2694	924	278	87	76	10	33262
1.00 - 1.99	4874	6368	3010	2686	1997	991	321	81	35	1	20364
2.00 - 2.99	.	998	1244	612	513	224	134	17	5	.	3747
3.00 - 3.99	.	27	212	343	146	62	11	10	4	.	815
4.00 - 4.99	.	.	6	78	58	42	9	6	10	.	209
5.00 - 5.99	.	.	.	2	19	13	3	.	4	.	41
6.00 - 6.99	.	.	.	.	1	1	.	.	.	.	2
7.00 - 7.99	.	.	.	.	.	.	.	.	.	.	0
8.00 - 8.99	.	.	.	.	.	.	.	.	.	.	0
9.00 - GREATER	.	.	.	.	.	.	.	.	.	.	0
TOTAL	15535	11504	12366	10248	5428	2257	756	201	134	11	58440

STATION: 66

OCCURRENCES OF WIND SPEED BY MONTH FOR ALL YEARS

WS (m/sec)	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
0.00 - 2.49	121	153	209	313	478	597	765	954	565	449	199	175	4978
2.50 - 4.99	722	682	838	1053	1403	1560	1908	1853	1666	1207	799	713	14404
5.00 - 7.49	1551	1430	1550	1611	1924	1942	1851	1723	1749	1766	1635	1543	20275
7.50 - 9.99	823	839	876	763	676	473	338	323	510	697	841	966	8125
10.00 - 12.49	991	809	885	710	387	205	93	91	257	646	860	940	6874
12.50 - 14.99	350	293	281	222	71	16	2	6	35	136	297	313	2022
15.00 - 17.49	276	191	218	114	21	6	2	3	12	48	125	207	1223
17.50 - 19.99	82	70	51	13	.	1	1	5	2	6	33	64	328
20.00 - GREATER	44	53	52	1	.	.	.	2	4	5	11	39	211
TOTAL	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440

STATION: 66

OCCURRENCES OF WIND DIRECTION BY MONTH FOR ALL YEARS

WD(deg) DIRECTION BAND & CENTER	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
337.50 - 22.49 (0.0)	688	753	814	638	627	516	472	612	732	702	654	755	7963
22.50 - 67.49 (45.0)	432	415	425	352	451	338	247	556	776	710	408	348	5458
67.50 - 112.49 (90.0)	280	285	367	256	349	314	254	451	542	401	376	227	4102
112.50 - 157.49 (135.0)	224	286	421	531	473	343	366	368	393	380	316	256	4357
157.50 - 202.49 (180.0)	495	473	712	830	932	1007	859	788	589	644	585	447	8361
202.50 - 247.49 (225.0)	593	592	621	657	891	1087	1317	1042	710	675	678	809	9672
247.50 - 292.49 (270.0)	763	577	519	614	542	601	794	633	536	600	746	785	7710
292.50 - 337.49 (315.0)	1485	1139	1081	922	695	594	651	510	522	848	1037	1333	10817
TOTAL	4960	4520	4960	4800	4960	4800	4960	4960	4800	4960	4800	4960	58440

WIS ATLANTIC UPDATE -- WITH HURRICANES 1976 - 1995
LAT: 38.75 N, LONG: 75.00 W, DEPTH: 18 M
OCCURRENCES OF WAVE HEIGHT AND PEAK PERIOD FOR 22.5-DEG DIRECTION BANDS

STATION: 66

SUMMARY OF MEAN Hmo(m) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1976	1.28	1.26	1.24	1.04	1.07	0.94	0.56	0.96	0.78	1.30	1.05	1.12	1.05
1977	1.25	1.14	1.30	1.07	0.73	0.72	0.57	0.60	0.71	1.28	1.65	1.32	1.03
1978	1.63	1.00	1.41	1.21	1.23	0.82	0.61	0.63	1.01	0.97	1.20	1.10	1.07
1979	1.66	1.49	1.52	1.26	1.06	0.71	0.63	0.70	1.27	0.89	1.21	1.29	1.14
1980	1.38	1.10	1.63	1.31	0.78	0.73	0.64	0.79	0.92	1.16	1.20	1.54	1.10
1981	1.12	1.90	1.33	1.26	1.28	0.63	0.84	0.68	1.21	1.11	1.26	1.16	1.14
1982	1.20	1.19	0.95	1.08	0.77	0.81	0.50	0.45	0.81	1.00	1.09	0.96	0.90
1983	1.32	1.35	1.53	1.24	1.07	0.73	0.57	0.54	0.92	1.41	1.20	1.56	1.12
1984	1.22	1.50	1.56	1.14	0.97	0.78	0.70	0.53	0.90	1.05	1.23	1.09	1.05
1985	1.23	1.21	1.17	1.07	0.88	0.59	0.64	0.62	0.95	0.92	1.56	1.04	0.99
1986	1.35	1.11	1.30	1.17	0.94	0.78	0.47	0.72	0.95	0.96	1.10	1.37	1.02
1987	1.38	1.08	1.30	1.45	0.95	0.69	0.52	0.68	0.77	0.97	1.15	0.95	0.99
1988	1.04	1.23	1.05	1.11	0.92	0.82	0.71	0.71	0.84	0.98	1.13	0.94	0.95
1989	1.03	1.10	1.51	1.04	1.05	0.74	0.66	0.93	1.65	0.98	1.29	1.13	1.09
1990	1.03	1.21	1.16	1.20	1.06	0.92	0.86	0.89	1.09	1.28	0.96	1.33	1.08
1991	1.14	1.01	1.28	1.08	0.76	0.72	0.68	0.85	1.05	1.08	1.11	1.09	0.99
1992	1.31	1.05	1.24	0.84	0.93	0.59	0.52	0.57	0.90	0.77	0.92	1.25	0.91
1993	1.24	1.26	1.27	1.39	0.84	0.67	0.57	0.67	0.84	0.96	1.24	1.18	1.01
1994	1.01	0.85	1.06	0.78	0.77	0.73	0.68	0.60	0.67	0.61	1.14	0.95	0.82
1995	1.14	0.84	0.80	0.76	0.68	0.76	0.55	1.00	0.94	0.97	1.08	1.01	0.88
MEAN	1.25	1.20	1.28	1.12	0.94	0.74	0.62	0.71	0.96	1.03	1.19	1.17	

STATION: 66

MAX Hmo(m)*10 WITH ASSOCIATED Tp(sec) AND Dp(deg/10) BY MONTH AND YEAR

YEAR	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MAX
1976	35 819	38 918	39 9 9	27 8 9	29 816	24 8 6	16 622	421113	20 712	301013	231011	26 631	421113
1977	41 914	441014	421015	36 917	19 533	21 536	15 620	16 617	16 810	30 910	39 911	4011 8	441014
1978	491115	29 634	451112	501412	431016	27 818	23 712	13 618	221214	27 8 7	28 713	35 813	501412
1979	561113	491310	571214	391014	25 717	17 616	16 616	15 7 8	36 916	21 718	371016	31 817	571214
1980	471017	31 9 7	4514 9	41 917	28 816	16 519	17 612	241714	211612	531312	31 813	551312	551312
1981	271011	591314	38 733	29 8 9	34 916	16 522	37 913	25 8 9	311713	22 718	3912 9	29 819	591314
1982	34 919	5010 8	21 713	34 918	17 522	22 713	10 421	15 813	201210	451312	251512	27 816	5010 8
1983	3410 8	4410 9	511211	39 915	30 818	20 619	15 533	14 6 5	481113	401112	35 815	481211	511211
1984	341111	411013	461012	451015	26 818	22 532	17 520	12 4 1	28 812	451412	31 816	25 715	461012
1985	30 631	461013	32 914	36 819	18 532	14 530	20 719	16 6 6	661412	351010	551212	30 630	661412
1986	281410	39 9 9	481018	29 818	22 717	19 914	11 519	41 815	251612	32 8 8	34 818	591412	591412
1987	41 9 9	3510 9	441114	35 812	211010	18 718	12 433	16 519	17 615	24 718	28 810	26 718	441114
1988	261510	33 818	27 817	3410 9	20 7 8	20 8 8	21 619	23 717	25 719	29 712	33 917	21 520	3410 9
1989	231213	4412 7	3410 8	30 918	39 916	18 618	24 715	291112	521913	31 810	33 917	32 9 8	521913
1990	27 819	31 818	33 910	32 918	30 919	25 818	19 718	221613	211512	3410 7	24 714	381015	381015
1991	37 912	24 719	29 919	24 9 7	21 7 8	20 7 6	17 617	451313	471813	261012	3310 7	23 615	471813
1992	441412	21 716	281112	26 717	25 8 8	15 812	13 519	18 618	31 9 8	24 8 8	30 718	441012	441012
1993	37 918	35 8 9	481012	401016	20 618	17 618	16 618	211314	451414	331110	521114	30 9 6	521114
1994	28 717	23 912	401010	22 718	281012	27 718	19 718	15 619	35 812	221011	421311	341212	421311
1995	37 917	21 530	25 712	25 718	22 718	17 8 9	10 519	341313	221314	37 916	451015	20 7 8	451015
MAX	561113	591314	571214	501412	431016	27 718	37 913	451313	661412	531312	551212	591412	

MAX Hmo(m): 6.6   MAX Tp(sec): 14.   MAX Dp(deg): 122.   DATE(gmt): 1985092715

MAX WIND SPEED(m/sec): 34.   MAX WIND DIRECTION(deg): 25.   DATE(gmt): 1985092712

MEAN Hmo(m): 1.0   MEAN Tp(sec): 7.

STANDARD DEVIATION Hmo(m): 0.6   STANDARD DEVIATION Tp(sec): 3.1

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