SEDIMENTATION STUDY OF SUMMIT NORTH MARINA

by

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ABSTRACT

Possible sedimentation problems at the site of the Summit North Marina are investigated in this study. The marina will be developed at the Lums Pond Lagoon which is connected to the Chesapeake and Delaware (C & D) Canal. First, available data on the environmental factors affecting the sedimentation processes in the marina area is collected and synthesized. The environmental factors considered in this study are geology, precipitation, wind, tides, hydrography and navigation in the C & D Canal. Additional data on the site-specific characteristics of the marina area is obtained by conducting three field surveys. The additional data includes the measurements of surface currents, sediment size distributions and beach profile changes as well as photographs showing the sedimentation processes in the marina area. Second, simple analyses are performed to determine the sediment transport patterns at the marina area. The sediment deposited at the marina entrance is found to originate from the eroding bluff along the northern shore of the Lums Pond Lagoon. The bluff erosion due to the combined effects of precipitation, slope instability, groundwater seepage and wave action at the toe of the bluff introduces sediment into the beach along the northern shore of the Lums Pond Lagoon. The introduced sediment is then transported eastward along the northern shore and deposited at the marina entrance mainly by the combined action of vessel-generated waves and wind-driven currents. The volumetric rate of the sediment deposition in the

vicinity of the marina entrance is estimated to be 5×10^5 ft^3/yr prior to the dredging conducted in November, 1984 and January, 1985. The sedimentation rate in the dredged navigation channel connecting the marina site to the C & D Canal is predicted to vary along the channel in the range 1 - 5 ft/yr. Third, different sedimentation mitigation measures are examined to identify the most effective and feasible means for maintaining the marina at Lums Pond Lagoon. To maintain the navigation channel connecting the marina site to the C & D Canal, annual maintenance dredging will likely be required if no other measures are taken. One way to reduce the dredging requirement is to reduce the supply of sediment resulting from the bluff erosion. Sand-filled tubes could be placed at the toe of the eroding bluff to protect the toe from the attack of vessel-generated waves. The sand-filled tubes are regarded to be temporary and may remain effective only for several years although they are used as low-cost shore protection devices. In order to provide more permanent protection along the shore of the marina area, the riprap revetment along the C & D Canal may be extended at least along the eroding bluff exposed to the direct wave attack. The riprap revetment may eventually need to be extended all the way around the marina area if long-term erosion of the unprotected shore inside the marina entrance is found to be unacceptable. Alternatively, a rubble-mound jetty may be constructed normal to the northern shore to trap the sediment transported toward the marina entrance. However, the updrift fillet formed on the western side of the jetty is predicted to be filled to its capacity within 3-4 years and become ineffective in trapping the sediment and mitigating the sedimentation in the dredged navigation channel.

ACKNOWLEDGMENTS

The sedimentation study of the Summit North Marina reported herein was sponsored by the Office of State Park Planning and Development, Division of Parks and Recreation, State of Delaware. The authors would like to thank Messrs. Wilkie, Adams and Corazza for initiating and supporting this project and providing the maps and information related to the marina development. Mr. Williams of the Division of Soil and Water Conservation supplied the data on the estimated volume of sediment dredged during the period of November, 1984 to January, 1985.

The authors initially contacted Mr. Tomilin of U. S. Army Corps of Engineers at Chesapeake City, Maryland, who provided the information regarding the maintenance of the C & D Canal and directed us to the following personnel at the Philadelphia District, U. S. Army Corps of Engineers. Mr. Rose gave the 1965 hydrographic map showing the marina area during the enlargement work of the C & D Canal. Mr. Uibel supplied the 1974 topographic map of the marina area and the test pit data obtained

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in 1968. Messrs. Lipski and Gebert provided the reports and documents concerning the tides and flows in the C & D Canal. Ms. Armstrong sent the navigation data for the C & D Canal. The authors would like to acknowledge their full cooperation for this study. In addition, Mr. Gredell of Gredell & Paul Inc., Newark, Delaware provided the test boring data obtained in 1985 for the marina construction.

The authors would also like to acknowledge that Professor Dalrymple and Professor Trowbridge participated in the initial phase of this project. The students of the Ocean Engineering Program who participated in the first field survey conducted on July 3, 1985 are Messrs. Cannon and Otta and Mses. Contos and Frankenstein.

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1. INTRODUCTION

The Office of State Park Planning and Development, State of Delaware, requested the Ocean Engineering Group, Department of Civil Engineering to investigate possible sedimentation problems at the site of the Summit North Marina which will be developed at the Lums Pond Lagoon connected to the Chesapeake and Delaware (C & D) Canal. The Ocean Engineering Group visited the marina site on March 13, 1985 and submitted the proposal entitled, "Sedimentation Study of Summit North Marina." Fig. 1 depicts the approximate location of the Summit North Marina situated along the C & D Canal which is a sea-level canal between the Delaware River and the Chesapeake Bay. Before describing the objectives and scope of the proposed study, a brief history of the C & D Canal and the Lums Pond Lagoon is given so as to provide the background information.

1.1 Brief History of Lums Pond Lagoon

The original sea-level canal completed in 1927 connecting the Delaware River and the Chesapeake Bay was 90 ft wide and 12 ft deep. In 1935 Congress authorized enlargement of the canal to 250 ft wide and 27 ft deep. This enlargement work was completed in 1954. A further enlargement of the canal to 450 ft wide and 35 ft deep was authorized in 1954 and started in 1956. The enlargement work started in 1956 also included modification at all bends to obtain a minimum radius of curvature of 7,000 ft and the rebuilding of all bridges to obtain a minimum vertical clearance of 135 ft, along with stabilization and revetment of the banks (3). The present 450 ft wide and 35 ft deep canal was

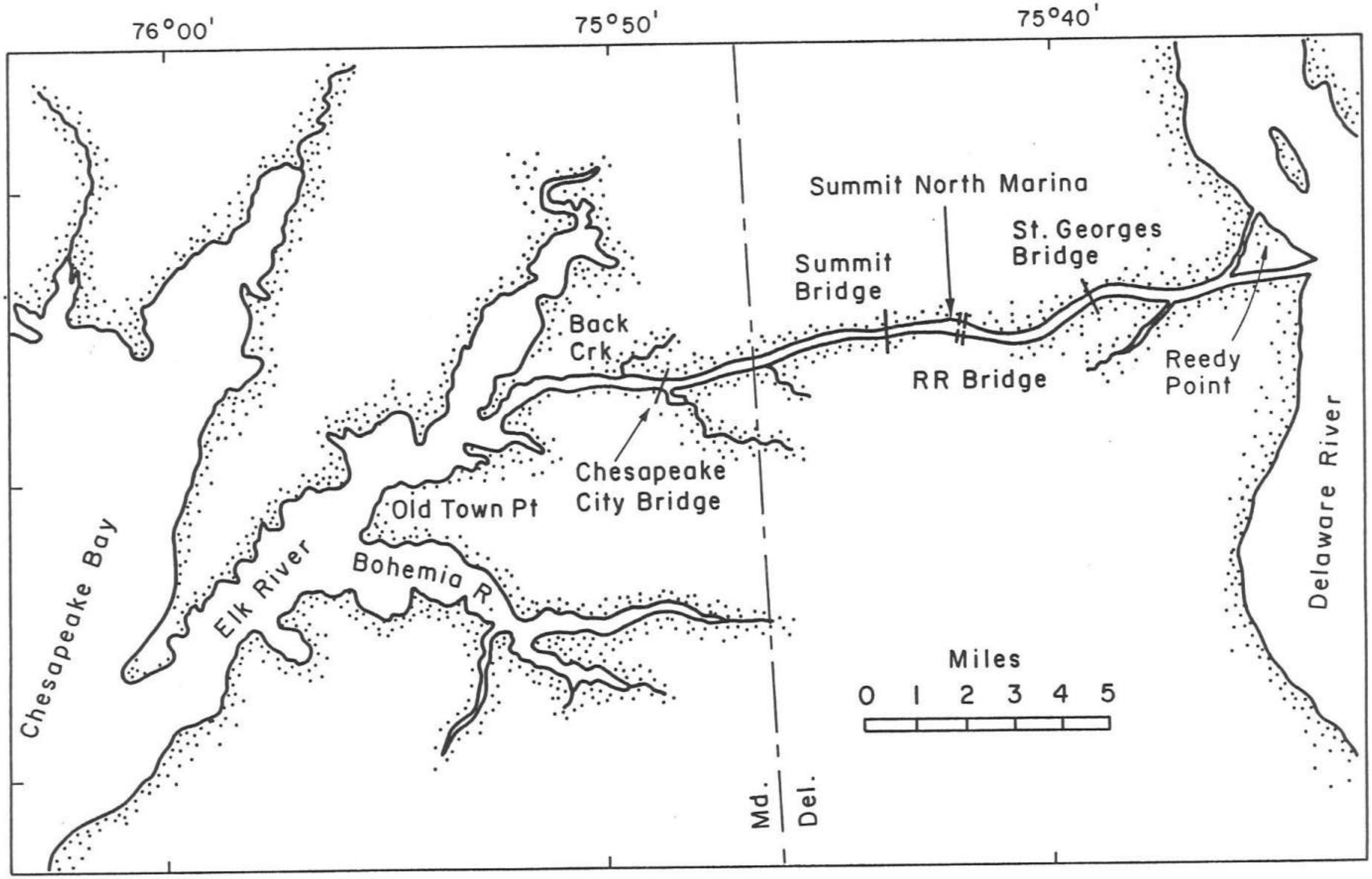

Fig. 1. C & D Canal and Location of Summit North Marina

completed in 1975. Extensive hydrographic and ecological studies were also performed (3, 4, 10, 16, 18). However, these studies do not deal with the Lums Pond Lagoon specifically.

The 1965 hydrographic map and the 1974 topographic map provided by the Philadalphia District, U. S. Army Corps of Engineers indicated that the Lums Pond Lagoon was a segment of the old 250 ft wide and 27 ft deep channel and the new 450 ft wide and 35 ft deep channel was dredged south of the old channel so as to increase the radius of curvature of the new channel. The 1965 map shows the new dredged channel together with the old channel. The 1974 map indicates the completion of the enlargement and realignment work in the vicinity of the new railroad bridge built over the new channel. As a result, the Lums Pond Lagoon was created between 1965 and 1974 by closing the segment of the old channel on its eastern end along the old railroad bridge and maintaining the connection of the segment with the new channel on its western end. The 1980 hydrographic map provided by the Office of State Park Planning and Development indicated that the Lums Pond Lagoon was filled with sediment almost completely. Two dredging operations were conducted in November, 1984 and January, 1985 to increase the water depth of the marina site and connect the marina site with the C & D Canal through a dredged navigation channel. The delay between the two dredging operations was caused by bad weather and freezing. The hydrographic survey of the dredged area was made in March, 1985. The 1985 hydrographic map was provided by the Division of Soil and Water Conservation. Comparing the 1980 and 1985 maps, Williams (personal communication, 1985) estimated that approximately 48,000 yd^3 of sediment was dredged from the marina site.

1.2 Objectives and Scope of the Study

On the basis of the findings of the site visit made on March 13, 1985 as well as the maps and information provided by the Office of State Park Planning and Development, the tasks of the sedimentation study of the Summit North Marina have been specified as follows:

(1) Collection and synthesis of available data on the hydraulic and hydrographic characteristics of the Lums Pond Lagoon.

(2) Identification of the sources of the sediment deposited at the marina site.

(3) Determination of the sediment transport patterns at the marina site.

(4) Estimation of the sedimentation rate in the dredged marina area.

(5) Recommendation and evaluation of sedimentation mitigation measures.

The present study is limited to the hydraulic aspects of the marina construction. The constructional, environmental and legal aspects of the marina construction are beyond the scope of this study.

The contents of this report essentially follow the specified tasks in sequence. Section 2 summarizes the available data on the environmental factors affecting the sedimentation of the marina area. Section 3 discusses the findings of three field surveys conducted on July 3, July 30 and August 28, 1985. Section 4 analyzes the bluff erosion and subsequent sediment transport and deposition in the marina area on the basis of the available data and the field survey findings. Section 5 examines four different sedimentation mitigation measures, that is, maintenance dredging, low-cost bluff protection and riprap revetment extention and construction of a jetty. Section 6 gives the conclusions and recommendations of this study.

2. ENVIRONMENTAL FACTORS

The environmental factors affecting the sedimentation of the marina area include geology, precipitation, wind, tides and navigation in the C & D Canal. The available hydrographic data for the marina area is summarized in Section 2.6.

2.1 Geology

The data on the geology of the C & D Canal area is available from the Delaware Geological Survey. Fig. 2 shows the Cretaceous Formation present in the vicinity of the Lums Pond Lagoon (17). The Potomac Formation was the deltaic deposition of clays and sands transported by streams. The Magothy Formation consists of well-sorted clean quartz sand with beds of gray and black clayey silt which were deposited in a shoreline environment. The Merchantville, Marshalltown and Mount Laurel sediments were probably deposited in fairly shallow, open marine, perhaps embayed areas. The Merchantville Formation is comprised of sandy silt and silty fine sand. The Marshalltown Formation consists of very silty fine sand. The Mount Laurel Formation is comprised of fine to medium quartz sand with some silt. On the other hand, the Englishtown Formation

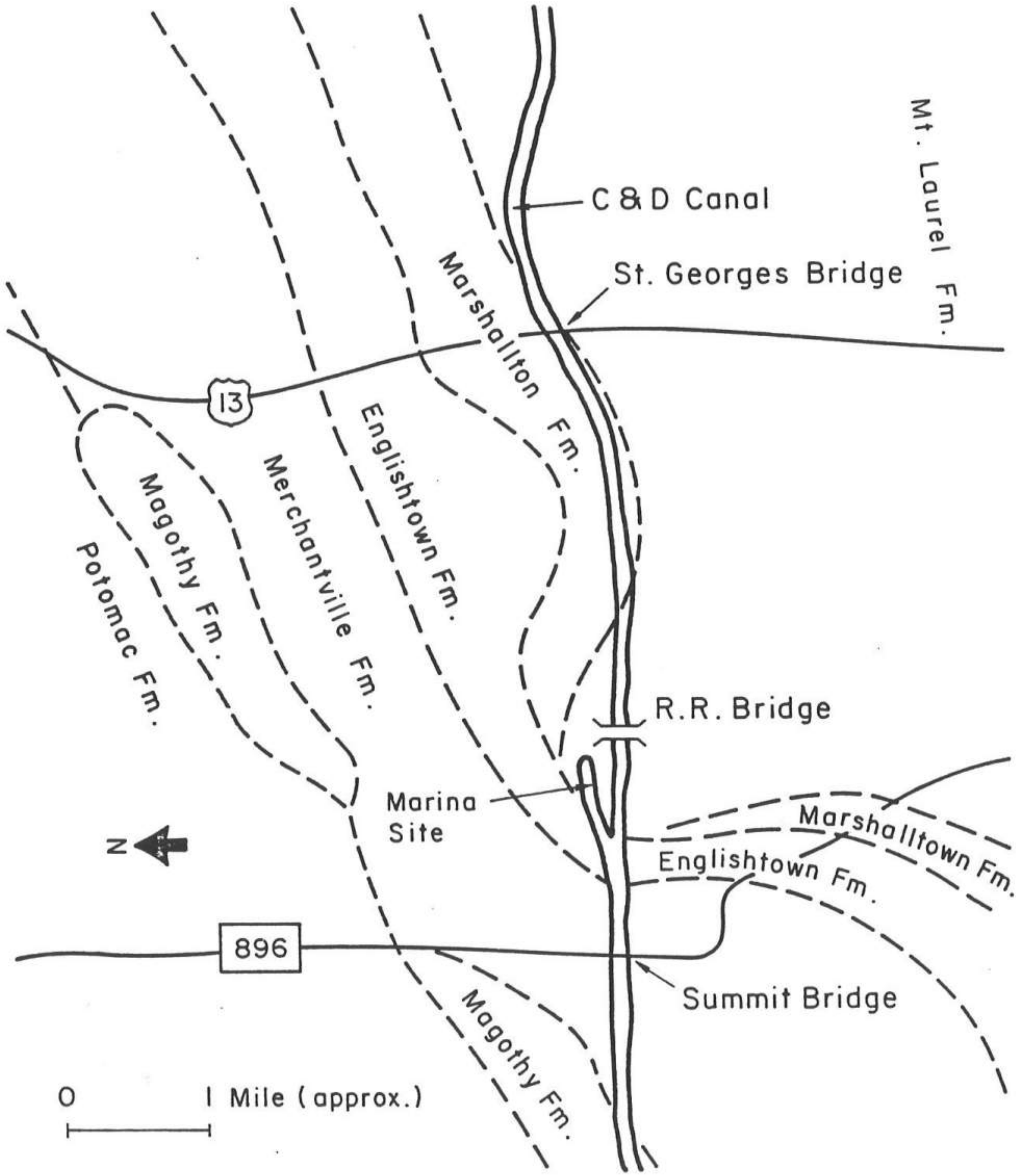

Fig. 2. Geology in the Vicinity of Lums Pond Lagoon

represents a shoreline environment in which sea level was dropping. The Englishtown sediment is fine sand with thin interbedded layers of silty sand. As a summary, the sediments in the vicinity of the marina area are essentially sands and silts.

2.2 Precipitation

Table 1 summarizes the precipitation data at Wilmington, Delaware, compiled by the National Climatic Data Center (13). Rainfall distribution throughout the year is fairly uniform in Delaware. However, the greatest amounts normally come during the summer months in the form of thunderstorms. The rainfall impact and surface runoff may cause soil detachment and transport from the bluff surrounding the marina area. The effects of the rainfall on the bluff erosion are reduced when the vegetative cover is the densest in summer. The site visit on March 13, 1985 indicated exposure of the bare bluff soil to precipitation. On the other hand, the bluff soil was partially protected by the dense vegetative cover during the field surveys conducted in July and August, 1985. Consequently, precipitation and vegetation influence the bluff erosion although it is extremely difficult to analyze their effects.

2.3 Wind

Wind generates waves and currents. Wind-generated waves and currents cause sediment transport. Table 2 summarizes the wind data at Wilmington, Delaware, compiled by the National Climatic Data Center (13). The mean monthly wind speed in winter is slightly greater than 10 mph, while the mean monthly wind speed in summer is approximately

TABLE 1: Precipitation Data at Wilmington, Delaware

Month	Precipitation (inches) (Water Equivalent)	
	Normal	Maximum in 24 Hrs.
JAN	3.11	2.12
FEB	2.99	2.29
MAR	3.87	3.11
APR	3.39	2.56
MAY	3.23	2.35
JUN	3.51	4.35
JUL	3.90	6.24
AUG	4.03	4.11
SEP	3.59	5.62
OCT	2.89	3.88
NOV	3.33	3.83
DEC	3.54	2.22

TABLE 2: Wind Data at Wilmington, Delaware

Month	Mean Speed (mph)	Prevailing Direction through 1963	Fastest Obs. 1 Min.	
			Speed (mph)	Direction (degrees)
JAN	10.0	WNW	46	290
FEB	10.5	NW	46	290
MAR	11.3	WNW	43	69
APR	10.6	WNW	45	290
MAY	9.1	S	46	300
JUN	8.4	S	40	230
JUL	7.8	NW	48	270
AUG	7.5	S	46	350
SEP	7.9	S	40	70
OCT	8.3	NW	58	200
NOV	9.2	NW	46	160
DEC	9.4	WNW	44	290

NOTE: Wind Direction in Tens of Degrees (True)

16 Points of Compass	Degrees	16 Points of Compass	Degrees
N	0, 10, 350	S	170, 180, 190
NNE	20, 30	SSW	200, 210
NE	40, 50	SW	220, 230
ENE	60, 70	WSW	240, 250
E	80, 90, 100	W	260, 270, 280
ESE	110, 120	WNW	290, 300
SE	130, 140	NW	310, 320
SSE	150, 160	NNW	330, 340

8 mph. The prevailing wind direction is from the northwest (NW) in winter and from the south (S) in summer. The fastest observed wind speed of 1 minute duration for each month is in the range 40-58 mph and may be regarded as the extreme upperlimit of the wind speed expected in the marina area although the 1 minute duration is too short to use for predicting wind-generated waves and currents. It should be noted that the relationship of wind direction in tens of degrees and in terms of 16 points of the compass is explained below Table 2.

2.4 Tides

The flow in the C & D Canal is dominated by tidal effects although it is often modified by meteorological effects. The maximum flow in the C & D Canal is in the range 80,000 - 100,000 ft^3/sec and the maximum surface velocity is typically 3.7 ft/sec (4). Table 3 summarizes the water level data for the C & D Canal on the basis of the documents provided by the Philadelphia District, U. S. Army Corps of Engineers and the tide tables compiled by the National Ocean Survey (14). The locations of the places listed in Table 3 are shown in Fig. 1. The tide tables compiled by the National Ocean Survey are based on the chart datum of soundings which is the same as the mean low water level. The mean low water level between Reedy Point and Chesapeake City is above the C & D Canal datum as shown in the last column of Table 3. It should be noted that the tide range in the C & D Canal decreases westward from Reedy Point to Chesapeake City. The mean tide level above the mean low water is equal to one half of the mean tide range.

TABEL 3: Summary of Tide Data for C & D Canal

Place	Tide Ranges*		Mean Tide Level* above Mean Low Water (ft)	Mean Low Water above C&D Canal Datum (ft)
	Mean (ft)	Spring (ft)		
Reedy Point	5.5	6.0	2.7	0.5
Biddle Point	5.1	5.5	2.5	0.9
Summit Bridge	3.5	3.9	1.7	1.7
Chesapeake City	2.7	3.0	1.4	2.3

*Heights are referred to mean low watch which is the chart datum of soundings.

TABLE 4: Summary of Tide Data for Lums Pond Lagoon

Water Level	Height above C&D Canal Datum (ft)	Height above Mean Sea Level (ft)
Mean High Water (MHW)	5.1	2.1
Mean Tide Level (MTL)	3.25 = (5.1 + 1.4)/2	0.26
Mean Sea Level (NGVD)*	2.99	0.0
Mean Low Water (MLW)	1.4	-1.6
C&D Canal Datum*	0.0	-2.99

*C&D Canal Datum is 2.99 ft below the Mean Sea Level Datum 1929 General Adjustment of the U.S. Coast and Geodetic Datum (NGVD).

Table 4 summarizes the tide data for the Lums Pond Lagoon. The heights of the mean high water (MHW), the mean tide level (MTL) and the mean low water (MLW) are given relative to the C & D Canal Datum and the Mean Sea Level Datum (NGVD). The C & D Canal Datum is 2.99 ft below the Mean Sea Level Datum. Table 4 shows that the mean tide range, that is, the difference between the mean high and low waters, is 3.7 ft. Table 3 indicates that the spring tide range in the vicinity of the Lums Pond Lagoon is 0.4 ft greater than the mean tide range. Consequently, the spring tide range in the marina area is 4.1 ft.

2.5 Navigation in C & D Canal

The vessels navigating in the C & D Canal generate waves and affect the erosion and sedimentation in the marina area. The vessel-generated wave characteristics in the C & D Canal depend on the vessel configuration and speed (19). These waves propagate into the marina area. The resulting wave field in the marina area depends on the direction (eastbound or westbound) of the vessel navigation since the western end of the Lums Pond Lagoon is connected to the C & D Canal at an angle. On the basis of the information provided by the Philadelphia District, U. S. Army Corps of Engineers, Table 5 summarizes the number of vessels for given draft which navigated eastbound or westbound in the C & D Canal in 1982. On the average, the number of eastbound vessels is approximately the same as the number of westbound vessels. Approximately 31 vessels per day navigated in the Canal in 1982. An analytical determination of the wave characteristics in the marina area resulting from these vessels is extremely difficult and beyond the scope of this study. Alternatively, the vessel-generated waves were visually observed during the field surveys for this study.

TABLE 5: Trips and Drafts of Vessels in C & D Canal in 1982

Draft (ft)	Eastbound	Westbound	Draft (ft)	Eastbound	Westbound
36	2	4	23	119	102
35	4	2	22	79	103
33	2	3	21	64	87
32	1	3	20	65	91
31	83	53	19	59	87
30	94	76	18	74	120
29	87	81	17	131	109
28	108	148	16	214	189
27	155	176	15	345	296
26	141	113	14	253	289
25	118	145	13	296	318
24	169	139	12 and less	2,977	2,871
			TOTAL	5,640	5,605
			Daily Average	15.5	15.4

2.6 Hydrographic Data

In order to estimate the bathymetry changes in the marina area, the following maps are used in this study:

1. 1965 Hydrographic Map (Inland Waterway, C & D Canal, Station 45+000 to Station 49+400, November, 1965).
2. 1974 Topographic Map (Inland Waterway, C & D Canal, Maintenance Dredging, Station 24+585 to Station 244+400, Penn Central Cutoff Disposal Area, April, 1974).
3. 1980 Hydrographic Map (Hydrographic Survey of Lums Pond Lagoon, March 27, 1980).
4. 1985 Hydrographic Map (Post Dredge Contours, Lums Pond Lagoon, August 8, 1985).

The 1965 and 1974 maps were provided by the Philadelphia District, U. S. Army Corps of Engineers. The vertical elevations of these maps are based on the C & D Canal Datum. On the other hand, the 1980 and 1985 maps were supplied by the Office of State Park Planning and Development, State of Delaware. The 1980 and 1985 maps use the Mean Sea Level Datum (NGVD). The Mean Sea Level Datum is used in this study. Since the C & D Canal Datum is 2.99 ft below the Mean Sea Level, the vertical elevations of the 1965 and 1974 maps are adjusted by 2.99 ft so as to make these maps compatible with the 1980 and 1985 maps. The maps used in this study are not included in this report since these maps are too large to be folded up in the report.

3. FIELD SURVEYS

In order to obtain additional data on the sedimentation processes in the marina area, three field surveys were conducted using simple equipment available at the Department of Civil Engineering. The field surveys were limited to the area along the northern shore of the marina area which could be surveyed without a boat. During the first field survey conducted on July 3, 1985, five stations were established and beach profiles were measured using a transit and poles with scales. In addition, surface current velocities were measured and sediment samples were collected. The second field survey was performed on July 30, 1985 to observe and photograph the sedimentation processes in the marina area. A tape was used to measure approximate distances between objects quickly. The third field survey was conducted on August 28, 1985, and beach profiles were measured along the same straight lines as those established in the first field survey. The findings of these field surveys are summarized in the following sections.

3.1 Description and Photos of Lums Pond Lagoon

Figs. 3 and 4 show the locations of the photographs shot in the directions indicated by arrows during the second low tide period of July 30, 1985. The photographs without arrows show the close views of the photograph locations. The approximate high and low water shorelines indicated in Figs. 3 and 4 were determined in the first and second field surveys. The dotted line indicating the low water shoreline in Figs. 3 and 4 is not drawn along the shore where the difference between the low and high water shoreline is not large because of dredging or riprap revetment. The

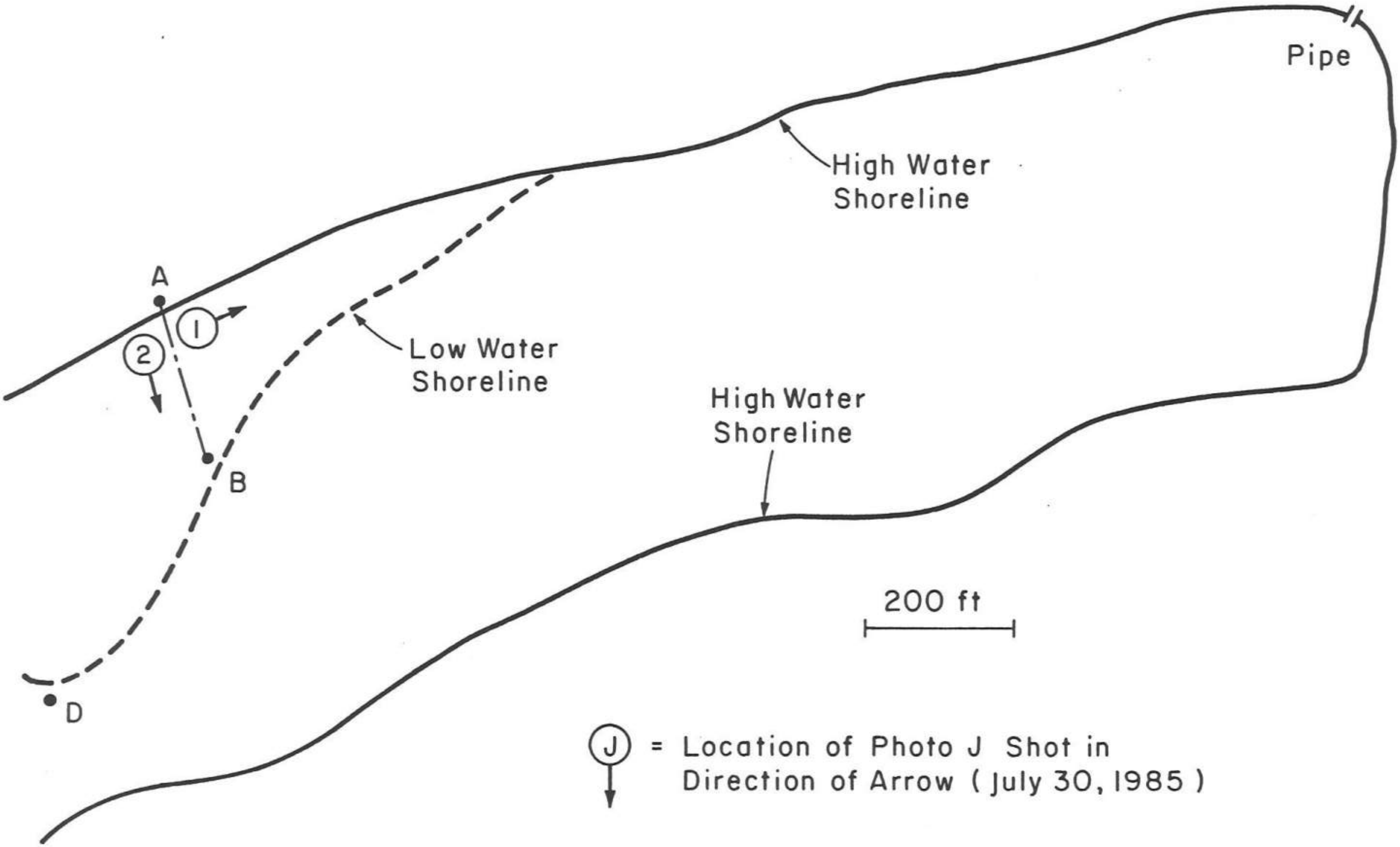

Fig. 3. Locations of Photo 1 - 2
Shot on July 30, 1985

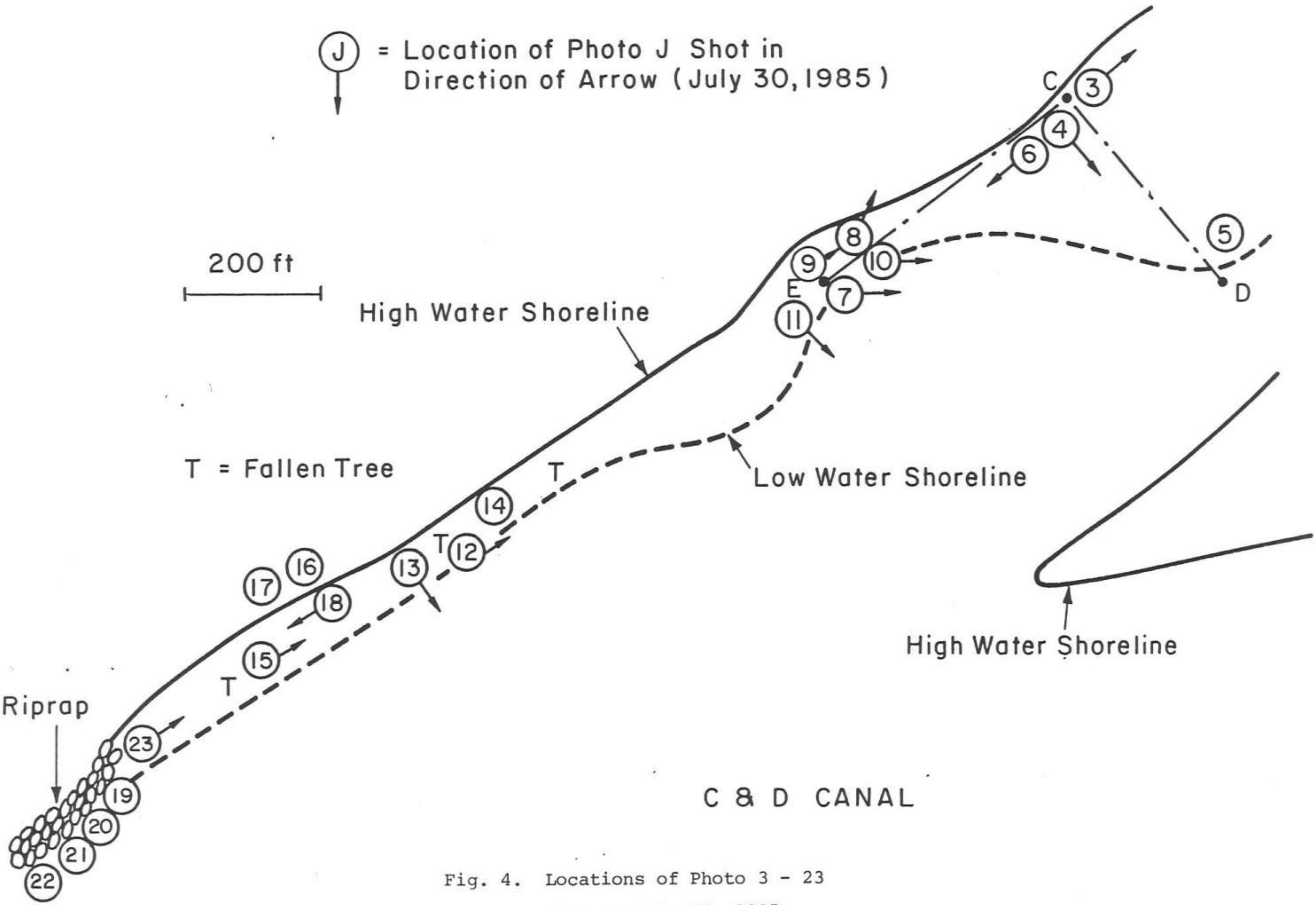

Fig. 4. Locations of Photo 3 - 23
Shot on July 30, 1985

locations of the Stations A, B, C, D and E relative to the pipe location shown in Figs. 3 and 4 were established in the first field survey. Stations A and C were marked with stakes. The beach profiles along the straight lines A-B, C-D and C-E were measured in the first and third field surveys. Three fallen trees were found between the marina site and the riprap revetment along the northern shore of the C & D Canal as indicated by the letter "T" in Fig. 4. The photographs taken at the locations specified in Figs. 3 and 4 are shown in Photos 1-23 with comments explaining each of the photographs. Photo 24 shows the transit used for measuring the beach profiles along the straight lines A-B, C-D and C-E indicated in Figs. 3 and 4.

3.2 Surface Current Measurements

Surface current velocities were measured on July 3, 1985, by tracking ten colored floats thrown on the water surface in the marina area. The float was a square wooden block whose length and thickness were 7 in. and 0.5 in., respectively. The distance traveled by each fload per minute was estimated by following the float and counting the number of steps per minute of an assigned tracker whose step length was measured in advance. The float was almost submerged and appeared to move with the surface water. As a result, the estimated float velocity may be assumed to be approximately the same as the surface current velocity.

Fig. 5 shows the observed path of each of the ten floats tracked on foot. All the floats traveled eastward toward the eastern end of the marina area. The only exception was that Floats 3 and 4 reversed their directions and then surged back eastward under the influence of waves of

Photo 1. From Station A toward the Pipe at the Eastern End of Marina Area. Note the growth of green alga on the beach indicating little wave action on the beach inside the shoal located at the entrance of the marina site.

Photo 2. From Station A toward Station B. Note that the area covered with water approximately corresponds to the area dredged to connect the marina site to C & D Canal.

Photo 3. From Station C toward Station A. Note that the area covered with grass tends to promote the sediment deposition and be located at a higher elevation than the area without grass.

Photo 4. From Station C toward Station D. Note that the grass promotes sediment deposition but creates a drainage channel between the two areas covered with the grass.

Photo 5. Cobbles near Station D on the Shoal. Note that these cobbles are likely to be originated from the eroding bluff surrounding the marina area. The cobbles may be transported by the action of ship-generated large waves. The role of ice in transporting the cobbles is not investigated.

Photo 6. From Station D toward Station E. The length of the bluff which appears to be eroding rapidly between Stations D and E is approximately 110 ft.

Photo 7. Obliquely Incident Waves Near Station E. These obliquely incident waves are observed to cause longshore sediment transport along the shoreline from Station E toward Station D.

Photo 8. Bluff Erosion near Station E. Note that the trees are falling into water because of the bluff erosion. The tree leaves tend to protect the eroding bluff against precipitation in summer by intercepting rainfall and reducing the impact of raindrops.

Photo 9. From Station E toward Station C. The eroding bluff in Photo 8 is located on the left hand side of Photo 9. The beach in front of this eroding bluff is strewn with cobbles and devoid of grass.

Photo 10. From Station E toward Station D. Note the extent of the shoal at the entrance of the marina site. The southern shore of the marina area is covered with trees and appears to be more stable than the northern shore.

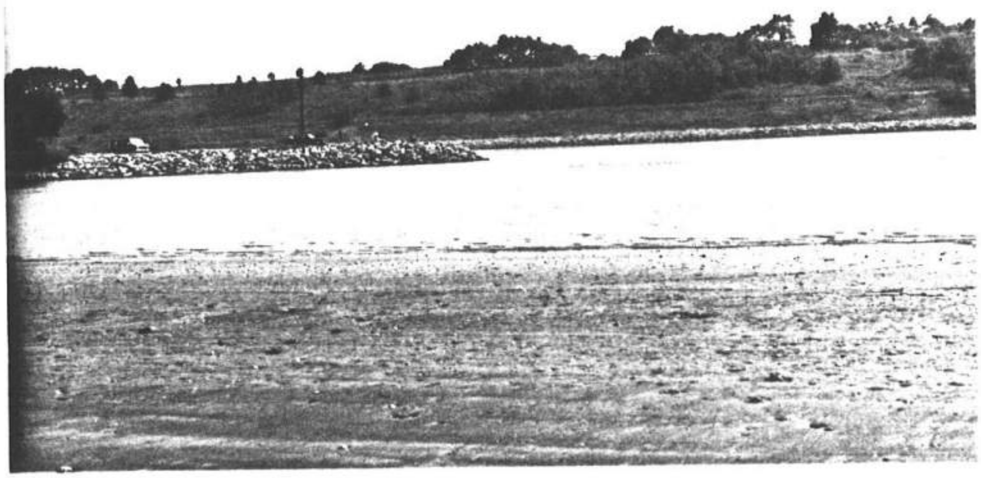

Photo 11. From Station E toward Riprap Revetment on the Opposite Shore. The cars in this picture were parked on the access road built for the placement of the riprap stones.

Photo 12. From Second Fallen Tree toward First Fallen Tree. The shore-normal distance from the eroding grass roots, shown in Photo 14, to the first fallen tree in this picture is approximately 40 ft.

Photo 13. Waves Generated by Two Boats Passing in C & D Canal. The wave height was approximately one foot and the waves lasted for a few minutes.

Photo 14. Eroding Grass Roots due to Wave Action. The grass roots are exposed to wave and current action during high tides and eroding although the roots tend to reduce the erosion rate.

Photo 15. From Third Fallen Tree toward First and Second Fallen Trees. The first fallen tree is shown in Photo 12. The second fallen tree lying on the beach in the distance still has green leaves. The leaning tree on the upper left hand side of this picture is located in the vicinity of the eroding bluff shown in Photos 16 and 17.

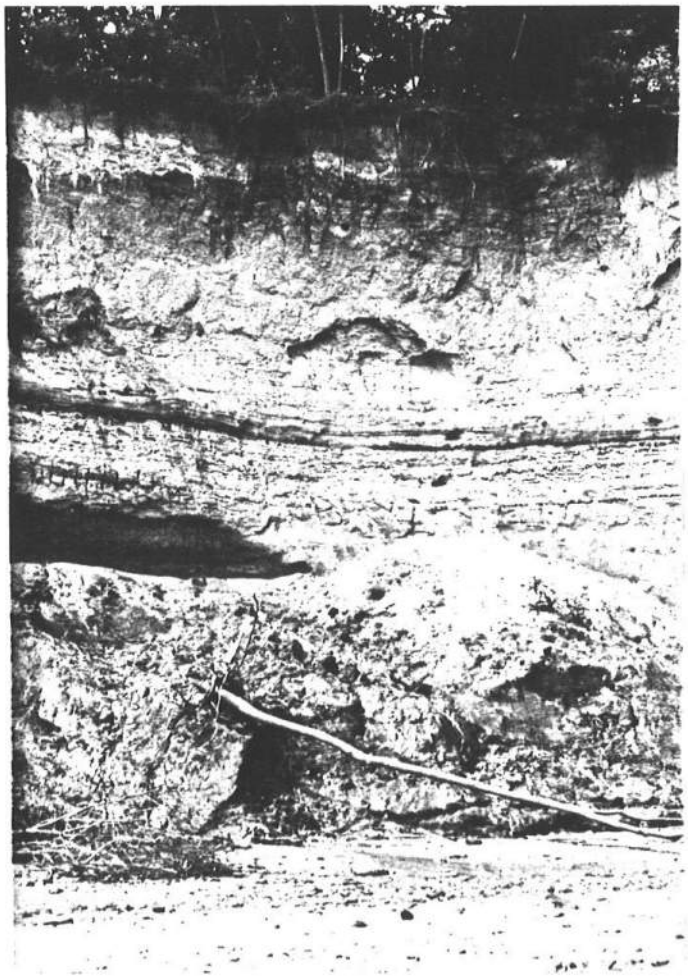

Photo 16. Eroding Bluff without any Vegetation. The height of this eroding bluff is approximately 60 ft. The hanging tree roots and fallen branches indicate active erosion.

Photo 17. Seepage of Groundwater from Eroding Bluff. The groundwater seepage appears to have created the large hole shown in this picture.

Photo 18. From Eroding Bluff toward Third Fallen Tree. The tree trunk standing vertically next to the root of the fallen tree is located approximately 25 ft from the vegetation growing at the toe of the bluff.

Photo 19. Gwen Charles of Height 5'8" in Front of Riprap Revetment. The height of the riprap revetment is approximately 9 ft. The revetment slope is approximately 1 on 1.5 as shown in Fig. 39. The horizontal line between gray and brown stones indicates the elevation of the high water level.

Photo 20. Exposed Underlayer Quarry Stone. The smaller underlayer quarry stone placed underneath the primary quarry stone is exposed and may eventually be dislodged by large waves generated by vessels passing in C & D Canal during high tides.

Photo 21. Primary Quarry Stone Fallen into Water. The displacement of the primary quarry stone was probably caused by large waves generated by vessels. However, the displacement could also be caused by ice or during the construction of the revetment.

Photo 22. Exposure of Soil Underneath Quarry Stone. The quarry stone appear to have slid down and the soil underneath the quarry stone was exposed. The access road built above the riprap revetment for the revetment construction is 25 ft wide.

Photo 23. From Riprap Revetment toward Marina Site. Note the consolidated clay in front of the riprap revetment which looks darker than the beach materials along the eroding bluff. This indicates that the sediment originated from the eroding bluff is transported toward the marina site, resulting in lack of cohesionless sediment in front of the riprap revetment.

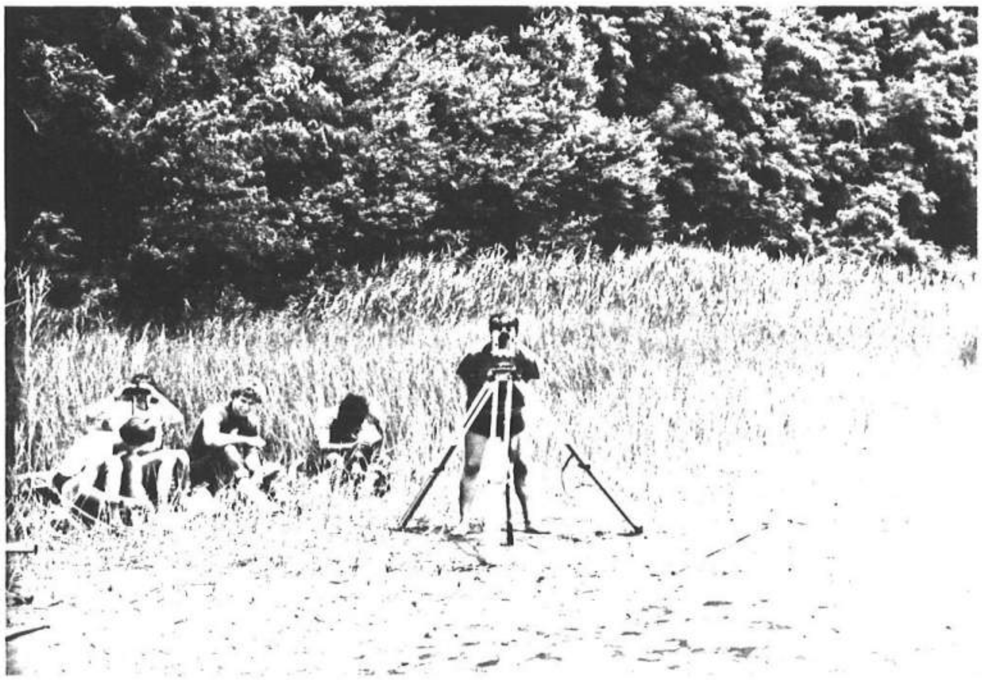

Photo 24. Transit Used for Field Surveys. The transit was placed at Station A at the high tide during the first field survey.

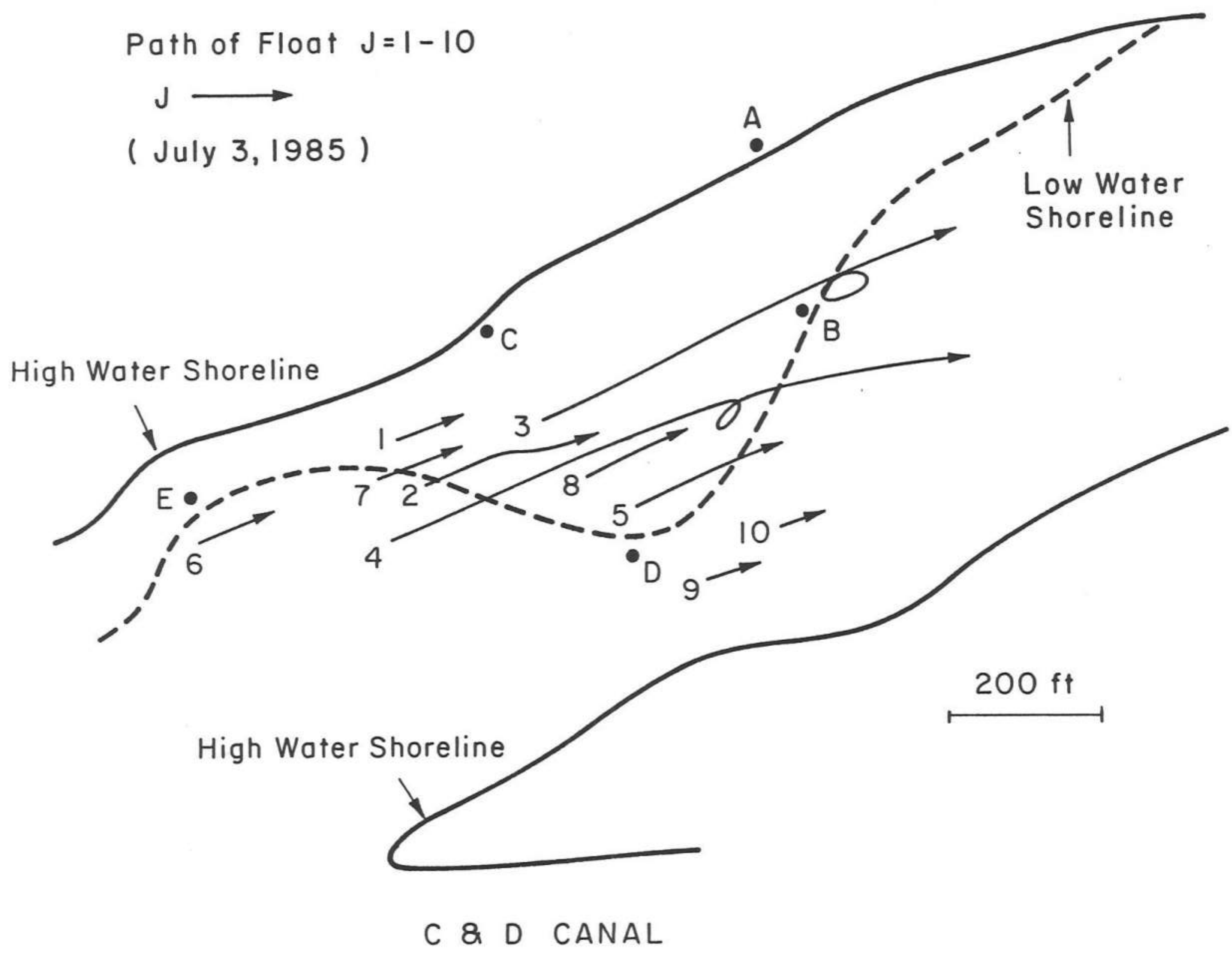

Fig. 5. Observed Paths of 10 Wooden Floats on July 3, 1985

2 ft height generated by a very large cargo vessel navigating westward in the C & D Canal. The effects of this cargo vessel lasted for three minutes. Table 6 summarizes the average velocities of the floats whose paths are depicted in Fig. 5. The average float velocity was calculated by dividing the travel distance of the float by the tracking time. Table 6 also shows the time of tracking each float and the corresponding tidal conditions. On July 3, 1985, the high water occurred at 12:20 and the low water was estimated to occur at 18:25 shortly after the survey team left the marina site. Consequently, Floats 1-5 were tracked during the flood tide when the inflow of the water into the marina area raised the water level. On the other hand, Floats 6-10 were tracked during the ebb tide when the outflow of the water out of the marina area lowered the water level. In order to explain the eastward movement of the floats during the ebb tide, the wind data at Wilmington, Delaware, on July 3, 1985, was obtained from the National Weather Service and listed in Table 6. The westerly wind was blowing at the speed of 15-22 ft/sec (10-15 mph) at the Wilmington weather station. The wind speed in the marina area appeared to be slightly less than the reported wind speed since the marina area is partly shielded by the surrounding bluff. The wind speed of 10-15 mph was greater than the mean wind speed of 7.8 mph for July as listed in Table 2. Table 6 shows that the float velocities during the flood tide were in the range 0.32-0.66 ft/sec, corresponding to 2.1-4.4 percent of the wind speed. On the other hand, the float velocities during the ebb tide were in the range 0.16-0.40 ft/sec, corresponding to 0.94-1.8 percent of the wind speed. These measurements indicate that the tidal current velocities in the marina area are relatively small and should be on the order 0.1 ft/sec.

TABLE 6: Summary of Surface Current Measurements Using Wooden Floats

Float Number	Time	Tide	Average Float Velocity (ft/sec)	Wind at Wilmington, Delaware		$\frac{\text{Float Velocity}}{\text{Wind Speed}} \times 100$ (%)
				Speed (ft/sec)	Direction	
1	10:20 - 10:23	Flood	0.44	15	270	2.9
2	10:23 - 10:33	Flood	0.35	15	270	2.3
3	10:39 - 11:10	Flood	0.32	15	270	2.1
4	10:48 - 11:10	Flood	0.45	15	270	3.0
5	10:39 - 10:44	Flood	0.66	15	270	4.4
6	14:30 - 14:35	Ebb	0.33	22	260	1.5
7	14:40 - 14:45	Ebb	0.33	22	260	1.5
8	14:50 - 14:55	Ebb	0.40	22	260	1.8
9	16:40 - 16:45	Ebb	0.20	17	210	1.2
10	17:10 - 17:15	Ebb	0.16	17	210	0.94

Furthermore, the current patterns in the marina area are mainly determined by the wind whose prevailing directions are from the west and the south as shown in Table 2. The resulting surface currents in the marina area are expected to flow mostly toward the eastern end of the marina site as observed in Fig. 5.

3.3 Sediment Samples and Size Distributions

During the first field survey, sediment samples were collected using a small garden trowel from the surfaces of the beach and bluff in the marina area. The sediment samples were placed in zip lock plastic bags and brought back to the Soil Mechanics Laboratory, the Department of Civil Engineering. The size distributions of the sediment samples were obtained by performing the standard sieve analysis recommended by U. S. Army Engineer Waterways Experiment Station (20). Fig. 6 shows the locations of the surface sediment samples collected on July 3, 1985. Samples 1-9 collected from the surface of the shoal at the entrance of the marina site represent the sediments deposited on the shoal. Sample 10 collected from the base of eroding grass roots represents the sediment remaining at the eroding base. Samples 11 and 12 collected from the eroding bluff represent the sediments introduced to the marina area due to the bluff erosion. Table 7 shows the precent of the sediment by weight retained in the specified sieve openings for each of the Samples 1-12. Based on the Unified Soil Classification (5), sediments with sizes between 0.074 mm and 0.42 mm are fine sands. Sediments passed through the 0.074 mm sieve openings are silt and clay. Table 7 indicates that the sediment deposited on the shoal at the entrance of the marina

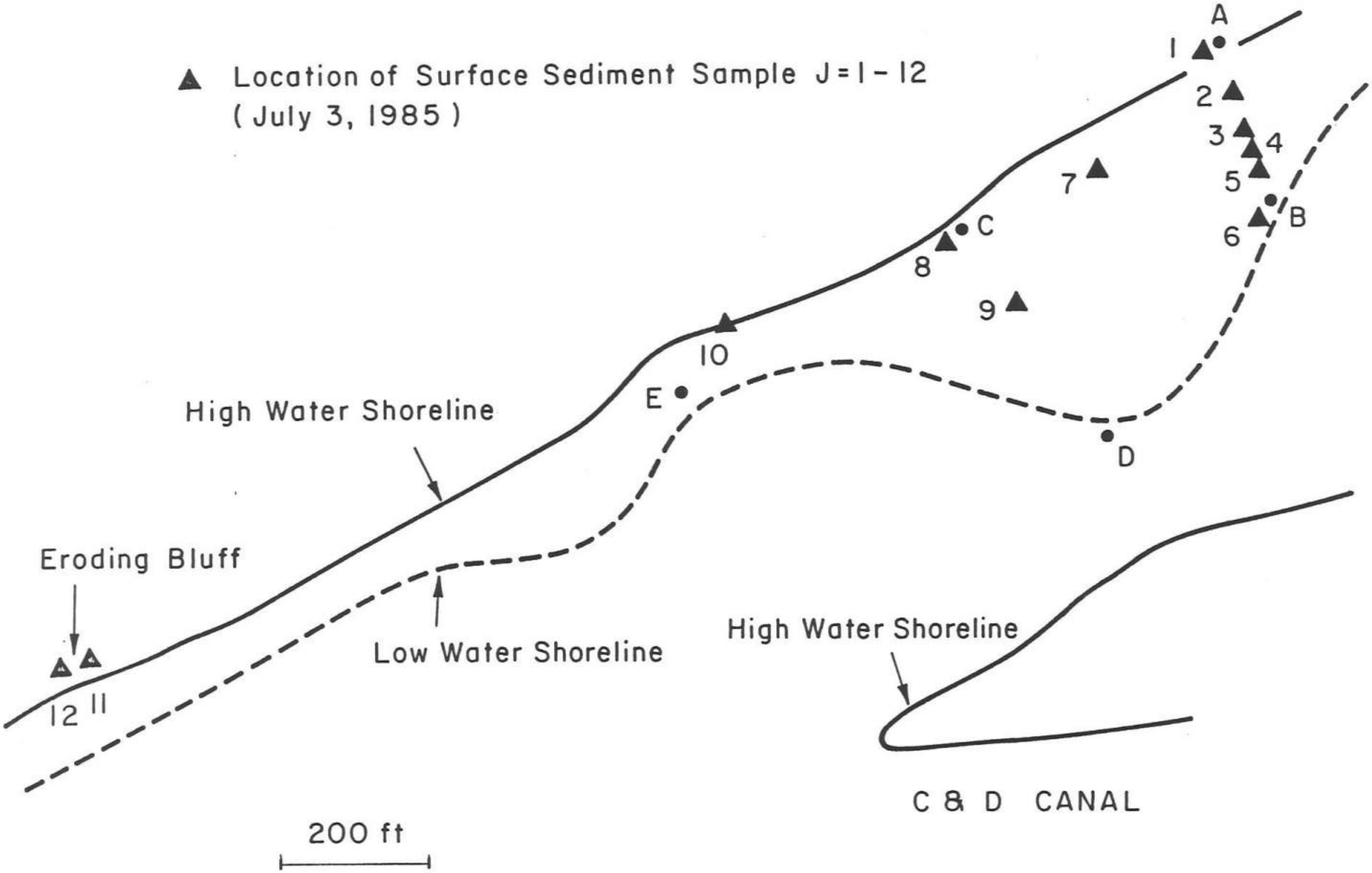

Fig. 6. Locations of Surface Sediment Samples Collected on July 3, 1985

TABLE 7: Summary of Surface Sediment Samples and Size Distributions in Percentage

Sieve Openings (mm)	Sample Number											
	1	2	3	4	5	6	7	8	9	10	11	12
≥0.59	0.1	0.4	1.3	4.5	6.4	0.0	0.1	7.6	1.6	25.0	0.2	0.6
0.42	1.1	3.9	4.5	1.8	10.2	0.2	0.8	6.7	6.1	8.3	1.1	1.2
0.297	7.1	14.4	12.5	6.0	13.5	2.6	5.7	12.9	13.1	12.6	3.4	3.0
0.210	24.7	33.5	21.0	18.0	21.9	15.0	19.1	18.4	24.8	16.4	5.5	7.1
0.149	40.9	32.3	29.5	35.8	26.3	45.3	41.2	34.4	29.5	24.2	37.1	39.9
0.105	18.0	4.8	18.3	21.1	3.2	8.0	15.0	13.5	3.0	8.5	34.4	23.8
0.074	5.8	5.3	7.8	9.1	13.8	27.0	9.7	3.6	12.8	2.6	7.3	7.0
<0.074	2.3	5.4	5.1	3.7	4.7	1.9	8.4	2.9	9.1	2.4	11.0	17.4

site is basically fine sand except that gravels and cobbles are also present on the shoal as shown in Photo 5. Samples 1-9 also indicate the degree of the spatial variations of the sediment characteristics on the shoal. On the other hand, Sample 10 contains more coarse sediment and Samples 11 and 12 contain more fine sediment in comparison to Samples 1-9. Consequently, coarse sediments tend to remain on the base of eroding grass roots or bluff and fine sediments tend to be transported further away. Table 7 indicates that Samples 1-12 are essentially the same sediment except that the sorting effects tend to produce spatial variations.

Test borings at the marina site were made in May and June, 1985. The test boring data provided by Gredell & Paul, Inc., Newark, Delaware is summarized in Fig. 7 and Table 8. The test boring locations shown in Fig. 7 were close to the eastern end of the Lums Pond Lagoon. Table 8 indicates that the bottom sediments at the test boring locations are essentially soft silt and river mud. This indicates that only silt and clay are transported eastward over the shoal at the entrance of the marina site although the land surrounding the eastern end of the Lums Pond Lagoon may also supply some silt and clay. On the other hand, the test pit data provided by the Philadelphia District, U. S. Army Corps of Engineers is summarized in Fig. 8 and Table 9. The locations of the test pits in the C & D Canal are shown in Fig. 8. The test pit data was obtained in 1968 when the water depth at this segment of the C & D Canal was approximately 29 ft as shown in Table 9. The test pit data suggests that the bottom sediment was essentially composed of clay, silt and fine sand. It is hence possible to assume that some of this sediment was originated from the eroding bluff on the north.

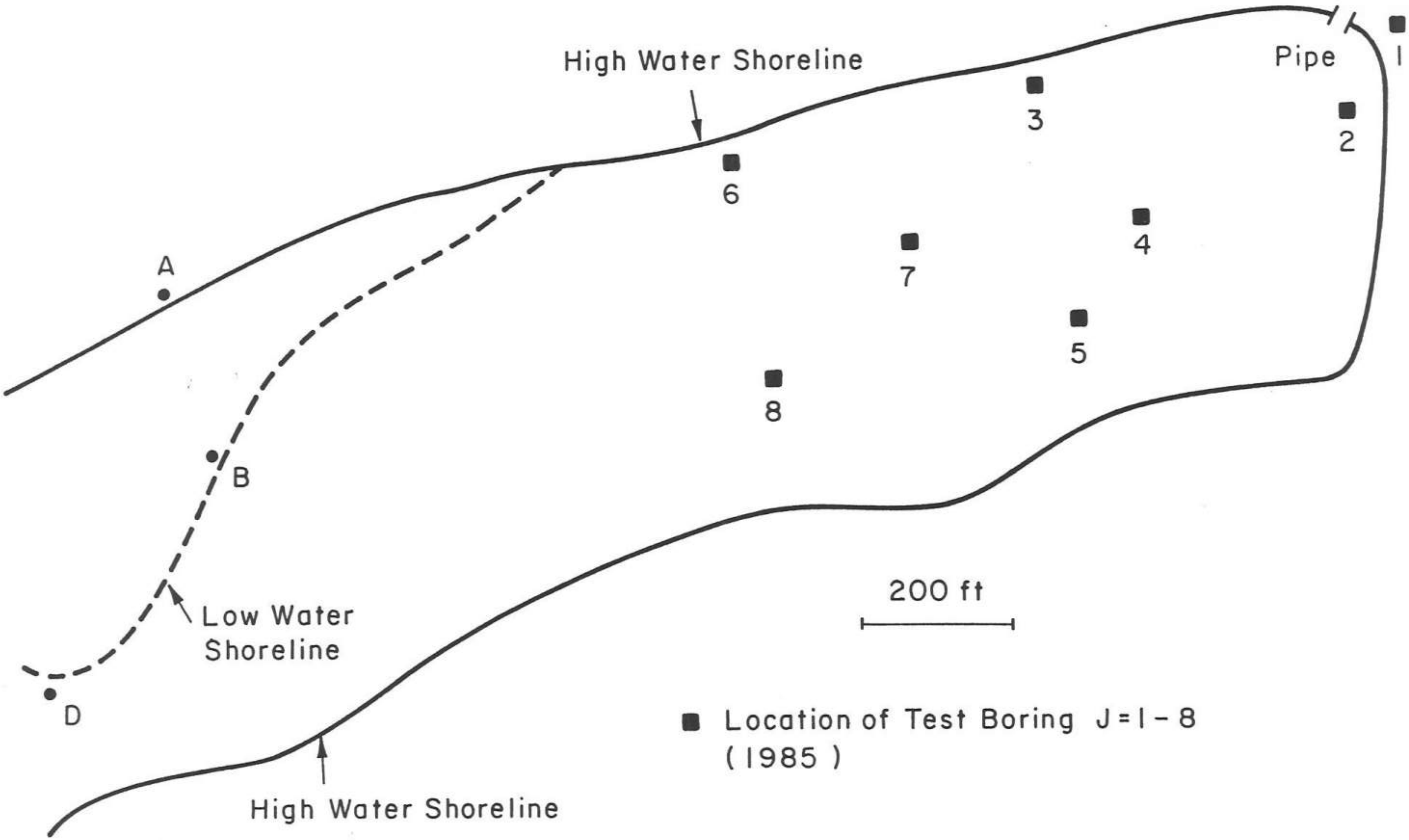

Fig. 7. Locations of Test Borings Conducted in 1985
for Summit North Marina Site

TABLE 8: Summary of Test Boring Data (1985) for

Summit North Marina Site

Boring Location	Depth Range (ft)	Description of Materials
1	0 - 2.8 2.8 - 6.0 6.0 - 30.0	Brn. Silt F/C Sand & Grav. Fill Brn. Silt F/M Sand w/Some Grav. Gray Silt F Sand
2	0 - 8.0 8.0 - 9.5 9.5 - 40.0	Water Gray Silt Gray Silty F Sand or Sandy Silt
3	0 - 10.5 10.5 - 24.0 24.0 - 43.0	Water & Soft Silt Gray Silt & Sand Tr. Mica Gray Silty Clay Tr. of Sand
4	0 - 24.0 21.0 - 35.0 35.0 - 55.5	Water Silt & Mud Gray Silt & Clay
5	0 - 19.2 19.2 - 36.0 36.0 - 62.0	Water River Mud Gray Silt & Clay
6	0 - 10.6 10.6 - 31.0	Water & Soft Silt Gray Silt & Sand
7	0 - 22.0 11.0 - 40.0 40.0 - 43.0	Water Soft Silt Gray Silt and Sand
8	0 - 18.0 18.0 - 36.0 36.0 - 39.0	Water River Muck Gray Sand, Silt & Clay

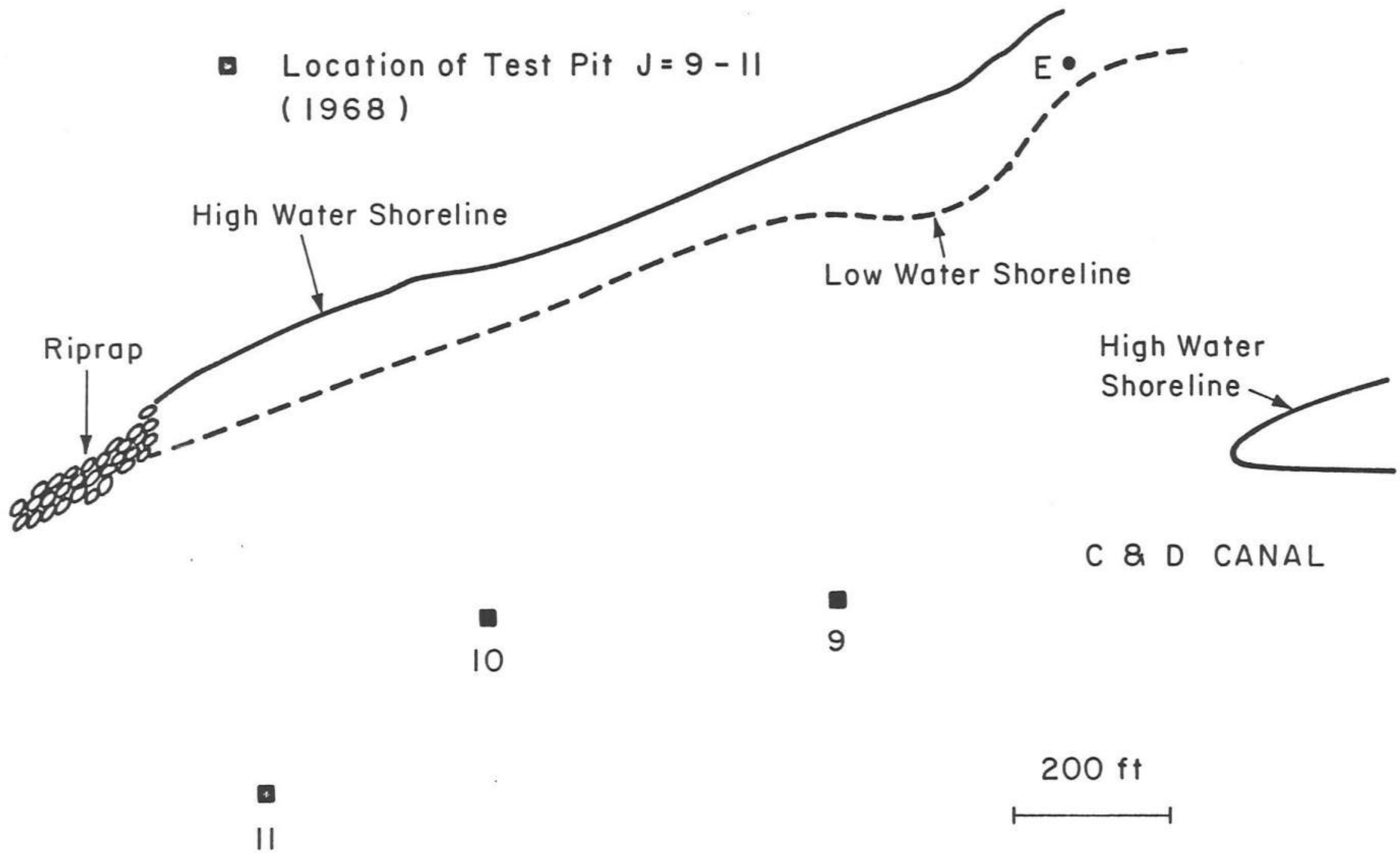

Fig. 8. Locations of Test Pits Conducted in 1968 in C & D Canal

TABLE 9: Summary of Test Pit Data (1968) in C & D Canal

Test Pit Location	Depth Range (ft)	Description of Materials
9	0 - 29.1 29.1 - 32.8 32.8 - 39.7	Water Gray Org. Clay w/Some Sand F Sandy Clayey Silt (Stiff)
10	0 - 29.4 29.4 - 32.4 32.4 - 35.0	Water Gray Org. Silty F Sand Silty Very F Sandy Clay (Stiff)
11	0 - 28.9 28.4 - 31.9 31.9 - 36.2	Water Gray F Sand w/Tr. Org. Silt F Sandy Clayey Silt (Stiff)

3.4 Beach Profile Changes

The beach profiles along the straight line A-B, C-D and D-E shown in Figs. 3 and 4 were measured on July 3, 1985 and on August 28, 1985. The beach profile changes over the interval of approximately two months indicate the degree of sedimentation and erosion in the neighborhood of the shoal at the entrance of the marina site. The measured profile changes relative to the beach profiles observed on July 3, 1985 are shown in Figs. 9-11. Fig. 9 indicates that the beach profile in the vicinity of Station A remained essentially the same while the neighborhood of Station B was eroding probably because of the falling and sliding of the sediment into the dredged channel located south of Station B. Fig. 10 shows that the western side of the shoal along the line C-D was accreting except that the grass in the vicinity of Station C created local erosion and accretion patterns as shown in Photos 3 and 4. The accretion in the vicinity of Station D was large probably because of the deposition of the sediment transported along the western side of the shoal near Station D which was located at the tip of the shoal. Fig. 11 also indicates the accretion on the western side of the shoal in the neighborhood of Station C. On the other hand, erosion occurred slightly east of Station E in the vicinity of the base of the eroding bluff shown in Photo 8. Figs. 9-11 suggest a rapid adjustment of the beach profiles following the dredging conducted in November, 1984 and January, 1985. The sedimentation rate at the tip of the shoal was observed to be as large as 0.7 ft during the interval of approximately 2 months. However, longer and underwater bathymetric surveys are required to estimate the sedimentation rate more precisely.

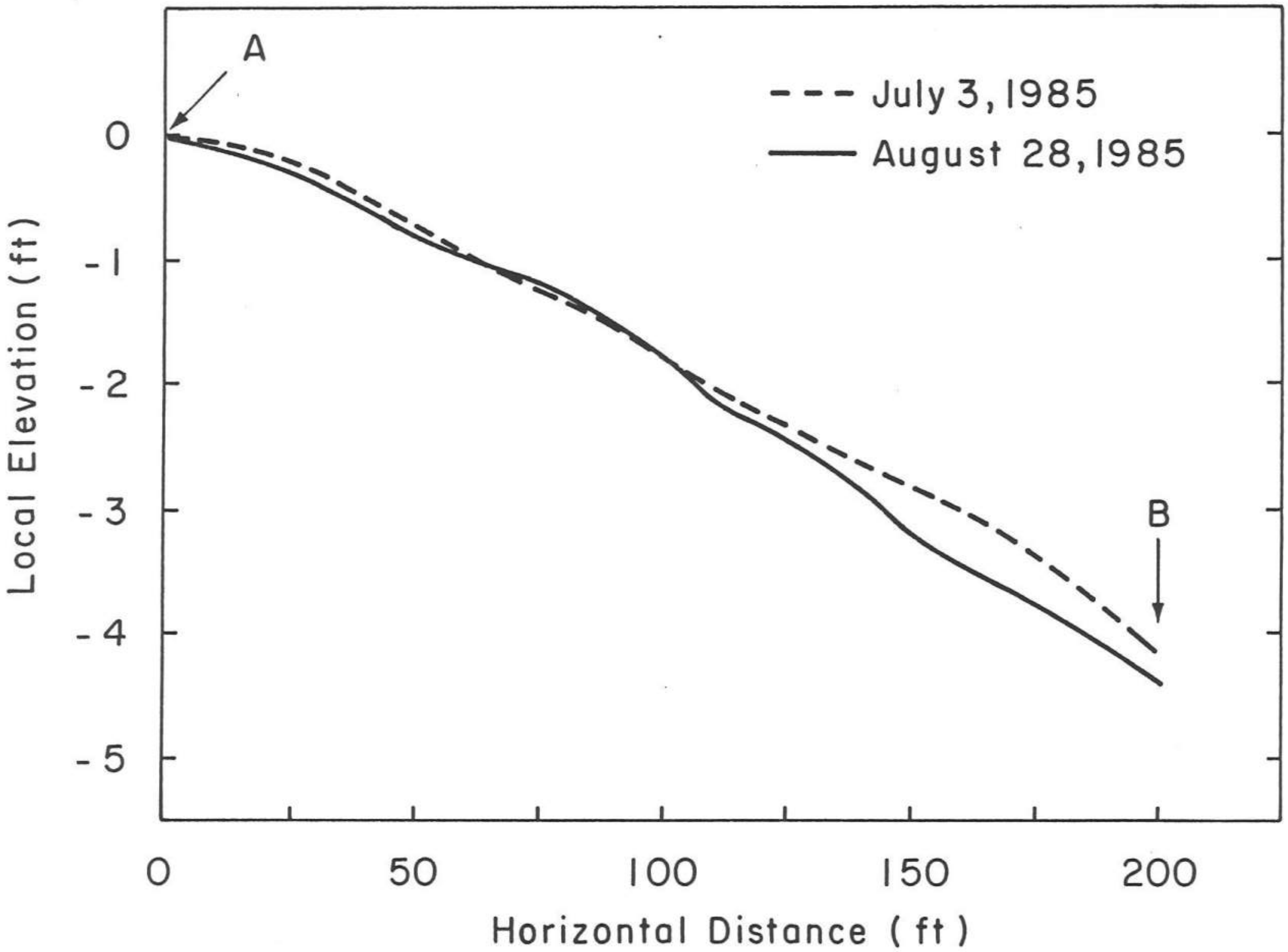

Fig. 9. Measured Beach Profile Changes between Station A and B in Summer, 1985

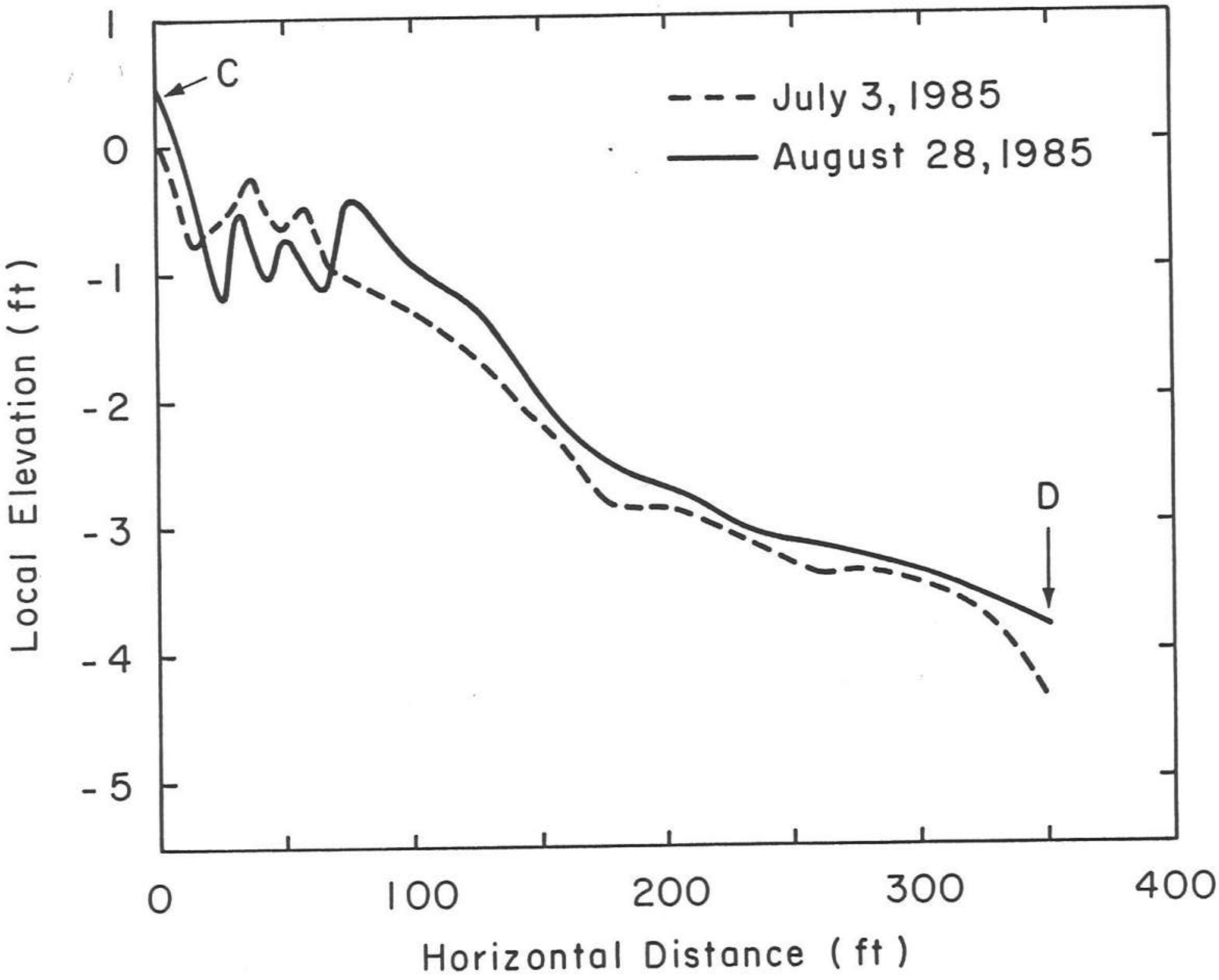

Fig. 10. Measured Beach Profile Changes between Station C and D in Summer, 1985

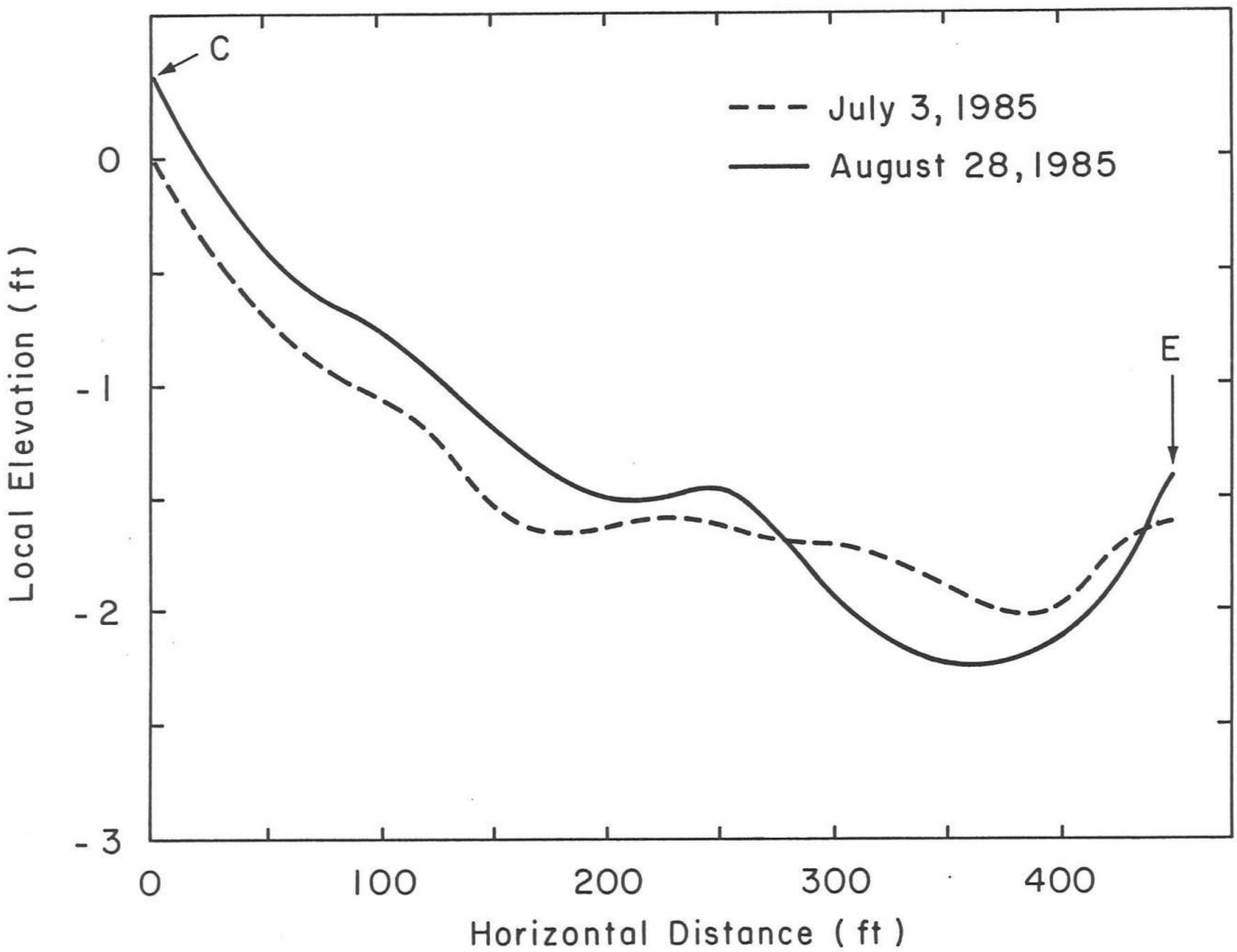

Fig. 11. Measured Beach Profile Changes between Station C and E in Summer, 1985

4. SEDIMENT TRANSPORT PATTERNS

In order to predict the severity of the sedimentation problems at the marina site, the sediment transport patterns in the marina area before and after the dredging conducted in November, 1984 and January, 1985 are estimated in the following.

4.1 Bathymetric Changes from 1965 to 1980

Comparing the 1965 and 1980 maps discussed in Section 2.6, the bathymetric changes from 1965 to 1980 are obtained. Fig. 12 shows the locations of the 21 cross sections compared in this study. The comparison was limited to the area surrounded by the cross sections 1 and 21 and the 1980 shoreline based on the Mean Sea Level Datum (NGVD) where both 1965 and 1980 maps give sufficient bathymetric information. The longitudinal spacings of the cross sections are determined on the basis of the contour variations in 1980, while the 1965 map indicates small longitudinal variations of the cross sections. Table 10 lists the longitudinal length represented by each of the 21 cross sections. Figs. 13-33 show the profile changes at the cross sections 1-21, respectively. Table 10 lists the net change of the cross section area for each of the 21 cross sections. The net area change is equal to the volume of the sediment deposited per unit longitudinal length. The net area change increased with the cross section number, that is, westward except for cross sections 1, 2 and 3. The net area changes at these cross sections shown in Figs. 13-15 must have been affected by the construction of a road next to the eastern end of the marina area. Figs. 16-33 indicate that the sedimentation depth for each of the cross sections 4-21 was the maximum in the middle of the channel.

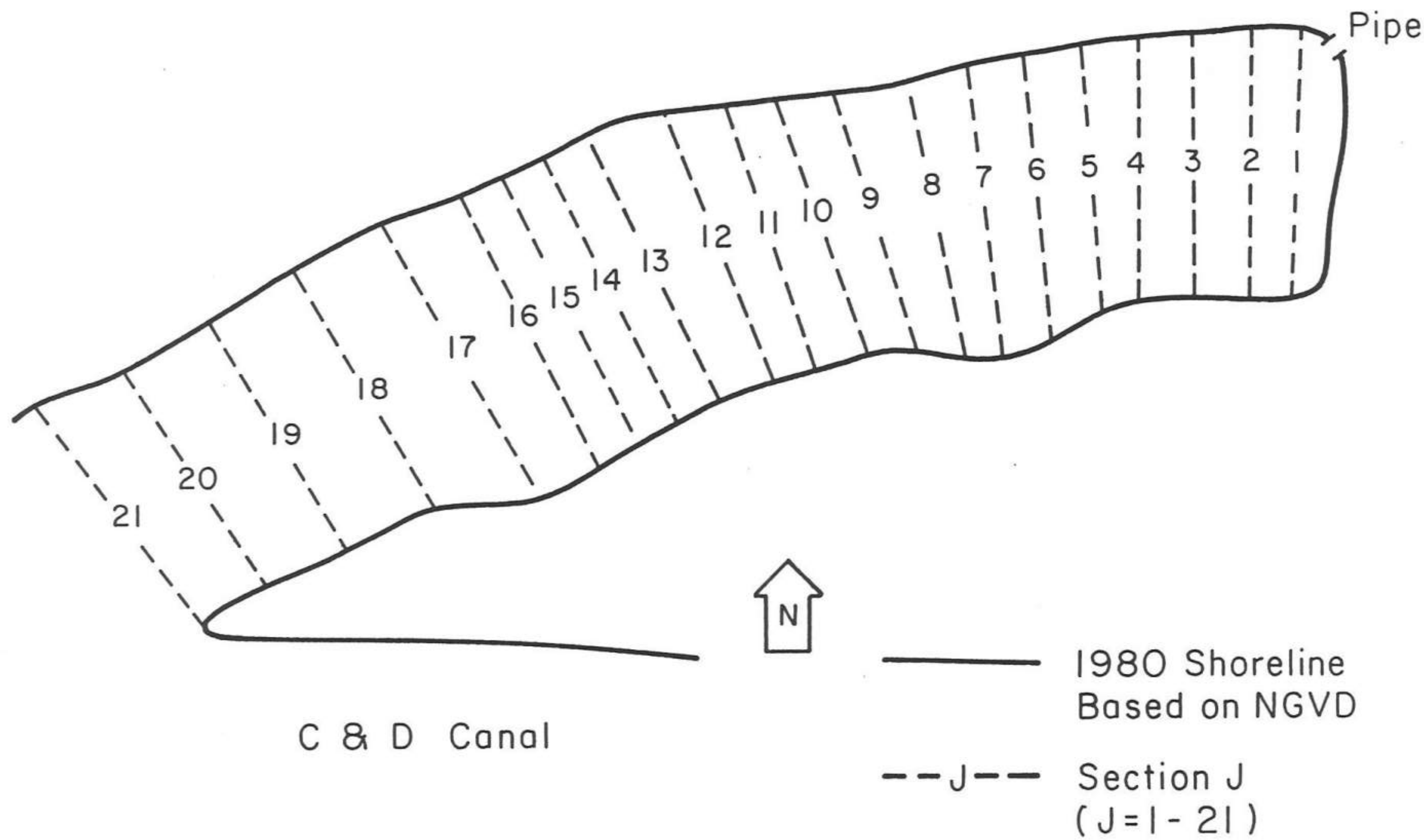

Fig. 12. Locations of 21 Cross Sections Plotted in Figs. 13 - 33

TABLE 10: Net Volume Change Between Section 1 and 21 from 1965 to 1980

Section Number	Net Change of Cross Section Area (ft^2)	Longitudinal Length of Section (ft)	Net Volume Change From 1965 to 1980 (ft^3)
1	7.54×10^3	50	3.77×10^5
2	5.39×10^3	100	5.39×10^5
3	3.57×10^3	100	3.57×10^5
4	2.71×10^3	100	2.71×10^5
5	2.85×10^3	100	2.85×10^5
6	2.74×10^3	112.5	3.08×10^5
7	3.105×10^3	100	3.11×10^5
8	3.29×10^3	112.5	3.70×10^5
9	3.37×10^3	100	3.37×10^5
10	3.380×10^3	100	3.38×10^5
11	3.51×10^3	100	3.51×10^5
12	3.78×10^3	112.5	4.25×10^5
13	4.96×10^3	100	4.96×10^5
14	5.68×10^3	75	4.26×10^5
15	6.71×10^3	100	6.711×10^5
16	7.12×10^3	85	6.05×10^5
17	7.13×10^3	200	1.43×10^6
18	7.57×10^3	200	1.51×10^6
19	7.56×10^3	185	1.39×10^6
20	8.03×10^3	200	1.61×10^6
21	7.99×10^3	100	7.99×10^5
		TOTAL	1.32×10^7

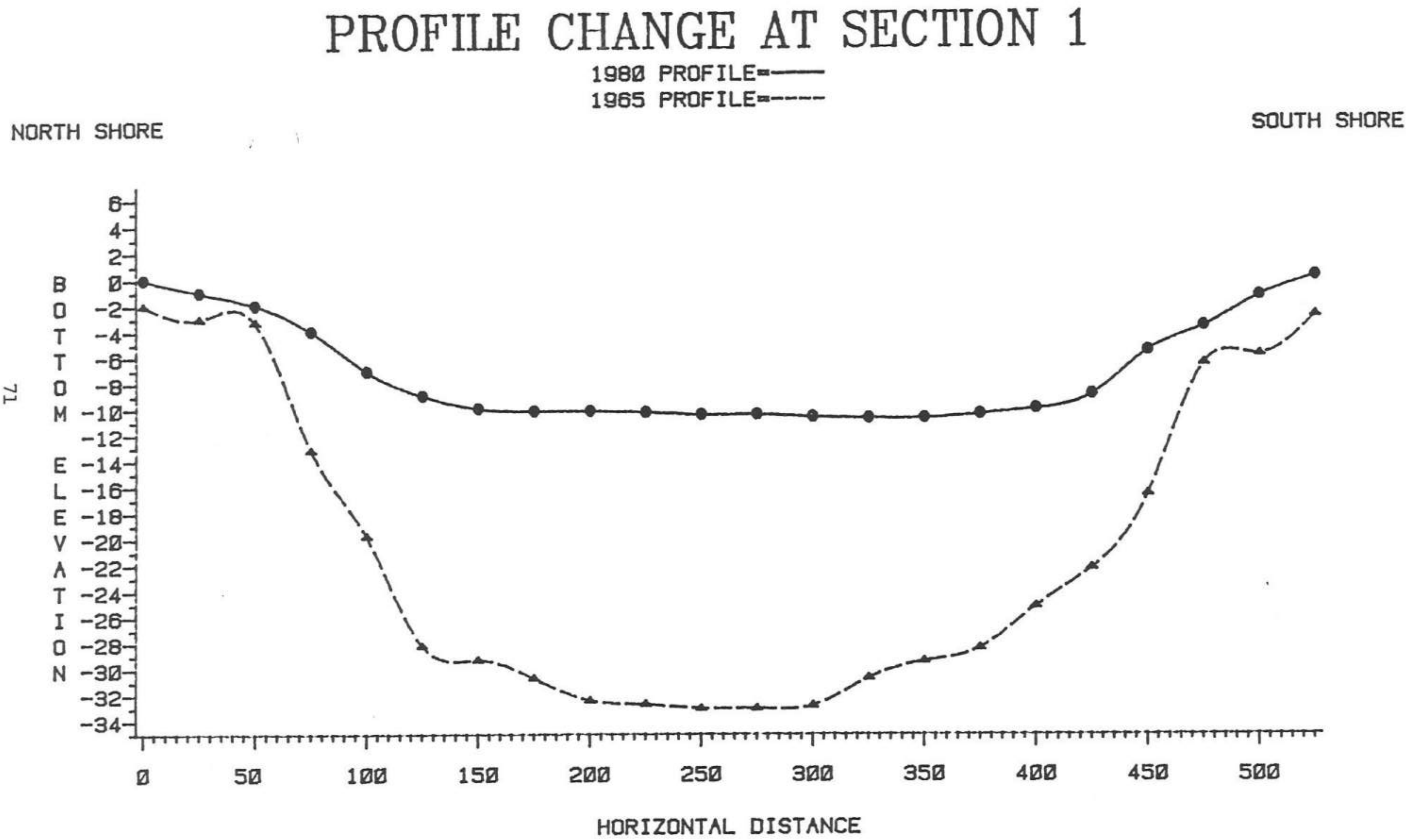

Fig. 13. Profile Change at Section 1 from 1965 to 1980

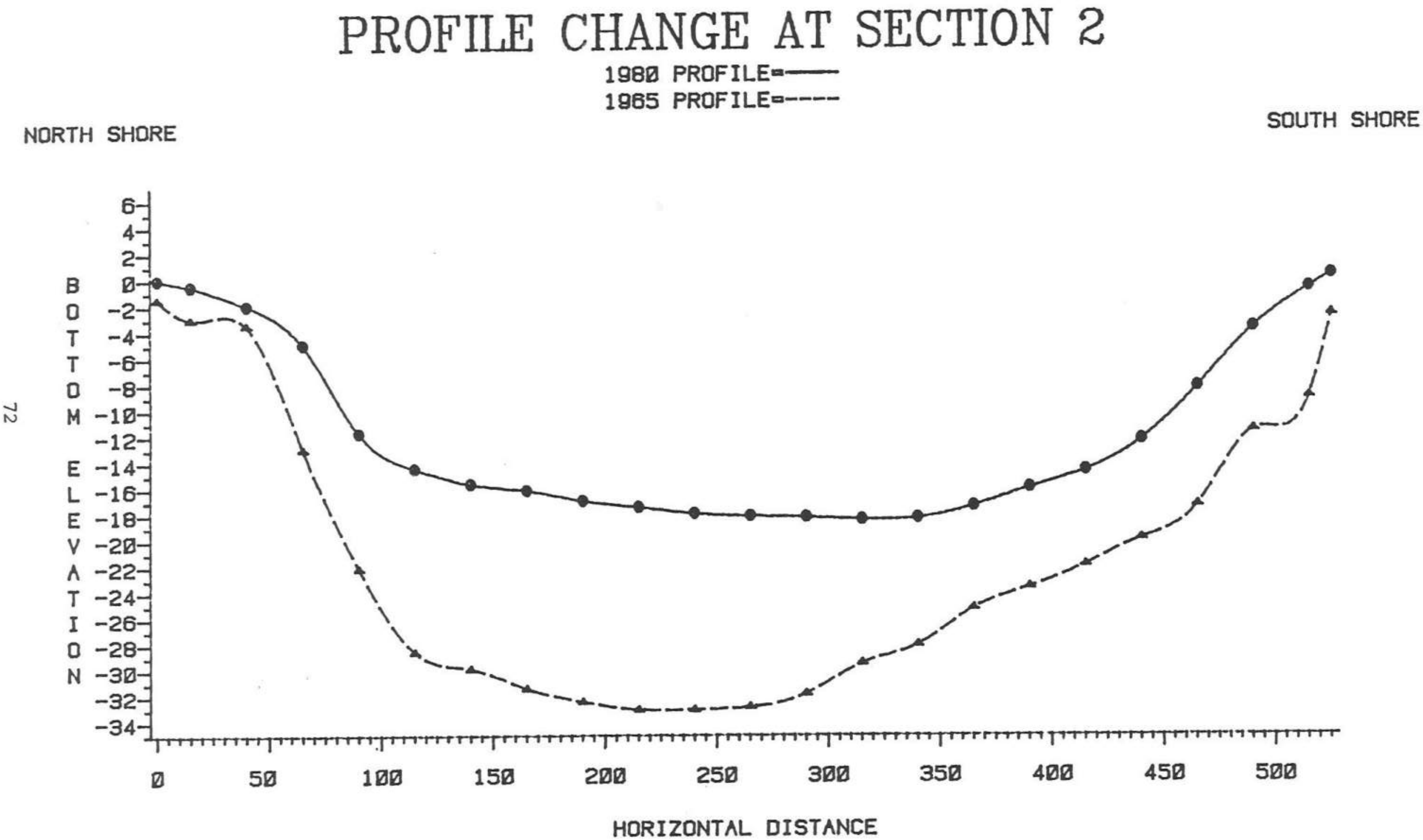

Fig. 14. Profile Change at Section 2 from 1965 to 1980

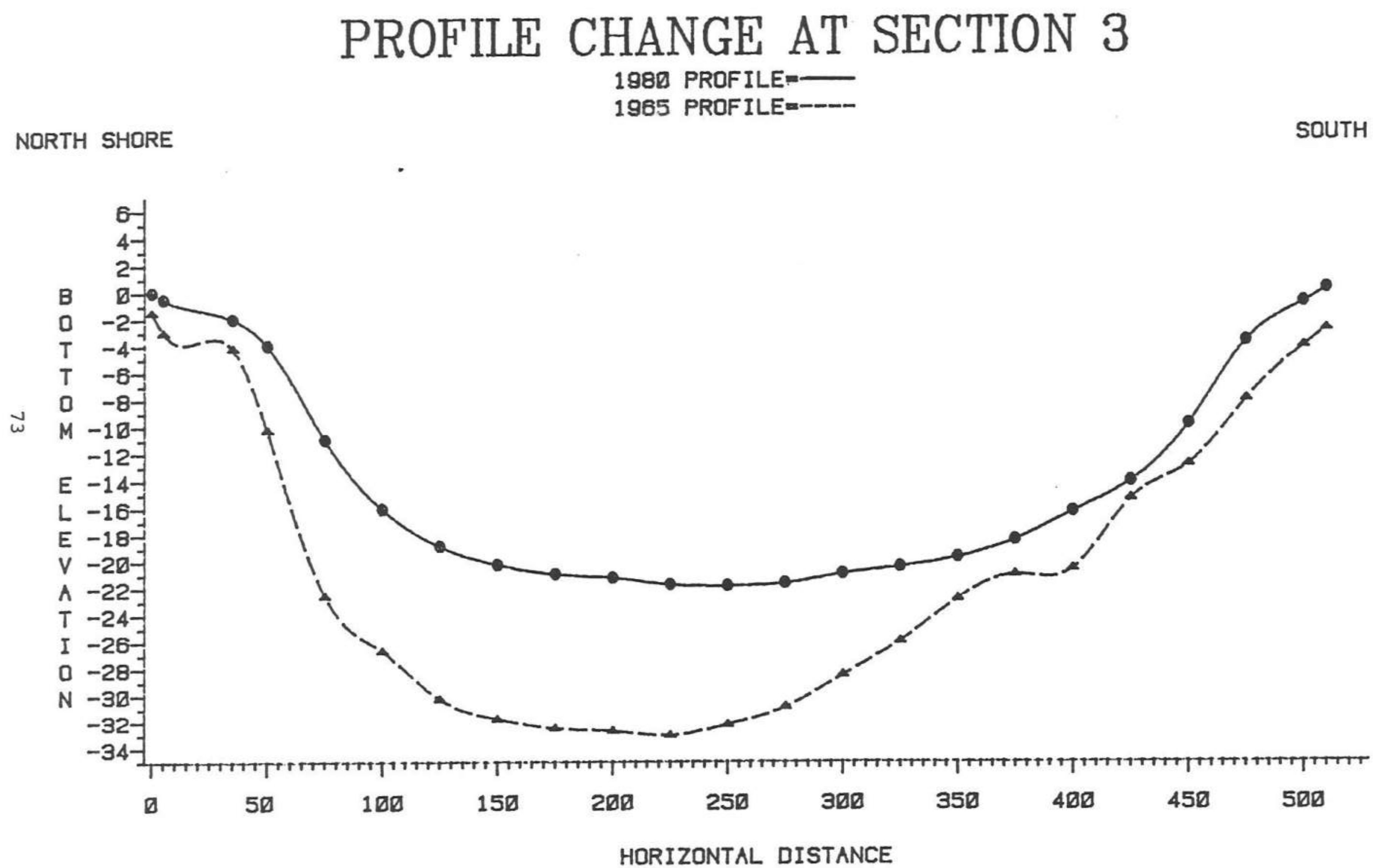

Fig. 15. Profile Change at Section 3 from 1965 to 1980

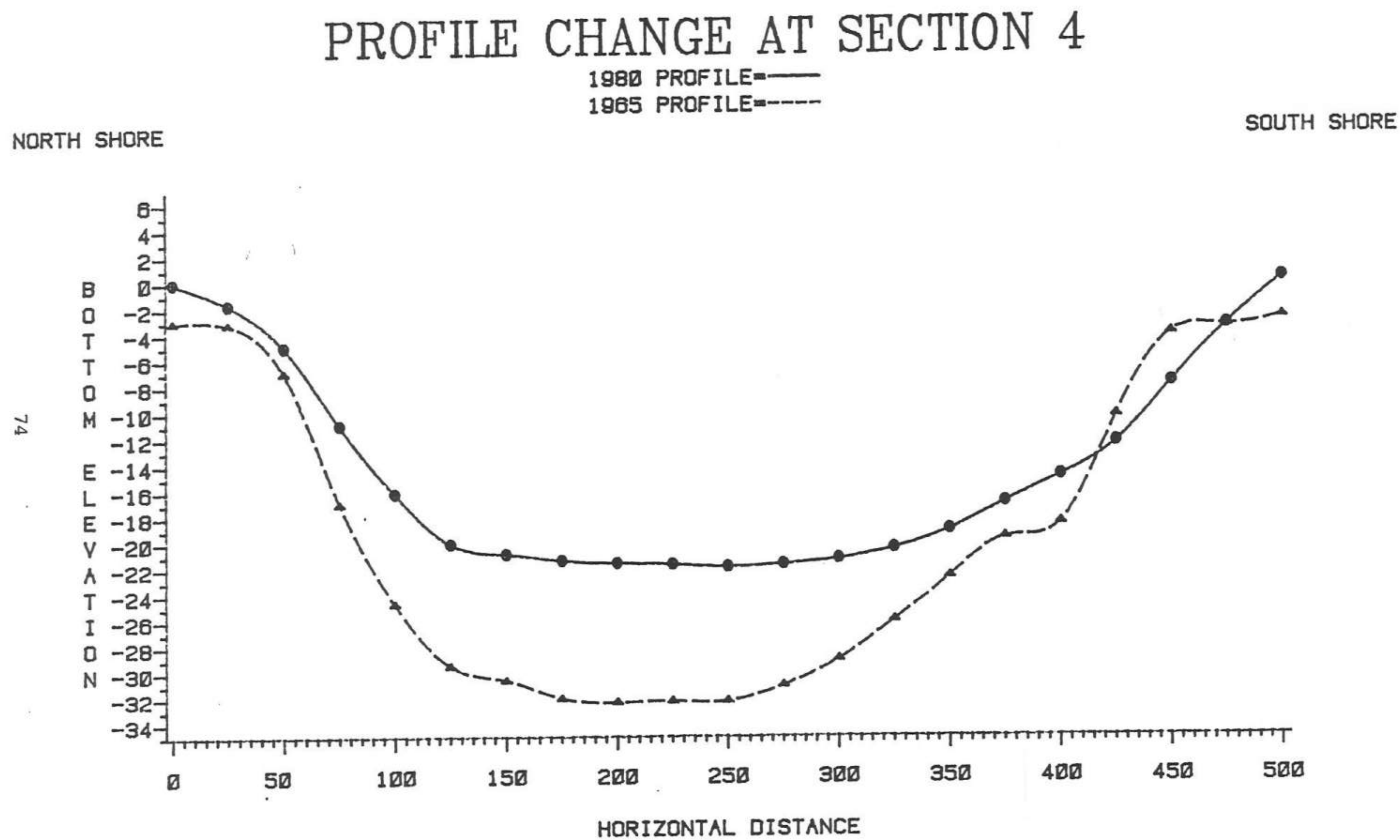

Fig. 16. Profile Change at Section 4 from 1965 to 1980

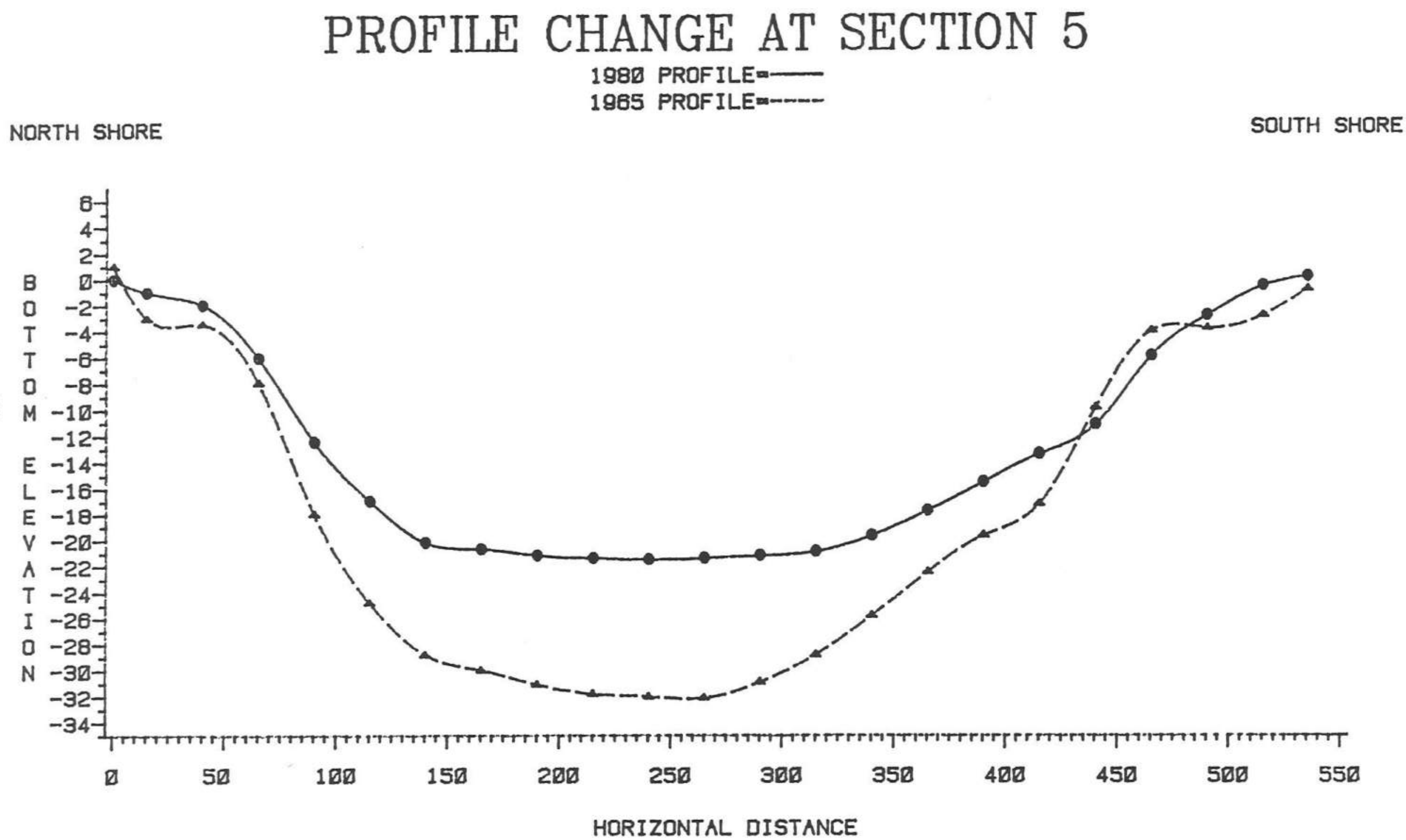

Fig. 17. Profile Change at Section 5 from 1965 to 1980

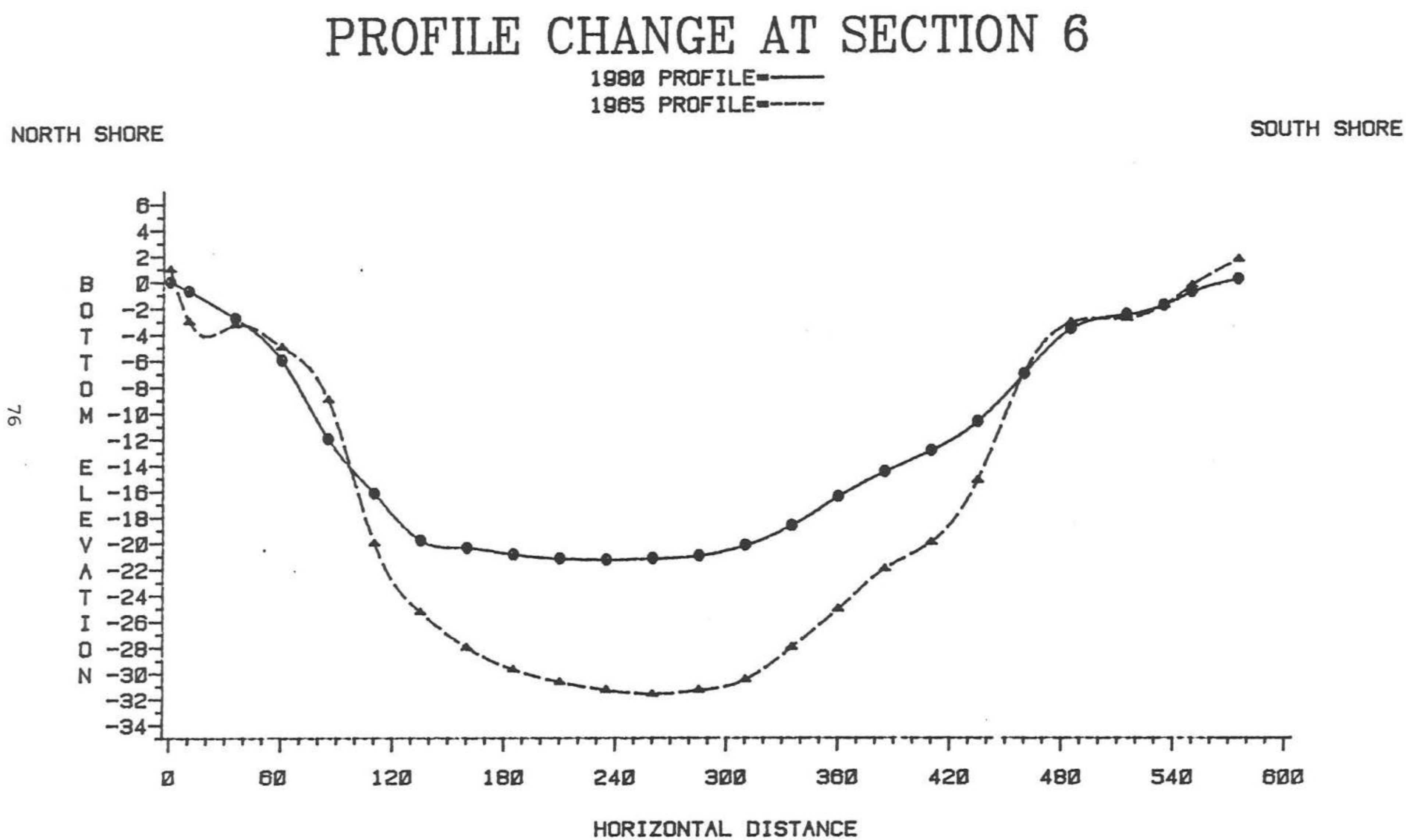

Fig. 18. Profile Change at Section 6 from 1965 to 1980

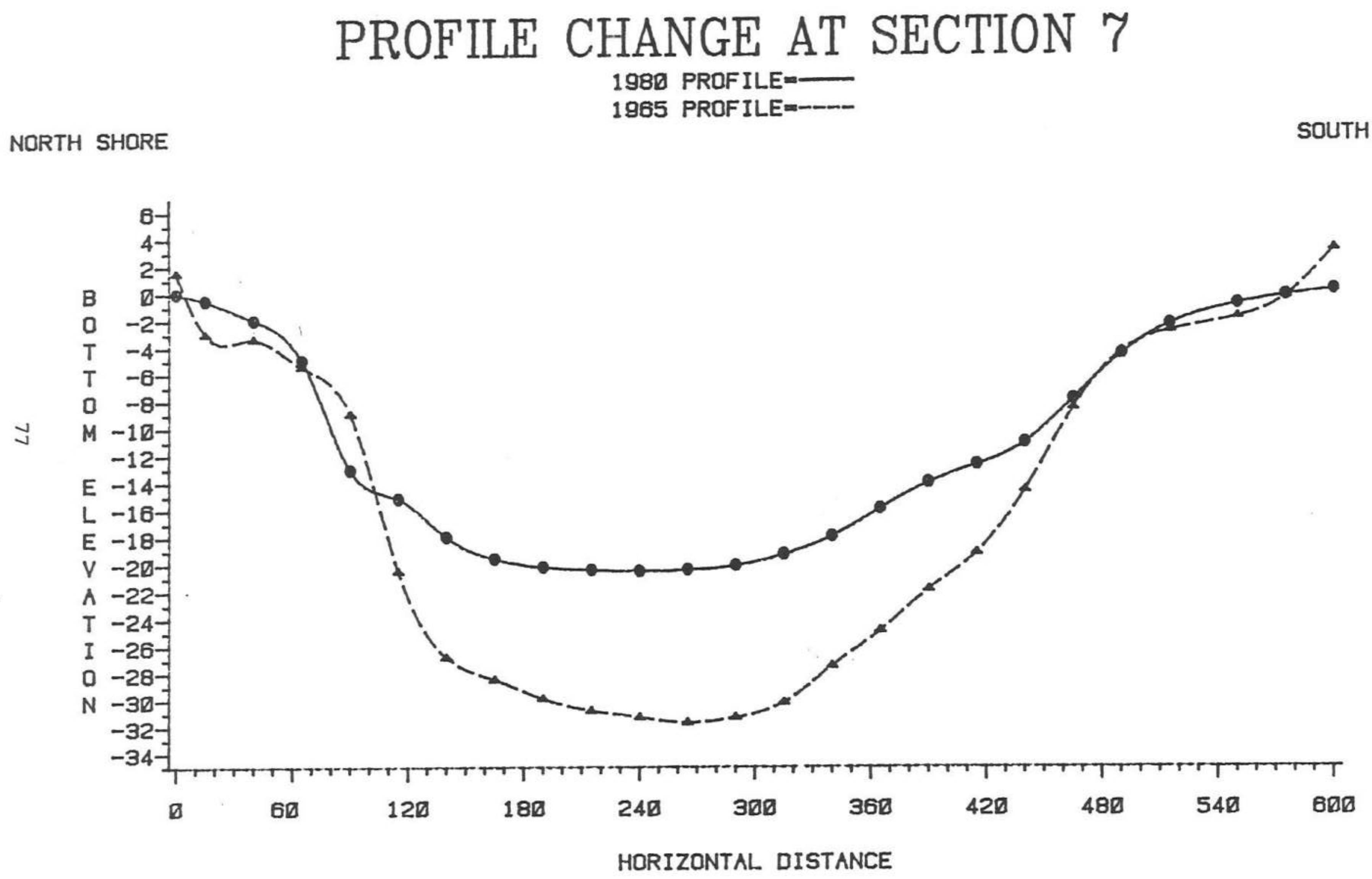

Fig. 19. Profile Change at Section 7 from 1965 to 1980

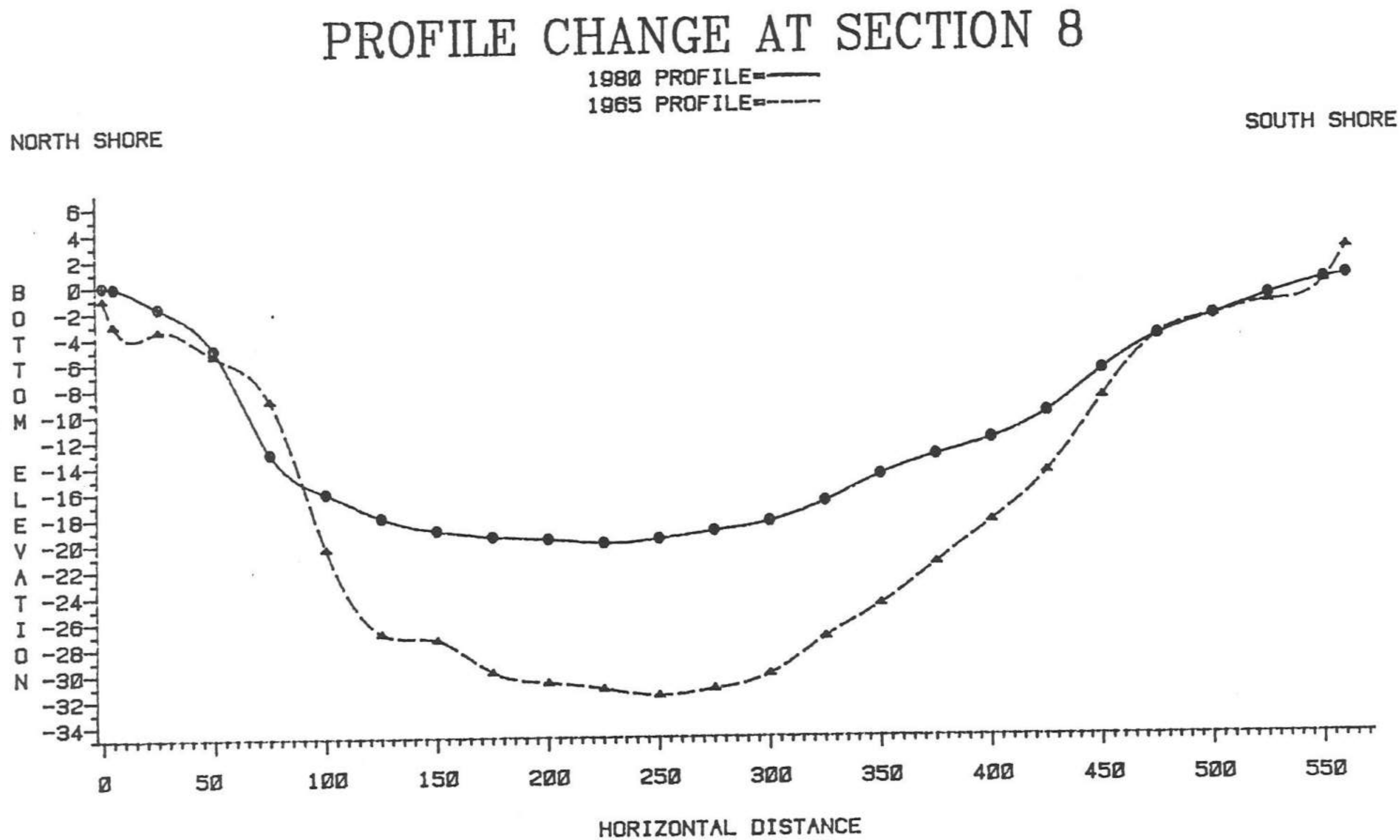

Fig. 20. Profile Change at Section 8 from 1965 to 1980

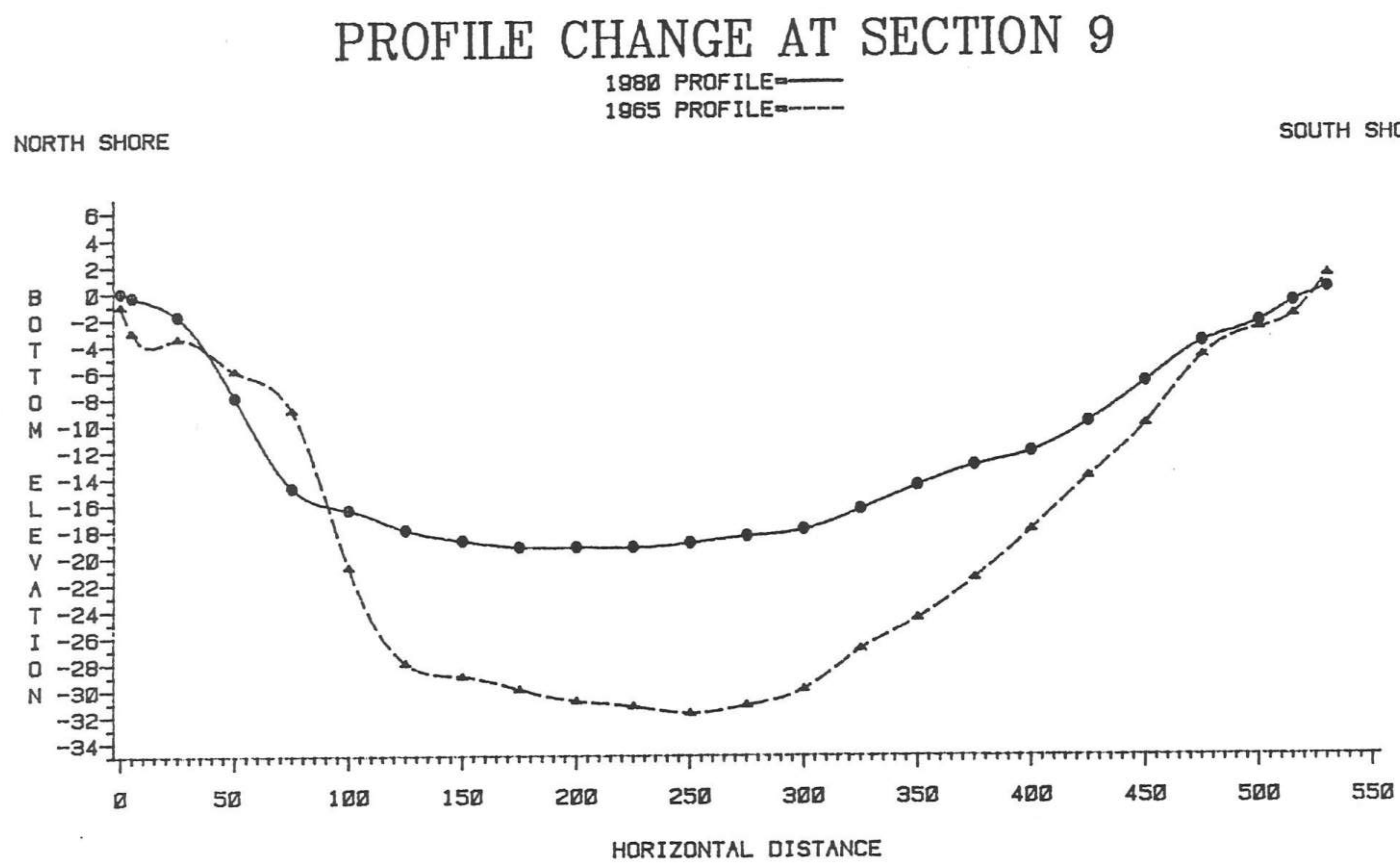

Fig. 21. Profile Change at Section 9 from 1965 to 1980

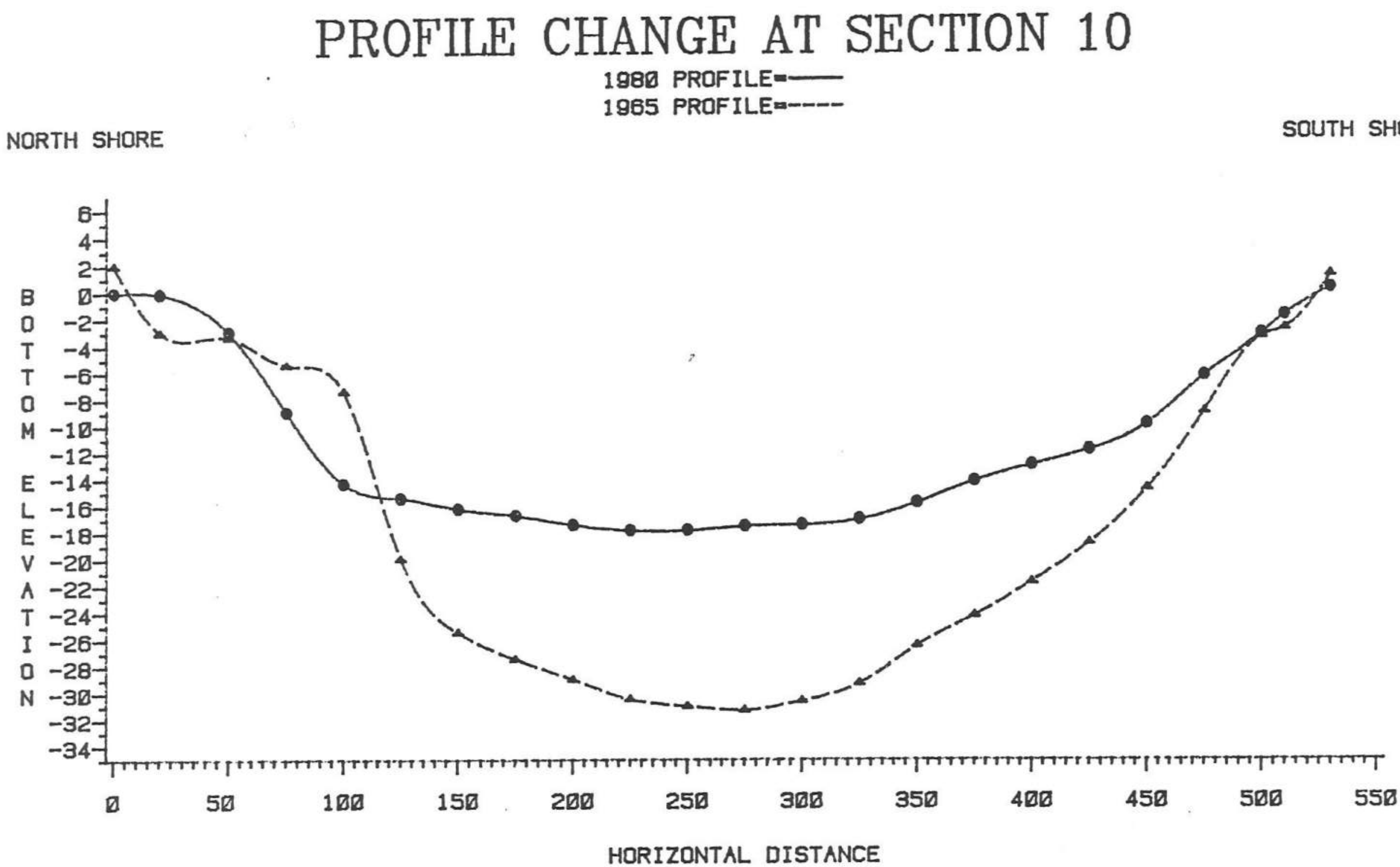

Fig. 22. Profile Change at Section 10 from 1965 to 1980

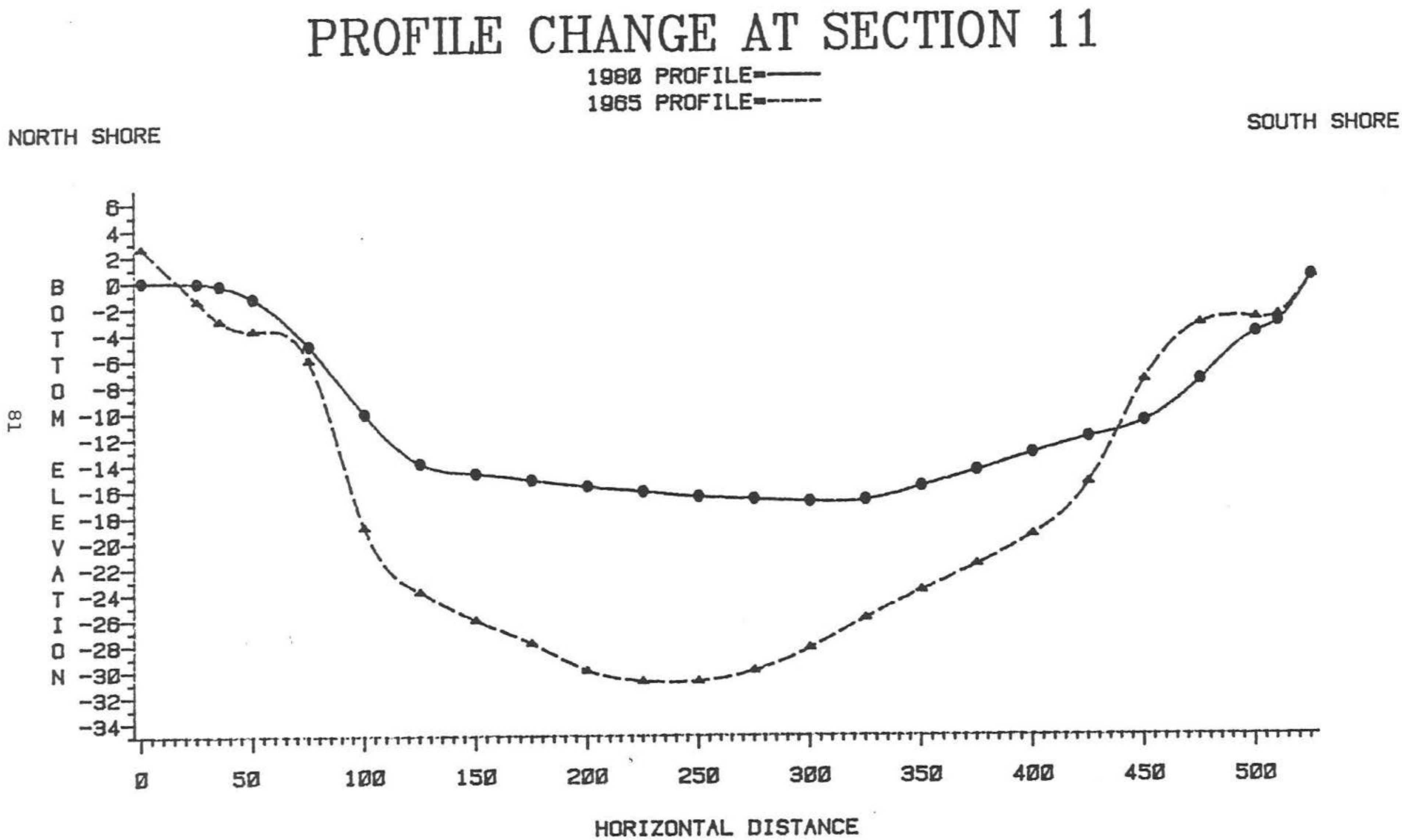

Fig. 23. Profile Change at Section 11 from 1965 to 1980

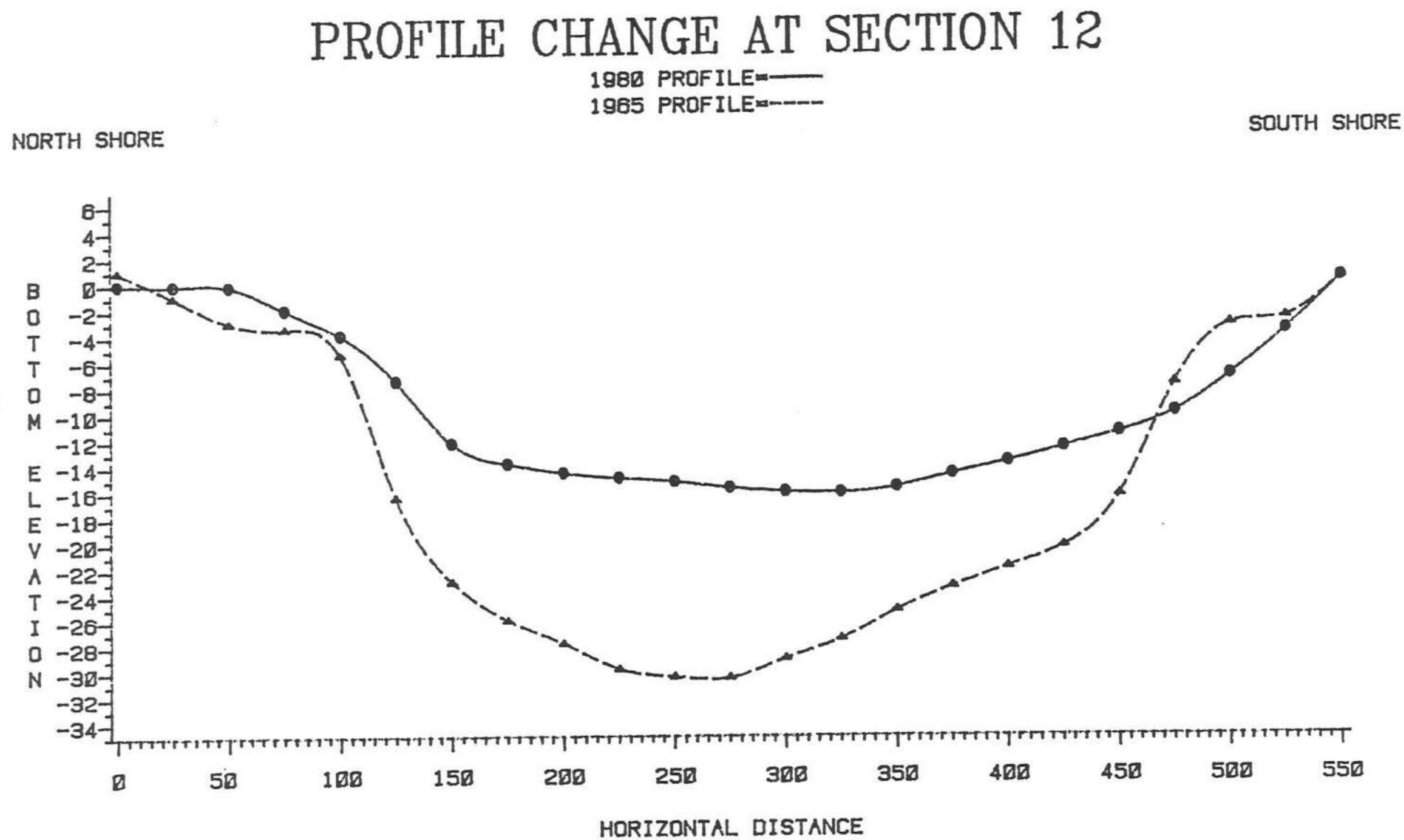

Fig. 24. Profile Change at Section 12 from 1965 to 1980

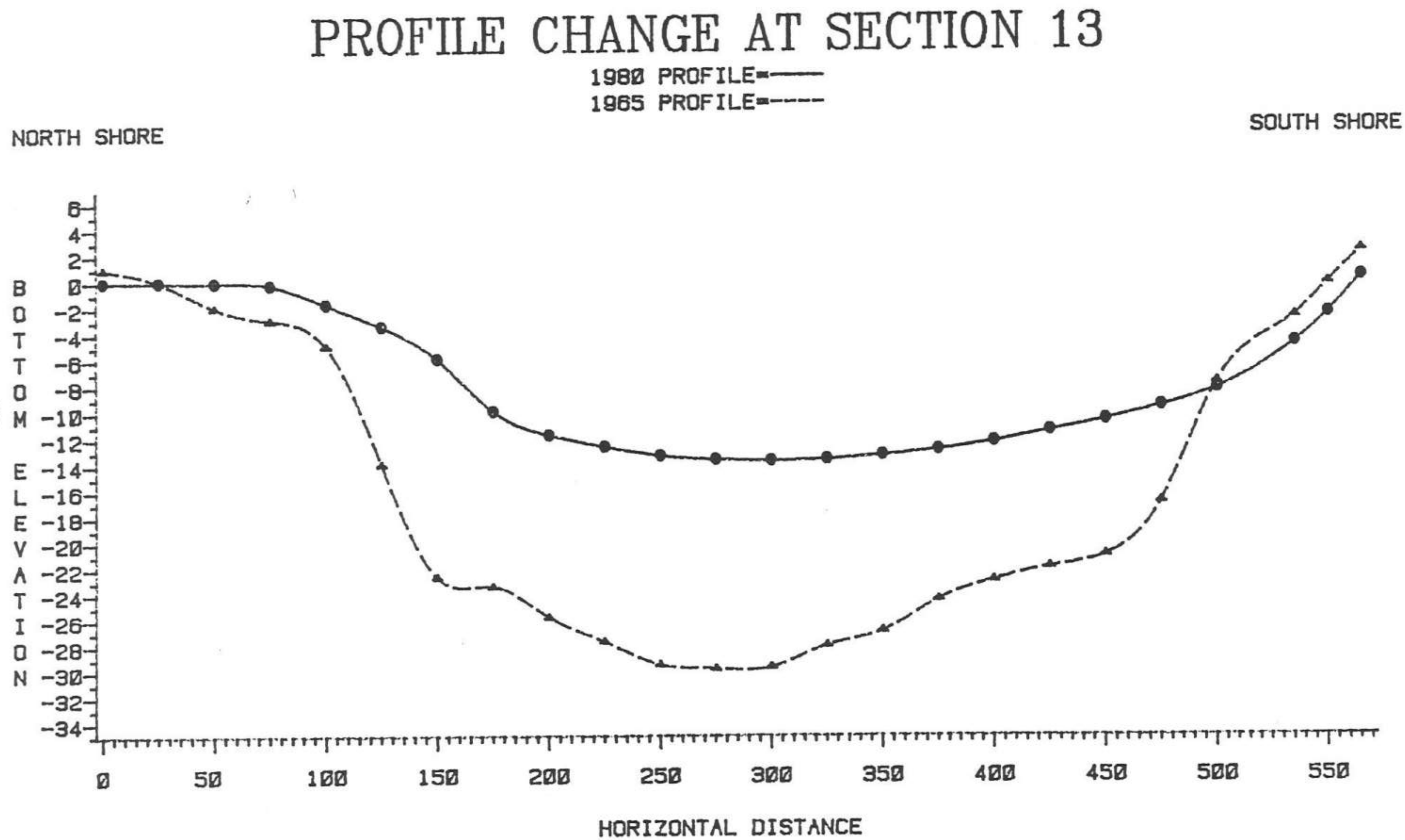

Fig. 25. Profile Change at Section 13 from 1965 to 1980

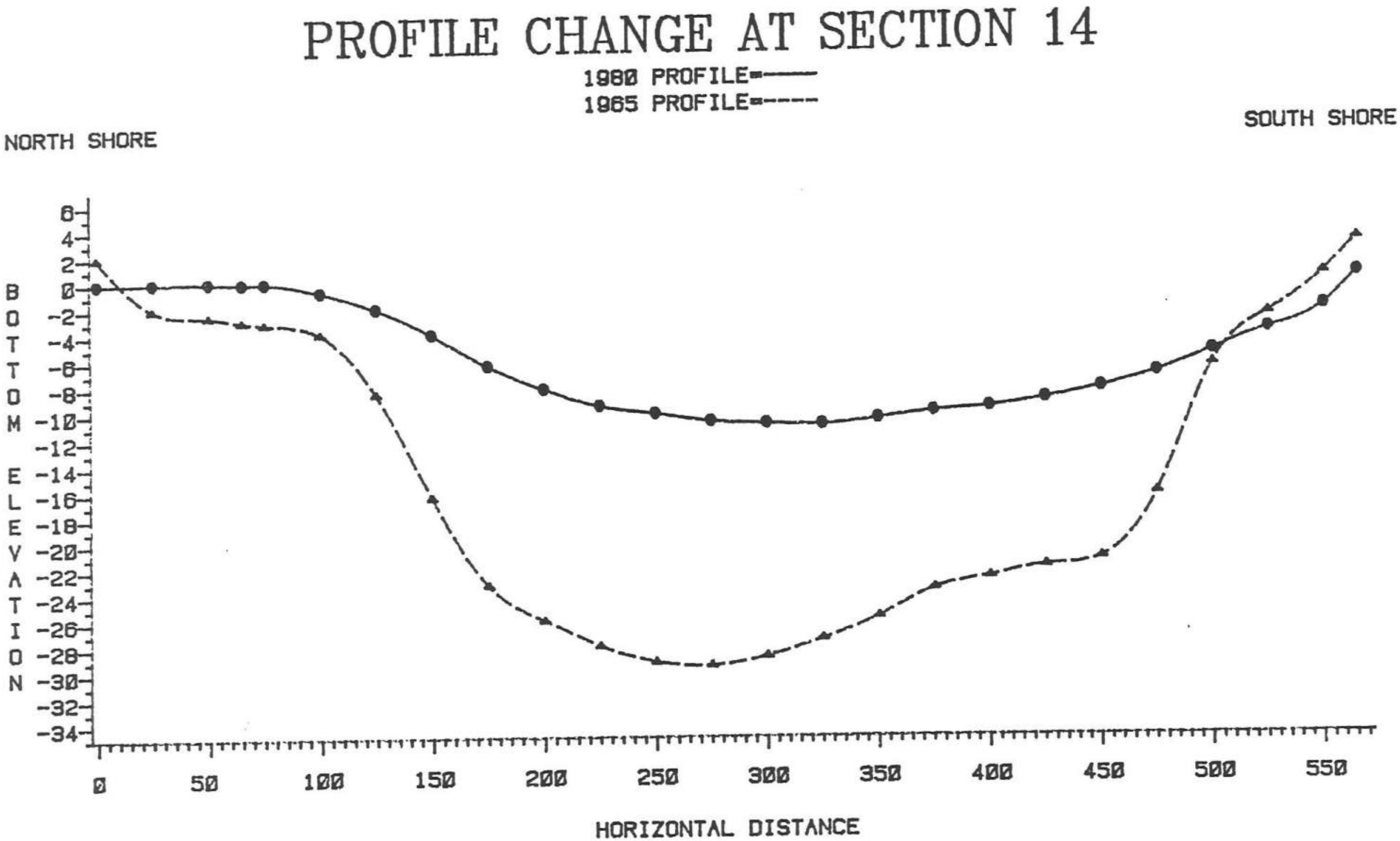

Fig. 26. Profile Change at Section 14 from 1965 to 1980

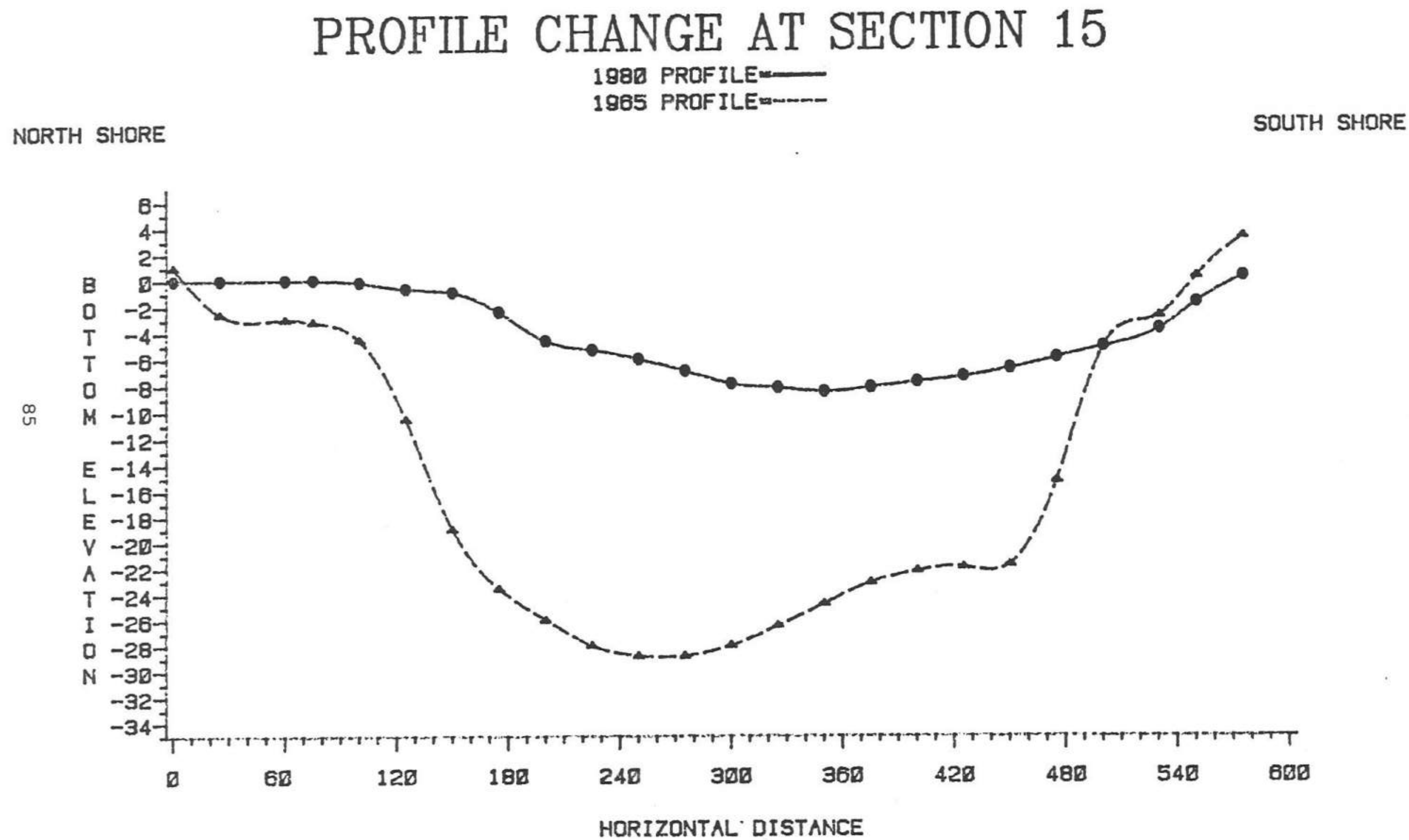

Fig. 27. Profile Change at Section 15 from 1965 to 1980

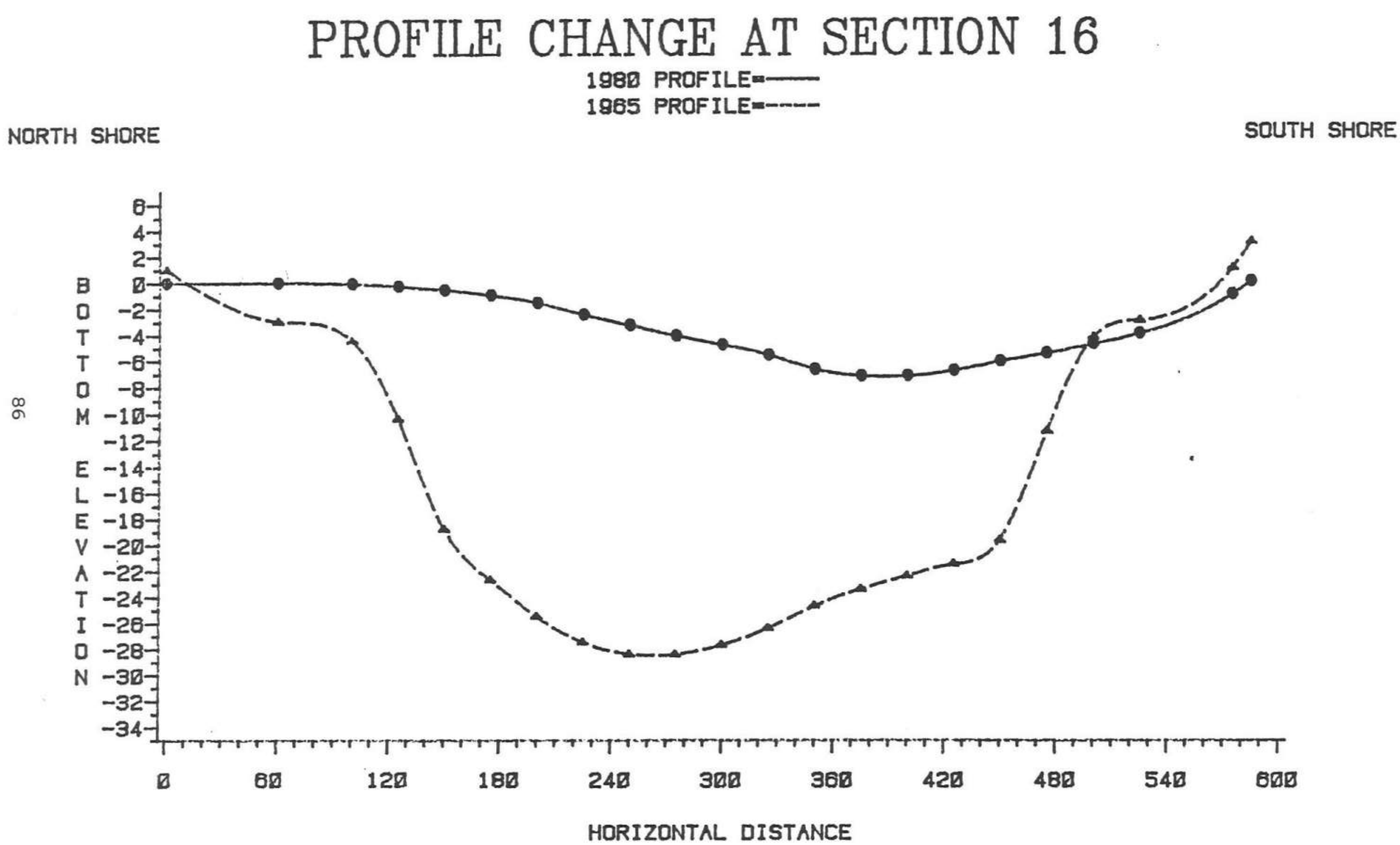

Fig. 28. Profile Change at Section 16 from 1965 to 1980

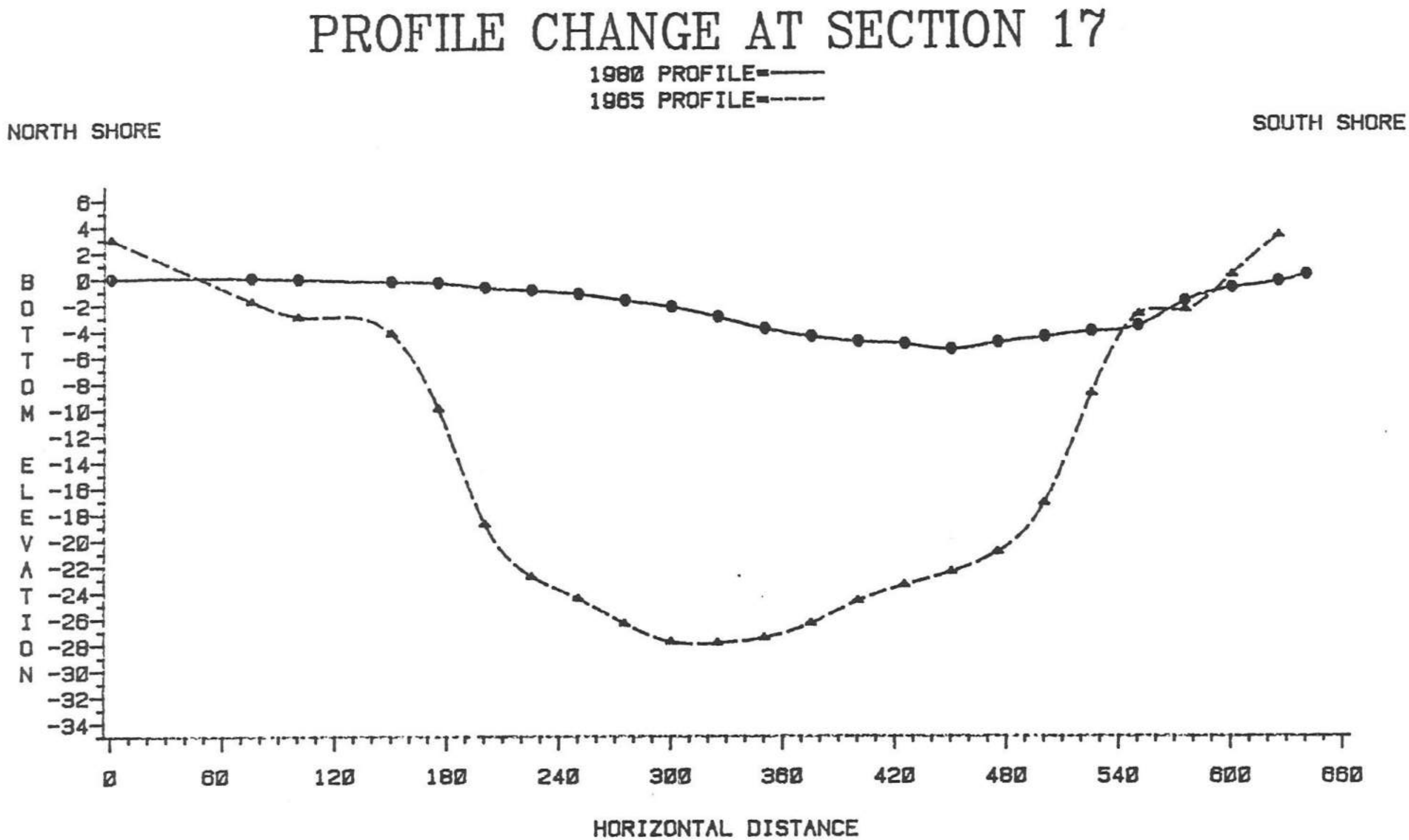

Fig. 29. Profile Change at Section 17 from 1965 to 1980

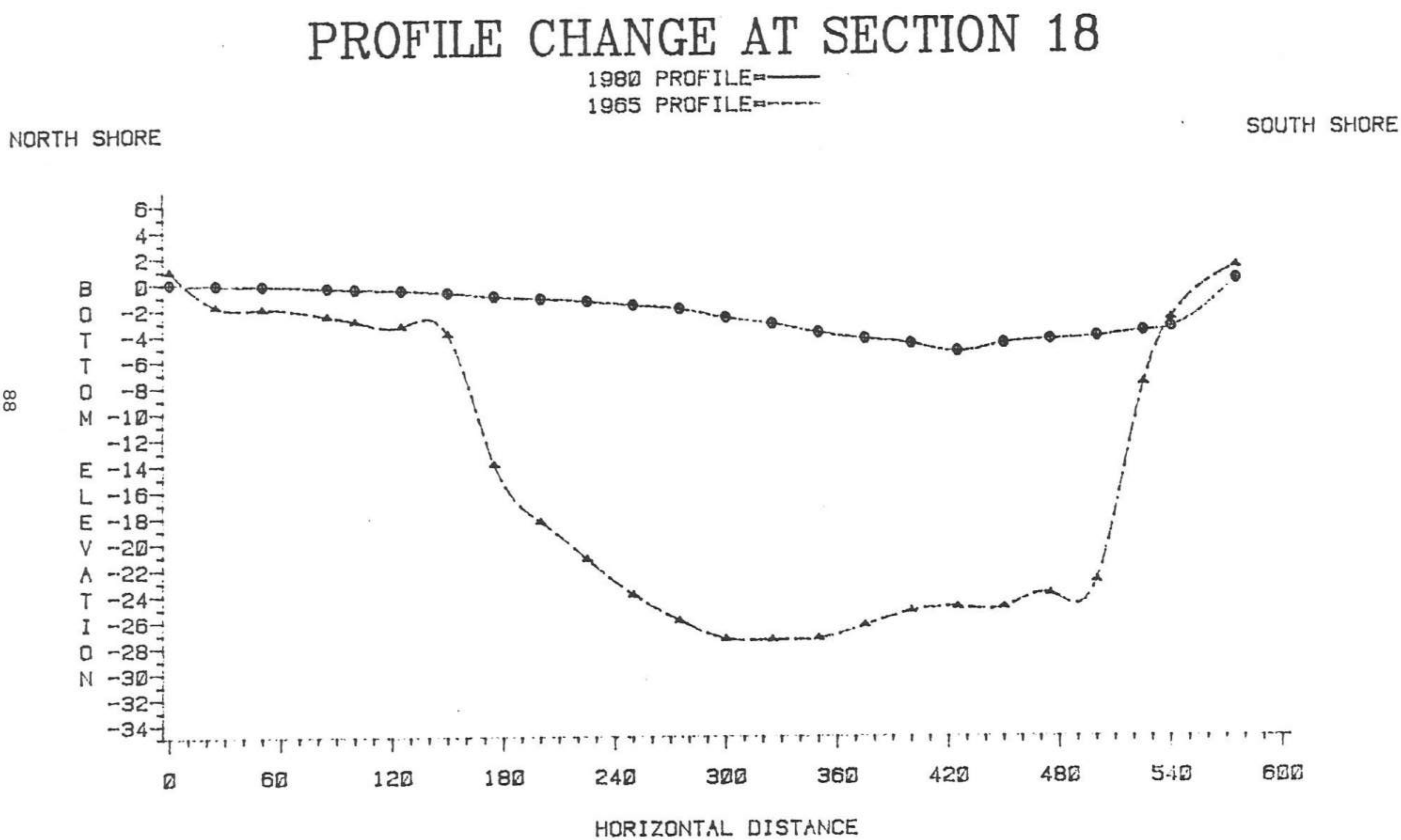

Fig. 30. Profile Change at Section 18 from 1965 to 1980

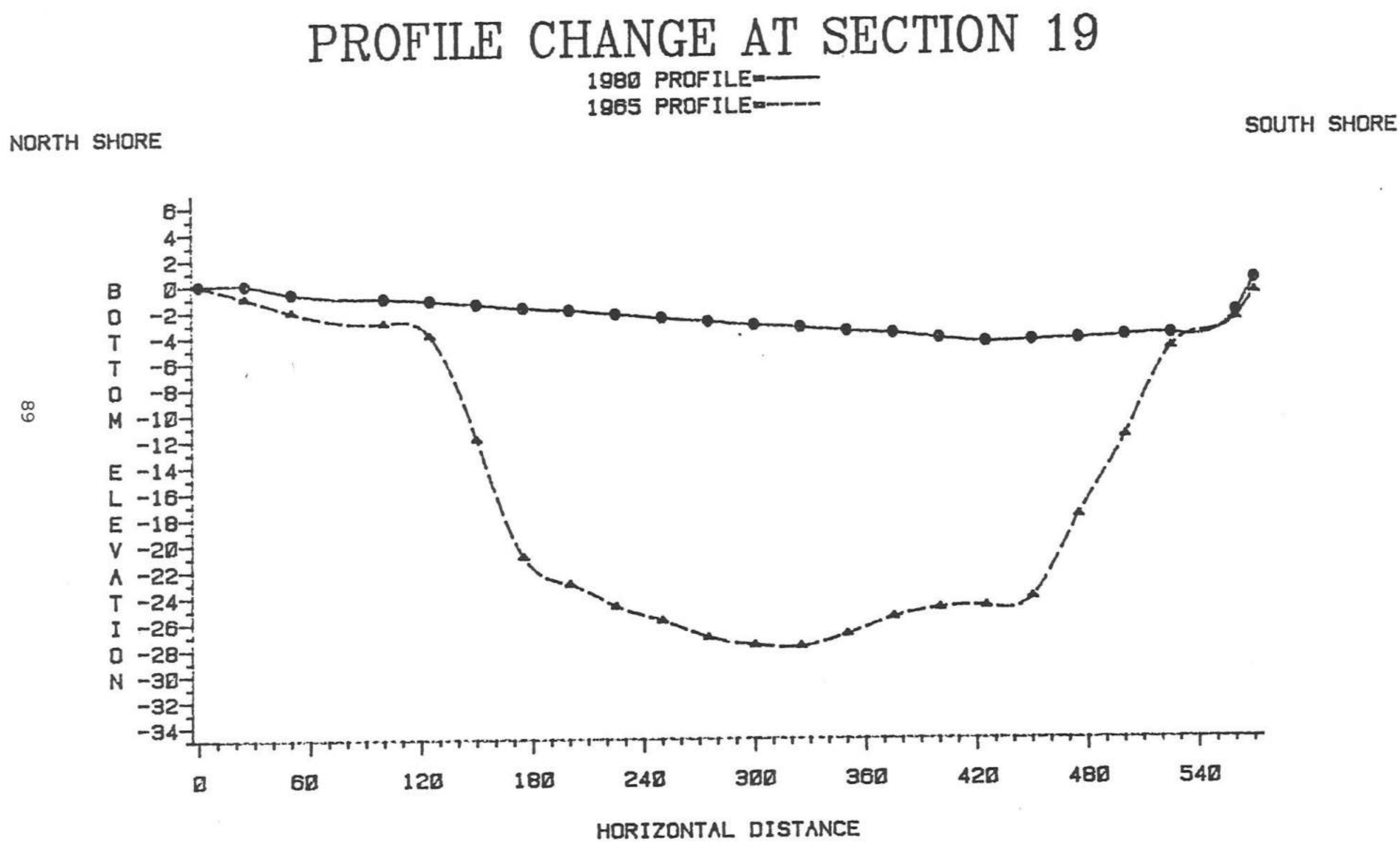

Fig. 31. Profile Change at Section 19 from 1965 to 1980

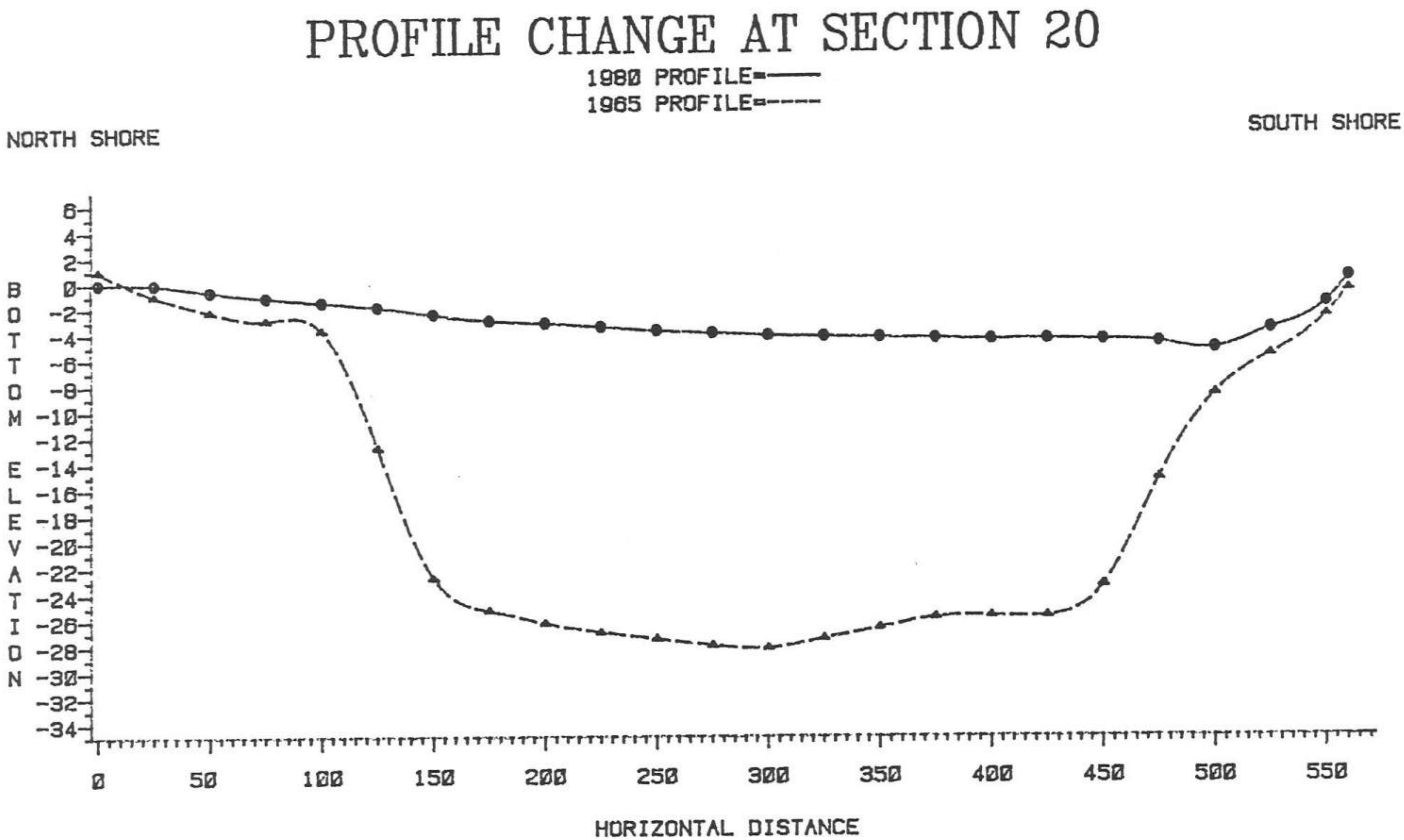

Fig. 32. Profile Change at Section 20 from 1965 to 1980

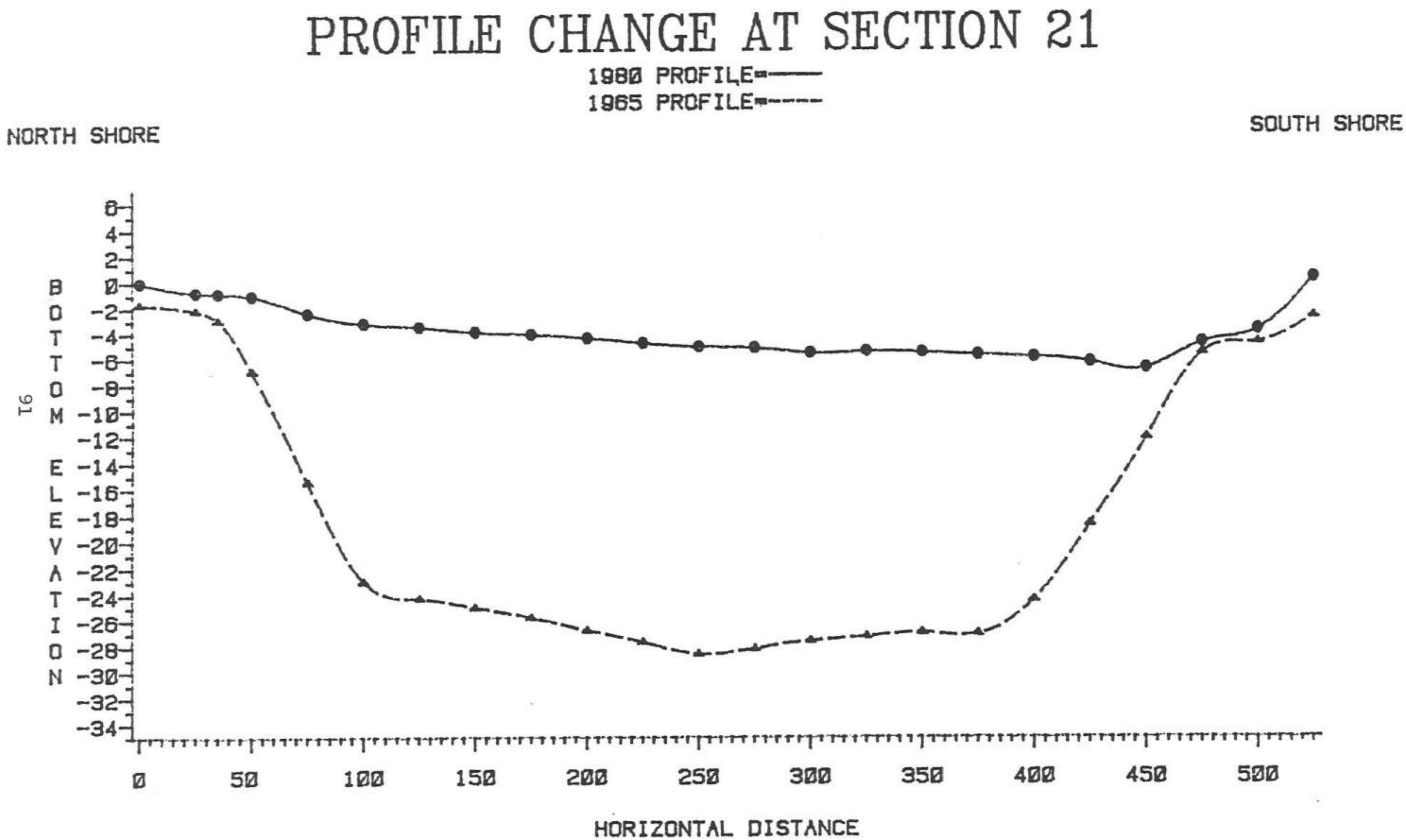

Fig. 33. Profile Change at Section 21 from 1965 to 1980

The maximum sedimentation depth along the middle of the channel increased from about 11 ft at cross section 4 to about 25 ft at cross section 21. Figs. 28-33 show that the area between cross sections 16 and 21 was almost completely filled to its capacity. This area was approximately 1,000 ft long and 350 ft wide. The volume of the sediment deposited between cross sections 16 and 21 was approximately 7.3×10^6 ft^3 on the basis of Table 10. On the other hand, the volume of the sediment deposited between cross sections 1 and 21 was estimated to be 1.3×10^7 ft^3.

As a summary, the bathymetric changes from 1965 to 1980 indicate that the volume of the sediment deposited in the marina area specified in Fig. 12 was approximately 1.3×10^7 ft^3 over 15 years, that is, 8.8×10^5 ft^3/yr. The maximum sedimentation depth in the vicinity of the marina entrance was approximately 25 ft over 15 years, that is, 1.7 ft/yr. Furthermore, the 1,000 ft long and 350 ft wide area of the 1965 channel at the marina entrance was almost completely filled to its capacity. The volume of the sediment deposited in this area was approximately 7.3×10^6 ft^3 over 15 years, that is, 4.9×10^5 ft^3/yr.

4.2 Currents and Waves

Currents and waves cause sediment transport. Simple analyses are performed to examine whether the volume of the sediment deposited in the marina area could be transported by currents and waves.

First, the surface current measurements described in Section 3.2 have indicated that the tidal current velocities in the marina area are relatively small and of the order 0.1 ft/sec. Consequently, wind-induced currents and resulting sediment movement are analyzed in the following.

The shear stress, τ_s, exerted by wind on the water surface may be expressed as

$$\tau_s = K \rho U_{10}^2 \tag{1}$$

in which K = an empirical parameter, ρ = the density of the water, and U_{10} = wind speed at 10 m above the water surface. The value of K may be estimated by (21)

$$\begin{aligned} K &= 0.4\sqrt{U_{10}} \times 10^{-6} \qquad &\text{for } U_{10} < 31 \text{ mph} \\ K &= 3.1 \times 10^{-6} \qquad &\text{for } U_{10} > 31 \text{ mph} \end{aligned} \tag{2}$$

On the other hand, the initiation of sediment movement results from the shear stress, τ_b, acting on the botton sediment. The critical condition for the initiation of sediment may be estimated using the Modified Shields Diagram (12) shown in Fig. 34 where the critical shields parameter Ψ_c is expressed as a function of the parameter S_* defined by

$$S_* = \frac{d}{4\nu} \sqrt{(s-1)gd} \tag{3}$$

in which d = the sediment diameter, ν = the kinematic viscosity of the water, s = the specific gravity of the sediment, and g = the gravitational acceleration. The Shields parameter Ψ is defined as

$$\Psi = \frac{\tau_b}{\rho g(s-1)d} \tag{4}$$

If $\Psi > \Psi_c$, sediment movement will occur. It may be assumed in shallow water such as on the shoal at the marina entrance that $\tau_b \simeq \tau_s$ neglecting wind setup (5). Then, the wind speed U_{10} corresponding to the critical condition for the initiation of sediment movement is given by

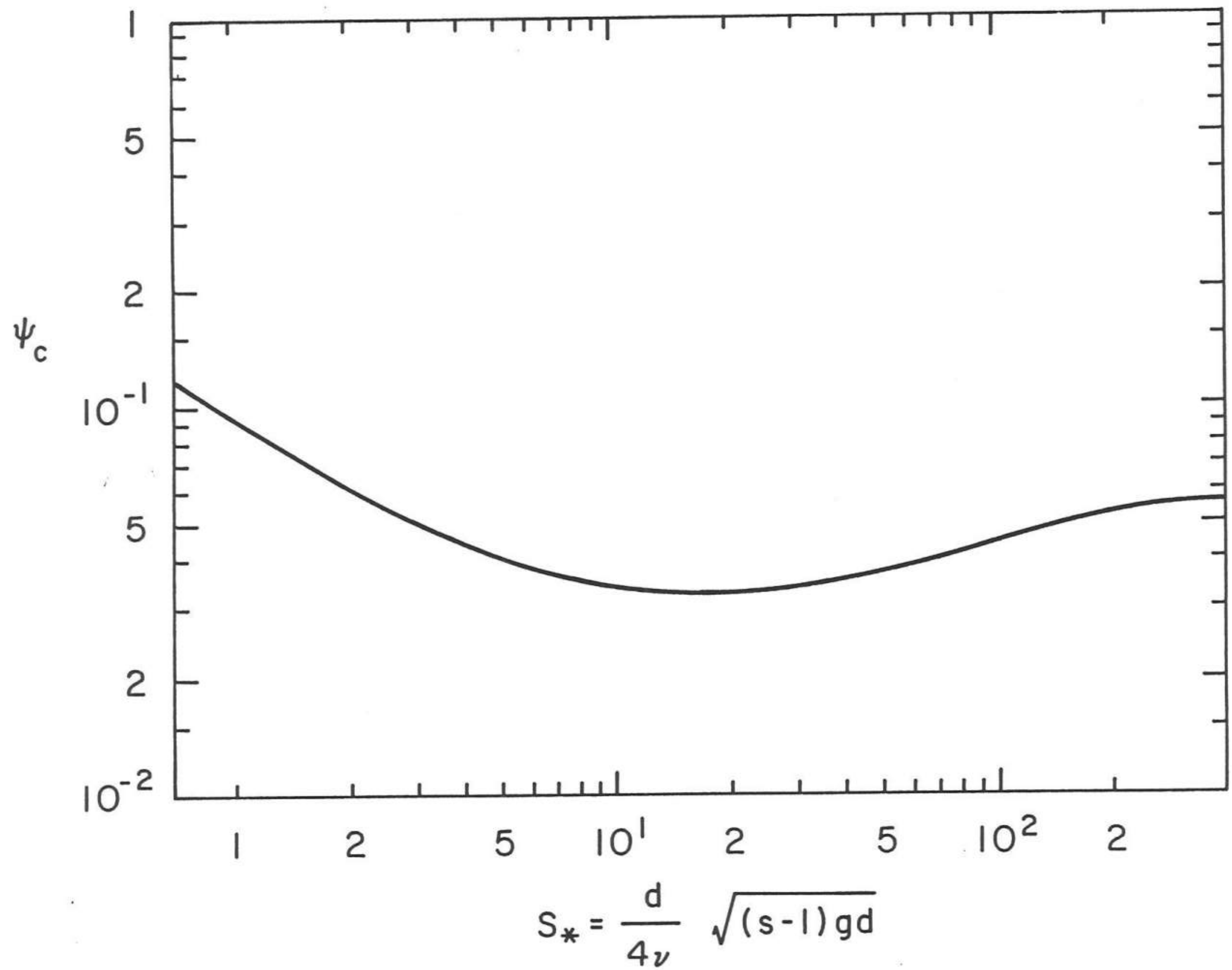

Fig. 34. Modified Shields Diagram for Initiation of Sediment Movement

$$U_{10} = \left[\frac{g(s-1)\ d\ \Psi_c}{K}\right]^{1/2} \quad (5)$$

For quartz sand in fresh water $S \simeq 2.65$ and $\nu \simeq 1.22 \times 10^{-5}\ ft^2/sec$. It should be noted that the salinity of the water in the C & D Canal was observed to be small (4). For given sediment diameter Eq. (3) yields S_* and hence Ψ_c by use of Fig. 34. Eq. (5) predicts the minimum wind speed required for the initiation of movement of the sediment for given diameter where K is given by Eq. (2). Table 11 shows the calculated minimum wind speed for specified particle diameter. The fall velocity of the specified particle is calculated assuming a spherical particle in quiet water (12). Table 11 indicates that wind speed greater than 20 mph is required to initiate the movement of fine sand. Table 11 also lists the velocity corresponding to 3 percent of the minimum wind speed. This velocity is roughly equal to the surface current velocity induced by the specified wind speed. Comparing Tables 2, 6 and 11, it may be concluded that wind-induced currents alone are normally too weak to transport even fine sand. This may explain the deposition of fine sand at the marina entrance and the transport of silt and clay toward the eastern end of the marina area. However, waves need to be taken into account to explain the transport of the fine sand to the marina entrance.

Second, waves can be generated by wind as well as by vessels navigating in the C & D Canal. Wind-generated waves may be predicted using the simple method described in the Shore Protection Manual (5). The length of a straight-line fetch associated with the wind blowing

TABLE 11: Minimum Wind Speed Required for Initiation of Sediment Movement

Particle Diameter (mm)	Fall Velocity (ft/sec)	Minimum Wind Speed Required (mph)	3% of Minimum Wind Speed (ft/sec)
0.074	0.015	20	0.91
0.1	0.026	21	0.92
0.2	0.065	22	0.96
0.3	0.13	23	1.0
0.4	0.18	25	1.1
0.5	0.24	26	1.2
0.7	0.34	30	1.3
1.0	0.49	33	1.4
1.5	0.72	39	1.7
2.0	0.92	46	2.0

toward the eastern end of the marina area is approximately 3,000 ft. Using this fetch length, the wave height and period in deep water for specified wind speed are calculated and tabulated in Table 12. Since the predicted wave period is very small, the assumption of deep water is appropriate except that the predicted wave will refract and shoal as it approaches the beach of the Lums Pond Lagoon. Furthermore, the calculated minimum wind durations required for fetch-limited conditions are so short that wind-induced waves in the marina area are most likely fetch-limited. Wind speed of a sufficient duration in the marina area is expected to be less than 40 mph on the basis of the wind data shown in Table 2. Consequently, the heights of wind-induced waves seldom exceed 0.1 ft and are much less than the heights of waves generated by vessels navigating in the C & D Canal. The heights of vessel-generated waves were observed to exceed 1 ft frequently during the three field surveys conducted in this study. Tomilin (personal communication, 1985) indicated that the design wave height for the riprap revetment along the C & D Canal is 4 ft. As a result, vessel-generated waves are dominant in the marina area. Navigation effects on the bank and shore erosion of inland waterways have been investigated (2, 8, 11, 15). However, there is no method for predicting the amount of the bank and shore erosion due to navigation. In this study, a crude estimate of the amount of sediment transported along the shoreline by the vessel-generated waves in the marina area is made using the empirical formula for the longshore sediment transport rate proposed in the Shore Protection Manual (5). This empirical formula can be shown to be simplified as

$$Q \simeq 0.035 \ H_b^2 \sqrt{gH_b} \sin 2\alpha_b \qquad (6)$$

TABLE 12: Wind-Generated Waves for 3,000 ft. Fetch in Deep Water

Wind Speed (mph)	Wave Height (ft)	Wave Period (Sec)	Minimum Wind Duration (hr)
5	0.010	0.75	0.68
10	0.023	0.96	0.51
20	0.053	1.3	0.38
30	0.088	1.6	0.33
40	0.13	1.8	0.29
50	0.16	1.9	0.26
60	0.21	2.1	0.24

where Q = the longshore sediment transport rate in terms of the volume of deposited sand, H_b = the height of breaking waves, α_b = the angle bewteen the breaking wave crest and the shoreline, and g = the gravitational acceleration. Fig. 35 shows Q (ft^3/hr) as a function of H_b (ft) for α_b = 5°, 15° and 45° where Eq. (6) predicts that Q is the maximum at α_b = 45° for given H_b. The sedimentation rate in the marina area from 1965 to 1980 has been estimated to be 8.8×10^5 ft^3/yr, that is, 2,400 ft^3/day. If we assume that only the sediment deposited in the vicinity of the marina entrance was transported into the marina area, the sedimentation rate is 4.9×10^5 ft^3/yr, that is, 1,300 ft^3/day. Table 5 indicates that on the average 31 vessels per day navigate in the C & D Canal. The waves generated by each vessel are observed to last a few minutes. As a result, the beach in the marina area is exposed to the wave action of approximately one hour on the avearge day. The breaker height H_b is observed to be typically 1 ft as shown in Photo 13. The associated angle α_b appears to be relatively large as shown in Photo 7. Fig. 35 suggests that the corresponding longshore sediment transport rate Q is of the order 500 ft^3/day due to the estimated wave action of one hour duration per day. This estimated rate is the gross longshore sediment transport rate which does not account for the direction of the longshore sediment transport. However, the field observations made in this study have indicated that the vessel-generated waves tend to cause the sediment transport along the shoreline toward the marina site.

As a summary, wind-induced currents and vessel-generated waves are dominant in the marina area. Wind-induced currents alone are normally too weak to transport even find sand. Vessel-generated waves alone may not

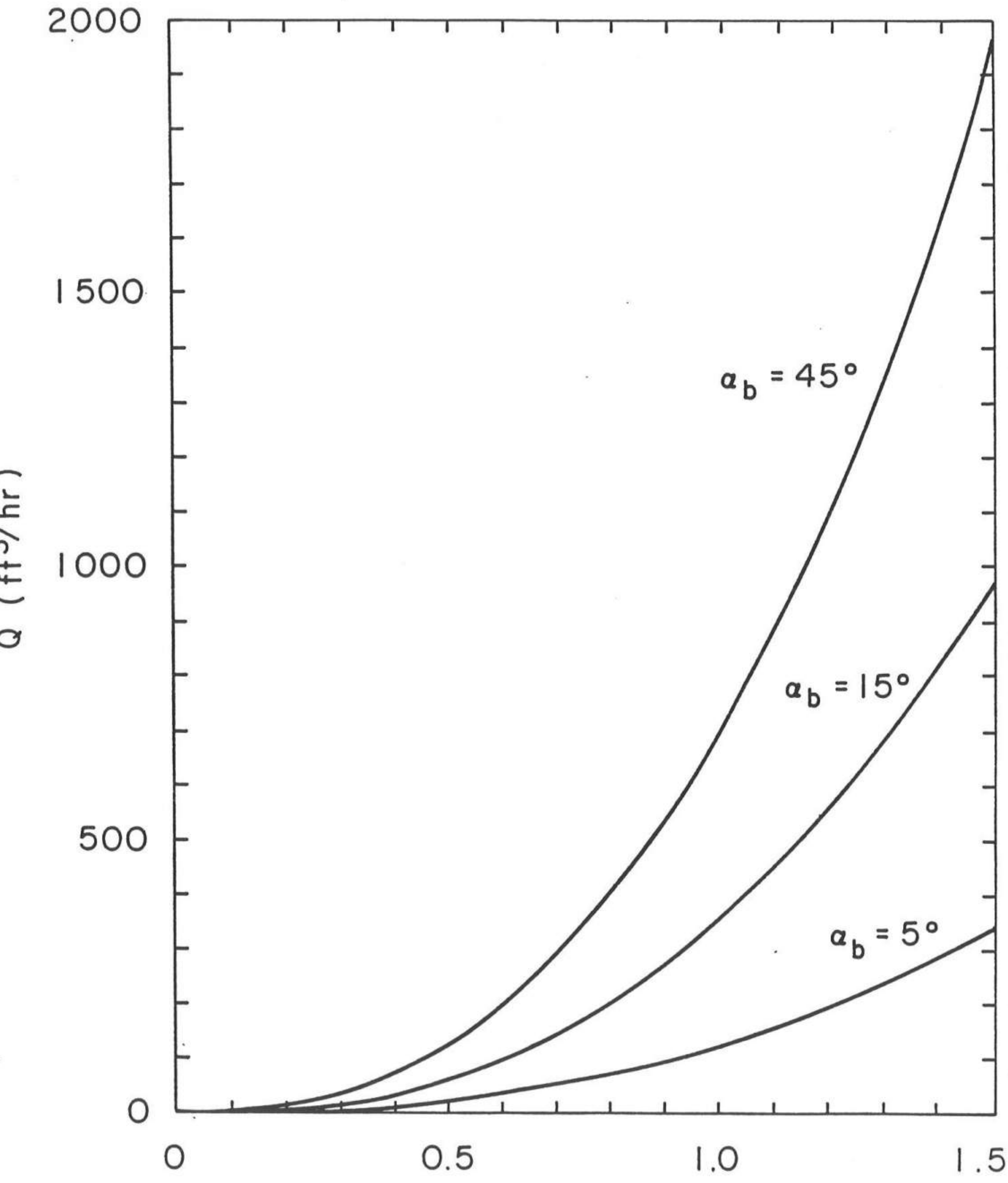

Fig. 35. Longshore Sediment Transport Rate Q as a Function of Breaker Hieght H_b for $\alpha_b = 5°$, 15° and 45°

be sufficient to transport the amount of sediment deposited in the marina area. However, the combined action of wind-induced currents and vessel-generated waves together with tidal currents and wind-generated waves appears to be adequate for transporting the amount of sediment deposited in the marina area. More extensive field data will be required to verify this conclusion.

4.3 Bluff Erosion

The description and photos of the Lums Pond Lagoon given in Section 3.1 and the sediment samples and size distribution discussed in Section 3.3 indicate that the sediment deposited in the marina area has originated mostly from the eroding bluff on the northern shore of the Lums Pond Lagoon. The simple analyses of the sediment transport by currents and waves given in Section 4.3 suggest that the combined wave and current action has apparently transported the sediment eroded from the bluff toward the marina entrance. The bluff erosion is caused by the combined effects of precipitation, slope instability, groundwater seepage, and wave and current action at the toe of the bluff. The field observations have revealed that the toe of the bluff is submerged during the period of high water and occasionally attacked by the vessel-generated waves.

An approximate estimate of the rate of the bluff retreat may be made assuming that the volume of the sediment deposited in the marina area is the same as the volume of the sediment eroded from the bluff. The volume of the deposited sediment between cross sections 1 and 21 shown in Fig. 12 has been estimated to be 1.3×10^7 ft^3 over 15 years in Section 4.1. This volume does not include the volume of the sediment deposited outside cross section 21. Consequently, the length of the eroding bluff is assumed to be approximately 3,000 ft, corresponding to the bluff length along the northern side of the marina area starting from the present riprap revetment of the C & D Canal. In other words, the bluff erosion behind the present riprap revetment is assumed to have resulted in the sedimentation outside cross section 21. The height of the eroding bluff is typically 60 ft on the basis of the 1974 topographic map provided by the Philadelphia District, U. S. Army Corps of Engineers. The corresponding retreat of the eroding bluff is estimated to be 73 ft over 15 years, that is, 4.9 ft/yr. This retreat rate may appear to be large but some of the bluff along the Great Lakes has been observed to retreat at the rate of 30 ft/yr (1). The fallen trees shown in Fig. 4 and Photos 12, 15 and 18 suggest that the estimated bluff retreat is reasonable although the precise determination of the distance of these trees from the toe of the eroding bluff was not possible because of the growth of thick vegetation. It should be noted that the bluff between the riprap revetment and the marina entrance near Station E in Fig. 4 is more exposed to the attack of vessel-generated waves and appears to be eroding faster than the more sheltered bluff east of the marina entrance. The sediment volume of 7.3×10^6 ft^3 deposited in the vicinity of the marina entrance as estimated in Section 4.1 may be assumed to be approximately the same as the volume of the sediment eroded from the exposed segment of the bluff whose length is approximately 1,500 ft. Then, the retreat of the exposed bluff is estimated to be 81 ft over 15 years, that is, 5.4 ft/yr.

4.4 Sedimentation Rates

Fig. 36 illustrates the sediment transport patterns in 1980 inferred from the findings discussed in Sections 4.1, 4.2 and 4.3. The sedimentation depth in the vicinity of the marina entrance has been estimated to be approximately 25 ft from 1965 to 1980, corresponding to the average sedimentation rate of 1.7 ft/yr. In 1980 the entrance area was almost completely filled to its capacity. The shoreline east of the filled entrance area was likely to become relatively stable since the filled entrance behaved like a submerged breakwater. The exposed bluff along the 1,500 ft long shore between the riprap revetment and the marina entrance was estimated to be retreating at the rate of approximately 5.4 ft/yr and supplying the sediment at the rate of approximately 4.9×10^5 ft^3/yr. The sediment eroded from the exposed bluff was probably transported eastward by the combined wave and current action. Most of the transported sediment was likely to be deposited in the vicinity of the marina entrance. Some of the eroded sediment was eventually transported into the C & D Canal. Some of the fine sediment such as clay and silt originating from the eroding bluff was transported in suspension over the filled marina entrance and settled in the enclosed area east of the filled entrance. The sedimentation depth in this enclosed area has been estimated to be approximately 11 ft from 1965 to 1980, corresponding to the average sedimentation rate of 0.7 ft/yr. Consequently, the average sedimentation rates from 1965 to 1980 were in the range 0.7 - 1.7 ft/yr for the area specified in Fig. 12. The sedimentation rates in 1980 for the same area were probably less than the average rates from 1965 to 1980. A crude estimate of the 1980 sedimentation rate in the vicinity of the

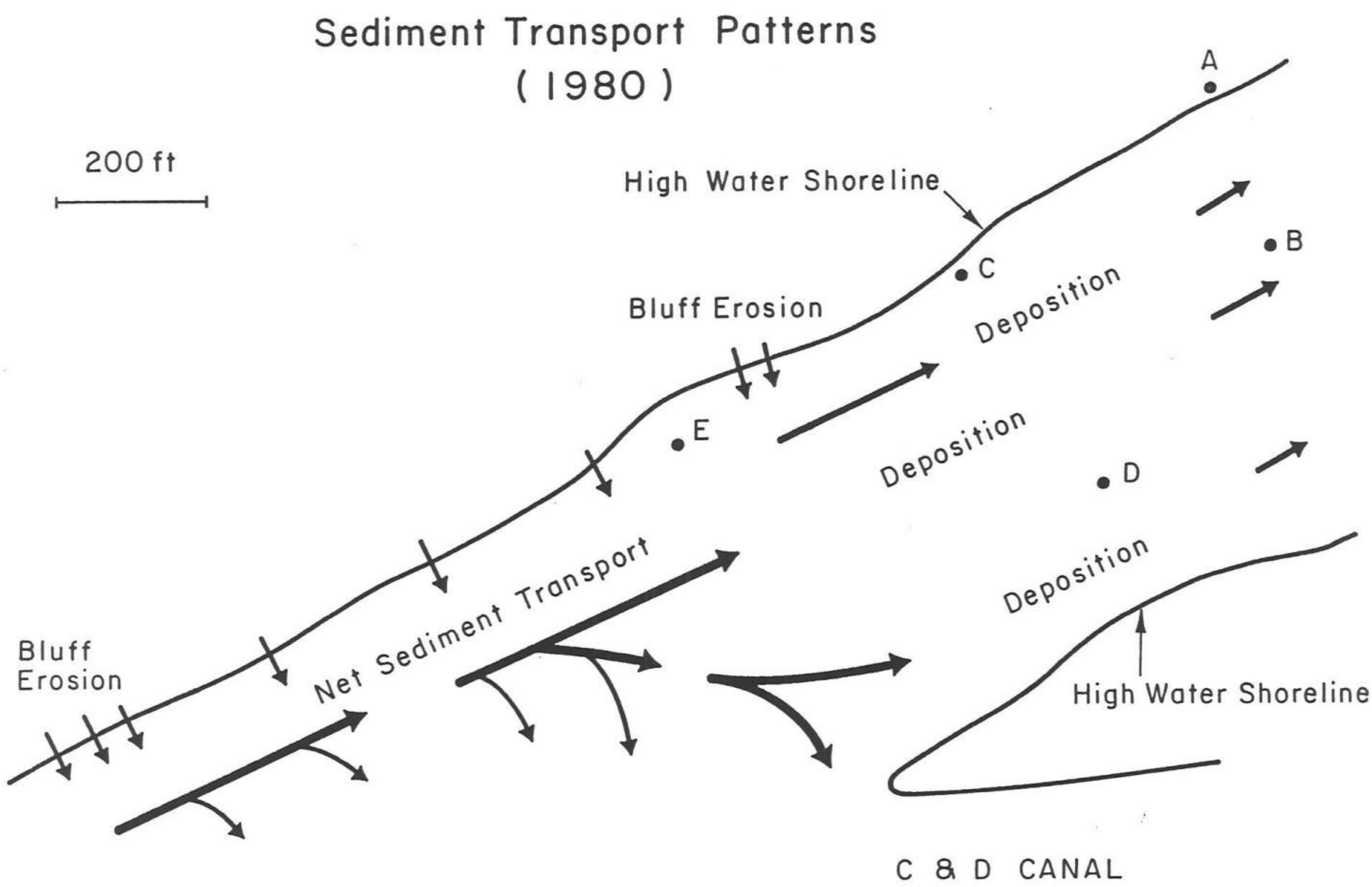

Fig. 36. Estimated Sediment Transport Patterns in Marina Area in 1980

marina entrance may be obtained assuming that the sediment eroded from the exposed bluff was deposited uniformly over the 1000 ft long and 600 ft wide area surrounded by cross sections 16 and 21 and the 1980 shoreline shown in Fig. 12. This assumption yields the 1980 sedimentation rate of 0.8 ft/yr. The sediment transport patterns and sedimentation rates after the dredging of the marina area will be discussed in connection with the maintenance dredging for the marina site.

5. SEDIMENTATION MITIGATION MEASURES

Two dredging operations were conducted in November, 1984 and January, 1985 to increase the water depth of the marina site and connect the marina site with the C & D Canal through a dredged navigation channel of approximately 100 ft width. The hydrographic survey of the dredged area was made in March, 1985. Comparing the 1980 and 1985 hydrographic maps, Williams (personal communication, 1985) estimated that the dredged sediment volume was approximately 1.3×10^6 ft^3. The 1985 map indicates that the water depth of the dredged navigation channel was approximately 10 ft below the Mean Sea Level Datum (NGVD).

5.1 Maintenance Dredging

The dredged marina site and navigation channel will require periodic dredging to maintain the required water depth. The amount and frequency of the maintenance dredging is evaluated in the following. On the basis of the sediment transport patterns in 1980 shown in Fig. 36 and the findings of the field surveys conducted in July and August, 1985, the sediment transport patterns in summer, 1985 are inferred and illustrated in Fig. 37. The major difference between Figs. 36 and 37 is that the dredged navigation channel acts as a sediment trap since the sediment transported along the western edge of the shoal tends to settle into the channel as shown in Fig. 37. Infilling of the dredged channel occurs since the sediment on the relatively steep side slopes of the channel slides down mostly under the action of vessel-generated waves. The measured beach profile changes along the straight lines A-B, C-D and C-E from July 3, 1985 to August 28, 1985 are shown in Figs. 9, 10 and 11, respectively. During this interval of approximately 2 months, the sedimentation along the western side of the shoal was in the range 0.1 - 0.3 ft except that the sedimentation at the tip of the shoal near Station D was large and approximately 0.7 ft. On the other hand, the sedimentation along the eastern side of the shoal was very small except that the sediment at the edge of the shoal tends to slide down into the dredged area.

The volume of the sediment deposited in the vicinity of the marina entrance after the dredging is expected to be approximately the same as that in 1980 since the amount of the sediment eroded from the exposed bluff and transported toward the marina entrance remains approximately the same. In other words, dredging the channel is predicted to have little effect on the erosion and transport processes in the marina area as long as the number of vessels navigating in the neighborhood of the marina area is the same. When the marina becomes operational, the number of vessels will increase but the effects of the small vessels navigating slowly in the marina area are expected to be minor relative to large cargo vessels and fast pleasure boats navigating

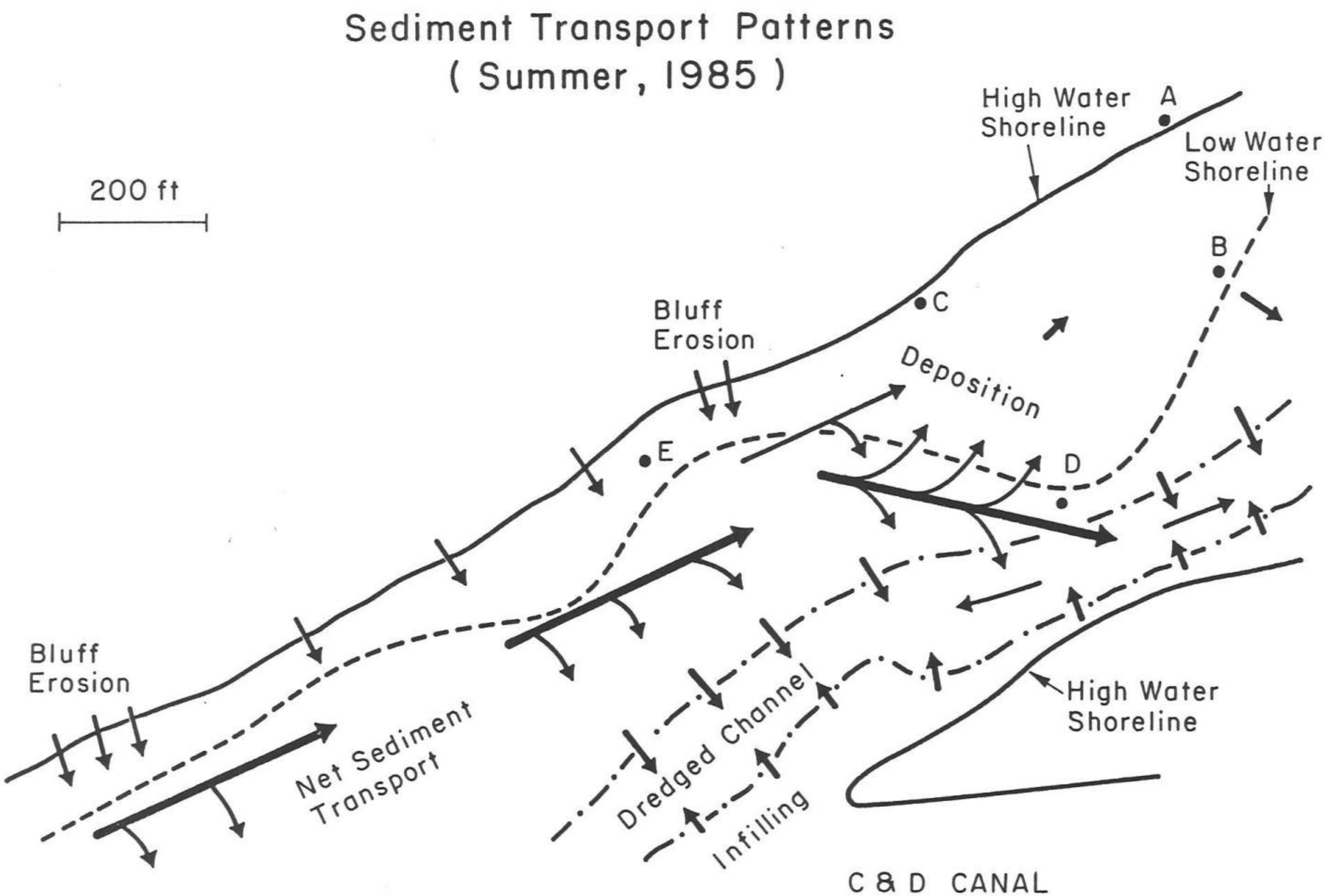

Fig. 37. Estimated Sediment Transport Patterns in Marina Area in Summer, 1985

in the C & D Canal at present. As a result, the volume rate of sediment deposition in the vicinity of the marina entrance is estimated to be approximately 4.9×10^5 ft^3/yr. This value should be interpreted as 5×10^5 ft^3/yr although two digits are specified to facilitate the comparison of the values given in Section 4. This volume of sediment will be deposited non-uniformly in the vicinity of the marina entrance because of the presence of the navigation channel. In the vicinity of the marina entrance, it has been estimated that the average sedimentation rate from 1965 to 1980 was 1.7 ft/yr and the sedimentation rate in 1980 was reduced to 0.8 ft/yr. Figs. 10 and 11 have indicated that the sedimentation rate on the western side of the shoal after the dredging was typically 0.2 ft from July 3, 1985 to August 28, 1985, corresponding to approximately 1.2 ft/yr. The sedimentation rate in the dredged channel will vary along the channel as illustrated in Fig. 37 and may be as large as approximately 5 ft/yr in the vicinity of Station D. Consequently, the sedimentation rate in the dredged channel is estimated to be in the range 1 - 5 ft/yr. The volume rate of sediment deposition in the channel will be less than 4.9×10^5 ft^3/yr since some of the sediment transported to the marina entrance will be deposited outside the dredged channel. However, the area outside the dredged channel will eventually be filled if no dredging is performed. As a result, all the sediment transported to the marina entrance will need to be dredged to maintain the marina entrance as it is.

The estimate of the sedimentation rates of 1 - 5 ft/yr in the dredged channel and the dredging requirement of 5×10^5 ft^3/yr need to be verified by comparing the bathymetric data at least from 1985 to 1986. Since the water depth of the dredged navigation channel was approximately 10 ft, the navigation channel may need to be dredged annually. The segment of the C & D Canal between the Railroad Bridge and the Summit Bridge has been dredged every 1 to 3 years (personal communication with Tomilin, 1985). The dredging of the marina area will reduce the volume of sediment transported to the C & D Canal and hence be beneficial to the maintenance of the C & D Canal.

5.2 Low-Cost Bluff Protection Measures

The exposed bluff along the northern shore of the marina area is presently eroding and supplying the sediment deposited in the vicinity of the marina entrance. As a result, the sedimentation problems associated with the marina operation will be mitigated if erosion of the exposed bluff is reduced. The toe of the exposed bluff may be protected against the combined wave and current action using low-cost shore protection devices suitable for relatively low-energy shorelines (1,6,7). A rule of thumb definition of the low cost is $50 per linear foot for materials if no heavy equipment is needed for installation, or $125 per linear foot for materials, labor and needed equipment at 1975 prices (7). The low-cost devices are regarded to be temporary and may be effective only for several years. Longard tubes or sand bags may be manufactured at the installation site by filling plastic tubes or bags with sand available at the marina area. Fig. 38 shows Longard tubes installed for protecting the toes of the eroding bluff and the eroding grass roots along the northern shore exposed to the waves generated by vessels navigating in the C & D Canal.

Longard Tube Shore Protection

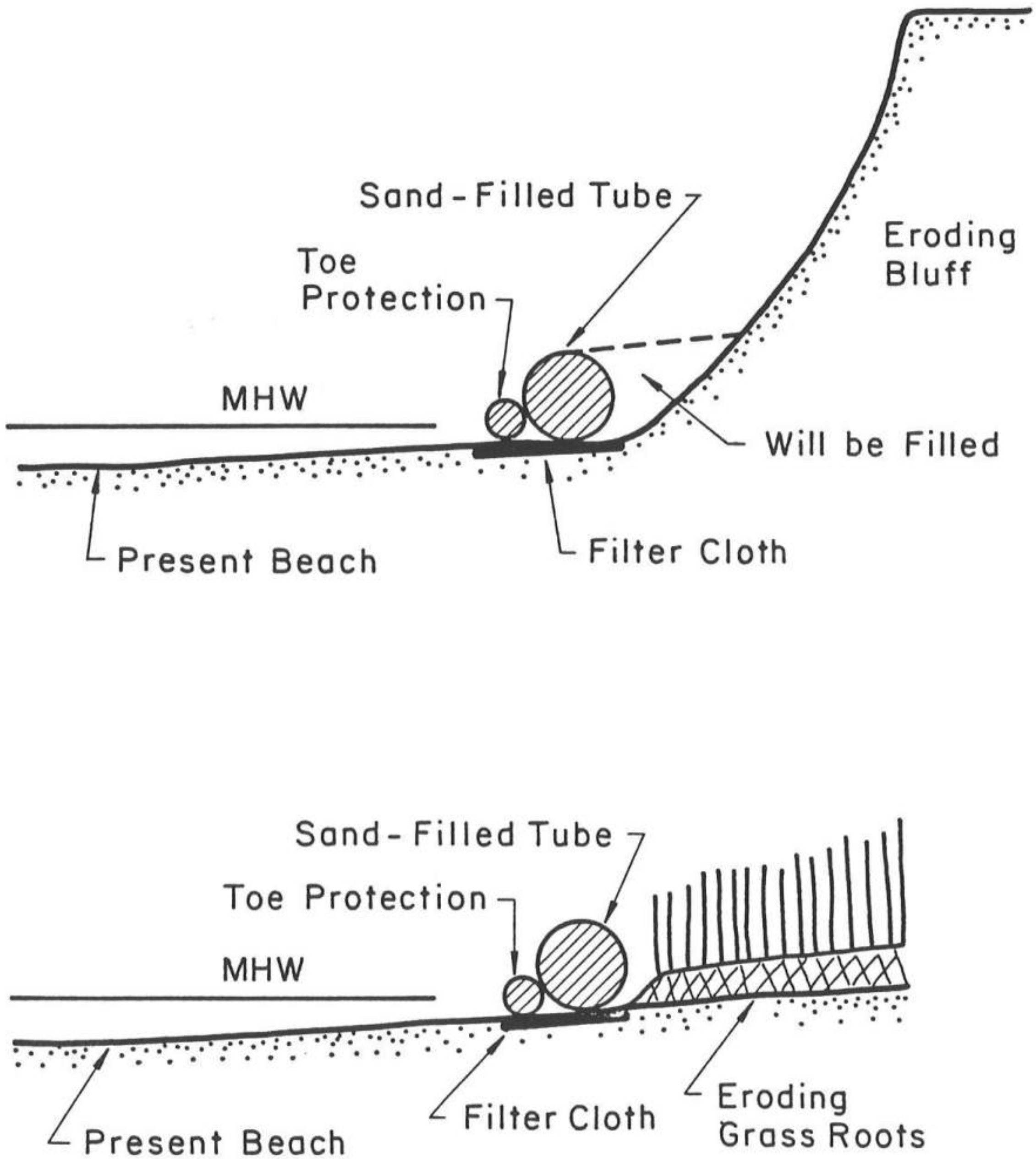

Fig. 38. Longard Tubes Used for Protecting Eroding Bluff and Eroding Grass Roots

The length of the exposed shoreline is approximately 1,500 ft. On the basis of the Longard tube experimental revetment installed in Del Mar, California during the period 1980-1983 (7), a 1.75-m (5.74 ft) Longard tube may be placed on a layer of filter cloth made of synthetic filter fabrics to prevent the settlement of the tube. Various types of synthetic filter fabrics are available for coastal engineering applications (9). A smaller tube may also be placed parallel to and in front of the main tube. The approximate cost for installing the Longard tubes at the marina area may be estimated using the cost of the Longard tube experimental revetment which was $150 per linear foot (1980 prices). The total cost for installing the Longard tube along the 1,500 ft long shore will be approximately $225,000. The Longard tubes will remian effective, that is, reduce the bluff erosion probably for several years. It should be noted that the maintenance dredging of the marina site will still be required although the amount and frequency of the dredging will be reduced significantly.

5.3 Extension of Riprap Revetment

In order to provide more permanent protection along the shore of the marina area, the riprap revetment along the C & D Canal may be extended at least along the exposed northern shore whose length is approximately 1,500 ft. The riprap revetment may eventually need to be extended all the way around the marina area if long-term erosion of the unprotected shore inside the marina entrance is found to be unacceptable.

A preliminary design of the riprap revetment is performed on the basis of the available information on the riprap revetment along the

C & D Canal as well as the design guidelines given in the Shore Protection Manual (5). The design wave height H is taken to be H = 4 ft which is the design wave height used for the riprap revetment along the C & D Canal (personal communication with Tomilin, 1985). A preliminary design of the weight of an individual riprap stone in the primary cover layer of the riprap revetment is made using the following empirical formula (5)

$$W = \frac{w_r H^3}{K_D (S_r - 1)^3 \cot \theta} \tag{7}$$

where W = the weight of an individual armor unit in the primary cover layer, w_r = the unit weight of the armor unit, H = the design wave height, S_r = the specific gravity of the armor unit, θ = the angle of the riprap revetment slope measured from horizontal in degrees, K_D = an empirical stability coefficient. $K_D \simeq 2$ for rough angular quarry stone with n = 2 placed randomly for $\cot \theta = 1.5 - 3$ in which n = the number of units comprising the thickness of the primary cover layer. In this preliminary design $\cot \theta = 1.5$ is assumed as shown in Fig. 39 since the total weight of riprap required for the riprap revetment decreases as $\cot \theta$ is decreased. The riprap revetment along the C & D Canal has the slope of approximately 1 on 1.5. The unit weight of the quarry stone is taken as $w_r = 165$ lb/ft^3. Correspondingly, $S_r = 2.64$ assuming that the water in the marina area is almost fresh water (4). Substitution of the assumed values into Eq. (7) yields $W \simeq 800$ lb. Tomilin (personal communication, 1985) indicated that quarry stone with $W \simeq 1{,}500$ lb was used for the recent maintenance of the riprap revetment along the C & D Canal. Eq. (7) indicates that $W \simeq 1{,}500$ lb approximately corresponds to $H \simeq 5$ ft. However, Eq. (7) was developed

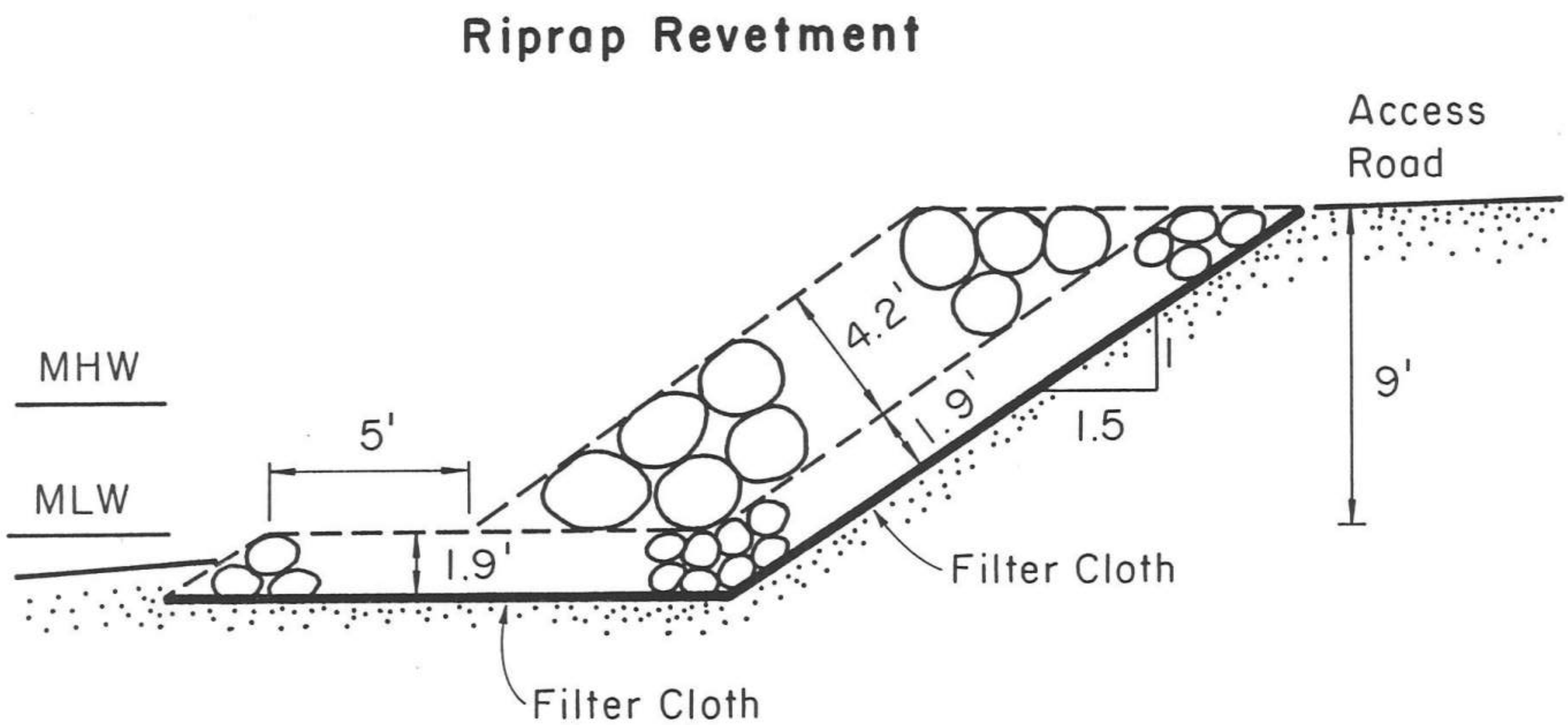

Fig. 39. Riprap Revetment for Protecting Northern and Southern Shores of Marina Area

for the design against wind-generated waves in the coastal environment rather than against vessel-generated waves in a canal. As a result, $W = 1{,}500$ lb is recommended to be used for the riprap revetment along the exposed northern shore of the marina area. Smaller quarry stone may be used inside the marina area where the action of vessel generated waves is less severe.

Fig. 39 shows the proposed cross section of the riprap revetment. The primary cover layer consists of quarry stone of $W = 1{,}500$ lb and its thickness is 4.2 ft. The average layer thickness, r, is estimated using the following formula (5).

$$r = n\, k_{\Delta} \left(\frac{W}{w_r}\right)^{1/3} \tag{8}$$

where n = the number of quarry stone in thickness comprising the layer and k_{Δ} = the layer coefficient which is unity for rough quarry stone. It should be noted that $n = 2$ is assumed to determine the value of K_D in Eq. (7). The weight of an individual quarry stone in the underlayer beneath the primary cover layer is taken to be W/10, that is, 150 lb. The thickness of the underlayer is calculated using Eq. (8) with $n = 2$, $k_{\Delta} = 1$, $W = 150$ lb and $w_r = 165$ lb/ft^3. The underlayer is extended 5 ft from the toe of the primary cover layer to protect the toe and prevent the quarry stone in the primary cover layer from sliding down. A layer of permeable filter cloth is placed between the underlayer and the bluff soil comprised of essentially fine sand.

The crest elevation of the riprap revetment is designed to be 5.3 ft above the Mean High Water (MHW) and 9 ft above the Mean Low Water

(MLW) where the mean tide range in the marina area is estimated to be 3.7 ft as shown in Table 4. The spring tide range is 0.4 ft greater than the mean tide range on the basis of Table 3. It is assumed in Fig. 39 that the beach erosion in front of the riprap revetment caused by termination of the sediment supply from the eroding bluff will eventually lower the beach elevation. The present beach in front of the riprap revetment along the C & D Canal is comprised of the consolidated clay which appears to resist erosion as shown in Photo 23. Test pits or borings need to be conducted for a more reliable determination of the equilibrium beach profile in front of the proposed riprap revetment.

The number , N_r, of individual quarry stones required for a specific layer is estimated using the following formula (5).

$$N_r = A\ n\ k_\Delta (1-P) \left(\frac{w_r}{W}\right)^{2/3} \tag{9}$$

where A = the surface area of the specific layer and P = the average porosity of the layer. It is assumed that $k_\Delta = 1$, $P = 0.37$ and $w_r = 165\ \mathrm{lb/ft^3}$ for rough quarry stone. For the primary cover layer per linear foot shown in Fig. 39, A = 16 ft, n = 2 and W = 1,500 lb, so that Eq. (9) yields $N_r = 4.7$ per linear foot. The required weight of the primary cover quarry stone is thus given by $WN_r = 3.5$ ton per linear foot. On the other hand, for the underlayer per linear foot, A = 25 ft, n = 2 and W = 150 lb, so that $N_r = 33.5$. The required weight of the underlayer quarry stone is 2.5 ton per linear foot. The cost of the 20,000 ton quarry stone purchased for the recent maintenance of the riprap revetment along the C & D Canal was $250,000, that is, $12.5 per ton (personal communication with Tomilin, 1985). This cost is less than the typical cost of quarry stone, that is, approximately

$20 per ton given in the Shore Protection Manual (5). Assuming that both 1,500 lb quarry stone and 150 lb quarry stone cost $12.5 per ton, the cost of quarry stone alone is $75 per linear foot of the proposed riprap revetment. Additional costs such as labor, needed equipment and site preparation must be included to estimate the total cost per linear foot which will exceed $150 per linear foot associated with the Longard tube revetment. The length of the revetment is approximately 1,500 ft along the exposed northern shore and about 6,000 ft along the entire unprotected shore. Since construction of the riprap revetment from the water side may be difficult because of shallow water depths along the northern shore of the marina area, construction of an access road above the riprap revetment may be required as shown in Fig. 39. This access road will also be useful for maintenance repairs of the constructed riprap revetment.

5.4 Construction of Jetty at Marina Entrance

Another alternative for mitigating the sedimentation problem at the marina site is to construct a rubble-mound jetty normal to the northern shore near the marina entrance as shown in Fig. 40. The jetty will trap the sediment transported toward the marina entrance. An updrift fillet of the deposited sediment will be formed on the western side of the jetty. After an approximate equilibrium shoreline is established, the sediment will be transported around the jetty. Some of the sediment will also be transported over and through the rubble-mound jetty. Fig. 40 indicates that the length of the shore-normal jetty should not exceed approximately 350 ft because of the presence of the dredged navigation channel. Since a longer jetty will trap more sediment, the length of the jetty is assumed to be 350 ft. The sediment will tend to be transported toward the dredged channel after the

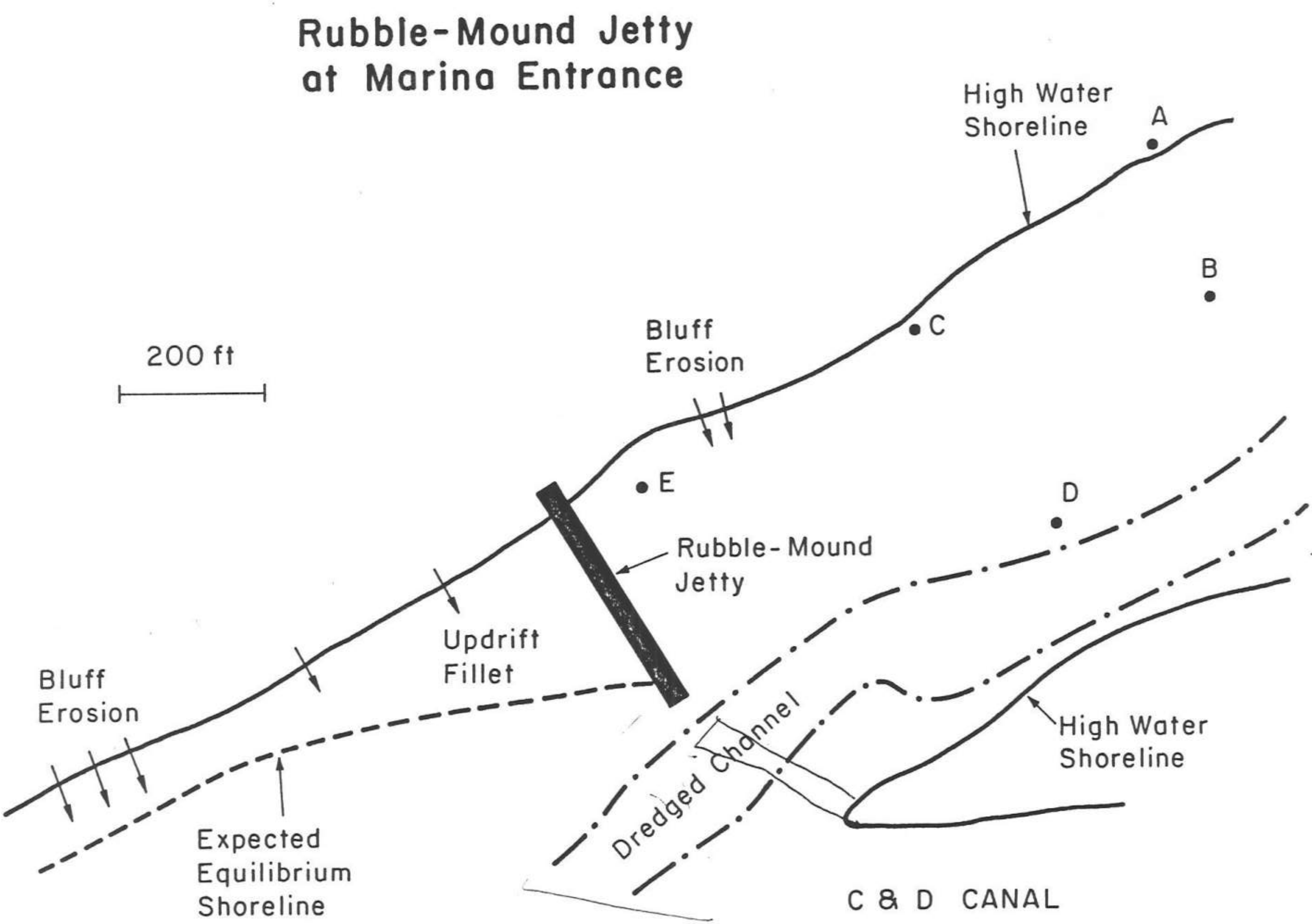

Fig. 40. Rubble-Mound Jetty at Marina Entrance

equilibrium shoreline is established. In order to reduce the volume of the sediment deposited in the dredged channel, the jetty may be constructed west of Station E located opposite to the tip of the southern shore of the marina entrance as shown in Fig. 40.

Fig. 41 shows the proposed profile of the rubble-mound jetty. The water depth at the tip of the 350 ft long jetty will be approximately 7 ft on the basis of the 1985 hydrographic map explained in Section 2.6. The crest elevation of the jetty is designed to be 5 ft above MHW so as to prevent significant overtopping of the design wave of 4 ft height. Fig. 42 shows the typical cross section of the rubble-mound jetty. Since the bottom elevation varies along the jetty, the cross section arrangements need to be adjusted. The proposed cross section of the jetty is basically the same as that of the riprap revetment discussed in Section 5.3 except that the core of the jetty consists of quarry run with the weight of individual units in the range 0.25 - 7.5 lb. The marina side of the jetty may be protected by smaller quarry stone if it reduces the cost. The minimum width of the crest of the jetty should be 6.3 ft corresponding to the combined widths of three stones (5). It should be emphasized that the preliminary design of the jetty shown in Figs. 40-42 does not consider soil mechanics aspects of the jetty design such as the bearing capacity and consolidation of the foundation underneath the jetty. Test borings must be made to ensure appropriateness of the foundation.

The jetty will be effective in reducing the sedimentation in the navigation channel as long as it traps the sediment on the updrift side. Consequently, the effectiveness of the jetty may be evaluated by estimating

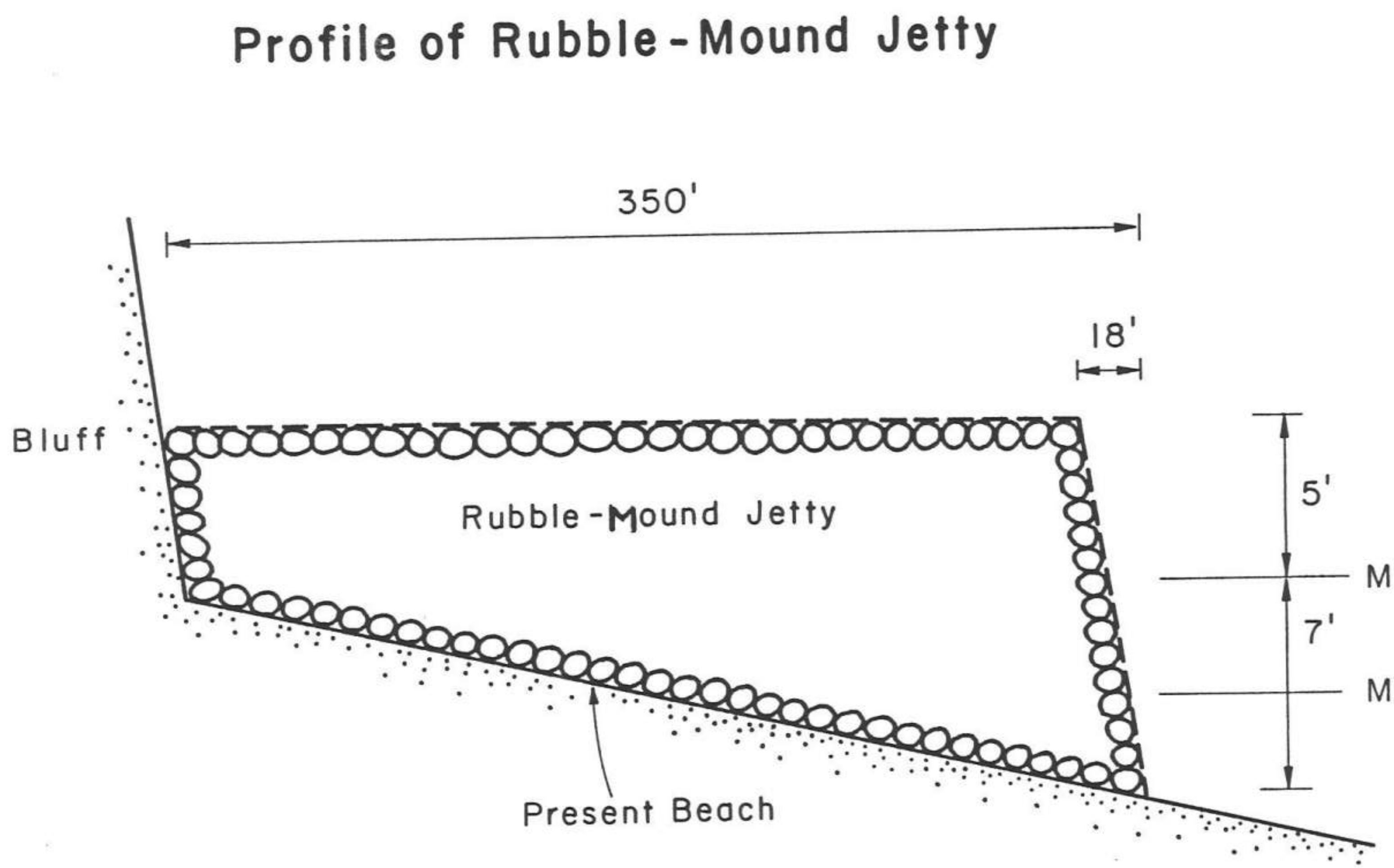

Fig. 41. Profile of Rubble-Mound Jetty

Cross Section of Rubble-Mound Jetty

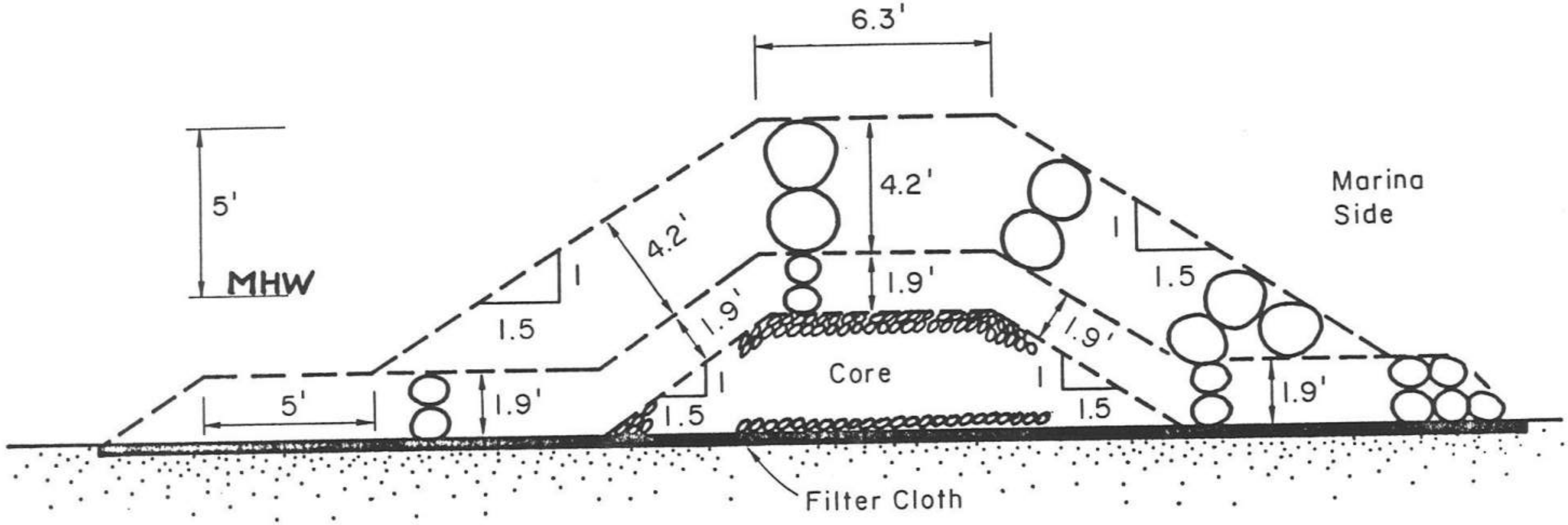

Fig. 42. Cross Section of Rubble-Mound Jetty

the maximum volume of the sediment trapped in the updrift fillet shown in Fig. 40. A crude estimate of this maximum volume may be made assuming a triangular fillet of uniform thickness. The shore-normal length of the triangular fillet is at most 350 ft. The associated alongshore length is expected to be less than approximately 1,000 ft since the riprap revetment along the C & D Canal limits this length. Fig. 41 suggests that the maximum increase of the beach elevation is likely to be less than 5 ft. As a result, the maximum volume of the updrift fillet will be less than 1.75×10^6 ft^3. Since the erosion rate of the exposed bluff has been estimated to be 5×10^5 ft^3/yr in Section 5.1, the updrift fillet will be filled to its capacity within approximately 3.5 years. Since the 350 ft long rubble-mound jetty roughly corresponds to the 700 ft riprap revetment in terms of required weight of quarry stone, it may be concluded that the rubble-mound jetty will not be as effective as the riprap revetment. It should be noted that the length of the eroding bluff surrounding the marina area is very short relative to the long shoreline along the Atlantic coast. Consequently, protection of the eroding bluff of relatively short length is more direct and reliable than trapping the eroded sediment without eliminating the source of the sediment supply.

6. CONCLUSIONS AND RECOMMENDATIONS

Available data on the environmental factors affecting the sedimentation of the Lums Pond Lagoon has been collected and synthesized in Section 2. The findings of three field surveys conducted on July 3, July 30 and August 28, 1985 have been summarized in Section 3. Using the results given in Sections 2 and 3, simple analyses have been performed to determine the sediment transport patterns in the marina area prior to the dredging of the marina site conducted in November, 1984 and January, 1985. The simple analyses have indicated that the sediment deposited at the marina entrance originates from the bluff along the northern shore of the Lums Pond Lagoon. The bluff erosion due to the combined effects of precipitation, slope instability, groundwater seepage and wave action at the toe of the bluff introduces sediment into the beach along the northern shore of the Lums Pond Lagoon. The sediment resulting from the bluff erosion is transported eastward along the northern shore and deposited at the marina entrance mainly by the combined action of vessel-generated waves and wind-driven currents.

The bathymetric changes from 1965 to 1980 have indicated that the volume of the sediment deposited in the marina area was approximately 1.3×10^7 ft^3 over 15 years, that is, 8.8×10^5 ft^3/yr. The corresponding sedimentation rate at the marina entrance was 1.7 ft/yr. This volume of sediment was equal to that associated with 73 ft retreat of the 60 ft high bluff over the 3,000 ft stretch on the northern side of the Lums Pond Lagoon. The 73 ft bluff retreat over 15 years corresponded to the average retreat rate 4.9 ft/yr. In 1980 the marina entrance was almost filled to its capacity and the approximately 1,500 ft stretch of the bluff between the marina entrance and the riprap revetment along the C & D Canal was exposed to the direct attack of vessel-generated waves. The retreat rate of the exposed bluff was estimated to be approximately 5.4 ft/yr by considering only the volume of sediment deposited in the vicinity of the marina entrance from 1965 to 1980. The 1,500 ft long exposed bluff retreating at the rate of 5.4 ft/yr supplied the sediment to the marina entrance at the rate 4.9×10^5 ft^3/yr and caused the sedimentation at the rate of approximately 0.8 ft/yr.

Two dredging operations were conducted in November, 1984 and January, 1985 to increase the water depth of the marina site and connect the marina site with the C & D Canal through a dredged navigation channel of approximately 100 ft width along the southern shore of the Lums Pond Lagoon. Since the dredged channel has little effects on the erosion of the exposed bluff, the volume rate of the sediment deposition in the vicinity of the marina entrance will remain essentially the same as that in 1980. As a result, the volume of sediment deposited at the rate of approximately 5×10^5 ft^3/yr will need to be dredged to maintain the marina entrance in the present conditions. The sedimentation rate in the dredged channel will vary along the channel and be in the range 1 - 5 ft/yr. These estimates should be verified by comparing the bathymetric data at least from 1985 to 1986. Since the water depth of the dredged channel was approximately 10 ft, the channel may need to be dredged annually. It should be noted that the segment of the C & D Canal between the Railroad Bridge and the Summit Bridge has been dredged every 1 to 3 years. Most of the sediment deposited at the marina entrance would have been transported to the C & D Canal if the marina entrance filled to its capacity had not been dredged. It is hence expected that the dredging requirement for the C & D Canal will be reduced because of the dredging of the marina entrance.

In order to reduce the volume and interval of the dredging required for the maintenance of the marina site, use could be made of low-cost shore protection devices suitable for relatively low-energy shorelines. Longard tubes could be manufactured by filling plastic tubes with sand available at the marina area and placed along the toe of the exposed bluff which is approximately 1,500 ft long. The Longard tubes are regarded to be temporary and may be effective only for several years. The cost for installing the Longard tubes at the marina area has been estimated to be approximately $150 per linear foot based on the Longard tube experimental revetment installed in Del Mar, California during the period 1980-1983. As a result, the total cost of the project will be approximately $225,000.

In order to provide more permanent protection along the shore of the marina area, the riprap revetment along the C & D Canal may be extended at least along the 1,500 ft long exposed bluff. The riprap revetment may eventually need to be extended all the way around the marina area if long-term erosion of the unprotected shore inside the marina entrance is found to be unacceptable. A preliminary design of the riprap revetment has been performed using the information available on the riprap revetment along the C & D Canal as well as the design guidelines given in the Shore Protection Manual (5). The cost of the quarry stone required for protecting the revetment slope has been estimated to be $75 per linear foot. This is the cost of the required quarry stone only. Additional costs such as labor, needed equipment and site preparation must be included to estimate the total cost which will be greater than that for the Longard tubes.

Alternatively, a rubble-mound jetty may be constructed normal to the northern shore near the marina entrance to trap the sediment transported toward the marina entrance. The presence of the dredged navigation channel will limit the length of the jetty to less than approximately 350 ft. An updrift fillet of the deposited sediment will be formed on the western side of the jetty. After the updrift fillet is filled to its capacity, the sediment will be transported toward the dredged channel and around the tip of the jetty. Some of the sediment will also be transported over and through the rubble-mound jetty. The maximum storage capacity of the updrift fillet has been estimated to be less than approximately 1.75×10^6 ft^3 by assuming a simple triangular fillet of uniform thickness. The updrift fillet will hence be filled to its capacity within approximately 3.5 years and become ineffective in trapping the sediment and mitigating the sedimentation in the dredged channel. As a result, protection of the eroding bluff of relatively short length is more direct and reliable than trapping the eroded sediment without eliminating the source of the sediment supply.

Finally, the estimates of the sedimentation rates and deposited sediment volumes given in this study need to be verified using long-term hydrographic data for the entire marina area. Consequently, it is recommended to conduct hydrographic surveys at least annually so as to ensure sufficient water depths at the marina site. Furthermore, the sedimentation mitigation measures proposed in this report will require more detailed design using additional data on the foundation and examining various construction methods if they are adopted.

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