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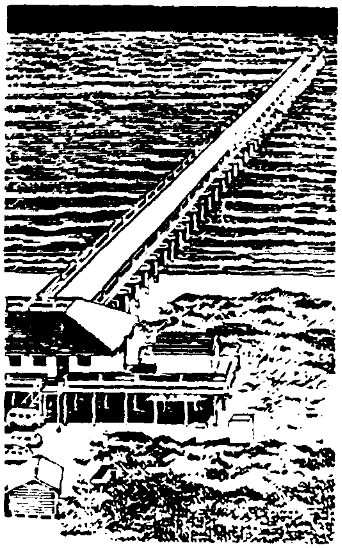

MONITORING COMPLETED COASTAL PROJECTS PROGRAM

MISCELLANEOUS PAPER CERC-92-7

MONITORING OF THE BEACH EROSION CONTROL PROJECT AT OAKLAND BEACH, RHODE ISLAND

by

Catherine LeBlanc

DEPARTMENT OF THE ARMY
US Army Engineer Division, New England
424 Trapelo Road, Waltham, Massachusetts 02254-9149

and

Robert R. Bottin, Jr.

Coastal Engineering Research Center

DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199

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USAE Division, New England. 424 Trapelo Road, Waltham, MA 02254-9149
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Under the Monitoring Completed Coastal Projects Program, an assessment of the Beach Erosion Control Project at Oakland Beach, Rhode Island was performed. The monitoring program extended over a 3-year period from April 1982 to April 1985. Monitoring included hydrographic and topographic surveys of the beach and nearshore area, aerial and ground photographs, wind data collection, littoral environment observations, sediment sampling, and site visits. Littoral transport, structure stability, and wind and wave analysis are evaluated and discussed. Data on the performance of the project and observations made in the prototype, as well as conclusions and recommendations based on the monitoring effort, are reported herein.

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Extremal analysis	Oakland Beach, Rhode Island	Waves
Groins	Stable shoreline	Wind

PREFACE

Funding for the study reported herein was provided through the Monitoring Completed Coastal Projects (MCCP) Program. The program entails intense monitoring of selected Civil Works coastal projects to collect data that can be used to improve project purpose attainment, design procedures, construction methods, and operation and maintenance techniques. Overall program management is by the Hydraulic Design Section of Headquarters, US Army Corps of Engineers (HQUSACE). The Coastal Engineering Research Center (CERC), US Army Engineer Waterways Experiment Station (WES), is responsible for providing technical and data management support, and for facilitating HQUSACE review and technology transfer. Technical Monitors for the MCCP Program are Messrs. John H. Lockhart, Jr., John G. Housley, and Barry W. Holiday. The Program Manager is Ms. Carolyn M. Holmes, CERC.

This report was prepared by Ms. Catherine LeBlanc, US Army Engineer Division, New England, and Mr. Robert R. Bottin, Jr., Wave Processes Branch, Wave Dynamics Division, under the general supervision of Mr. Charles C. Calhoun, Jr., and Dr. James R. Houston, Assistant Director and Director of CERC, respectively. This report was typed by Ms. Karen R. Wood, CERC, and was edited by Ms. Janean Shirley, Information Technology Laboratory, WES.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander and Deputy Director was COL Leonard G. Hassell, EN.

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CONVERSION FACTORS, NON-SI TO SI (METRIC) UNITS OF MEASUREMENT

Non-SI units of measurement used in this report can be converted to SI (metric) units as follows:

Multiply	By	To Obtain
cubic yards	0.02831685	cubic metres
degrees (angle)	0.01745329	radians
feet	0.3048	metres
feet per second	0.3048	metres per second
inches	2.54	centimetres
knots	1.8532	kilometres per hour
miles (US statute)	1.609347	kilometres
miles per hour	1.609347	kilometres per hour

MONITORING OF THE BEACH EROSION CONTROL PROJECT AT OAKLAND BEACH, RHODE ISLAND

PART I: INTRODUCTION

Project History

1. Oakland Beach is located in Warwick, RI, approximately 10 miles* south of Providence and 15 miles north of Newport. The beach is in the upper portion of Narragansett Bay at the southern extremity of a point of land known as Horse Neck. It faces Greenwich Bay to the south and is bordered by Warwick Neck to the east and Brush Neck to the west (Figure 1).

2. Presently, the beach area is divided into three distinct sections. The eastern section is a beach area approximately 500 ft long; the middle section is an area approximately 600 ft long adjacent to a parking lot and fronted by a revetment; and the western section is another beach area approximately 750 ft long.

3. Prior to 1938, Oakland Beach was a popular private saltwater recreational bathing beach area, visited by people from all parts of New England. The adjacent land area contained an amusement park, which attracted many visitors. The beach and amusement park were almost completely destroyed during a hurricane in 1938. Subsequent to the hurricane, the city of Warwick acquired the area and made some attempts to control erosion. These measures included the construction of a seawall fronting the parking lot, seven timber groins along the west beach, and one terminal wooden jetty at the eastern limit of the east beach. No maintenance was ever performed on these structures, and they eventually deteriorated to the point that they were ineffective. There is limited sediment in the littoral system in the area; therefore, when the structures deteriorated, the beach soon eroded to an unusable condition. Shoreline recession, due to storm waves and an inadequate supply of littoral material, reached 1-2 ft per year. In 1973, the City of Warwick requested assistance from the Corps of Engineers in solving the erosion problem that existed at Oakland Beach. An aerial photograph of the site in 1976 is shown in Figure 2.

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* A table of factors for converting non-SI units of measurement to SI (metric) units is presented on page 3.

4. At the time of the Corps' initial involvement, the eastern beach section was eroded to the point that no dry beach area existed above the mean high water (mhw) line, and the timber jetty was in a dilapidated state. The reinforced concrete seawall, fronting the parking lot in the middle section, was in poor condition, as were the seven timber groins along the west beach area. The west beach area contained the only available dry beach space, approximately 750 ft in length and 50 ft in width above the mhw line (approximately 4.0 ft above mean low water, mlw).*

5. In January 1980, the Corps completed its study of the erosion problem at Oakland Beach (US Army Engineer Division (USAED), New England 1980). The study included a thorough review of the history of erosion that had occurred at the beach and an evaluation of the existing conditions. No physical or mathematical model studies were conducted to aid in design of the project. Information utilized for design purposes is summarized in the following subparagraphs.

a. Tides. Tidal information for the project area was based on gage data from Providence and Newport, RI. The tides are semidiurnal with a mean range of 4.0 ft and a spring range of 5.0 ft. Based on this review of historic tidal records and an analysis of the design of other beaches in the area, a design tidal elevation (el) of +7.0 ft, with an associated return period of 7 years, was selected.

b. Winds. National Weather Service wind records for the T.F. Green State Airport in Warwick, located 3.5 miles northwest of Oakland Beach, were analyzed for the 10-year period of record from 1965-1974. This information indicated no predominant prevailing wind direction; however, it showed a significant percentage of winds approaching Oakland Beach from the south with an average speed of 8 mph. The area is also periodically subjected to hurricane winds (in excess of 75 mph) that approach from the south of Narragansett Bay and across Greenwich Bay. Hurricanes in 1938 and 1954 destroyed and seriously damaged homes and other shore structures, and caused extensive beach erosion at Oakland Beach. Winds from the south associated with the more frequent storms that occur during the winter months, however, are the chief cause of beach erosion and damage to shore structures in the area.

c. Waves. The configuration of the beach area and the surrounding land masses is such that only waves approaching from the southeast through southwest can substantially affect the shoreline at

* All elevations (el) cited herein are in feet referred to mean low water (mlw) unless otherwise noted.

Oakland Beach. It was determined that the water depth, as opposed to the fetch length or wind duration, would limit the size of the waves that would impinge on the beach, revetment, and groin structures. The design wave height was computed using the solitary wave formula, $H/d = 0.78$, where H is the wave height and d is the depth of water (with the design tide level). The design wave height for the revetment and sandfill was calculated to be 5.5 ft with a 3.5-sec period, and the design wave height at the head of the groins was 6.2 ft with a 3.5-sec period.

d. Littoral drift and currents. The littoral drift along Oakland Beach was investigated using historic data (old reports, aerial photographs, shoreline change maps), observations during site visits, and discussions with local people familiar with the area. The results indicated only a small amount of material moving in the area, and it appeared the net movement was slightly from east to west. It also appeared that the material moved more readily during flood tides and times when storm-driven waves approached from the south. Tidal current readings indicated, however, that the average maximum flood tide velocity was only about 0.3 to 0.7 fps.

e. Beach profiles and sand samples. Seven beach profiles were taken to determine the amount of beach fill needed to establish the slope fronting the beach and to establish a base for comparison purposes for future profiles where estimates of the rates of erosion/accretion could be determined in the area. A total of 13 sand samples were obtained along the seven profile lines to determine the composition of the native beachfill. The native material was composed of fine-grained sand and silt, which is easily moved by wave and tidal action. A medium-grained sand was selected for use in the beachfill project because it was assumed that it would prove more stable and less susceptible to erosion forces in the area.

It was determined that the best way to stabilize the shoreline and provide for recreational needs of the area was to raise and widen the beach above the mean high water line, construct intermediate and terminal groin structures to replace the dilapidated ones to help compartmentalize the sand, and provide for periodic beach nourishment.

6. A beach erosion control project for Oakland Beach was authorized by the Chief of Engineers on 30 April 1980, pursuant to the authority contained in Section 103 of the 1962 River and Harbor Act, as amended. The project, as constructed, included widening the beach by direct placement of suitable sandfill on either side of the existing seawall to a backshore elevation of 8.0 ft above mlw and construction of four high groin structures, one low-profile groin, and a rock revetment in front of the existing concrete seawall (Figure 3). This plan also provided a protective and recreational beach

averaging 100 ft in width above the mean high water line. Also included as part of the initial project were the removal of the seven existing, dilapidated timber groin structures, cleanup of debris (concrete foundations and slabs and rocks) and periodic nourishment for the 50-year economic life of the project (USAED, New England 1980).

7. Construction of the project was initiated in March 1981 and completed in August 1981. The total cost of the project was $738,700, with a Federal share of $557,500 and a non-Federal share of $181,200.

Monitoring Completed Coastal Projects Program

8. The Monitoring Completed Coastal Projects (MCCP) Program was initiated in 1981 with four projects. The erosion control project at Oakland Beach was subsequently selected for the program when funds became available.

9. The principal goal of the MCCP Program is to reduce the costs of operating and maintaining Corps coastal projects through the advancement of coastal engineering technology. Projects included in the program are analyzed to determine how well they are accomplishing their intended purposes, and resisting the attacks of the physical environment. These determinations, combined with existing knowledge, allow for more credibility in the design of future projects. Based on this information, future projects should have more cost-effective engineering solutions and improved design methods, construction practices, and maintenance techniques. The monitoring program will also identify areas that require more research attention.

10. The Corps of Engineers coastal offices are invited to nominate projects for inclusion in the monitoring program when funds are available. A selection committee, comprised of members of the MCCP Program Field Review Group (representatives of District and Division offices) and civilian members of the Coastal Engineering Research Board, reviews and prioritizes the projects nominated. When Oakland Beach was reviewed, it was prioritized according to how well it met criteria developed by a group of coastal engineers and scientists when the MCCP Program was originally formulated. The prioritized list is reviewed by the program's Technical Monitors at Headquarters, US Army Corps of Engineers (HQUSACE). Final selection is based on this prioritized list, national priorities, and the availability of funding. A prioritized listing of the program's area of interest is included in Table 1.

Table 1

MCCP Program Areas of Interest

Shoreline and nearshore current response to coastal structures.
Wave transmission by overtopping.
Prediction of controlling cross section at inlet navigation channels.
Wave attenuation by breakwaters (submerged and floating).
Bypassing at jettied and unjettied inlets.
Wave refraction and steepening by currents.
Beach fill project monitoring.
Stability of rubble structures - investigations to determine causes of failure.
Comparison of pre- and post-construction sediment budgets.
Wave and current effects on navigation.
Dynamics of floating structures.
Wave reflection.
Effects of construction techniques on scour and deposition near coastal structures.
Diffraction around prototype structures.
Wave runup on structures.
Onshore/offshore sediment movement near coastal structures.
Harbor oscillations.
Wave transmission through structures.
Material life cycle.
Ice effects on structures and beaches.
Model study verification.
Wave translation.
Construction techniques.

11. The overall monitoring program is under the management of the US Army Engineer Waterways Experiment Station's Coastal Engineering Research Center (CERC), with guidance from HQUSACE. Operation of the individual monitoring projects is a cooperative effort between the submitting District/Division office and CERC. Development of the monitoring plan and the conduct of data collection and analysis are dependent upon the combined resources of CERC and the Districts/Divisions.

PART II: MONITORING PROGRAM

Objective

12. The major objective of the Oakland Beach monitoring program was to determine the effectiveness of improvements designed by the USAED, New England (CENED) by evaluating the way in which the new beach and structures were functioning. If the project was found to be functioning as intended, the success of the design approach would be made known to other Division and District offices. On the other hand, if the project was not functioning properly, then, by using the monitoring program, the cause of the problem could be identified and the design methodology improved in future applications. The Oakland Beach monitoring program also allowed the Corps a unique opportunity to study a small self-contained beach type project. Much of the information presently available in the field of coastal engineering is based on large open ocean areas; little is available in sheltered areas such as Oakland Beach.

Data Collection

13. The monitoring program at Oakland Beach extended over the 36-month period from April 1982 through April 1985. The elements of work which comprised the monitoring program included hydrographic and topographic surveys of the beach and nearshore area; aerial and ground photographs; wind data collection at the T. F. Green Airport (approximately 3 miles from the site); wind data collection at the site (1 year of data); wave and tide data collection at the site (1 year of data); littoral environment observations (LEO); sediment sampling; and site visits.

14. It was planned to initiate the hydrographic and topographic surveys in April 1982, and continue them for each October and April in FY83, FY84, and FY85; however, funding and scheduling problems were encountered on several occasions. Therefore, surveys were actually performed on the following dates: September 1982, April 1983, September 1983, May 1984, September 1984, and March 1985. A survey performed in August 1977 (4 years prior to construction) was also available for comparison. There were no as-built surveys taken; therefore, an assumption was made that the project was constructed in accordance with the construction plans.

15. Thirteen profiles were surveyed during the September 1982 and April 1983 surveys. Profiles 1 through 8 repeated historic survey locations. Following the April survey, concerns were raised that the easternmost terminal groin may have been experiencing some settlement along its length. Therefore, two additional short profile lines were added during the September 1983 survey, along with provisions for a center-line profile and cross sections of the easternmost terminal groin. Grab samples were obtained at 12 of the 13 historic locations for sediment sampling. Sample location S-6 could not be used since it was now at the top of the revetment. Figure 4 shows the locations of profile lines and sediment samples.

16. Controlled vertical aerial photographs of the beach and backshore area were taken at a scale of 1 in. = 100 ft with 60 percent overlap for stereo viewing. The dates on which the work was performed were: October 1982; January, April, July, and October 1983; January, April, July, and November 1984; January, April, and July 1985. The photographs were taken at low tide as close to noon as possible.

17. An anemometer and a wave and tide gage were placed at the site so that verification of accuracy of the design conditions could be made. Due to funding constraints, the gages were scheduled to be used at the site for a period of only 1 year. Wind data from the site were compared with wind data from the National Oceanic and Atmospheric Administration gage at the T. F. Green Airport in Warwick, RI. The purpose of the wind gage at the site was to verify wind transformation techniques used in the coastal design. The techniques entail a manipulation of available wind data in order to predict actual winds at the site. With data from the airport and 1 year of data at the site, it was possible to convert the airport data and compare results to actual winds at the site.

18. The anemometer was installed at the site in September 1983, and was scheduled to be kept in operation for a period of 1 year. However, after approximately 9 months of continually recording data, the gage failed in May 1984. Attempts to repair it were unsuccessful. Therefore, only 9 months of wind data at the site were available for analysis.

19. Wind data collected at the site on pressure-sensitive strip charts were digitized by a private firm under contract to CENED. These data were compiled in tabular form, displaying the wind speed (in miles per hour) and direction (in degrees on the compass).

20. During the design stage, wind data were used to determine the wave climate in the area, although a depth-limited wave was eventually used for design. Using information on wave height and direction obtained from the wind data, the approximate natural alignment of the beach was found. It had been intended to use the wind and wave data acquired during this monitoring program to verify methods used in the design stage. As will be reported, though, the wave gage failed to collect adequate data for this comparison. Instead, the wind data were used to investigate the accuracy of the Shore Protection Manual (SPM 1984) method of relating winds measured over land to those over water at this site and the comparability of hindcast waves resulting from both predicted and measured winds.

21. A service contract was awarded in June 1983 for the installation, maintenance, servicing, and removal of a Sea Data 635-11 Wave and Tide Recorder at the mouth of Greenwich Bay. Wave and tide data were recorded on a cassette tape inside the recorder. Since the recorder was submerged, the contract required the services of a diver to periodically (usually on a 6-week basis) retrieve the cassette tape and install a new one. The gage was scheduled to be kept in service for a period of 1 year.

22. When the data tapes were analyzed, it was found that the gage had malfunctioned during several deployments. Even though the gage was replaced, only a short record of good wave data was obtained and the data were not used. When the gage deployment was planned, it was recognized that data recovery could be a problem. Because of the shallow nature of the bay, it was necessary to deploy the instrument in water much shallower than intended by the instrument manufacturer. Boat wakes were also expected to cause problems in the analysis of the data, since traffic was heavy in the area of gage deployment.

23. A LEO station was initiated on the eastern portion of Oakland Beach in August 1982. Had the program been successful, considerable additional information about structural performance, ice effects, and reflection from the revetment may have been obtained. Unfortunately, the observer was unable to continue data collection. After 6 months of sporadic collection, an attempt to find a new observer was unsuccessful. The LEO program was abandoned without obtaining any useful results.

PART III: RESULTS

Analysis of Survey Data

24. An analysis of the littoral transport at Oakland Beach was performed using survey data from August 1977; September 1982, 1983, and 1984; April 1983; May 1984; and March 1985, as well as aerial photographs taken in April 1976, July 1983, July 1984, and July 1985. Sand grab samples were obtained at various locations along the beach and offshore area during all of the above surveys, with the exception of August 1977.

25. Using the surveys, erosion and accretion volumes were determined during 1-year periods. The beach was split into three reaches, reach 1 being the east beach area, reach 2 the revetment area, and reach 3 the west beach area. A comparison of the August 1977 survey with the September 1982 survey was not used in this analysis due to the unusually long time span involved. During this time span, there was a major blizzard (February 1978) and the sandfill was not placed on the beach until the spring of 1980. The remaining surveys were compared using similar seasons. The September surveys were not compared to the April surveys since it would not be possible to account for normal seasonal changes. Analysis of the remaining pairs of surveys showed definite trends in erosion and accretion. Any deviation from the trends was explained based upon unusual occurrences during the year in question. Table 2 shows the erosion and accretion rates during the periods analyzed.

Table 2

Comparison of Surveys

Survey Dates	Reach 1	Reach 2	Reach 3	Net Change All Reaches
Sep 1982 and Sep 1983	Erosion 18,920*	Erosion 950	Accretion 8,530	Erosion 11,340
Sep 1983 and Sep 1984	Accretion 670	Erosion 1,560	Accretion 8,990	Accretion 8,100
Apr 1983 and May 1984	Erosion 6,770	Erosion 7,750	Erosion 4,760	Erosion 19,280
May 1984 and Mar 1985	Accretion 290	Accretion 4,210	Accretion 11,230	Accretion 15,730

* All volumes are given in cubic yards.

26. During the period from September 1982 through September 1983, erosion in reach 1 occurred mainly offshore in the easternmost area. Profiles for the September 1982 and 1983 surveys are shown in Figures 5-7. For reference, the profiles from the 1977 survey are included on these figures. This erosion may have been due in part to the influence of a channel directly adjacent to this area. The first set of surveys was performed within 3 years of construction of the beach, so it is likely that the beach was still attempting to reach a stable condition at the time of the surveys, which would also account for this erosion. Erosion in reach 2 was relatively minor, as would be expected, since there was no sandfill placed in this area. Reflection off the revetment probably would not yet be a problem because of the covering of sand over the revetment toe. Accretion in reach 3 occurred mainly along the nearshore area, which would be expected, since a terminal groin was located at the end of this reach.

27. The same general pattern was found during the September 1983 through September 1984 period (Figures 8-10); however, there was a small amount of accretion in reach 1 and an increase of erosion in reach 2. The accretion in reach 1 was probably due to the combination of erosion in reach 2 and the fact that the eastern offshore area had, by this time, most likely reached an equilibrium point with respect to the channel. Increased erosion in reach 2 was probably caused by the toe of the revetment becoming uncovered and because of reflection from the stone. As the revetment became uncovered from natural seasonal changes in the nearshore zone, the energy dissipation effects of the sand in front of the revetment disappeared. As a result, more of the revetment became uncovered, and the reflection forces increased, causing the loss of even more sand. Once again, the accretion in reach 3 was most likely due to the influence of the groin structure.

28. The period from April 1983 through May 1984 (Figures 11-13), with its high rate of erosion, appears at first not to fit the trends shown above; however, there was a major coastal storm in March 1984, which would explain the erosion along the entire beach. The period from May 1984 through March 1985 also does not support the trends (Figures 14-16); however, the winter of 1984 through 1985 was unusually mild, which would help to explain the accretion along the entire length of beach.

29. Analysis of this survey data suggests that Oakland Beach is approaching a near stable condition. Reach 1 suffered severe erosion from September 1982 to September 1983, but underwent mild accretion during the period September 1983 to September 1984, a strong indication of stability. Also, the erosion during the 1982 to 1983 period took place mainly beyond mean low water. Reach 2 experienced mild to moderate erosion throughout most of the 1982 through 1983 period, but this was balanced by moderate accretion during 1984 through 1985. This is a relatively high-energy area as evidenced by no appreciable sand buildup. Reach 3 experienced significant accretion during all periods of the study except the heavy storm year of April 1983 through May 1984. This accretion occurred because of the terminal groin located at the west end of the project area. As this groin-associated beach compartment becomes full, it is anticipated that the high annual net changes in sand volume recorded since 1982 will be significantly reduced. It is concluded, therefore, that Oakland Beach is relatively stable and that there are no measurable detrimental effects as a result of the design of the project. Figure 17 shows a comparison of shoreline changes during the monitoring period. It was noted during the period that ice cover in the winter months helped to reduce erosion by limiting the intensity of the waves acting on the beach during the most severe storm season.

30. Aerial photographs (Figures 18-20) also support the conclusion that the beach is stable. The photographs show the entire beach rather than the sections shown in the survey. Once again, an attempt was made to compare photographs taken during the same time frame, since the seasonal differences could not be accounted for in the analysis. One characteristic of the beach that is quite clear in the photos, but is not apparent in the survey profiles, is the sand retention capability of the groins. The survey profiles were taken in areas between the groins, therefore, the scallop being formed along the downdrift groin is not readily apparent in the analysis of the profiles. The buildup of sand fillets on the east side of the intermediate and westernmost terminal groins on the West Beach and similar buildup on the west side of the eastern terminal groin on the East Beach reveal that the movement of sand is away from a point (seaward of the revetment) and toward both the East and West Beaches. This indicated a transport nodal point seaward of the revetment. As can be seen in the photographs, the groins, particularly those to the west of the revetment, are holding a great deal of accreted material. This accretion will eventually reach the point where the groins will not be

able to retain any more material, and it will begin spilling over onto the beach downdrift of the structures. The beaches should be reshaped and graded in order to minimize sand loss.

31. Analysis of grain size distribution along the beach also supports the fact that the design of the beach and selection of the material were suited to the conditions of the area. Sand samples were taken at several locations along the beach and in the nearshore area during the time that surveys were performed (Figure 4). The mean particle size and sorting coefficient were determined for each sample and are shown in Table 3, during the April 1983 survey. The fill was coarser than the material that was present in the project area in 1977. The design specified a "well-graded material...with median diameter of not more than .40 mm and not to exceed 1.0 mm" (USAED, New England 1980). Figures 21-32 show typical gradation curves for the material.

32. For the most part, the mean particle sizes reflect what would be expected. The larger particle sizes are found along the shoreline and the finer particle sizes are found in the nearshore and offshore areas. Only in the area between mlw and mhw was there any significant variability. The mean particle size at each location did not change significantly over the years, which would indicate that the material was well-suited to this area. If the material was not suitable to the area, natural forces would have removed the unsuitable material until a point of equilibrium was established. There were some occasions when the values did not correlate with the rest of the data, and it was assumed that there was either an error in sampling or in the sieve analysis. For example, the September 1982 data for sample S-7 show a mean particle size of 9.0 mm. Since this value was significantly different from the results for other years in the same location, and there was evidence of shell and glass fragments in the sand sample, it was assumed that these fragments were most likely the reason for the large particle size reading.

33. The sorting coefficients for the various samples showed much the same results as the mean particle sizes, in that they reflected what would be expected and did not indicate any major changes over the years. The coefficients were found to get closer to 1.00 the farther offshore the sample was acquired. These results would indicate that the variation in the particle size distribution was greater for the samples taken along the shore than for those taken in the offshore zone. This would be expected since most the fine material would be carried into the offshore area and, therefore, the particle size distribution in that area would not vary to a great extent. As with the

Table 3

Sand Grain Analysis

Sample Results	Sep 1982	Apr 1983	Sep 1983	May 1984	Sep 1984	Mar 1985
Sample S-1, located on profile 2, 20 ft from the baseline						
Mean particle size, mm	1.15	1.05	0.68	0.85	0.70	0.75
Sorting coefficient	1.69	1.90	2.10	2.01	2.04	1.98
Sample S-2, located on Profile 2, 70 ft from the baseline						
Mean particle size, mm	1.00	0.68	1.10	0.87	0.85	1.05
Sorting coefficient	1.72	2.00	1.94	1.94	1.92	1.94
Sample S-3, located on profile 2, 150 ft from the baseline						
Mean particle size, mm	1.35	0.19	0.94	1.20	0.66	0.23
Sorting coefficient	1.44	1.46	1.86	2.02	2.14	1.65
Sample S-4, located on profile 2, 210 ft from the baseline (approx mlw)						
Mean particle size, mm	0.20	1.75	0.25	0.23	1.90	0.19
Sorting coefficient	1.86	1.49	1.83	1.81	1.35	1.65
Sample S-5, located on profile 2, 1,000 ft from the baseline						
Mean particle size, mm	0.16	0.16	0.15	0.15	0.20	0.16
Sorting coefficient	1.21	1.10	1.11	1.07	1.37	1.09
Sample S-7, located on profile 4, 133 ft from the baseline (approx. mlw)						
Mean particle size, mm	9.00	0.64	0.61	0.55	0.43	0.75
Sorting coefficient	6.20	1.72	2.90	1.73	1.52	4.53
Sample S-8, located on profile 4, 1,100 ft from the baseline						
Mean particle size, mm	0.14	0.19	0.17	0.18	0.18	0.19
Sorting coefficient	1.24	1.17	1.14	1.16	1.18	1.15

(Continued)

Table 3. (Concluded)

Sample Results	Sep 1982	Apr 1983	Sep 1983	May 1984	Sep 1984	Mar 1985
Sample S-9, located on profile 6, 60 ft from the baseline						
Mean particle size, mm	0.90	0.80	0.87	0.90	0.79	0.88
Sorting coefficient	1.98	2.04	1.95	2.05	2.09	2.03
Sample S-10, located on profile 6, 100 ft from the baseline						
Mean particle size, mm	1.25	0.70	0.74	0.69	0.82	0.68
Sorting coefficient	1.42	2.00	1.90	2.01	1.96	1.72
Sample S-11, located on profile 6, 132 ft from the baseline						
Mean particle size, mm	0.40	1.10	0.88	1.25	0.88	0.70
Sorting coefficient	1.27	1.35	1.51	1.21	1.43	1.92
Sample S-12, located on profile 6, 160 ft from the baseline (approx. mlw)						
Mean particle size, mm	1.50	0.77	0.50	1.55	1.40	0.80
Sorting coefficient	1.69	1.60	1.93	2.00	1.70	1.48
Sample S-13, located on profile 6, 1,000 ft from the baseline						
Mean particle size, mm	0.28	0.25	0.26	0.27	0.25	0.23
Sorting coefficient	1.14	1.36	1.34	1.26	1.28	1.22

mean particle size, the sorting coefficients did not change significantly over the years, except between mlw and mhw, which would indicate once again that the beach is relatively stable and is in a configuration that is compatible with the natural forces acting on the beach. The medium-grained sand fill material proved to be resistant to offshore loss; however, it is not known if the native sand would have acted similarly.

Structural Stability

34. Comparisons of cross sections and profiles of the eastern terminal groin, using the initial construction plans and survey results of September

1983 and May 1984, show essentially no change. The concerns that the structure had settled were unfounded. All of the structures at Oakland Beach, in fact, revealed no indications of settling or becoming unraveled and remained in excellent condition throughout the monitoring period, surviving both storms and ice with no adverse effects.

Wind and Wave Analysis

35. Wind data for use in wave hindcasting are generally assumed to be from a measurement elevation of 10 m over the water. When these assumptions are not valid, then corrections are available to compensate or adjust for land effects, heights other than 10 m, k, and air-sea temperature differences (SPM 1984). The SPM correction for location effects requires that land-based wind measurements be close enough to the body of water so that they result from the same pressure gradient as the over-water winds. The SPM location correction also requires that landscape roughness characteristics be similar to those for airport weather stations around the Great Lakes.

36. The data presented here consist of measurements taken from areas where the geography is complex and may violate the roughness assumption of the SPM correction. Winds measured at the Corps of Engineers site at Oakland Beach, Rhode Island, were obtained to represent the true over-water winds (unattenuated by effects present in the land data). Winds measured at the T. F. Green Airport were used to develop a prediction equation for the Oakland Beach location. The data consist of 748 observations of instantaneous wind speed and direction taken at 3-hr intervals between September 1983 and May 1984. Only winds approaching the area from 50 to 280 deg relative to true north were considered, since other directions could not generate waves affecting Oakland Beach.

37. Winds affecting an anemometer site at Oakland Beach will have passed over a significant land mass before reaching the measurement location. Because the winds that affect the site must pass over land, and due to the overall complexity of the location geography, the analysis was expected to produce results that differ from those of the Great Lakes region and, therefore, the SPM correction. The purpose of this study was to provide information to supplement that given by the SPM and to demonstrate the effect of an empirical wind speed prediction on extreme wave analyses. A brief discussion

of how the SPM correction relates to these data is found in the following paragraph.

38. Hindcast significant wave heights and periods were computed using SPM formulas (SPM pages 3-44). Analyses were based on deep-water formulas, since waves associated with the fetch and measured wind speeds for this location are generated in deep water relative to the maximum possible height and period for the conditions. Extremal analyses based on the data are presented for hindcast data from observed and predicted beach winds. The extremal analysis demonstrates the sensitivity of extremal methods to small errors in input data, such as those that arise from predicting winds using inland wind records.

Winds

39. Exploratory data analysis was performed to determine the basic relationship between winds measured at Oakland Beach and the T. F. Green Airport. In the following discussion, the Oakland Beach wind speed and direction are identified as U_w and D_w, or wind speed and direction over water, respectively, and the airport speed and direction are identified as U_l and D_l, or wind speed and direction over land. Summary statistics for the two locations are presented in Table 4.

40. The correlation coefficients given in Table 5 indicate a reasonably strong correlation between wind direction for the two sites ($r = 0.8$) and a more moderate correlation between wind speeds ($r = 0.69$). More detailed information including histograms, normal probability plots, and other descriptive statistics for the variables of Table 4 are presented in Figures 33-38. The figures include summary statistics for the variables of interest on this study. The portion entitled "Normal Probability Plot" contains the data plotted versus a standardized normal variate. If the data are distributed approximately normally, then the plot will look nearly linear. Looking at

Table 4

Summary Statistics for Oakland Beach and T. F. Green Airport Wind Measurements

Variable	Mean	Standard Deviation
D_w	184 deg	64 deg
D_l	169 deg	56 deg
U_w	8.8 knots	4.5 knots
U_l	9.4 knots	4.2 knots

Table 5

Correlation Coefficients for Oakland Beach and T. F. Green Airport Wind Measurements

Variable	U_1	D_1	$U_1 - U_w$
U_w	0.69	0.36	-0.47
D_w	0.08	0.80	-0.49
D_1	0.04	1.00	-0.42
$D_1 - D_w$	-0.07	0.10	0.23

Figure 33, the histogram for wind speed at the Oakland Beach location displays a distribution that is skewed toward high wind speeds. This skew toward high wind speeds is also apparent in the T. F. Green Airport data of Figure 35. The distribution of wind speed differences between the two locations (Figure 38) appears to be much more symmetric than the individual wind speed distributions. However, the wind speed difference distribution still displays some skew in the direction of higher wind speeds, as is apparent from the histogram and the deviation of the upper tail of the normal probability plot from linearity. The relatively large magnitude negative correlation between D_1 and $U_1 - U_w$ ($r = -0.42$), or wind direction over land and wind speed difference between land and water, indicates a possible relation between wind direction and wind speed attenuation between the two measurement sites. It is consistent with the geographic variability of the area that the relation between wind speeds for the two sites may vary with wind direction.

41. Least squares regression for different wind direction classes further demonstrates the dependence of wind speed attenuation on wind direction. The data were separated into direction classes as shown in Table 6 and least squares regressions of U_w on U_1 were computed. Intercept terms were not significant, as is expected if the winds at both locations result from the same pressure gradient (i.e., if $U_w = 0$, then $U_1 = 0$). Estimated slopes, denoted by a, and squared correlation coefficients for the equation

$$U_w = aU_1 \qquad (1)$$

are also presented in Table 6.

Table 6

Least Squares Regressions by Direction

Direction	Slope, a	r^2
50-90	0.586	0.78
90-130	0.654	0.50
130-170	0.682	0.58
170-210	0.984	0.48
210-250	1.080	0.59
250-280	1.070	0.66

42. Regressions with higher order quadratic and cubic terms did not yield significant coefficients, resulting in the conclusion that the linear model of Equation 1 is appropriate. The regression lines of Figures 39-44 also indicate that the linear model is appropriate. Note that since the regression intercepts were negligible, the regressions were forced through the origin, resulting in only a slope parameter.

43. The regression slopes in Table 6 exhibit an apparent trend as wind direction increases, suggesting that a model including wind direction as a parameter may be appropriate. Regression analyses including wind direction over land D_1 and the cross-product of wind direction and speed over land U_1D_1 resulted in no significant contribution by wind direction D_1 and a significant contribution by the cross-product term U_1D_1. The resulting model for predicting wind speed at the beach site is given by

$$U_w = U_1[0.4237 + (2.776\times10^{-3})D_1] \quad (2)$$

where the term in brackets represents the slope or wind speed attenuation as a function of wind direction. Equation 2 produces values similar to those in Table 6 for given values of wind direction D_1. The overall squared correlation for the model in Equation 2 is $r^2 = 0.61$, meaning that the right-hand side of Equation 2 accounts for 61 percent of the variability in wind speed at the beach site. Higher order cross-products with quadratic, cubic, and quartic terms in D_1 produced an overall squared correlation of $r^2 = 0.62$, indicating negligible improvement over the model of Equation 2.

Waves

44. Application of the SPM correction for location effects to the Oakland Beach data results in estimates for the regression of Equation 1, as listed in Table 7.

Table 7

SPM Corrected Least Squares Regressions, by Direction

Direction	Slope, a	r^2
50-90	0.47	0.76
90-130	0.49	0.51
130-170	0.54	0.59
170-210	0.77	0.49
210-250	0.85	0.57
250-280	0.86	0.64

45. Note that the squared correlations (r^2) indicate essentially the same degree of linear correlation between the two sites when the SPM correction is applied as when it is not. The slopes of Table 7 indicate that, for these data, the SPM correction is not appropriate (i.e., the slopes are closer to 1.0 for the data of Table 6, or the uncorrected data).

46. Hindcast significant wave heights and periods based on observed and predicted (Equation 2) wind speeds were computed using hindcast formulas given in the SPM. A fetch of 20,000 ft and a water depth of 20 ft were used to produce approximate wave conditions for the wave generation area offshore of Oakland Beach, between the north and west ends of Conanicut Island and the south end of Warwick Neck. The hindcast waves were not meant to represent an exact hindcast for the area, but to demonstrate the effect that the empirical correction of Equation 2 has on extreme wave predictions. Summary statistics for the hindcast results are shown in Figures 45 through 50. The mean wave height for the hindcast based on observed winds at Oakland Beach was 0.52 ft and the maximum height was 2.14 ft (Figure 45). The mean wave period was 1.7 sec with a range of periods from 0.9 sec to 2.9 sec. The hindcast from predicted (Equation 2) wind speeds resulted in a mean wave height of 0.49 ft and maximum wave height of 2.38 ft (Figure 47). The associated mean wave period was 1.7 sec, with a range of 0.9 sec to 3.0 sec. The mean difference

between observed wind hindcast and predicted hindcast wave heights was 0.03 ft with a standard deviation of 0.21 ft (Figure 49). The mean difference is significantly different from zero at the 0.0001 significance levels, implying that the empirical wind speed prediction results in a systematic underprediction on the average. Note also that the maximum height is larger for predicted winds than for observed winds. This indicates that the underprediction mentioned above may not generalize to extremes. The 99 percentile wave height for predicted winds is 1.46 ft, while the 99 percentile for observed winds is 1.52 ft, indicating that for near extremes the predicted wind speeds still produce smaller wave heights than observed wind speeds.

Extremal analyses

47. Extremal analyses were performed using the Extremal Type I and the Weibull distributions as possible choices for modeling extreme wave heights for the Oakland Beach area. Since the study site is depth-limited, it should be noted that any of the following results that exceed the depth-limited design wave conditions used in the original design study are purely academic and are presented here for the purpose of demonstrating the effect of input data errors on extremal predictions.

48. The Extremal Type I cumulative distribution function (CDF) has the form:

$$F(x) = \exp(-\exp(ax + b)) \tag{3}$$

and the Weibull CDF has the form:

$$F(x) = 1 - \exp(-(ax + b)^k) \tag{4}$$

where, for both equations, the quantities a and b are scale and location parameters and for the Weibull, k is a distribution shape parameter. Methods for selecting the appropriate model and for estimating the parameters are available in the literature on extremal analysis (Petrauskas and Aagaard 1971, Borgman and Resio 1982, Goda 1989, Andrew and Hemsley 1991). The method used he is outlined in a paper by Andrew and Hemsley (1991). This method was shown to provide objective means for selecting between the Extremal Type I and the Weibull, using criteria that are based on how well each model and set of parameter values predicts extremes in the measured data. The Extremal Type I model was rejected for hindcast waves from both observed and predicted wind speed records. Visual inspection of the data plotted on an Extremal Type I scale is sufficient to demonstrate this conclusion (Figures 51 and 52). The Weibull with shape parameter $k = 0.7$ produced the best fit for waves from predicted winds and the Weibull with $k = 0.8$ was best for waves from observed winds (Figures 53 and 54). The model selection and choice of the shape parameter k were based on the prediction bias. The prediction bias is defined to be the average amount by which the lowest 90 percent of the data underpredicts or overpredicts the upper 10 percent of the data for a proposed model (and choice of k if the model is Weibull). Table 8 contains values for the prediction bias for both models.

49. The Weibull with $k = 0.8$ produces minimum bias for waves from observed winds and $k = 0.7$ has minimum bias for waves from predicted winds. The extrapolated wave heights from predicted winds for return periods $R = 1$, 2, 5, and 10 years are 2.63, 3.04, 3.55, and 4.02 ft, respectively. The same return periods for waves from observed winds result in extrapolated wave heights of 2.15, 2.36, 2.62, and 2.85, respectively. For the range of return periods, the difference between the two predictions starts at 0.48 ft for $R = 1$ and is as much as 1.17 ft for $R = 10$ years. This divergence of the two predictions provides a good example of the sensitivity of extremal prediction methods to errors in input data. In general, it is accepted practice to avoid extrapolating beyond 2 or 3 times the time extent of the measured data.

Discussion

50. Data from the Corps of Engineers measurement site at Oakland Beach, Rhode Island and from T. F. Green Airport, 35 miles northwest of Oakland Beach, were analyzed using least squares multiple regression. The linear model of Equation 1 was found to explain the relationship between wind speeds at the two locations as well as any higher order nonlinear models. Wind speed attenuation between the two locations was found to be dependent on wind direction. This result is not surprising since the surrounding geography is complex, consisting of varying proportions of land and water and resulting in varying surface roughness. The model of Equation 2 describes the dependence of wind speed attenuation on wind direction. The overall model of Equation 2 explains 61 percent of the variability in wind speed at the Oakland Beach site.

51. Hindcast data were computed by means of standard SPM formulas for both observed and predicted (or corrected) wind speed data. Extremal analyses

Table 8

Model Selection Criteria

Model	k	Prediction Bias: Predicted Wind	Prediction Bias: Observed Wind
Extremal Type I:		0.370	0.200
Weibull:	2.0	0.450	0.270
	1.0	0.250	0.100
	0.8	0.135	-0.004
	0.7	0.041	-0.082
	0.6	-0.122	-0.197

computed for waves from both wind speed records and the predicted wind speed record were shown to overpredict extremes.

PART IV: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

52. Based on the results of the monitoring effort reported herein, it is concluded that:

- a. Oakland Beach is relatively stable. There have been no measurable detrimental effects as a result of the project design. While the initial design called for periodic renourishment, none has been required, although failure to reshape the beach may result in loss of material around the terminal groins.

- b. The sand fill material placed at the site has been resistent to offshore loss.

- c. The beaches appear to benefit from winter ice cover, since they are not subject to erosion during the most severe storm season.

- d. A transport node appears to exist seaward of the revetment, which results in sediment movement toward both the east and west beaches.

- e. All structures remained stable and in good condition throughout the monitoring period.

- f. The procedure to adjust winds measured over land to a site on the coast was developed for a situation in the Great Lakes. At Oakland Beach, because of the different nature of the site, the adjustment would have produced information noticeably different from that measured at the site.

- g. The use of the depth-limited design wave conditions has proven a good choice at Oakland Beach.

Recommendations

53. As a result of the monitoring effort, the following recommendations are offered:

- a. The City of Warwick should reshape and grade the beach to prevent the loss of beach material around the terminal groins.

- b. The use of fill material coarser than the native material worked well at Oakland Beach. It should be considered in the future in areas where a low wave climate exists and where the coarser material would be acceptable to the users of a recreational beach.

c. The SPM wind adjustment should be used with care in areas not similar to the Great Lakes regime where it was developed. When used, one must realize that actual winds at the coast may be over- or under-predicted.

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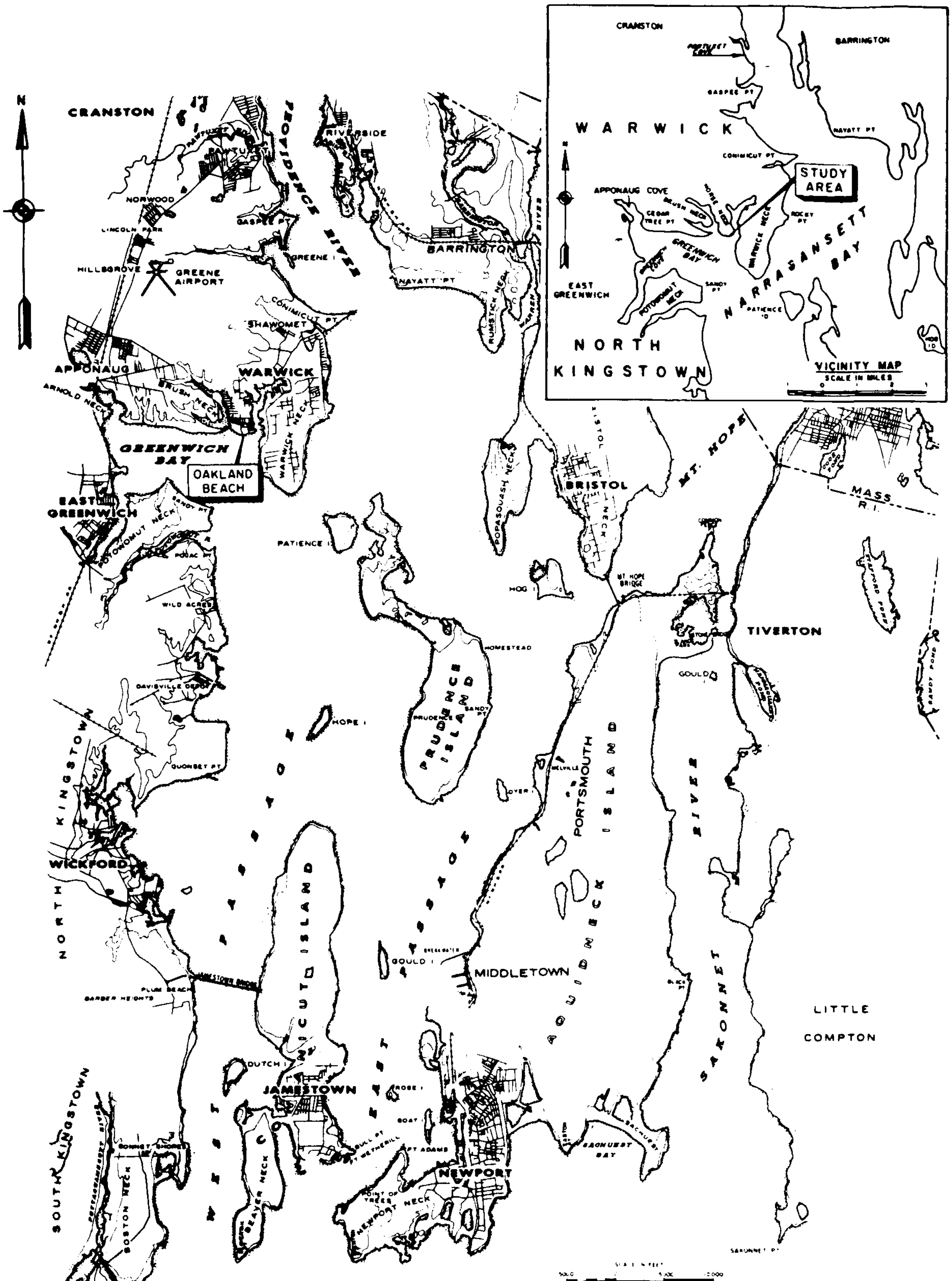

Figure 1. Project location

Figure 2. Aerial view of Oakland Beach, April 1976

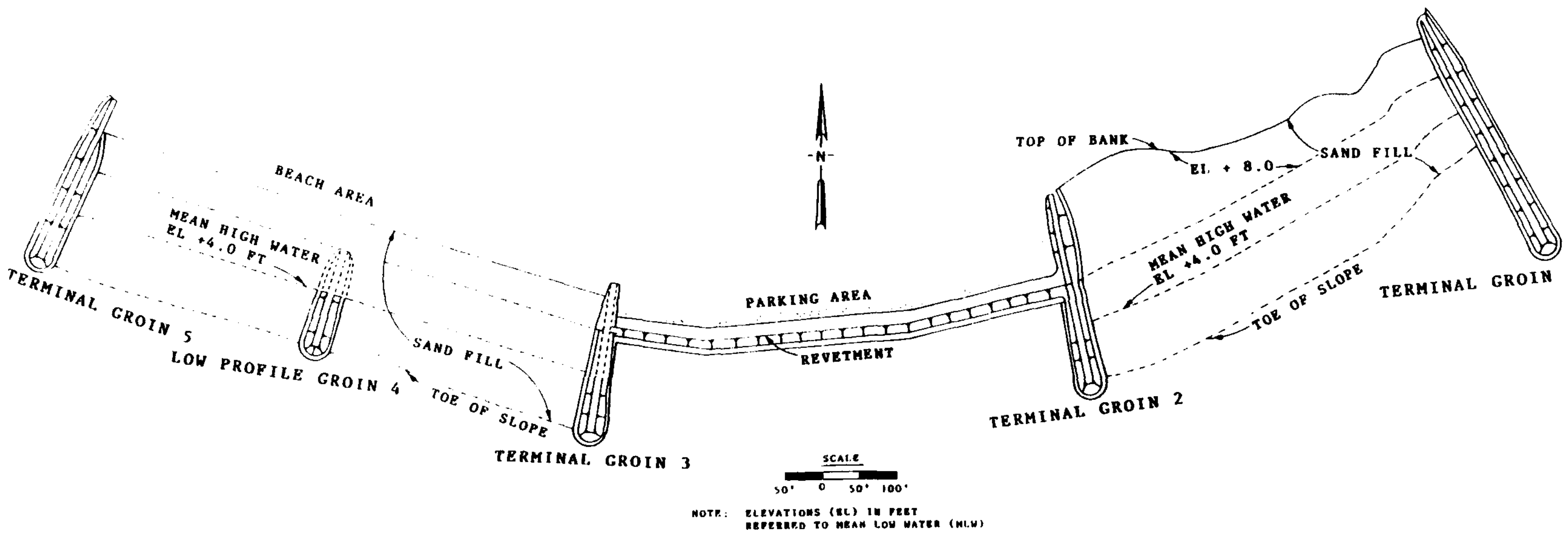

Figure 3. Elements of beach erosion project

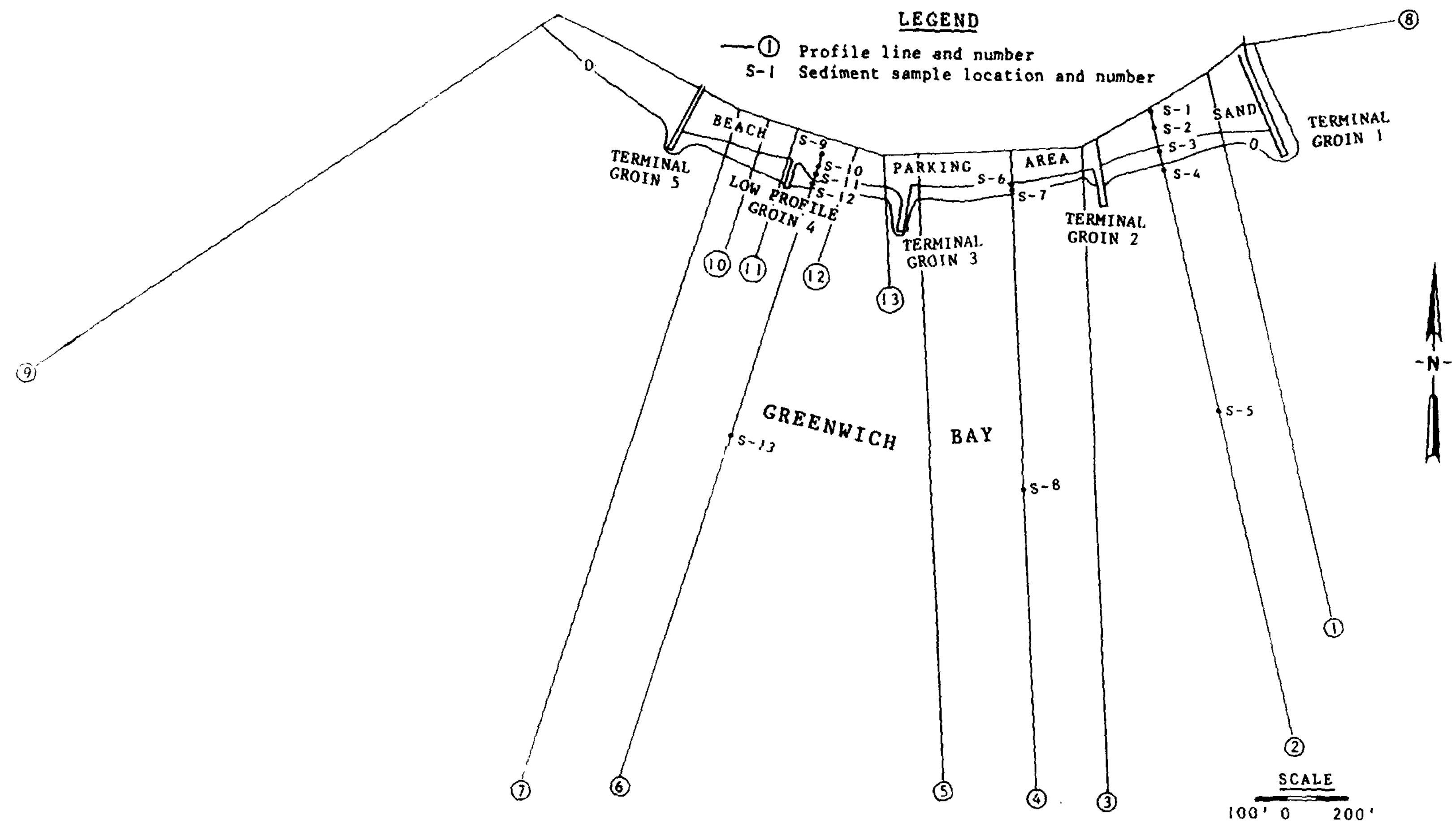

Figure 4. Profile lines and sediment sample locations

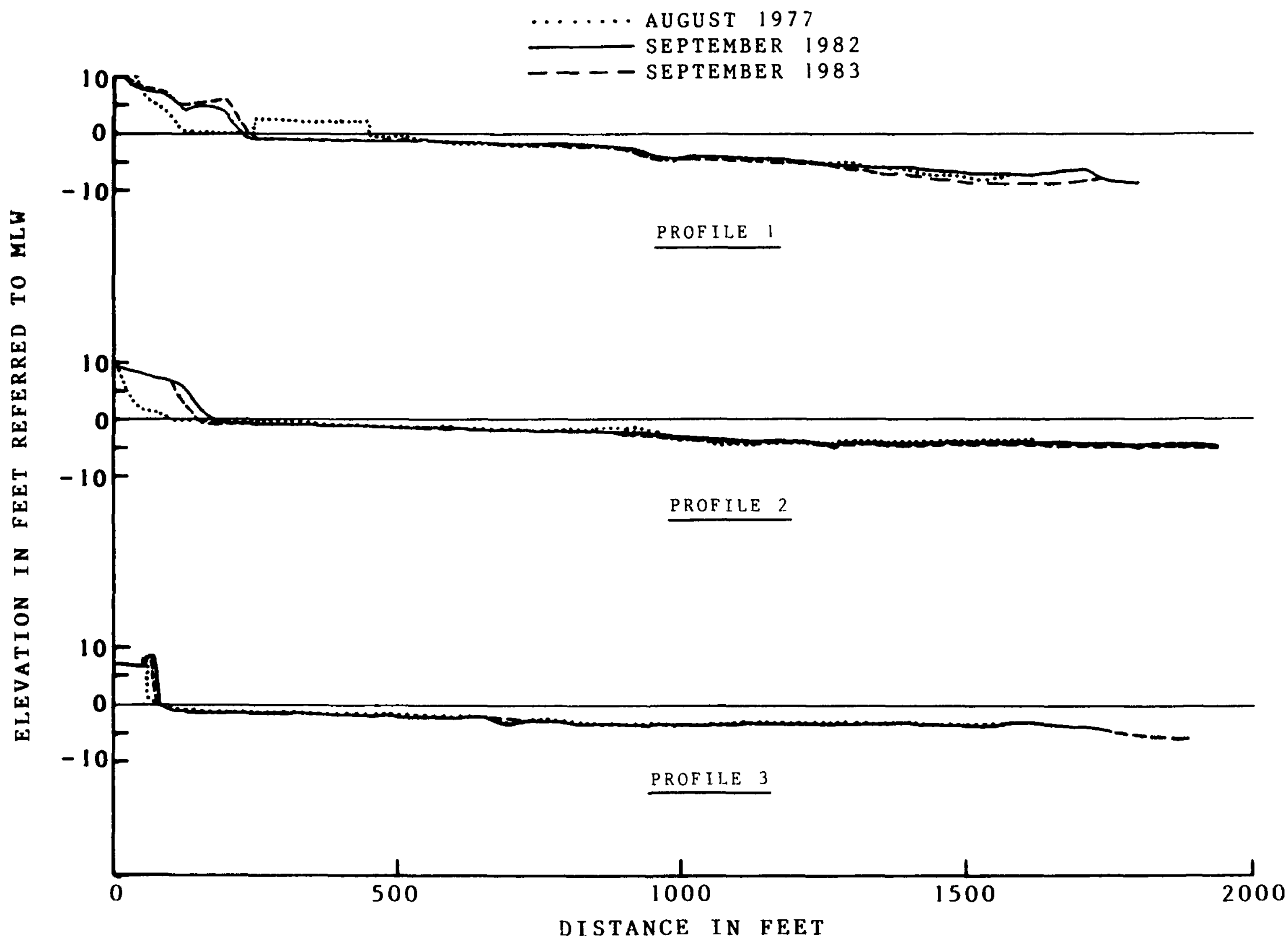

Figure 5. Profile changes, September 1982-September 1983, profiles 1-3 (1977 profiles included for reference)

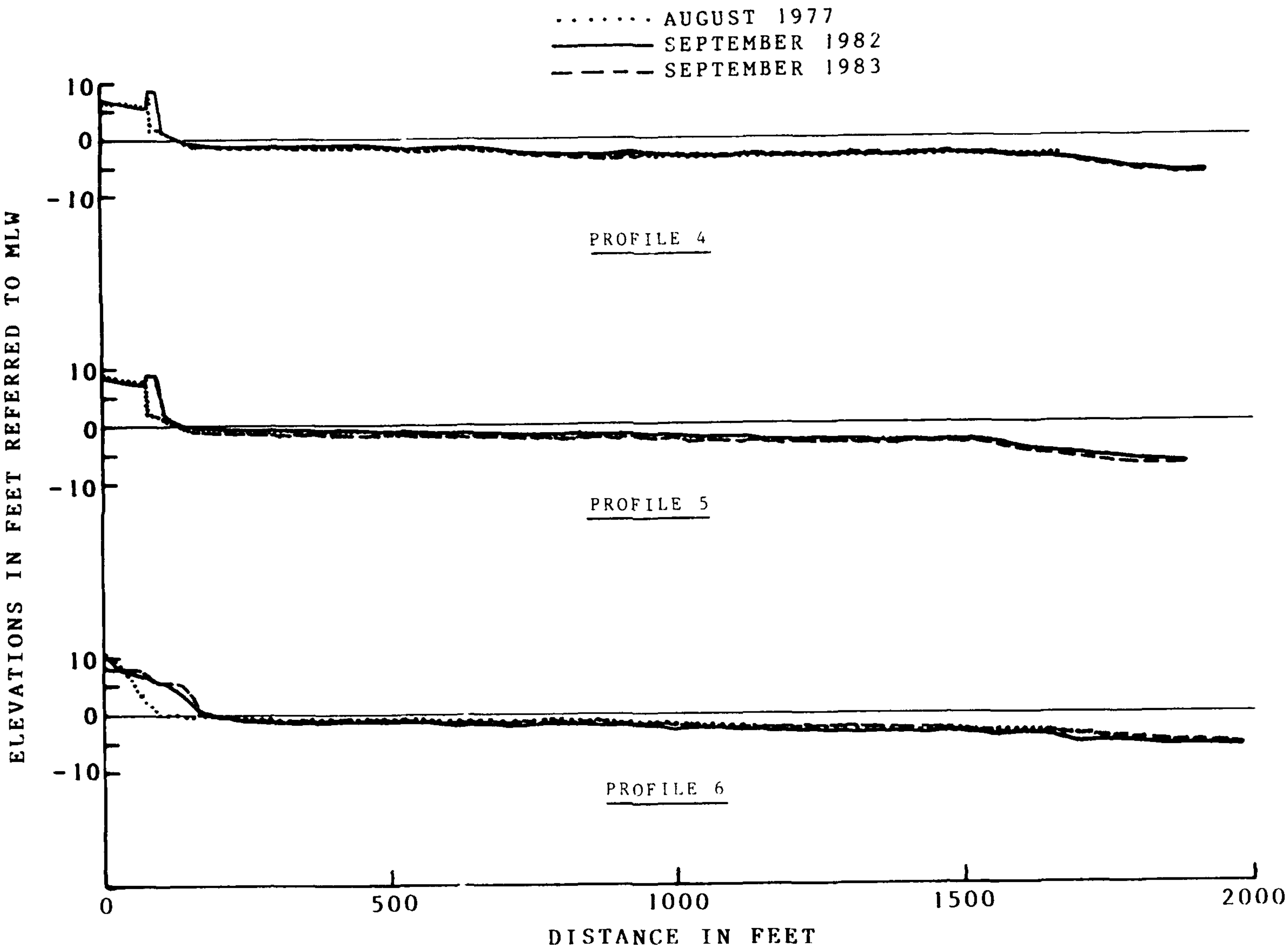

Figure 6. Profile changes, September 1982-September 1983, profiles 4-6 (1977 profiles included for reference)

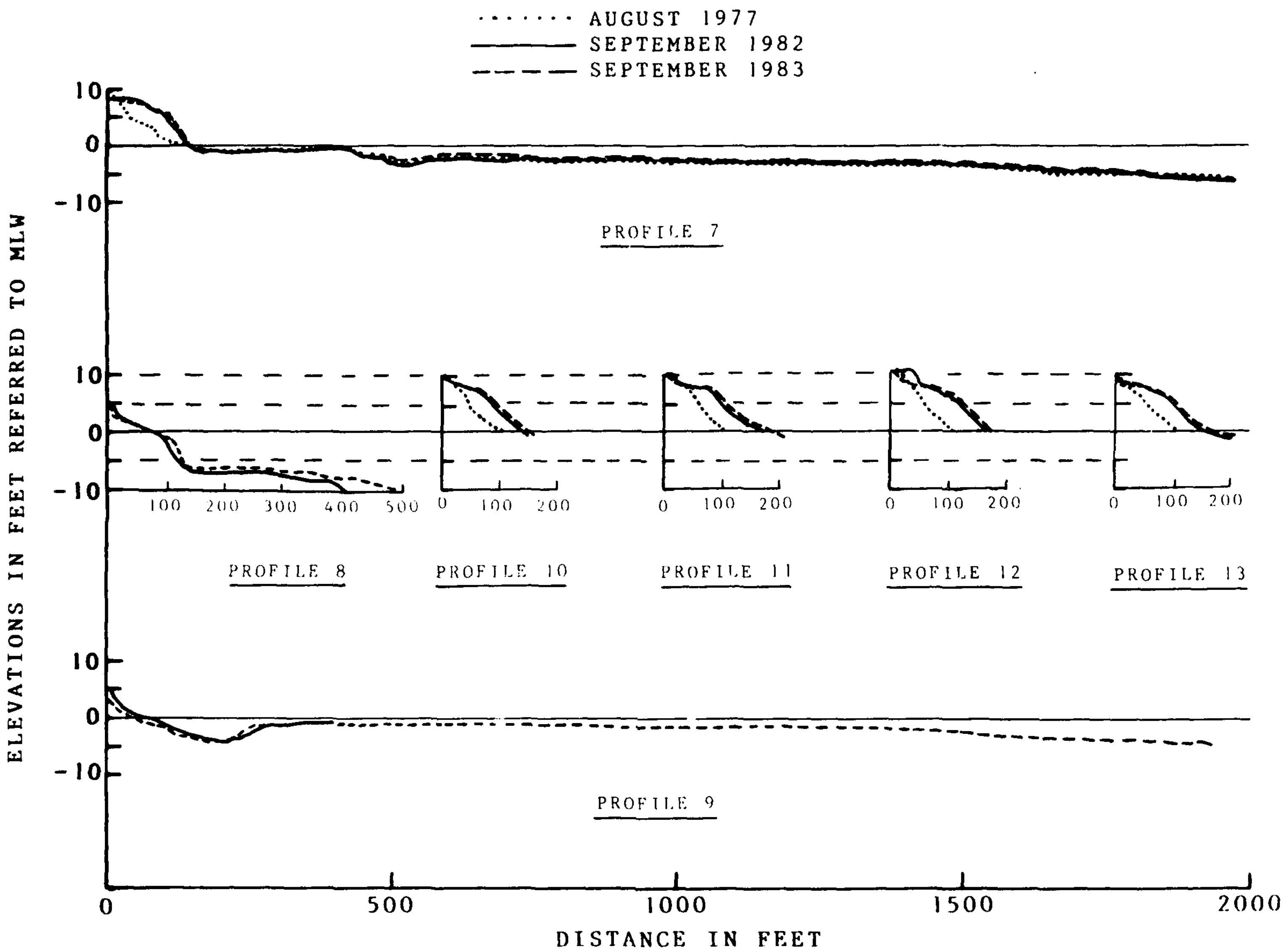

Figure 7. Profile changes, September 1982-September 1983, profiles 7-13 (1977 profiles included for reference)

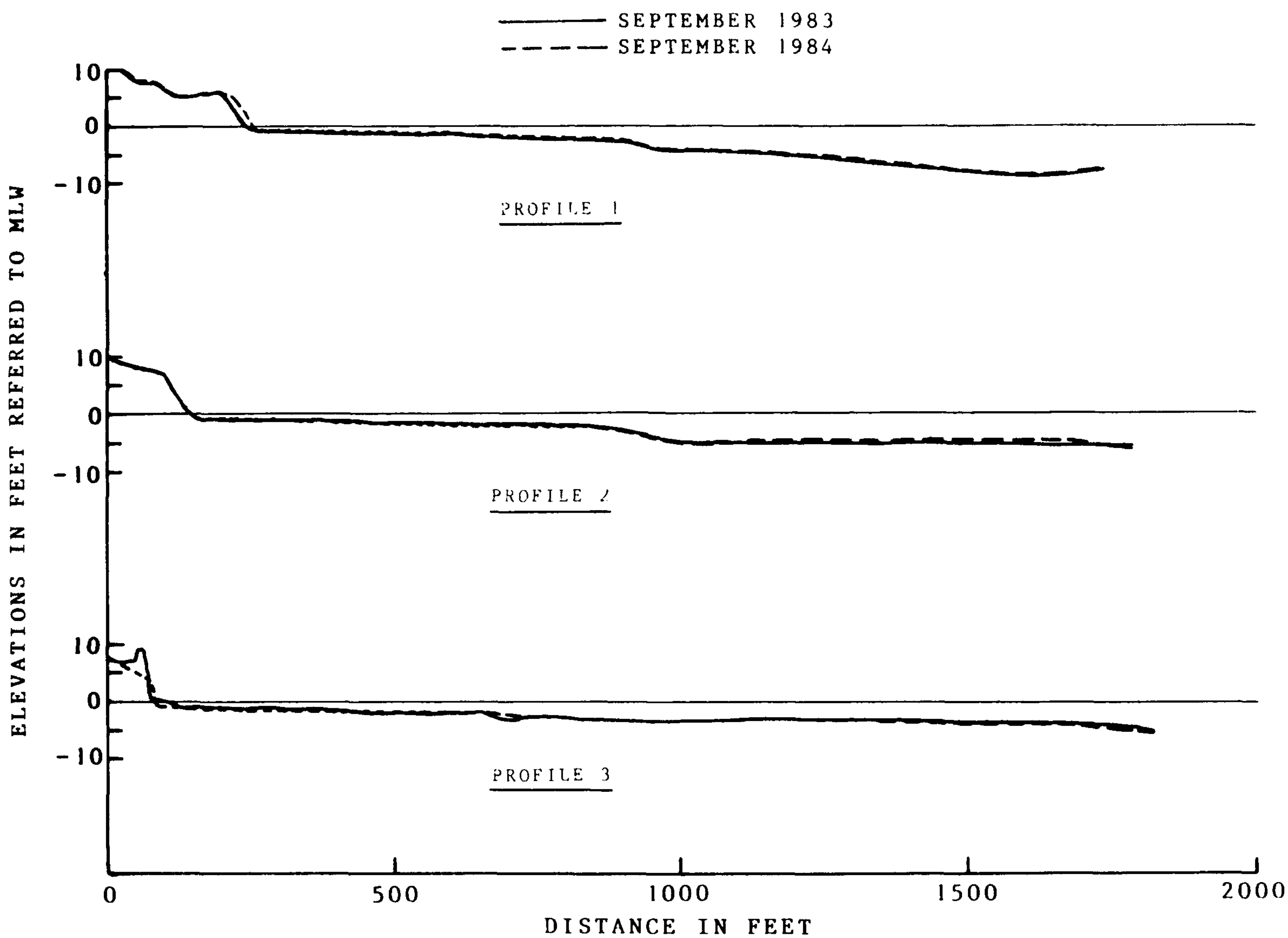

Figure 8. Profile changes, September 1983-September 1984, profiles 1-3

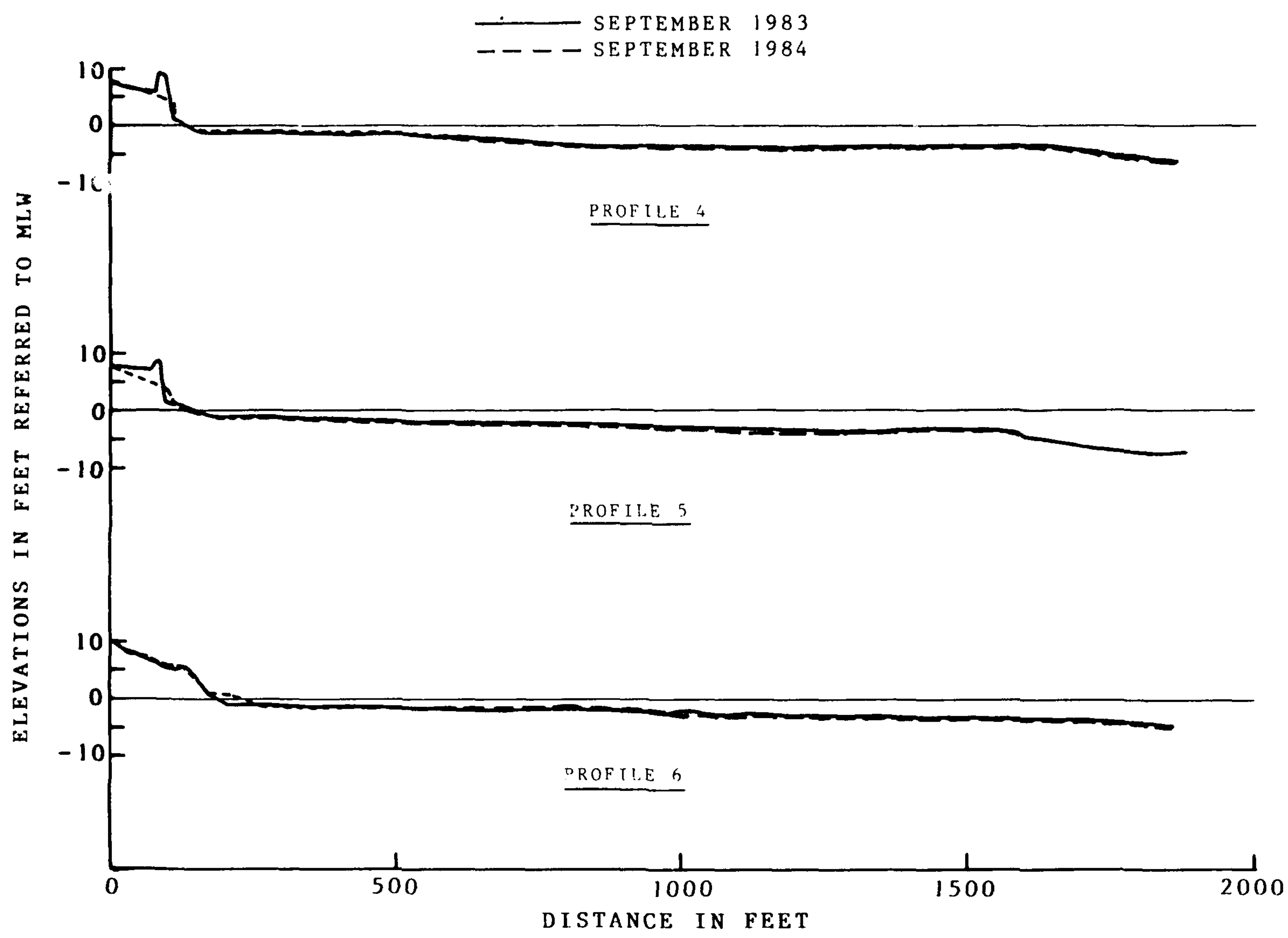

Figure 9. Profile changes, September 1983-September 1984, profiles 4-6

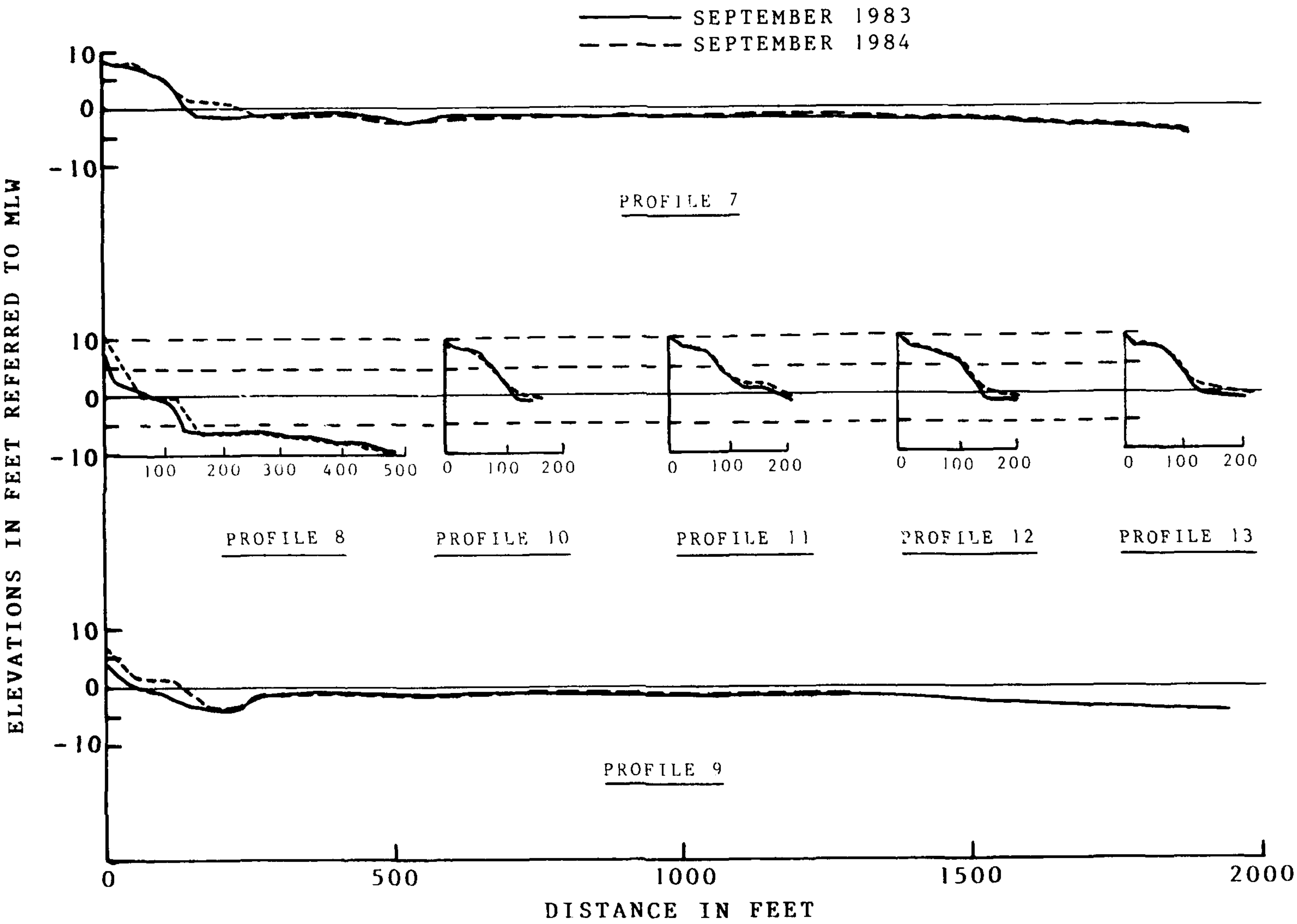

Figure 10. Profile changes, September 1983-September 1984, profiles 7-13

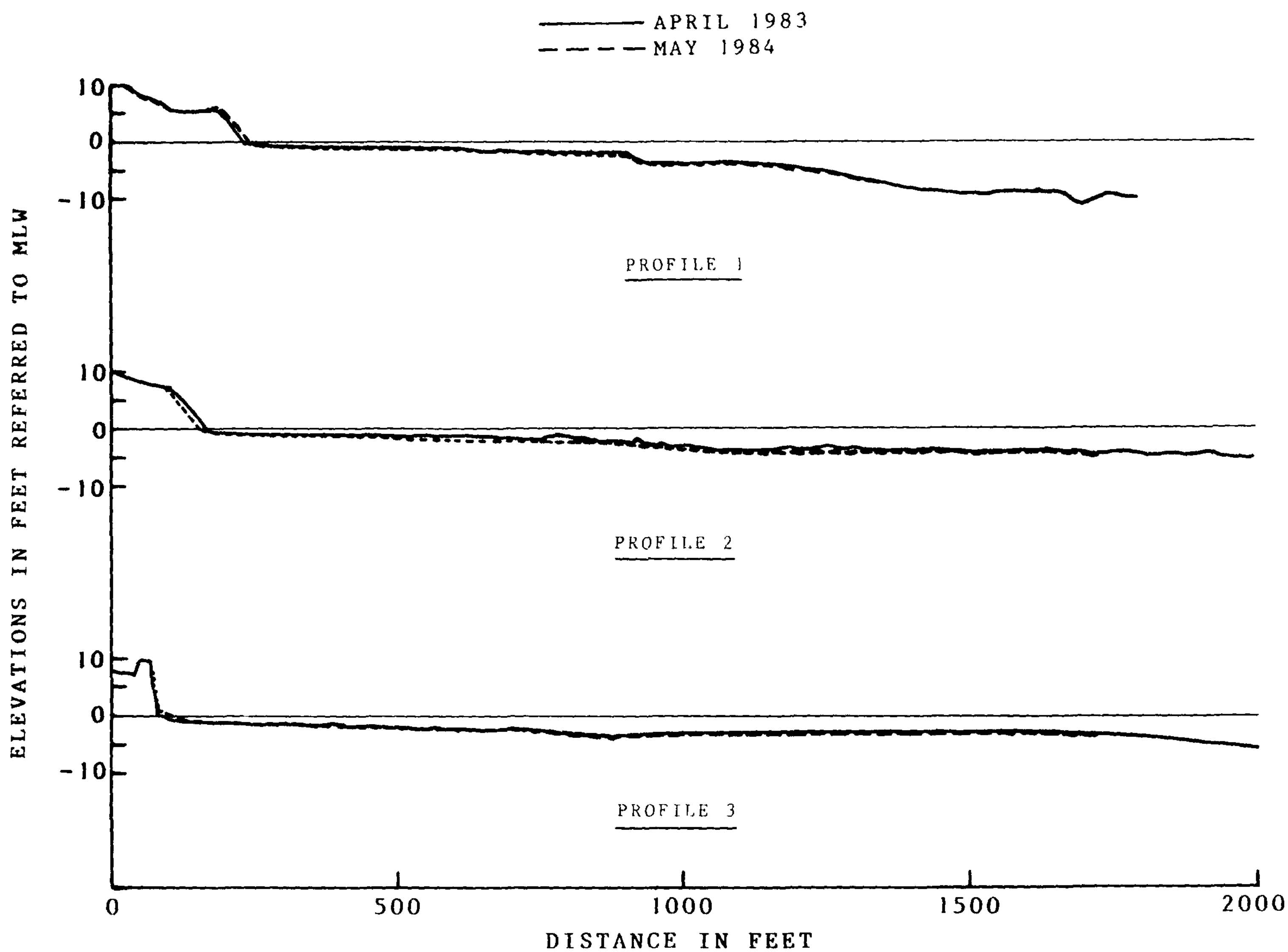

Figure 11. Profile changes, April 1983-May 1984, profiles 1-3

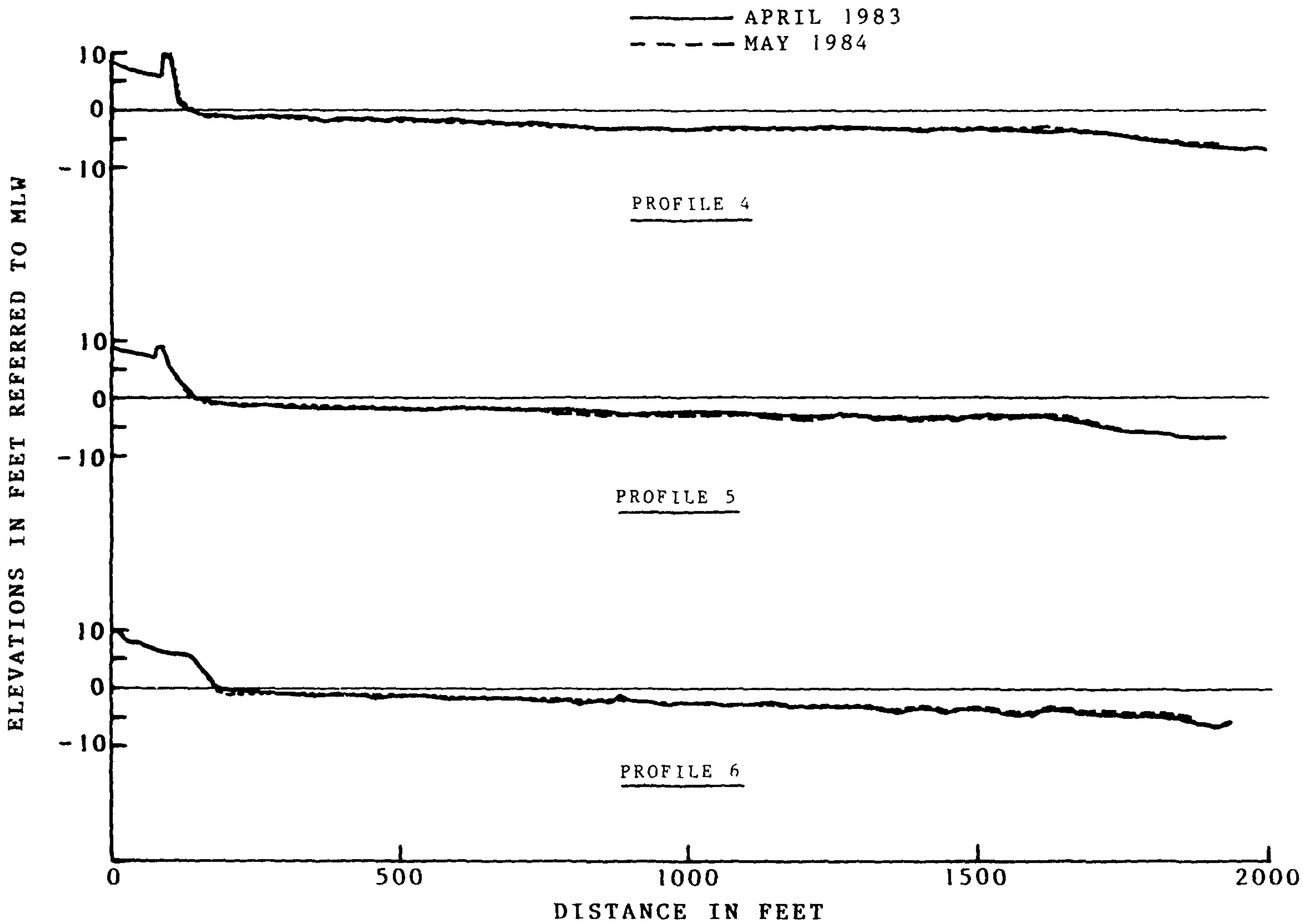

Figure 12. Profile changes, April 1983-May 1984, profiles 4-6

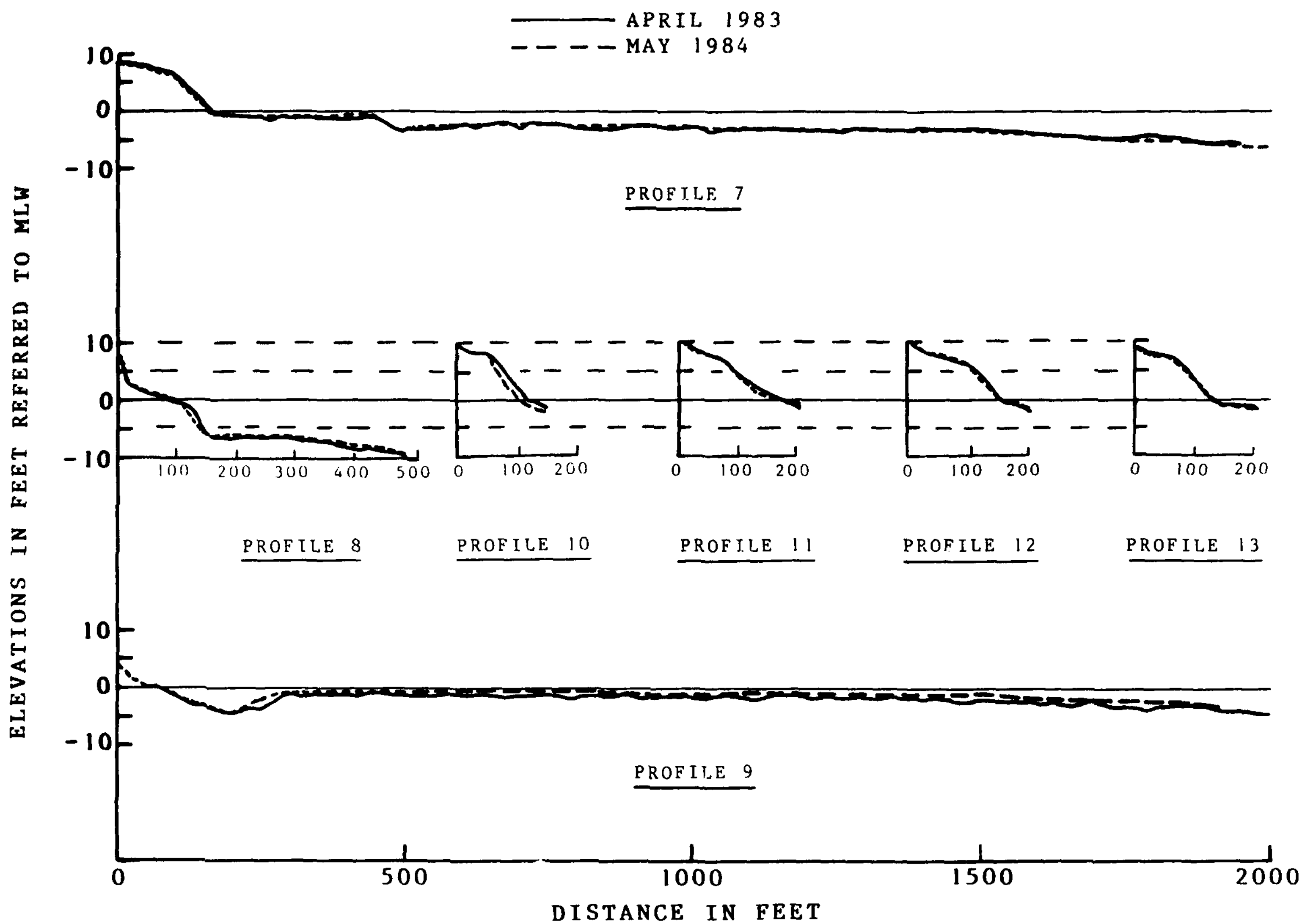

Figure 13. Profile changes, April 1983-May 1984, profiles 7-13

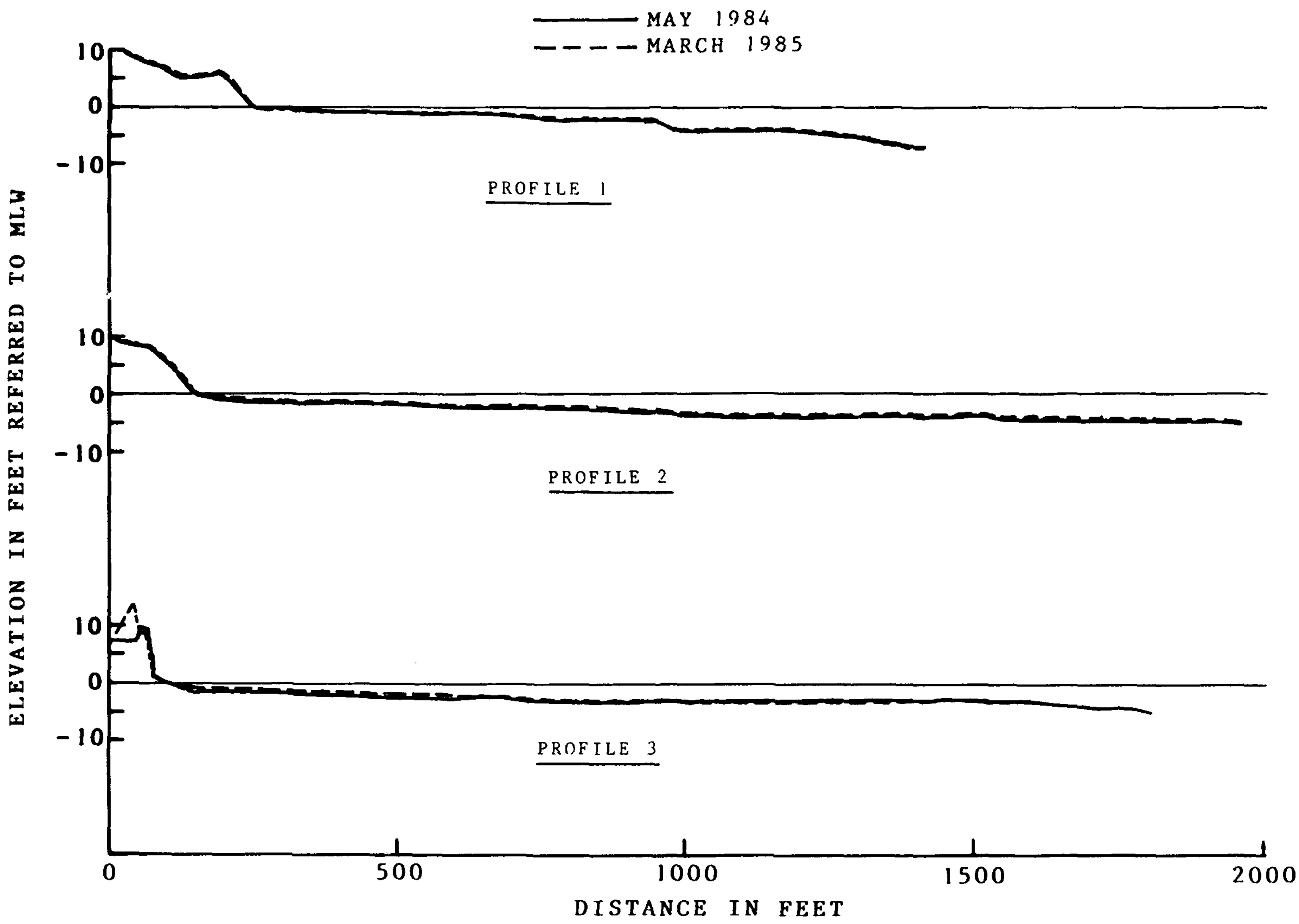

Figure 14. Profile changes, May 1984-March 1985, profiles 1-3

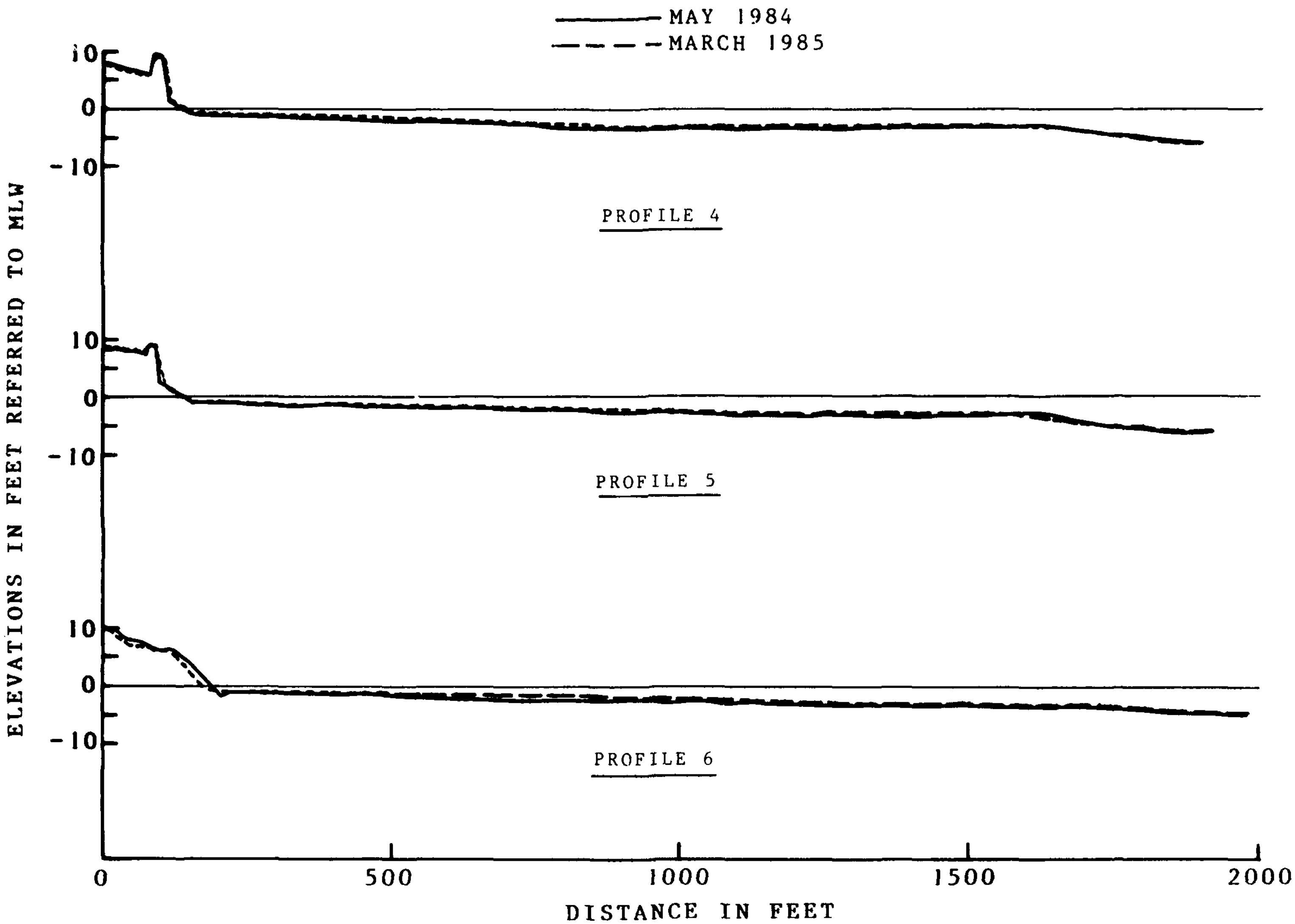

Figure 15. Profile changes, May 1984-March 1985, profiles 4-6

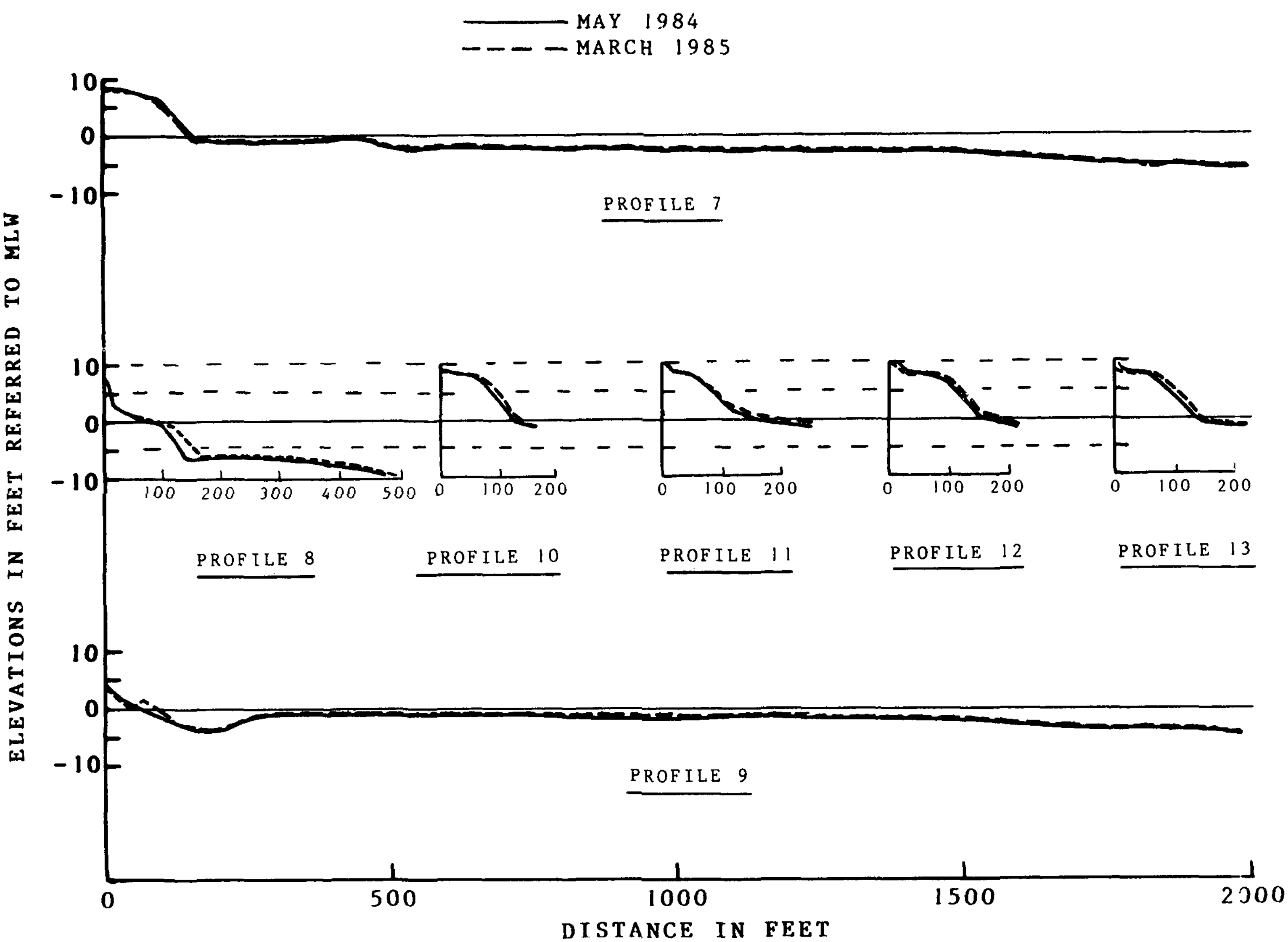

Figure 16. Profile changes, May 1984-March 1985, profiles 7-13

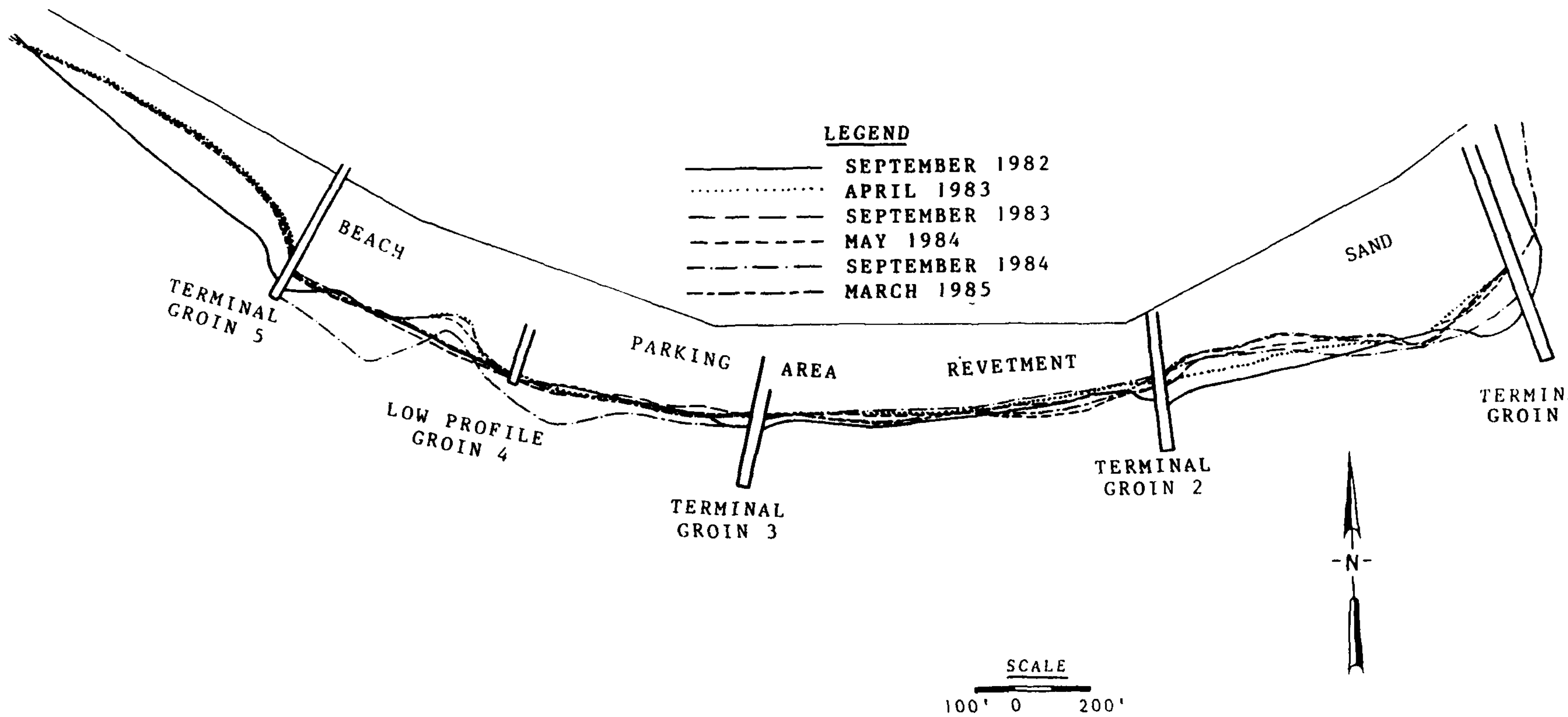

Figure 17. Comparison of shoreline changes (mean low water) during monitoring period

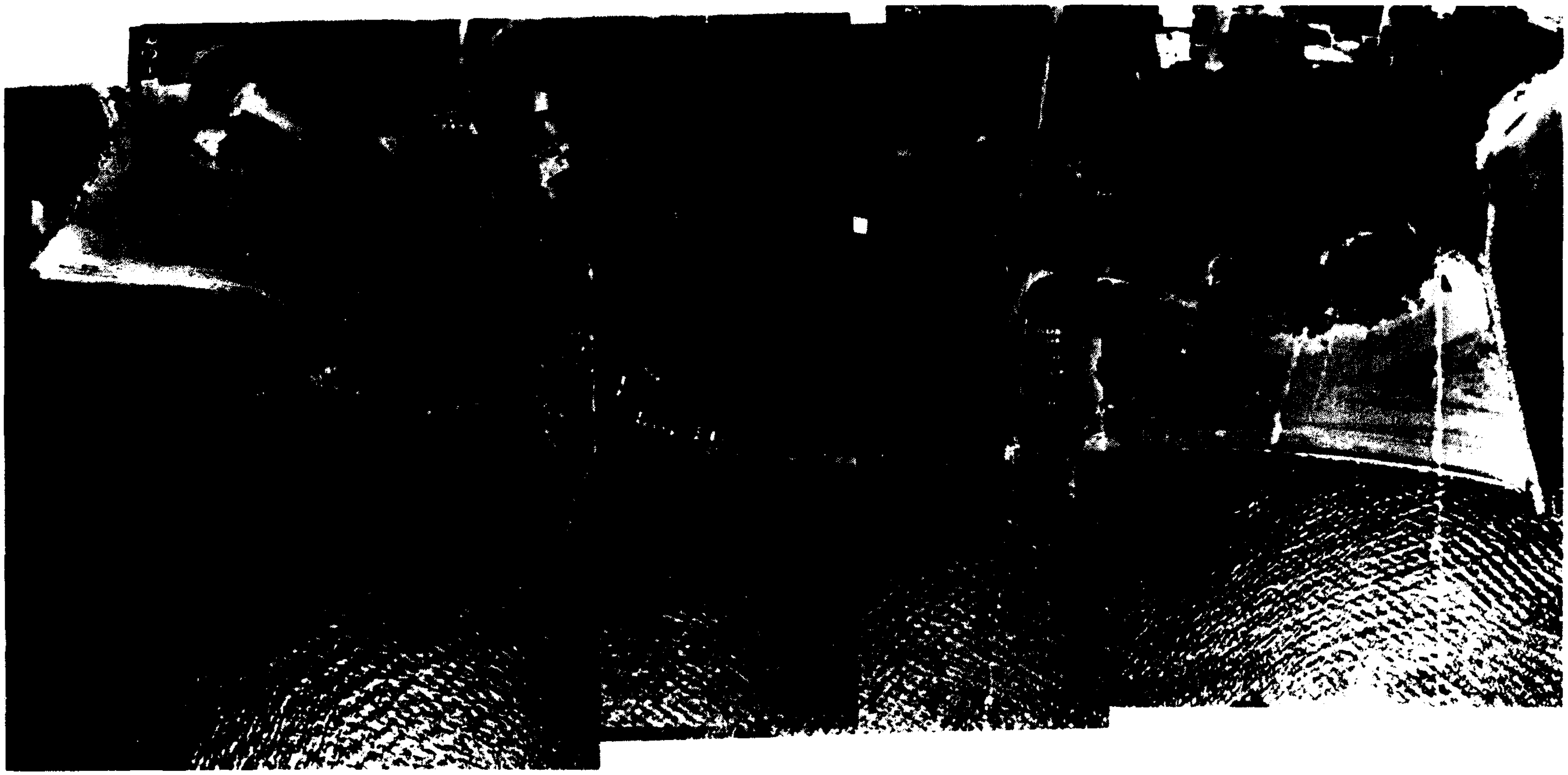

Figure 18. Aerial photo of Oakland Beach, July 1983

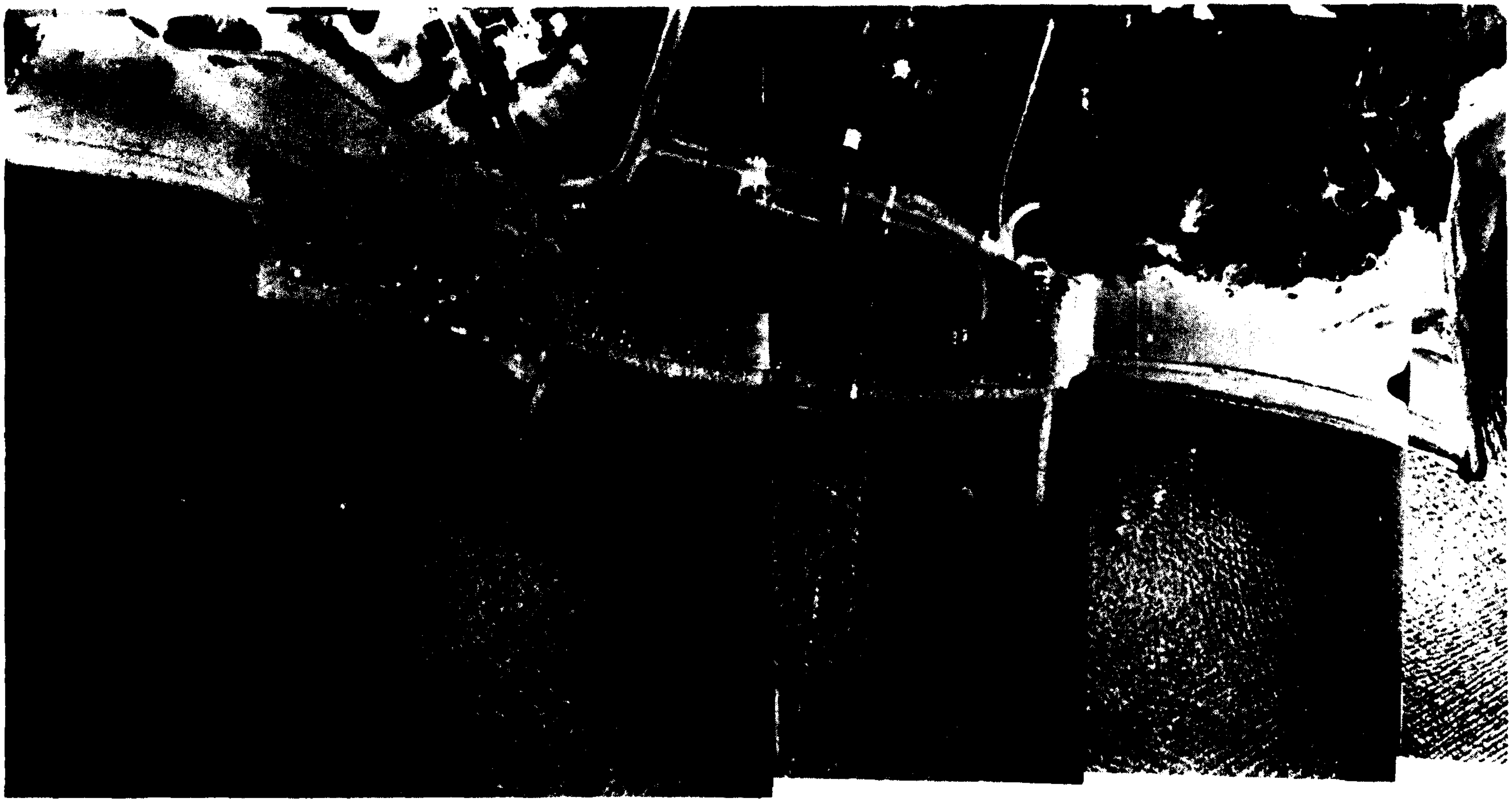

Figure 19. Aerial photo of Oakland Beach, July 1984

Figure 20. Aerial photo of Oakland Beach, July 1985

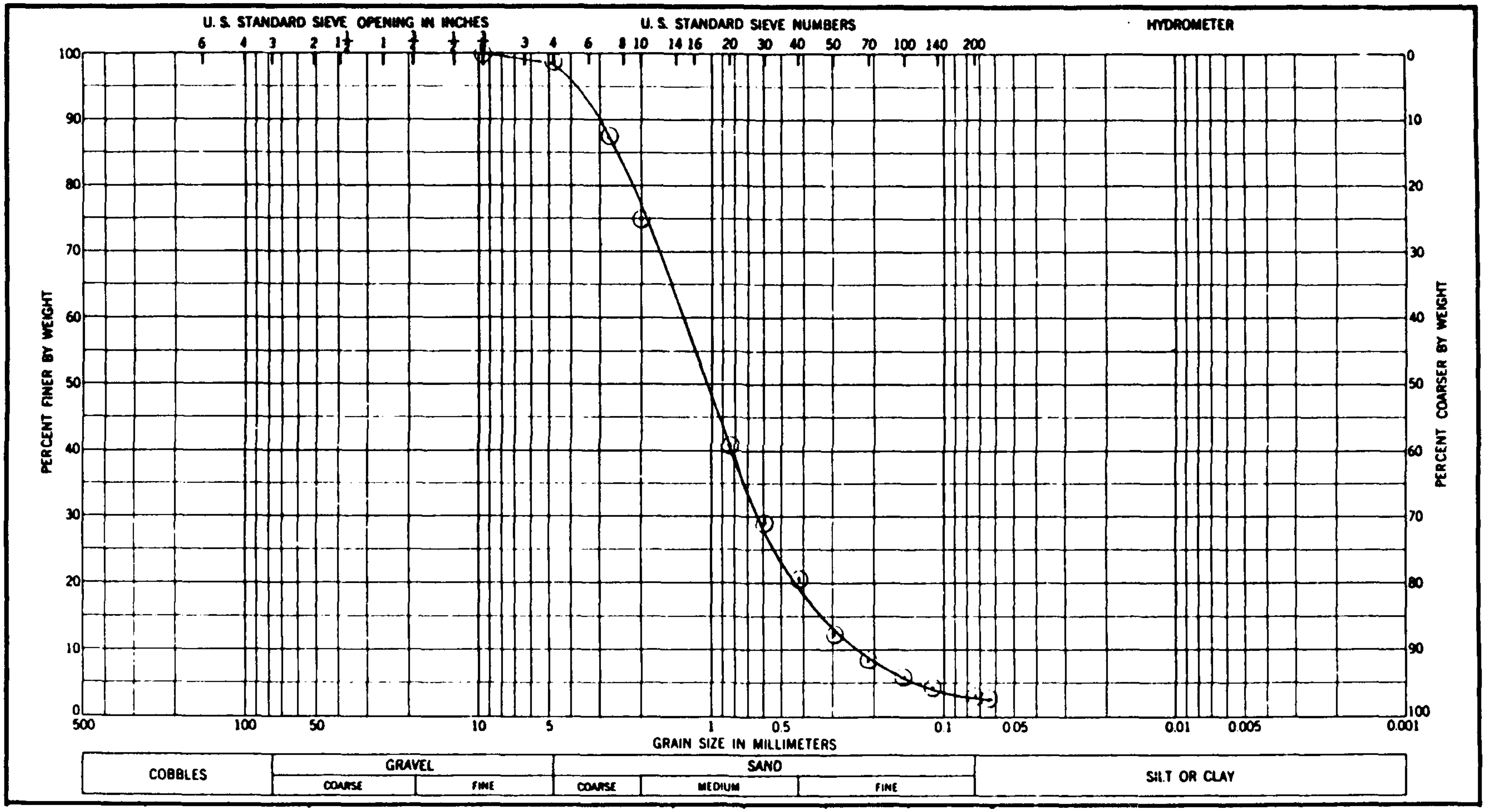

Figure 21. Sediment gradation curve for sample S-1, April 1983

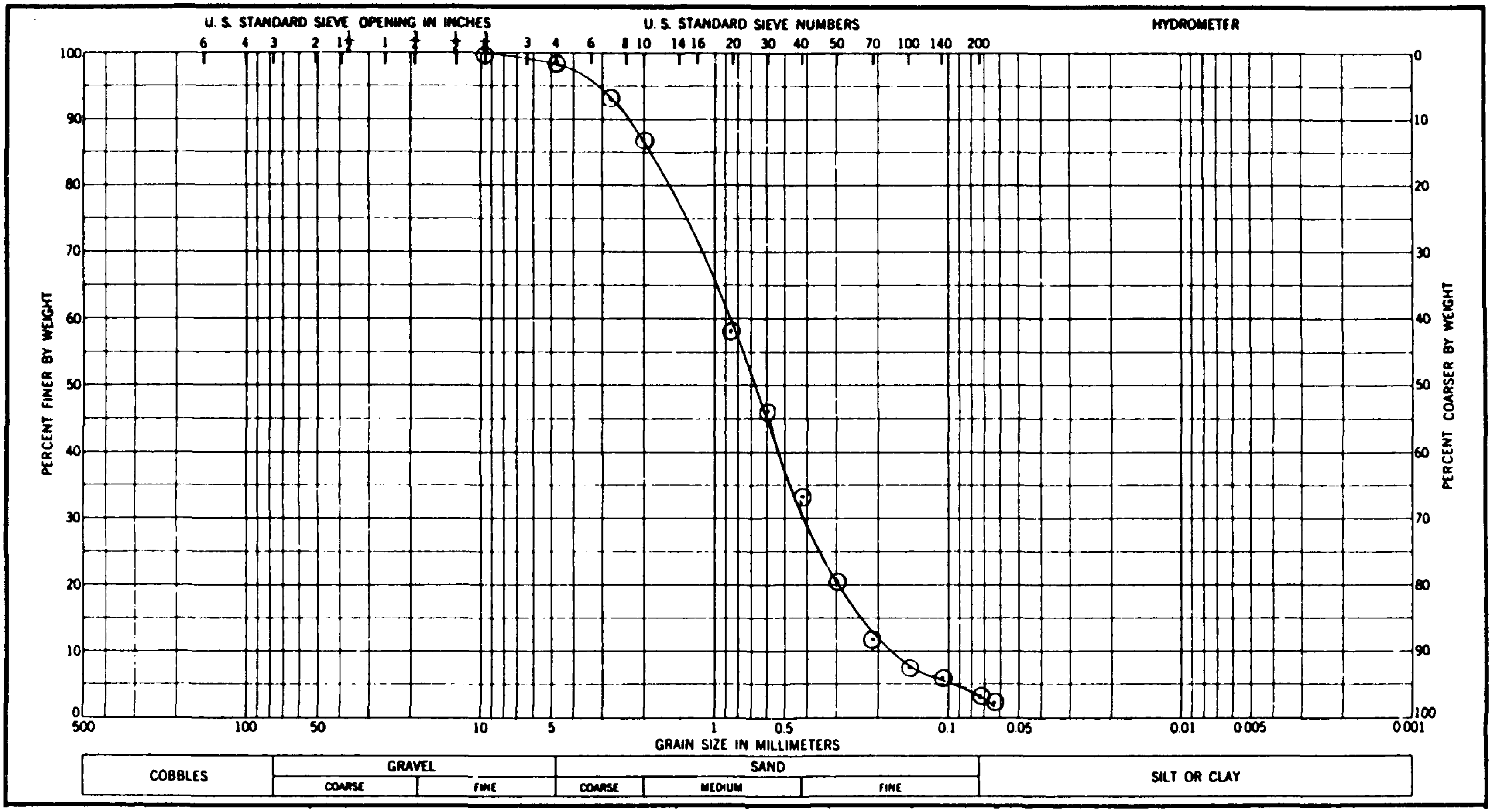

Figure 22. Sediment gradation curve for sample S-2, April 1983

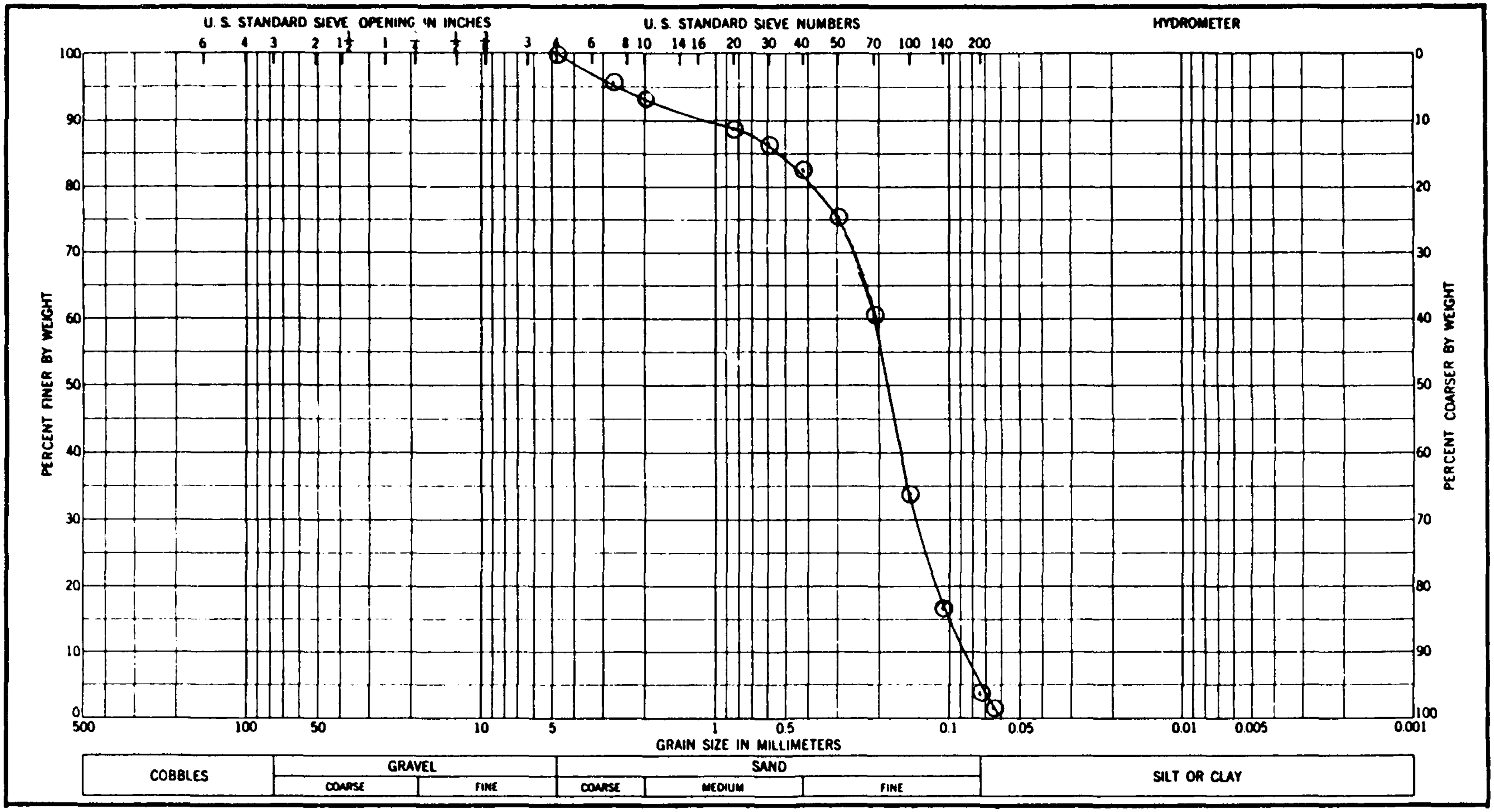

Figure 23. Sediment gradation curve for sample S-3, April 1983

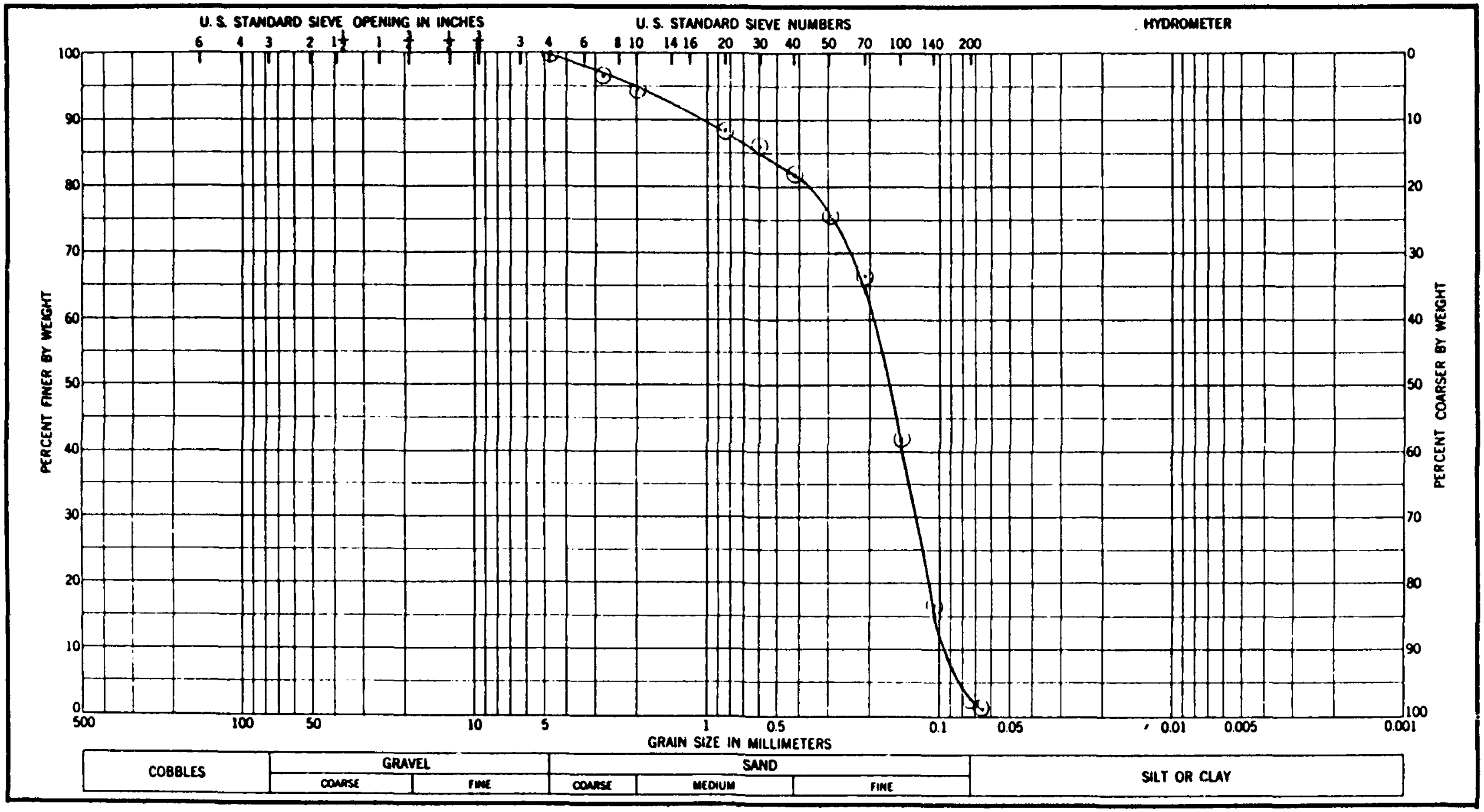

Figure 24. Sediment gradation curve for sample S-4, April 1983

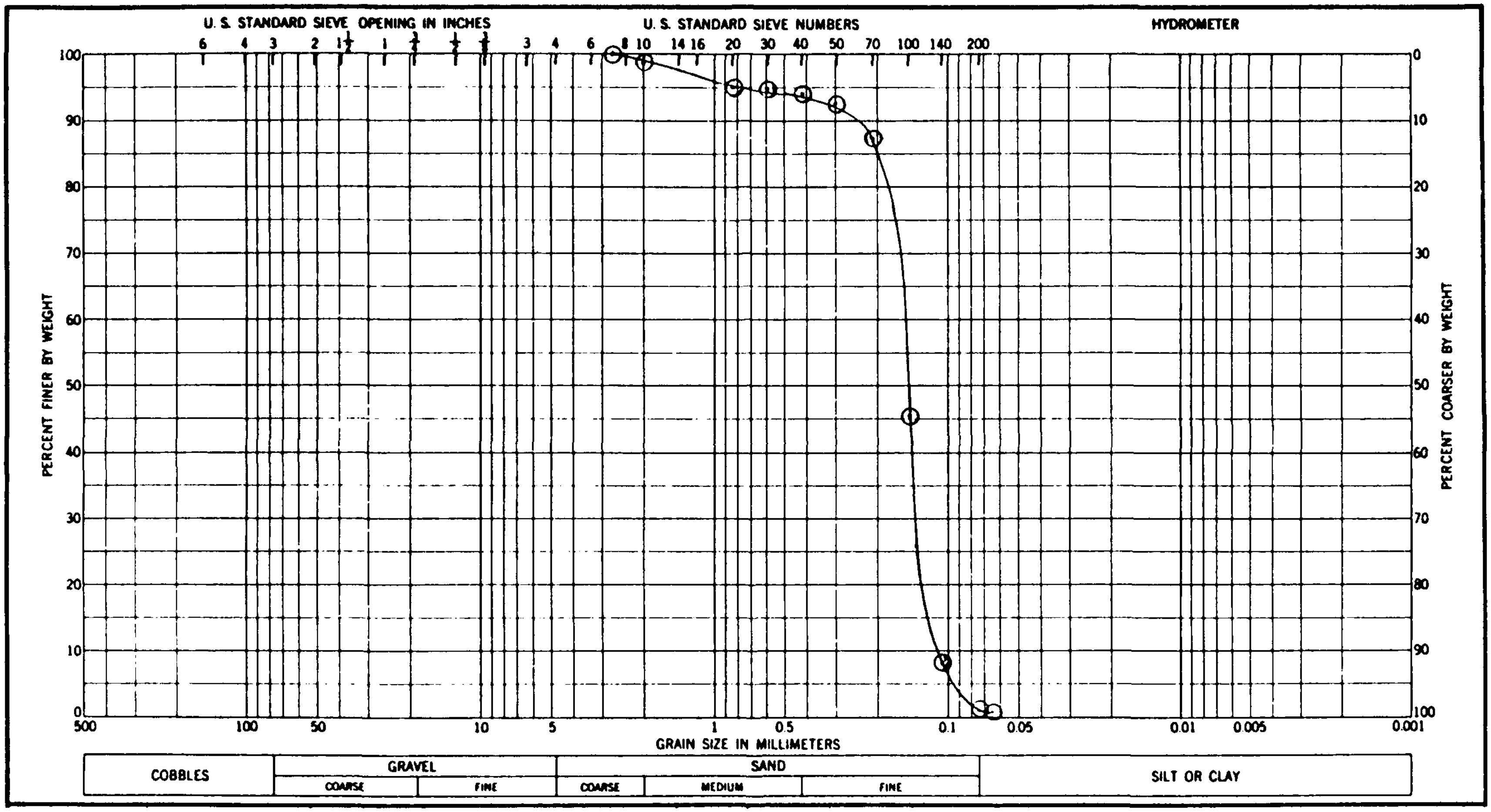

Figure 25. Sediment gradation curve for sample S-5, April 1983

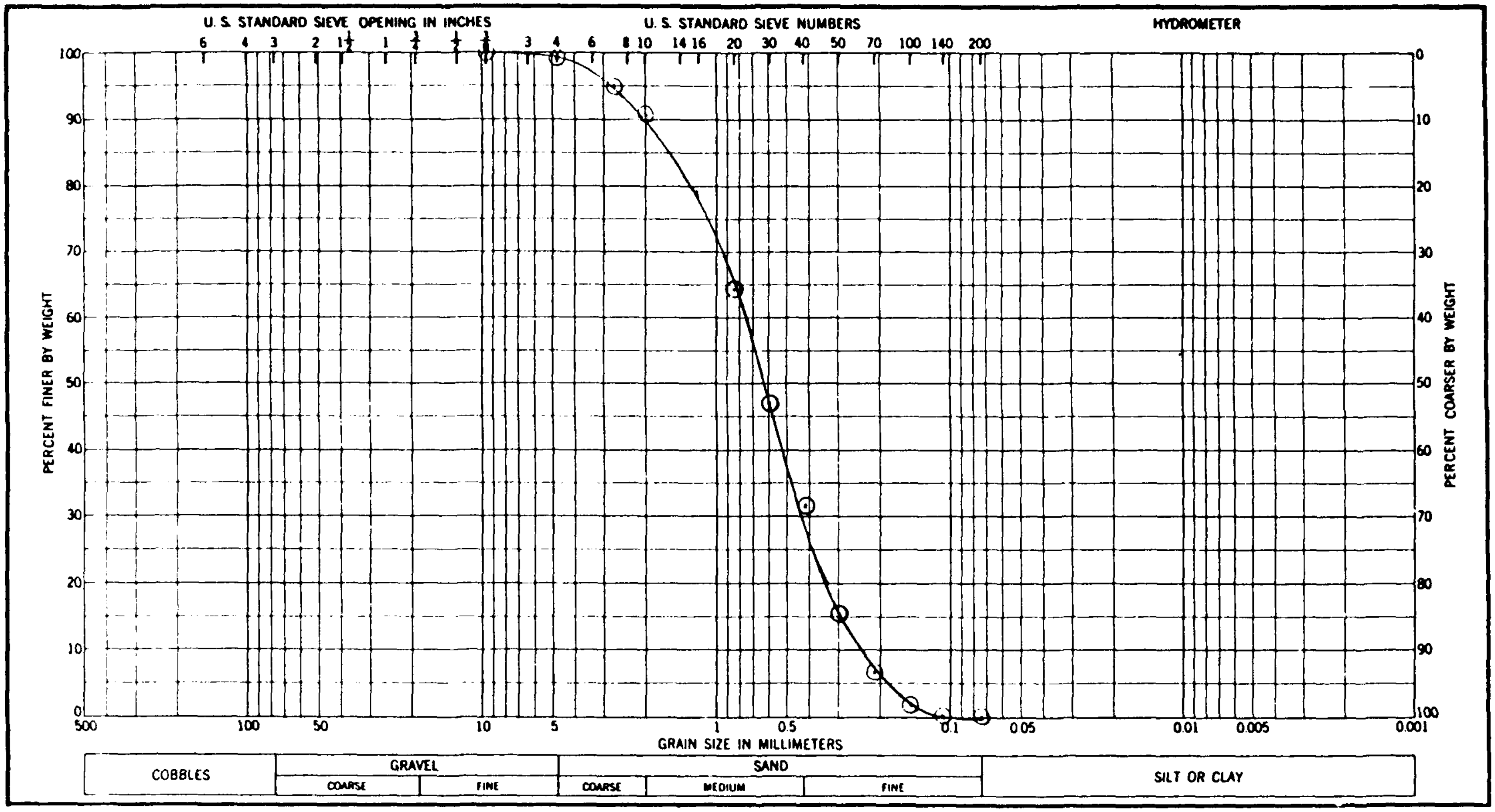

Figure 26. Sediment gradation curve for sample S-7, April 1983

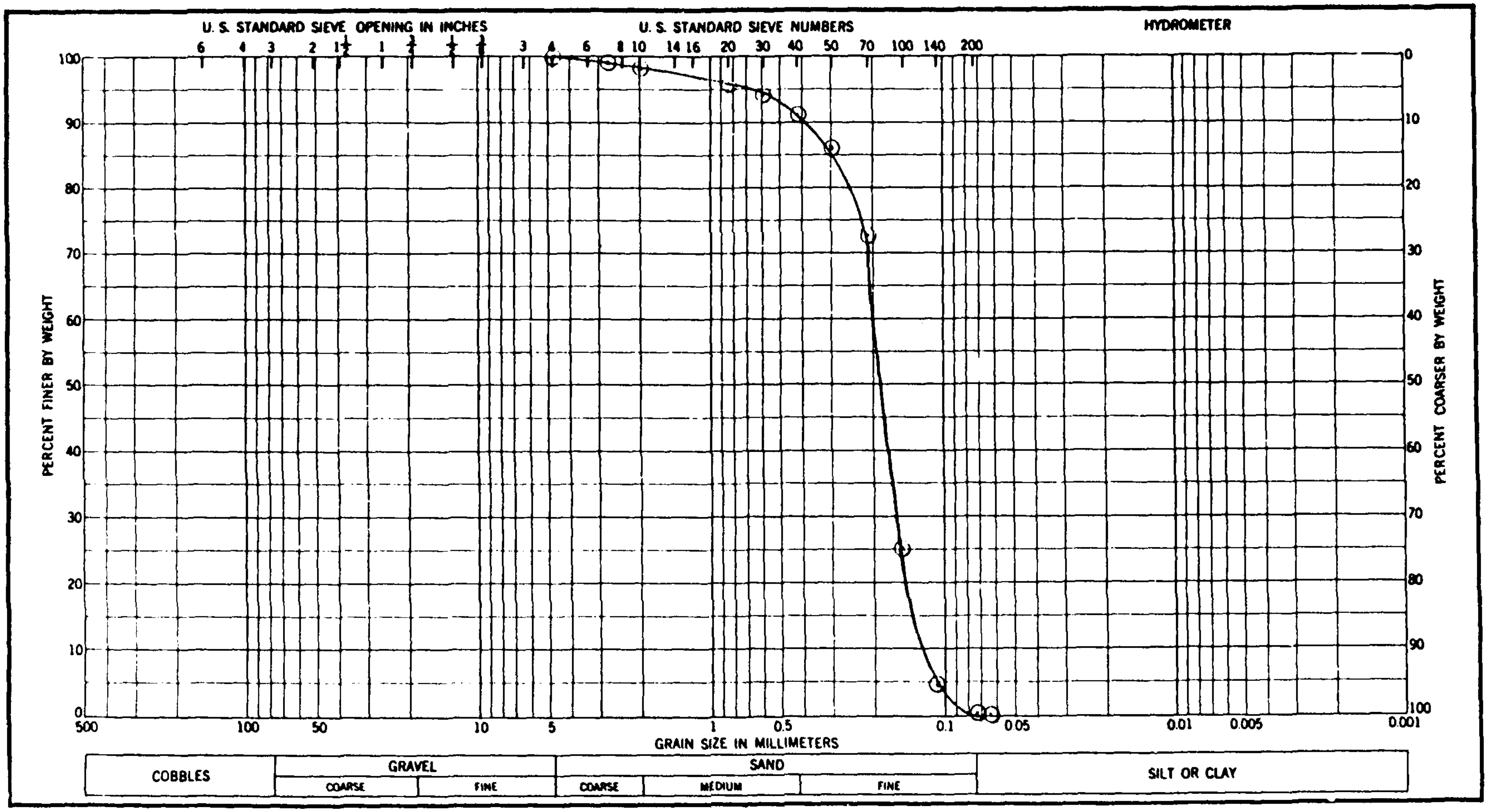

Figure 27. Sediment gradation curve for sample S-8, April 1983

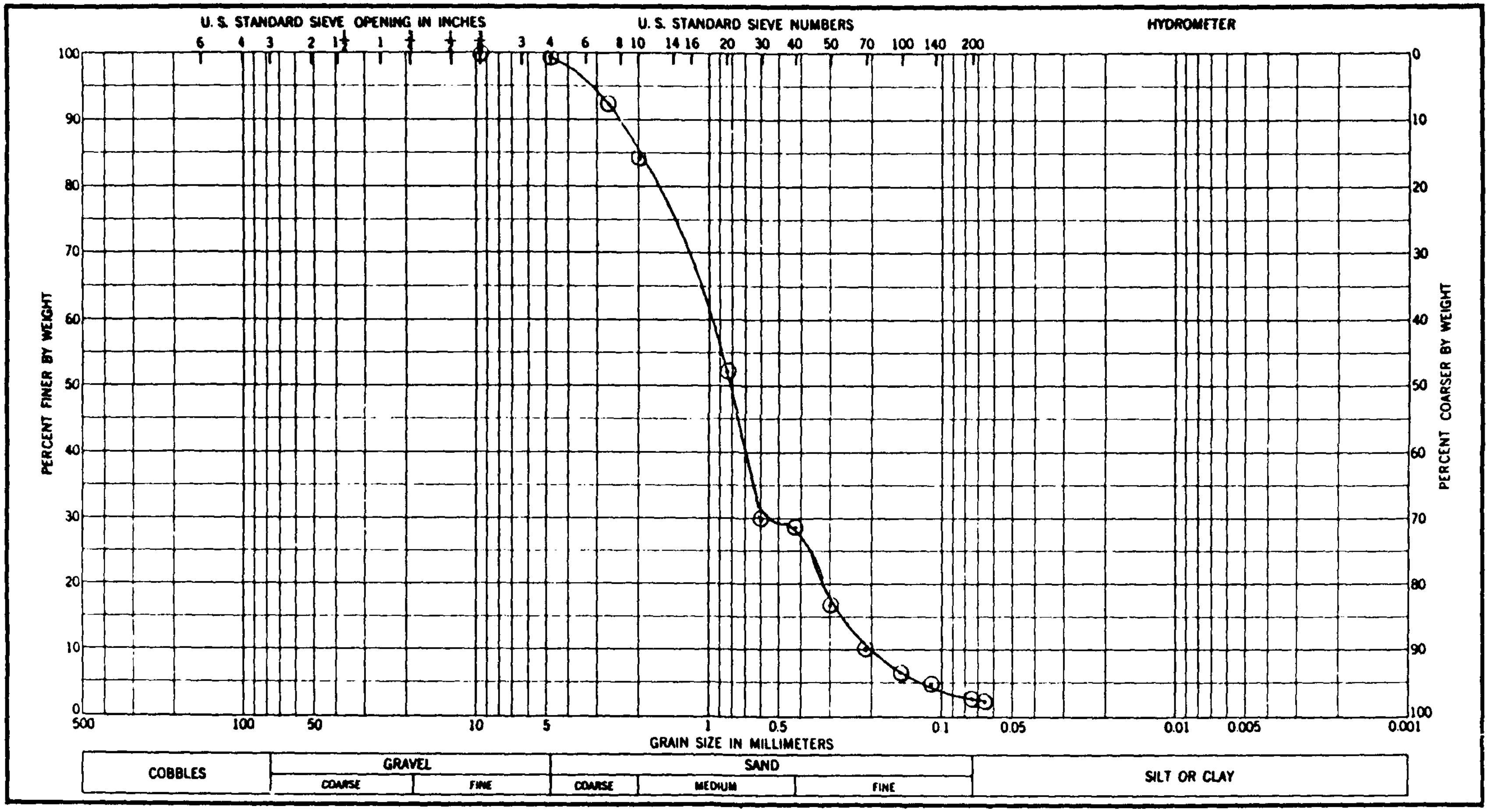

Figure 28. Sediment gradation curve for sample S-9, April 1983

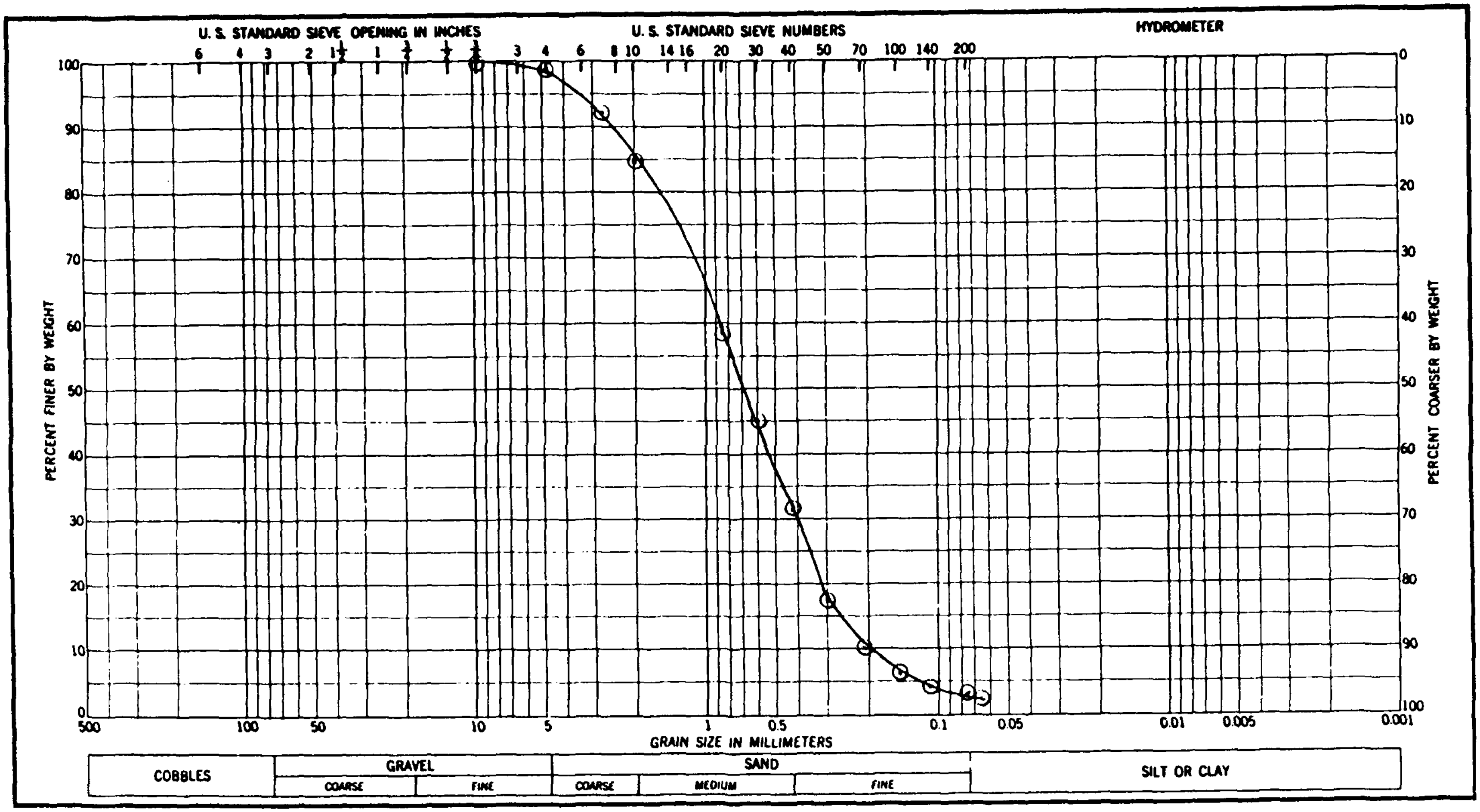

Figure 29. Sediment gradation curve for sample S-10, April 1983

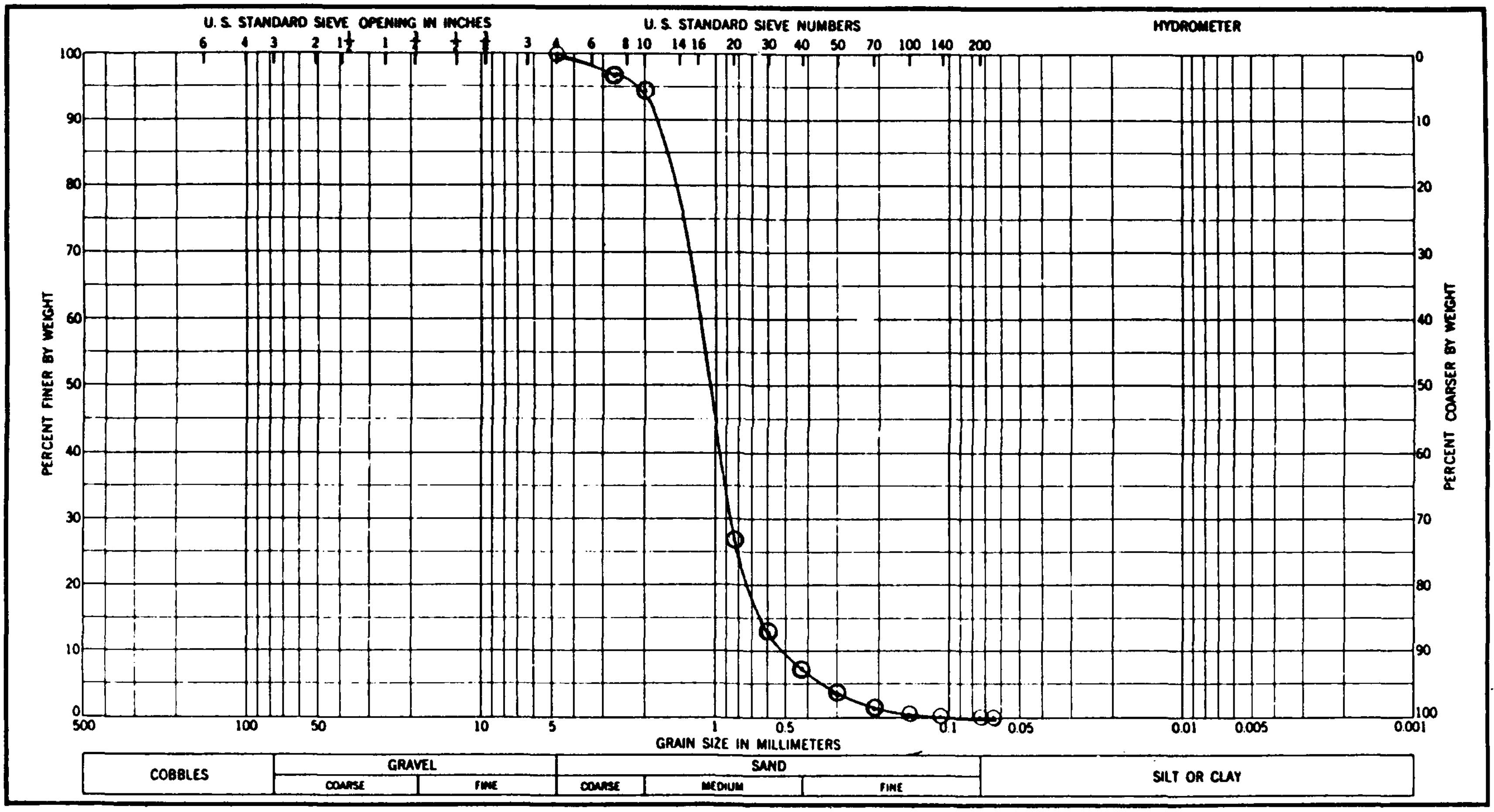

Figure 30. Sediment gradation curve for sample S-11, April 1983

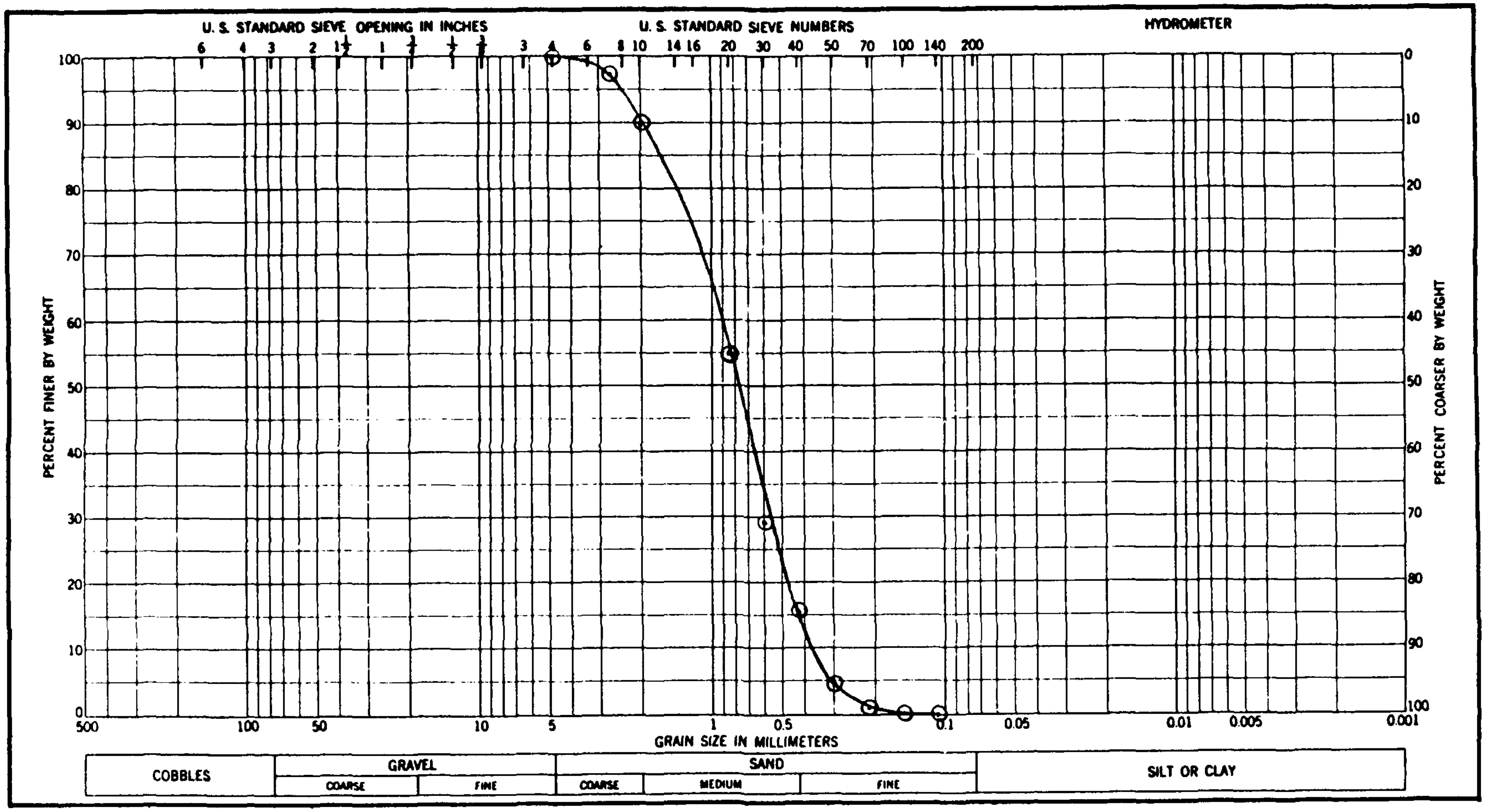

Figure 31. Sediment gradation curve for sample S-12, April 1983

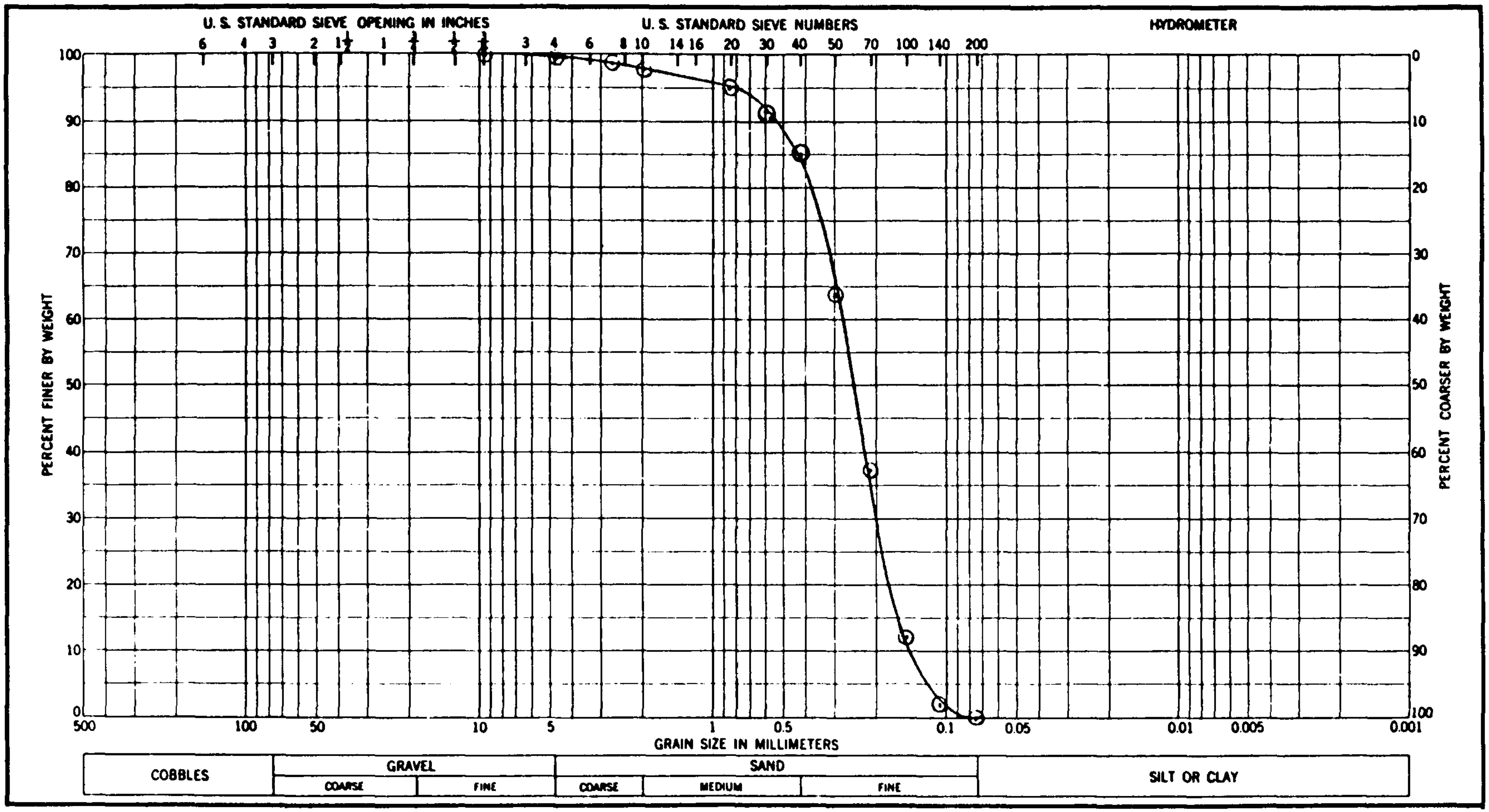

Figure 32. Sediment gradation curve for sample S-13, April 1983

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VARIABLE=CESP ACTUAL WIND SPEED,CE

MOMENTS

N	748	SUM WGTS	748
MEAN	8.84839	SUM	6618.59
STD DEV	4.54898	VARIANCE	20.6932
SKEWNESS	0.87692	KURTOSIS	0.833472
USS	74021.7	CSS	15457.8
CV	51.4103	STD MEAN	0.166327
T:MEAN=0	53.1987	PROB>|T|	0.0001
SGN RANK	140063	PROB>|S|	0.0001
NUM ^= 0	748		
D:NORMAL	0.104193	PROB>D	<.01

QUANTILES(DEF=4)

100% MAX	28.6707	99%	21.7202
75% Q3	11.2945	95%	17.3762
50% MED	7.81929	90%	14.7698
25% Q1	5.21286	10%	3.47524
0% MIN	1.73762	5%	2.60643
		1%	1.73762
RANGE	26.9331		
Q3-Q1	6.08167		
MODE	5.21286		

EXTREMES

LOWEST	HIGHEST
1.73762	23.4579
1.73762	24.3267
1.73762	26.9331
1.73762	26.9331
1.73762	28.6707

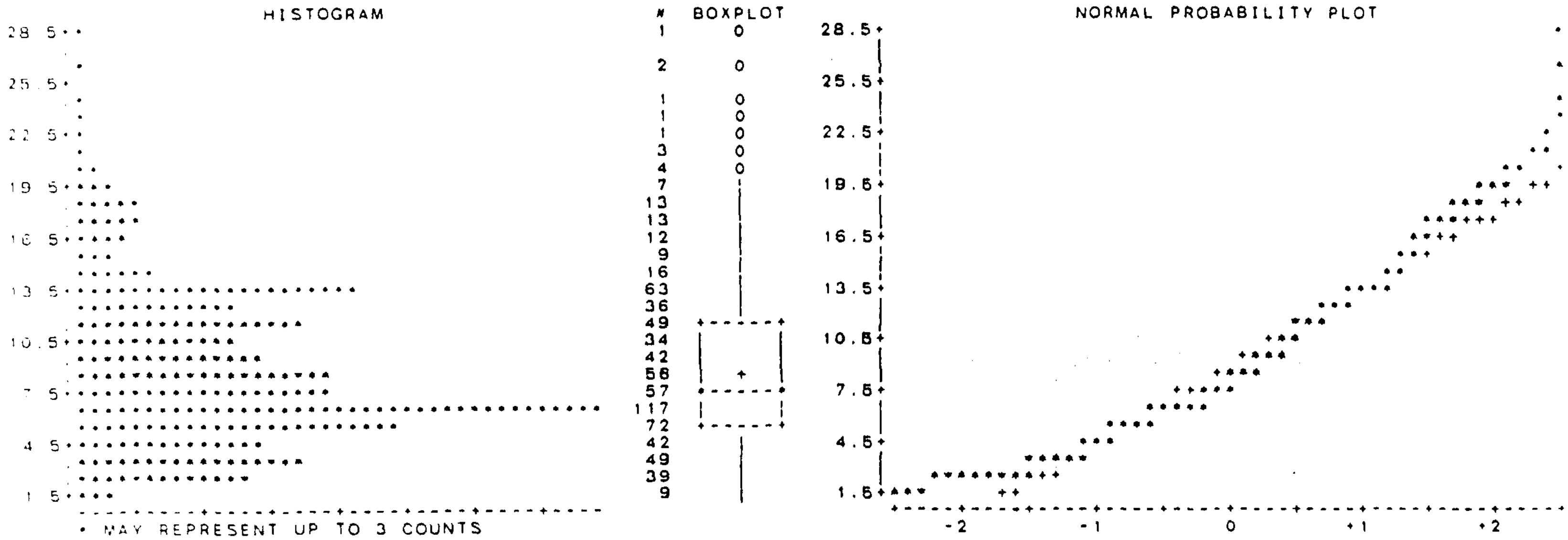

Figure 33. Oakland Beach wind speed summary statistics

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                                   SAS                    14:23 TUESDAY, AUGUST 15, 1989    4

                               UNIVARIATE

VARIABLE=CEO          ACTUAL WIND DIRECTION,CE

                MOMENTS                              QUANTILES(DEF=4)                          EXTREMES

N               748  SUM WGTS        748     100% MAX      280     99%     278      LOWEST    HIGHEST
MEAN        184.154  SUM          137747      75% Q3       233     95%     271           3        279
STD DEV     63.9927  VARIANCE    4095.07      50% MED      194     90%     261          14        279
SKEWNESS  -0.652994  KURTOSIS  -0.352935      25% Q1    143.75     10%    85.9          17        280
USS        28425641  CSS         3059015       0% MIN        3      5%    59.9          18        280
CV          34.7496  STD MEAN    2.33981                             1%   22.47          19        280
T:MEAN=0    78.7047  PROB>|T|     0.0001     RANGE         277
SGN RANK     140063  PROB>|S|     0.0001     Q3-Q1       89.25
NUM ^= 0        748                          MODE          187
D:NORMAL  0.0886389  PROB>D         <.01
```

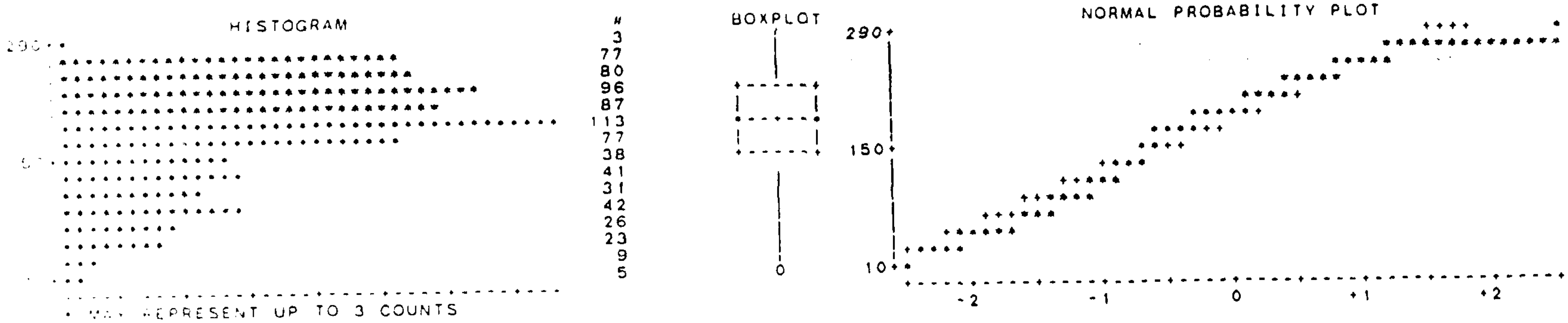

Figure 34. Oakland Beach wind direction summary statistics

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                                         SAS                     14:23 TUESDAY, AUGUST 15, 1989

                                      UNIVARIATE

VARIABLE=NWSP

              MOMENTS                                  QUANTILES(DEF=4)                          EXTREMES

N               748  SUM WGTS        748  100% MAX        32      99%   23.51          LOWEST    HIGHEST
MEAN        9.36364  SUM            7004   75% Q3         12      95%      17               3         25
STD DEV     4.21623  VARIANCE    17.7766   50% MED         9      90%      15               3         25
SKEWNESS    1.06551  KURTOSIS    2.09846   25% Q1          6      10%       5               3         26
USS           78862  CSS         13279.1    0% MIN         3       5%       4               3         28
CV          45.0277  STD MEAN    0.15416                           1%       3               3         32
T:MEAN=0    60.7395  PROB>|T|     0.0001   RANGE          29
SGN RANK     140063  PROB>|S|     0.0001   Q3-Q1           6
NUM ¬= 0        748                        MODE            8
D:NORMAL   0.113444  PROB>D         <.01
```

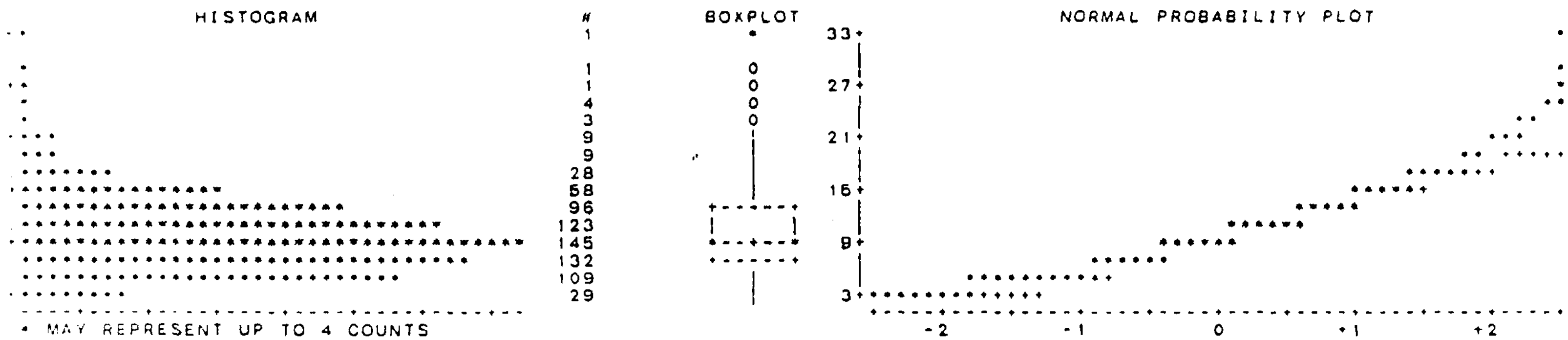

Figure 35. T. F. Green Airport wind speed summary statistics

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VARIABLE=NWD

MOMENTS

N	748	SUM WGTS	748
MEAN	169.345	SUM	126670
STD DEV	55.8946	VARIANCE	3124.2
SKEWNESS	-0.597881	KURTOSIS	-0.226427
USS	23784700	CSS	2333779
CV	33.0063	STD MEAN	2.04371
T:MEAN=0	82.8616	PROB>|T|	0.0001
SGN RANK	140063	PROB>|S|	0.0001
NUM ¬= 0	748		
D:NORMAL	0.11944	PROB>D	<.01

QUANTILES(DEF=4)

100% MAX	290	99%	275.1
75% Q3	210	95%	250
50% MED	180	90%	230
25% Q1	140	10%	70
0% MIN	40	5%	60
		1%	50
RANGE	250		
Q3-Q1	70		
MODE	180		

EXTREMES

LOWEST	HIGHEST
40	280
50	280
50	280
50	280
50	290

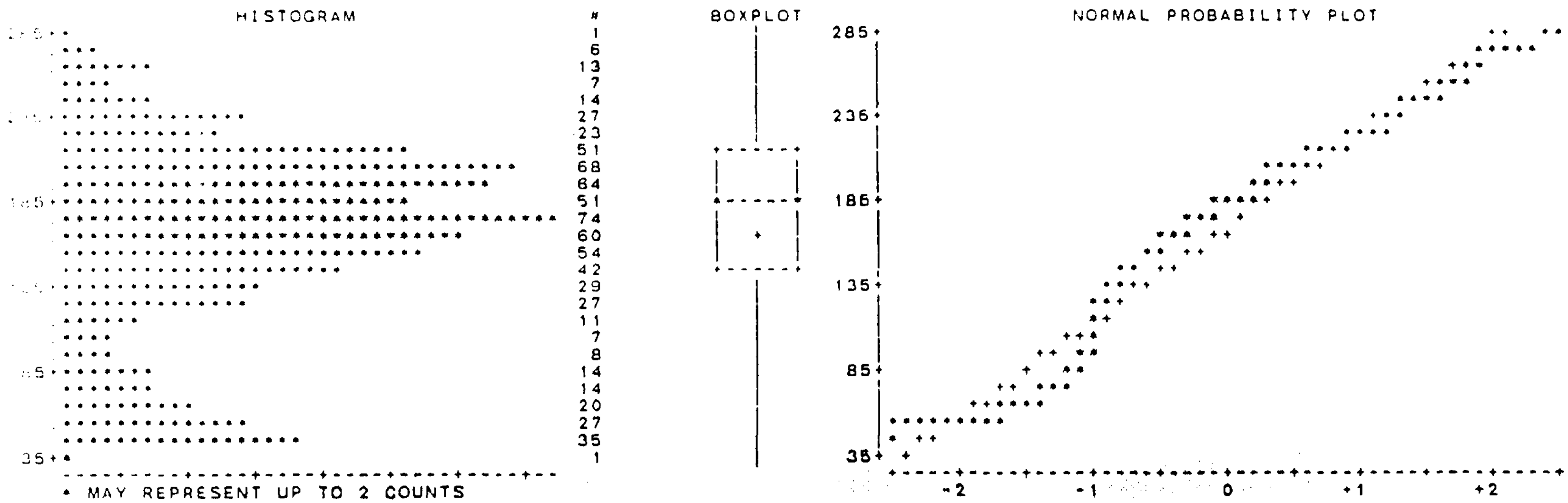

Figure 36. T. F. Green Airport wind direction summary statistics

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                                         SAS                          14:23 TUESDAY, AUGUST 15, 1989    6
                                      UNIVARIATE

VARIABLE=DD             DIRECTION DIFFERENCE NWS-CE

                MOMENTS                                  QUANTILES(DEF=4)                      EXTREMES

N              748  SUM WGTS         748     100% MAX       168     99%   152.57          LOWEST    HIGHEST
MEAN       -15.496  SUM           -11591      75% Q3      -2.25     95%    37.65            -178        157
STD DEV    34.7673  VARIANCE     1208.76      50% MED       -19     90%     16.1            -124        162
SKEWNESS   1.74796  KURTOSIS     8.20583      25% Q1        -34     10%      -47            -111        164
USS        1082561  CSS           902947       0% MIN      -178      5%      -59            -106        166
CV        -224.363  STD MEAN     1.27122                             1%   -80.02             -98        168
T:MEAN=0  -12.1899  PROB>|T|      0.0001     RANGE          346
SGN RANK  -88579.5  PROB>|S|      0.0001     Q3-Q1        31.75
NUM ^= 0       740                           MODE           -19
D:NORMAL  0.112649  PROB>D          <.01
```

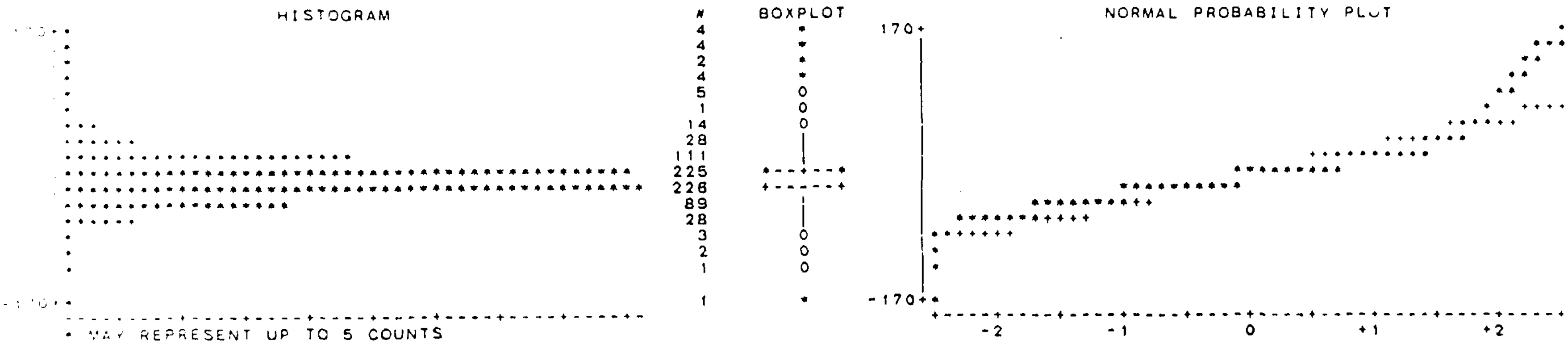

Figure 37. Wind direction difference summary statistics

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VARIABLE=SD   SPEED DIFFERENCE NWS-CE

MOMENTS

N	748	SUM WGTS	748
MEAN	0.515251	SUM	385.407
STD DEV	3.45721	VARIANCE	11.9523
SKEWNESS	0.302474	KURTOSIS	0.994973
USS	9126.95	CSS	8928.37
CV	670.976	STD MEAN	0.126408
T:MEAN=0	4.07609	PROB>|T|	0.0001
SGN RANK	22481	PROB>|S|	.000143018
NUM ¬= 0	748		
D:NORMAL	0.0379129	PROB>D	<.01

QUANTILES(DEF=4)

100% MAX	17.2302	99%	9.46048
75% Q3	2.65596	95%	6.04952
50% MED	0.524761	90%	4.6609
25% Q1	-1.90096	10%	-3.90096
0% MIN	-9.03215	5%	-5.03215
		1%	-7.3388
RANGE	26.2624		
Q3-Q1	4.55691		
MODE	2.78714		

EXTREMES

LOWEST	HIGHEST
-9.03215	10.099
-8.98262	11.099
-8.85143	13.656
-8.63858	14.7055
-8.03215	17.2302

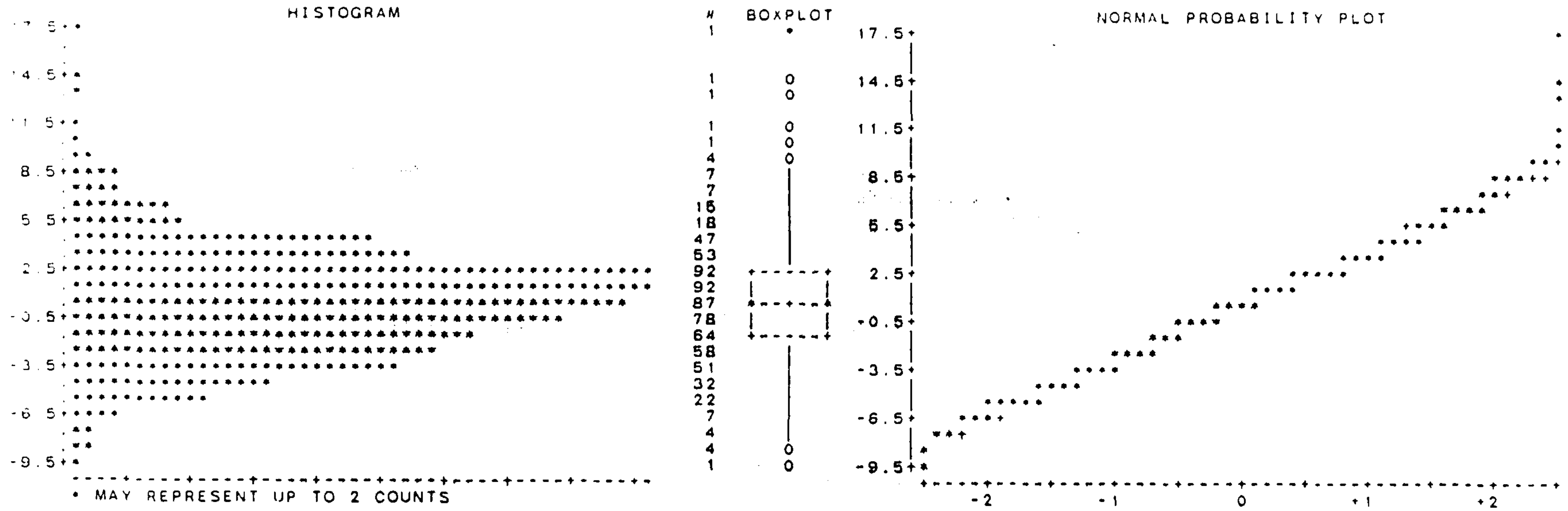


Figure 38. Wind speed difference summary statistics

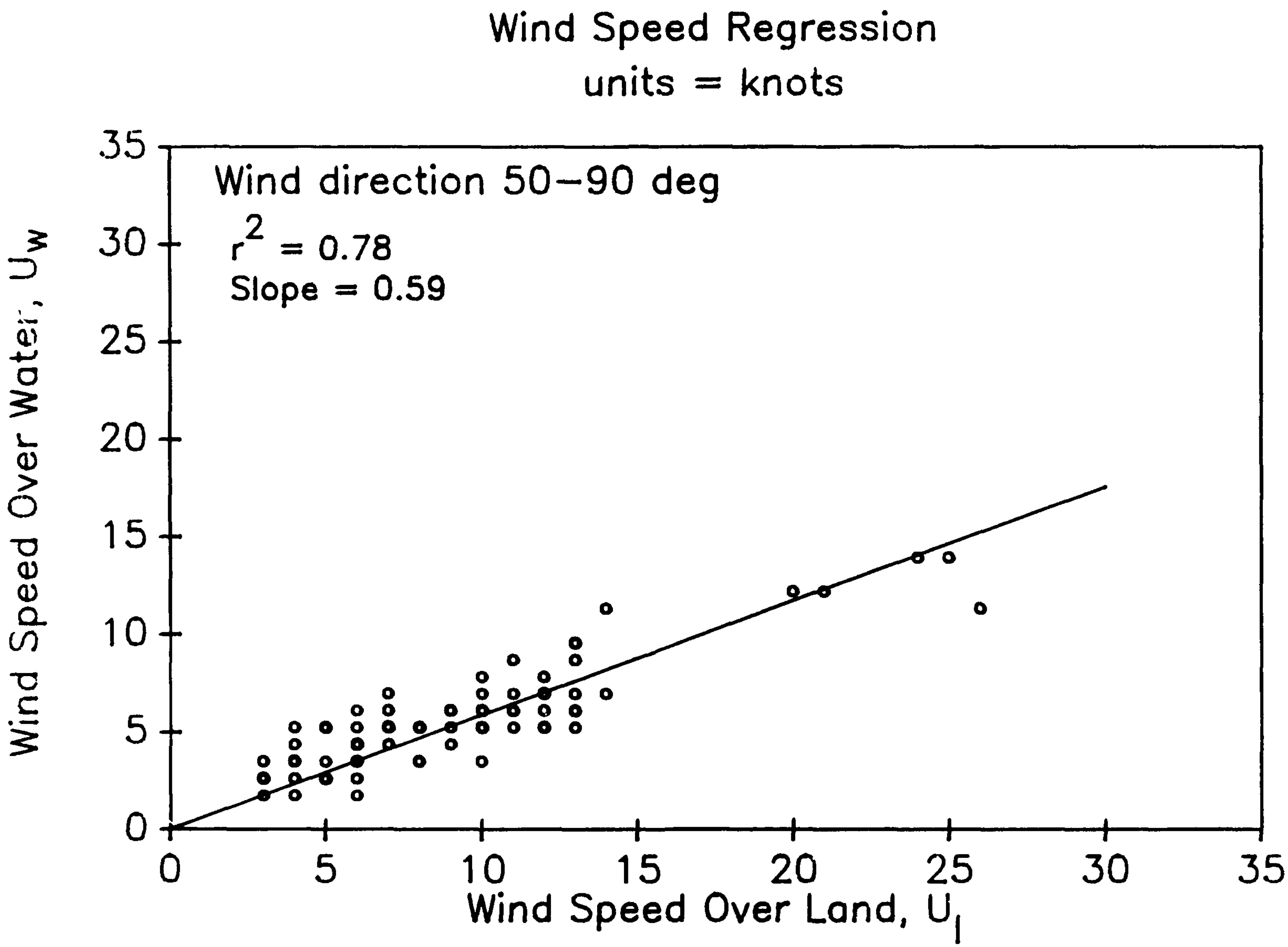


Figure 39. Wind speed regressions, 50-90 deg

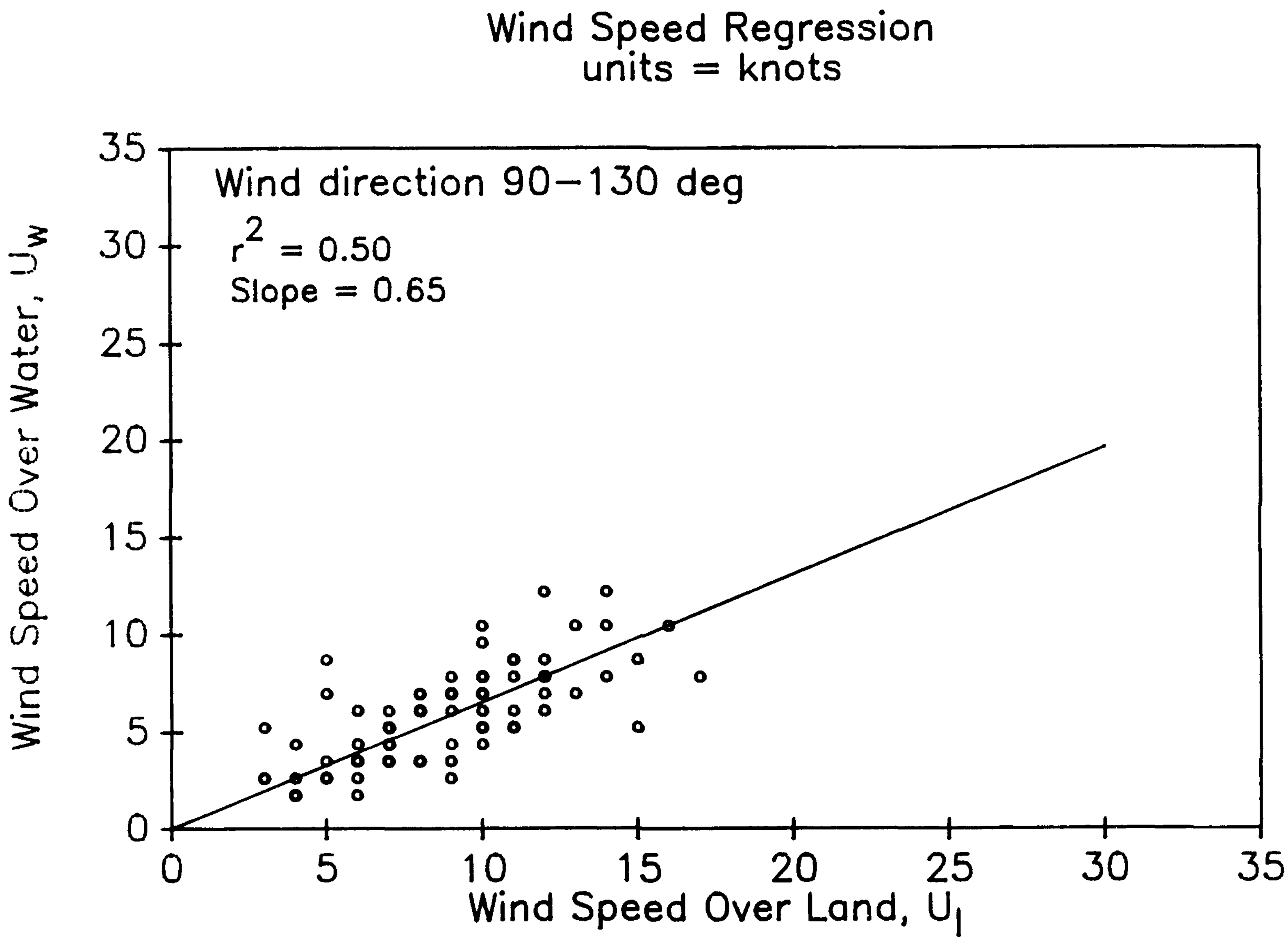


Figure 40. Wind speed regressions, 90-130 deg

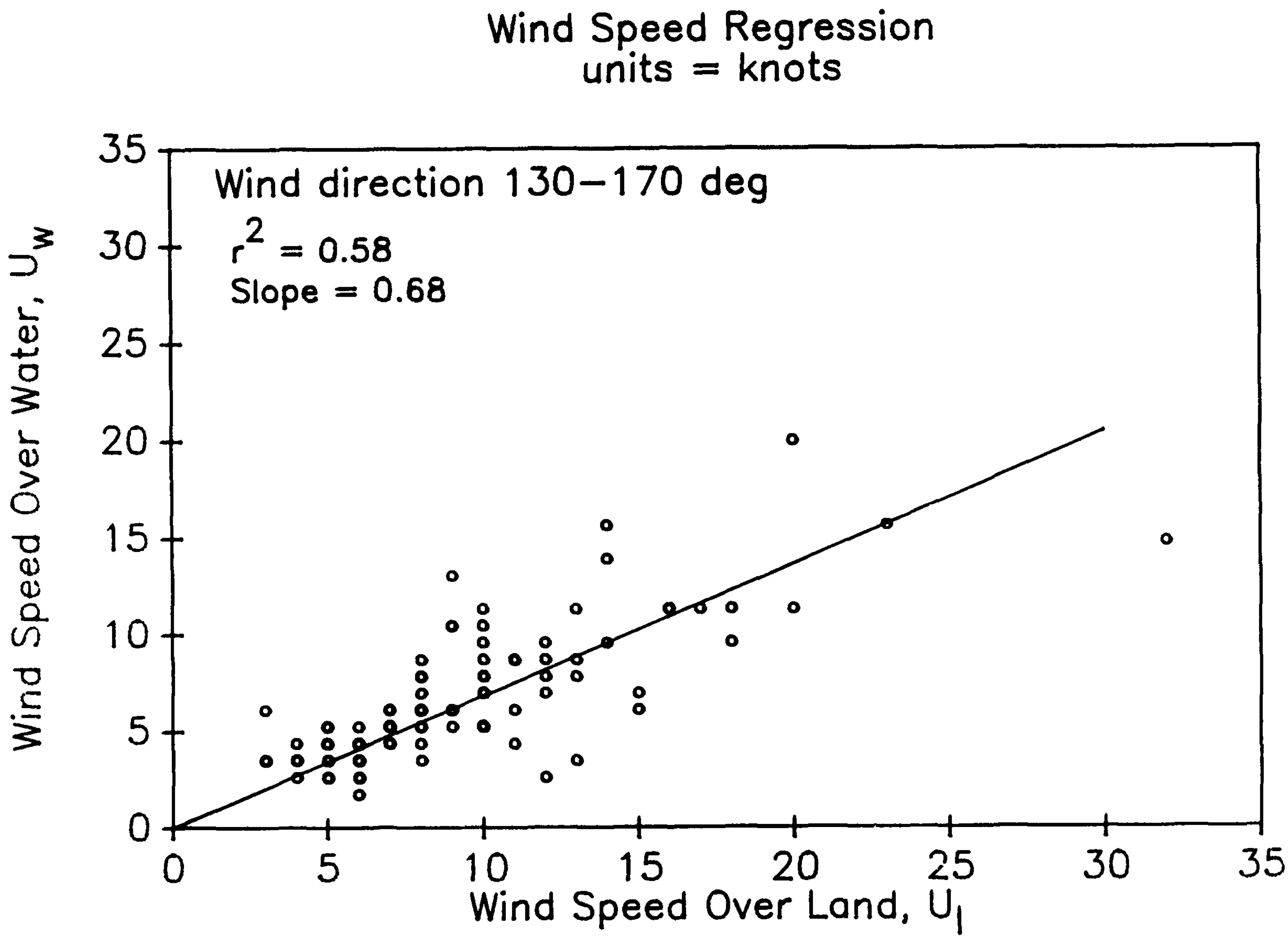


Figure 41. Wind speed regressions, 130-170 deg

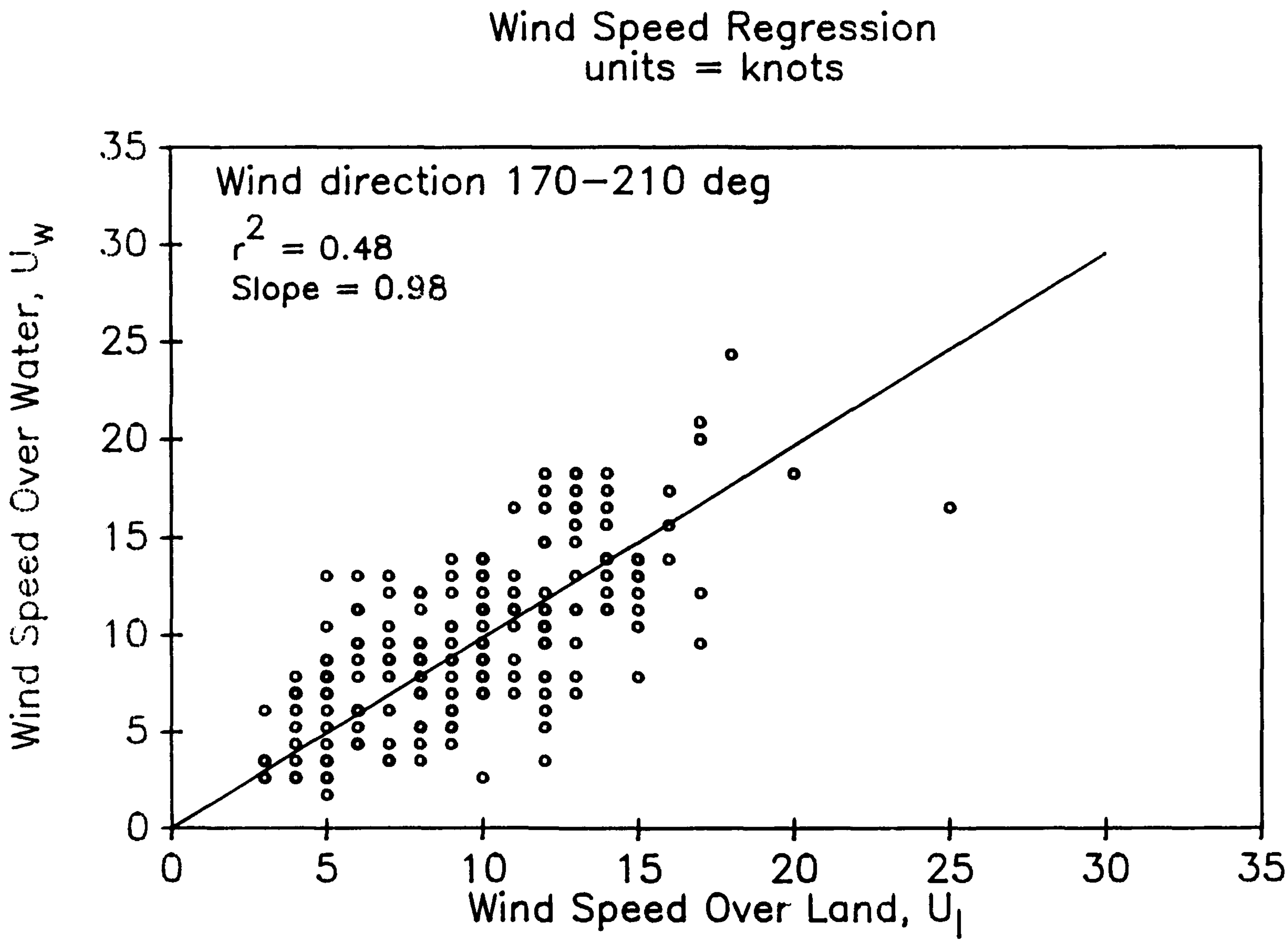


Figure 42. Wind speed regressions, 170-210 deg

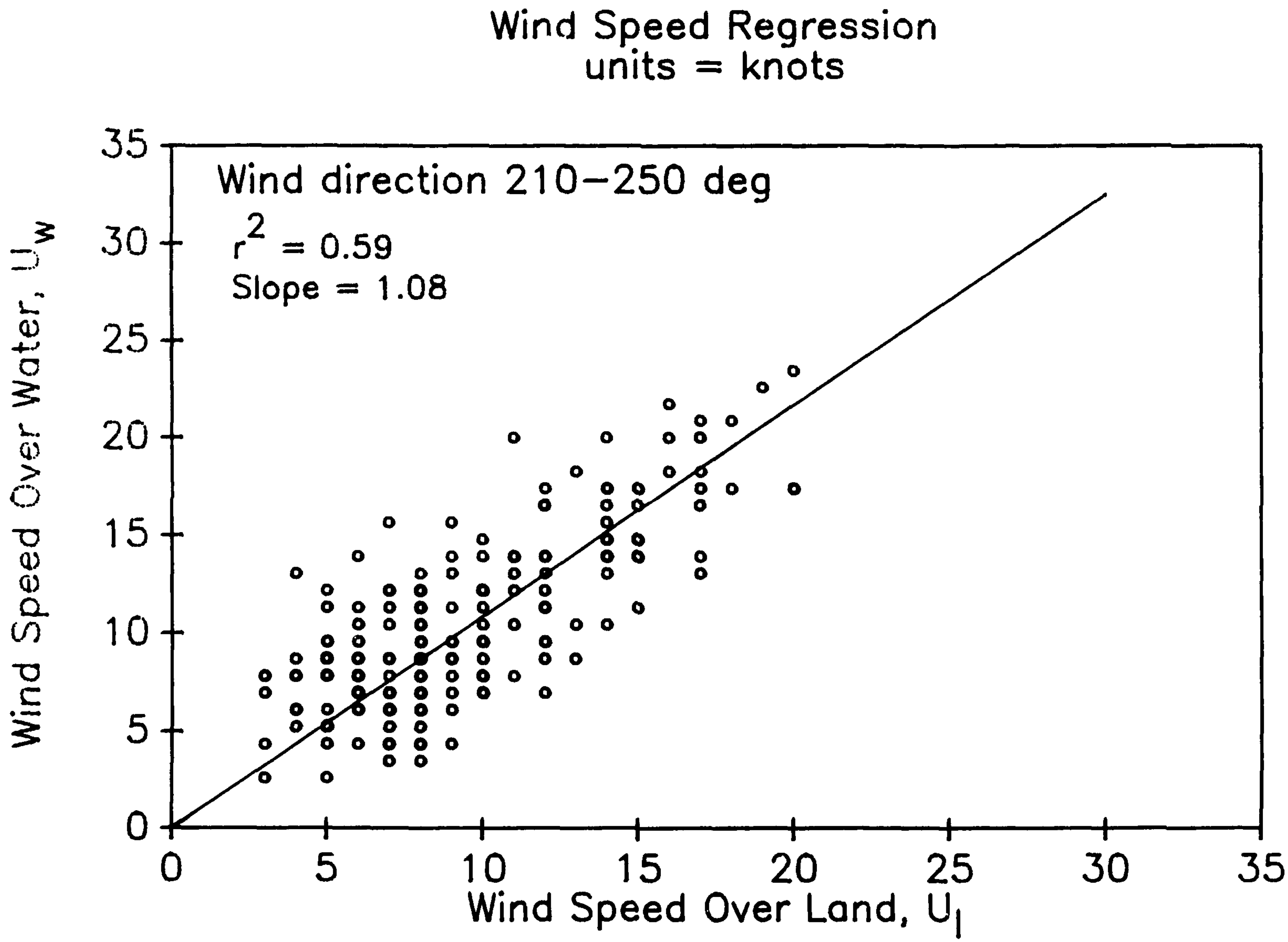


Figure 43. Wind speed regressions, 210-250 deg

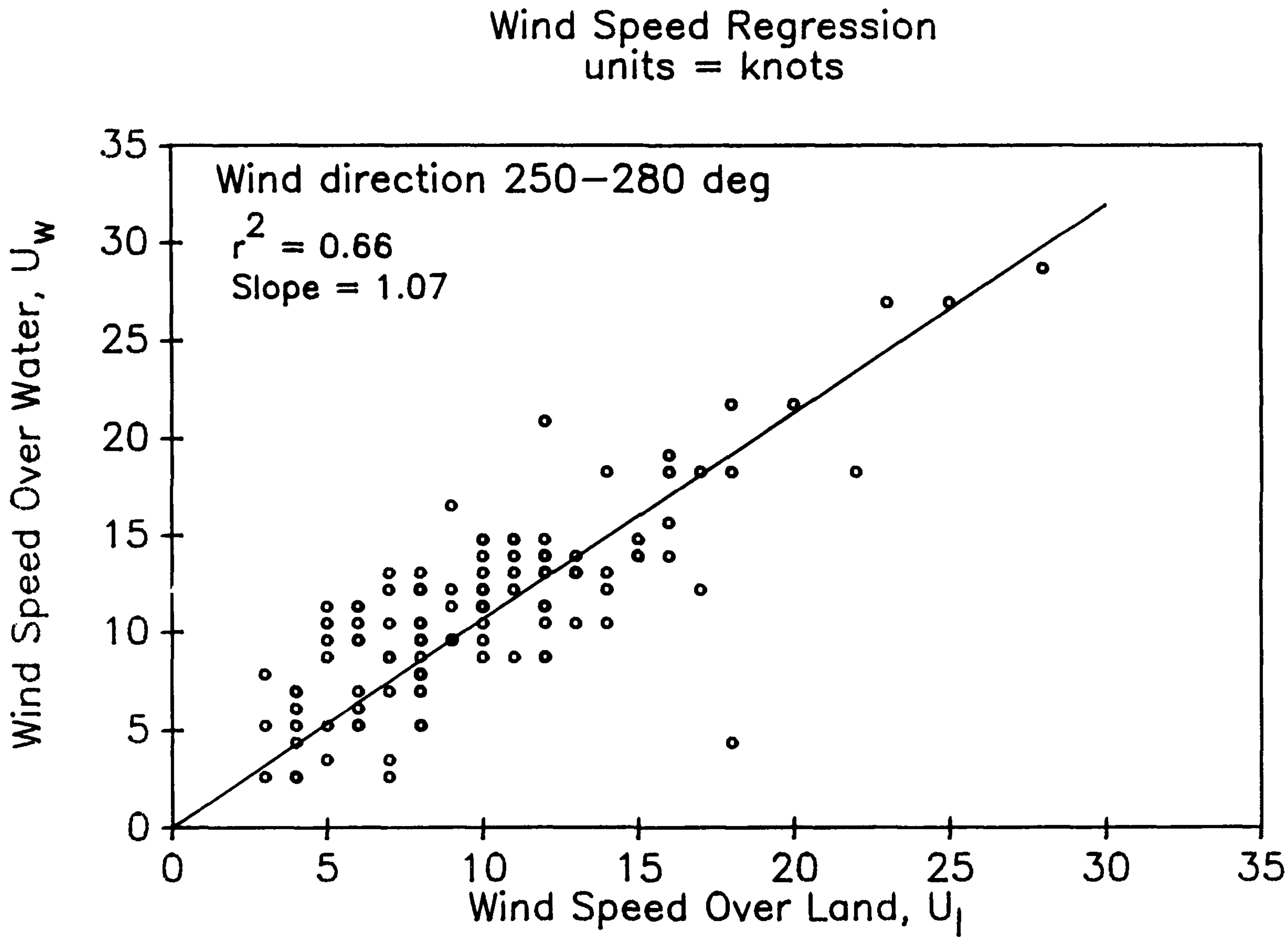


Figure 44. Wind speed regressions, 250-280 deg

HINDCAST HMO FROM ACTUAL WIND SPEEDS

MOMENTS

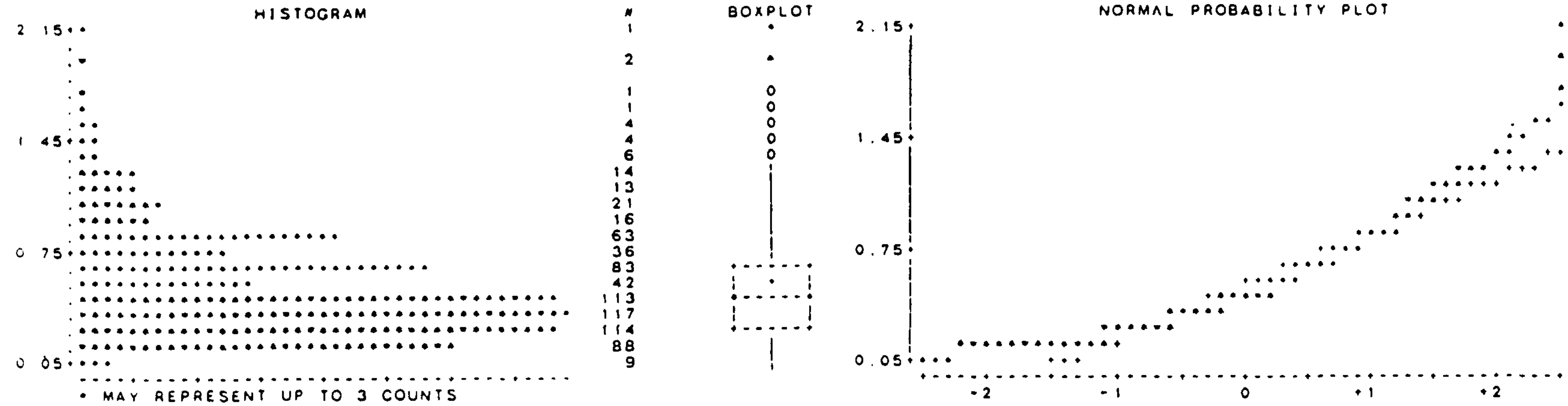
N	748	SUM WGTS	748
MEAN	0.521529	SUM	390.104
STD DEV	0.328573	VARIANCE	0.10796
SKEWNESS	1.185	KURTOSIS	1.86282
USS	284.097	CSS	80.6461
CV	63.0018	STD MEAN	0.0120138
T:MEAN=0	43.4108	PROB>|T|	0.0001
SGN RANK	140063	PROB>|S|	0.0001
NUM ¬= 0	748		
D:NORMAL	0.125151	PROB>D	<.01

QUANTILES(DEF=4)

100% MAX	2.13748	99%	1.51913
75% Q3	0.679642	95%	1.15451
50% MED	0.432363	90%	0.945327
25% Q1	0.262576	10%	0.159464
0% MIN	0.0679824	5%	0.111941
		1%	0.0679824
RANGE	2.0695		
Q3-Q1	0.417065		
MODE	0.262576		

EXTREMES

LOWEST	HIGHEST
0.0679824	1.66996
0.0679824	1.74636
0.0679824	1.97927
0.0679824	1.97927
0.0679824	2.13748



FREQUENCY TABLE

VALUE	COUNT	PERCENTS CELL	PERCENTS CUM
0679824	9	1.2	1.2
0.111941	39	5.2	6.4
0.159464	49	6.6	13.0
0.209828	42	5.6	18.6
0.262576	72	9.6	28.2
0.317395	56	7.5	35.7
0.374051	61	8.2	43.9
0.432363	57	7.6	51.5
0.492187	56	7.5	59.0
0.553405	42	5.6	64.6
0.615918	34	4.5	69.1
0.679642	49	6.6	75.7
0.744504	36	4.8	80.5
0.810442	29	3.9	84.4
0.877399	34	4.5	88.9
0.945327	16	2.1	91.0
1.01418	9	1.2	92.2
1.08392	12	1.6	93.9
1.15451	13	1.7	95.6
1.22591	13	1.7	97.3
1.2981	1	0.1	97.5
1.37105	6	0.8	98.3
1.44474	4	0.5	98.8
1.51913	3	0.4	99.2
1.59422	1	0.1	99.3
1.66996	1	0.1	99.5
1.74636	1	0.1	99.6
1.97927	2	0.3	99.9
2.13748	1	0.1	100.0

Figure 45. Summary statistics for Hmo hindcast from measured winds

HINDCAST PERIOD FROM ACTUAL WIND SPEEDS

MOMENTS

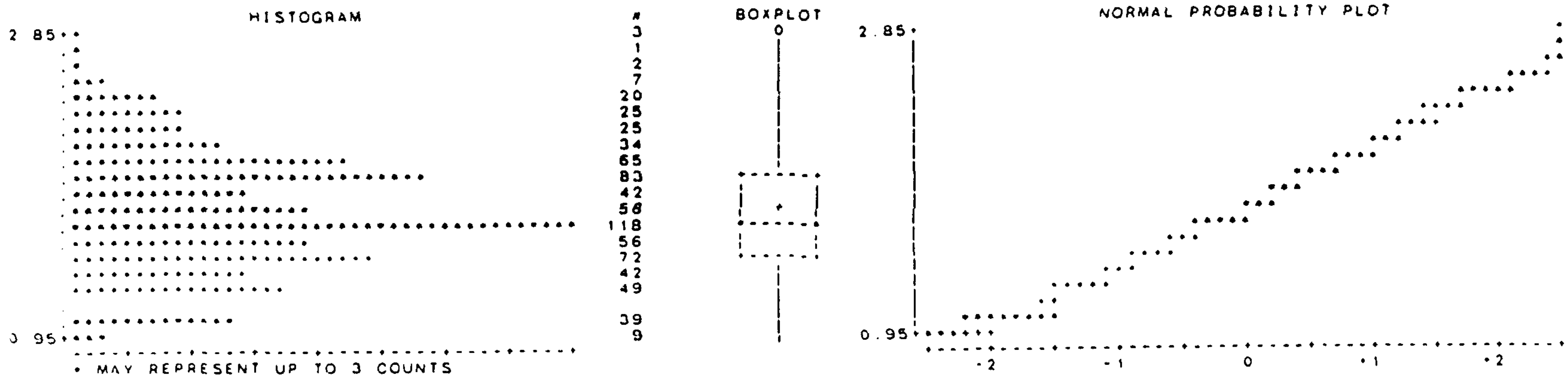
N	748	SUM WGTS	748
MEAN	1.72699	SUM	1291.79
STD DEV	0.374441	VARIANCE	0.140206
SKEWNESS	0.135663	KURTOSIS	-0.417879
USS	2335.65	CSS	104.734
CV	21.6817	STD MEAN	0.0136909
T:MEAN=0	126.141	PROB>|T|	0.0001
SGN RANK	140063	PROB>|S|	0.0001
NUM ¬= 0	748		
D:NORMAL	0.0634307	PROB>D	<.01

QUANTILES(DEF=4)

100% MAX	2.889	99%	2.57818
75% Q3	1.97185	95%	2.35277
50% MED	1.69589	90%	2.20111
25% Q1	1.43615	10%	1.21619
0% MIN	0.915334	5%	1.08088
		1%	0.915334
RANGE	1.97367		
Q3-Q1	0.535703		
MODE	1.43615		

EXTREMES

LOWEST	HIGHEST
0.915334	2.66083
0.915334	2.7008
0.915334	2.81589
0.915334	2.81589
0.915334	2.889



FREQUENCY TABLE

VALUE	COUNT	PERCENTS CELL	PERCENTS CUM
0.915334	9	1.2	1.2
1.08088	39	5.2	6.4
1.21619	49	6.6	13.0
1.33271	42	5.6	18.6
1.43615	72	9.6	28.2
1.52984	56	7.5	35.7
1.61584	61	8.2	43.9
1.69589	57	7.6	51.5
1.77075	56	7.5	59.0
1.84132	42	5.6	64.6
1.90819	34	4.5	69.1
1.97185	49	6.6	75.7
2.03268	36	4.8	80.5
2.091	29	3.9	84.4
2.14707	34	4.5	88.9
2.20111	16	2.1	91.0
2.2533	9	1.2	92.2
2.30381	12	1.6	93.9
2.35277	13	1.7	95.6
2.40031	13	1.7	97.3
2.44653	1	0.1	97.5
2.49153	6	0.8	98.3
2.53538	4	0.5	98.8
2.57818	3	0.4	99.2
2.61997	1	0.1	99.3
2.66082	1	0.1	99.5
2.7008	1	0.1	99.6
2.81589	2	0.3	99.9
2.889	1	0.1	100.0

Figure 46. Summary statistics for wave period hindcast from measured winds

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HINDCAST HMO FROM PREDICTED BEACH WIND

MOMENTS

N	748	SUM WGTS	748
MEAN	0.487268	SUM	364.477
STD DEV	0.301139	VARIANCE	0.0906847
SKEWNESS	1.59009	KURTOSIS	4.48478
USS	245.339	CSS	67.7415
CV	61.8015	STD MEAN	0.0110107
T:MEAN=0	44.2539	PROB>|T|	0.0001
SGN RANK	140063	PROB>|S|	0.0001
NUM ¬= 0	748		
D:NORMAL	0.11495	PROB>D	<.01

QUANTILES(DEF=4)

100% MAX	2.38066	99%	1.46265
75% Q3	0.639378	95%	1.01888
50% MED	0.418113	90%	0.892579
25% Q1	0.262223	10%	0.178131
0% MIN	0.0655786	5%	0.140666
		1%	0.087517
RANGE	2.31508		
Q3-Q1	0.377154		
MODE	0.448331		

EXTREMES

LOWEST	HIGHEST
0.0655786	1.64333
0.0655786	1.8238
0.0695817	1.8948
0.0695817	2.19814
0.0695817	2.38066

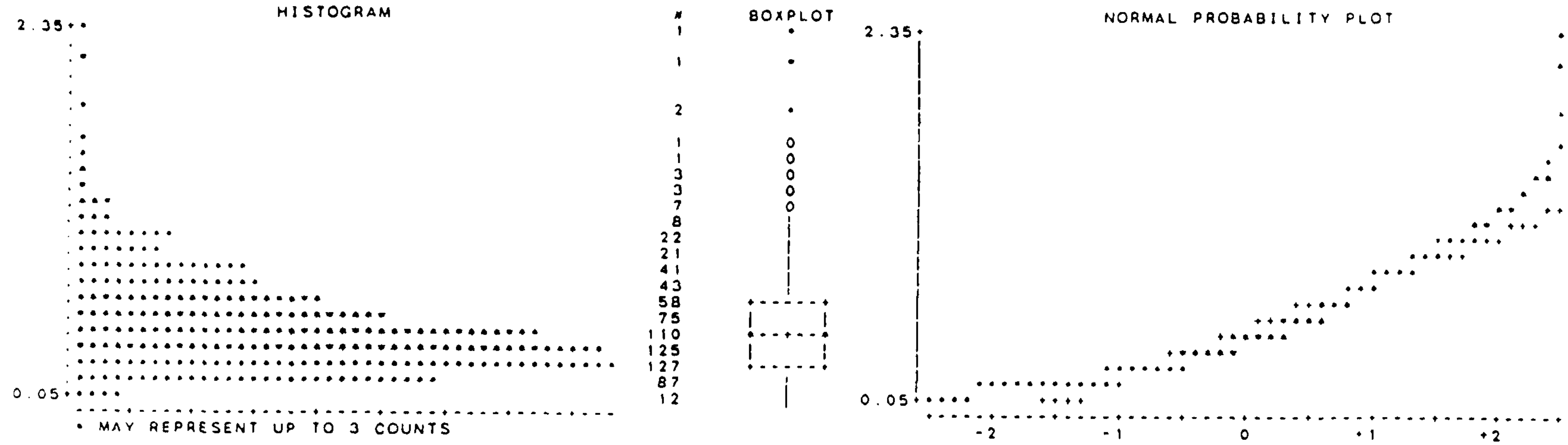

Figure 47. Summary statistics for Hmo hindcast from predicted (Equation 2) winds

HINDOCAST PERIOD FROM PREDICTED BEACH WIN

MOMENTS

N	748	SUM WGTS	748
MEAN	1.69679	SUM	1269.2
STD DEV	0.343129	VARIANCE	0.117738
SKEWNESS	0.324486	KURTOSIS	0.00329599
USS	2241.51	CSS	87.95
CV	20.2223	STD MEAN	0.012548
T:MEAN=0	135.245	PROB>|T|	0.0001
SGN RANK	140063	PROB>|S|	0.0001
NUM ¬= 0	748		
D:NORMAL	0.0405095	PROB>D	<.01

QUANTILES(DEF=4)

100% MAX	2.99465	99%	2.54582
75% Q3	1.93212	95%	2.25677
50% MED	1.67705	90%	2.15938
25% Q1	1.4355	10%	1.26191
0% MIN	0.904416	5%	1.16639
		1%	0.995254
RANGE	2.09024		
Q3-Q1	0.496812		
MODE	1.71651		

EXTREMES

LOWEST	HIGHEST
0.904416	2.6466
0.904416	2.74014
0.922456	2.77525
0.922456	2.91608
0.922456	2.99465

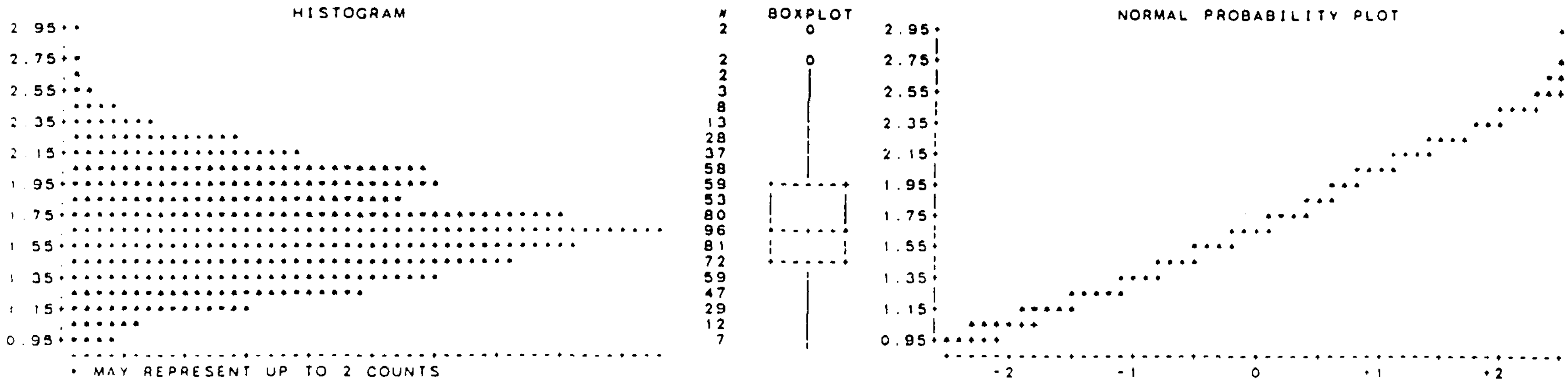

Figure 48. Summary statistics for wave period hindcast from predicted (Equation 2) winds

MOMENTS

N	748	SUM WGTS.	748
MEAN	0.034261	SUM	25.6272
STD DEV	0.208517	VARIANCE	0.0434792
SKEWNESS	0.0313584	KURTOSIS	1.31247
USS	33.357	CSS	32.4789
CV	608.613	STD MEAN	0.00762412
T:MEAN=0	4.49376	PROB>|T|	0.0001
SGN RANK	23882	PROB>|S|	0.0001
NUM ¬= 0	748		
D:NORMAL	0.0571214	PROB>D	<.01

QUANTILES(DEF=4)

100% MAX	0.733545	99%	0.578986
75% Q3	0.163569	95%	0.389745
50% MED	0.0146655	90%	0.301224
25% Q1	-0.0858187	10%	-0.199883
0% MIN	-0.949478	5%	-0.299996
		1%	-0.457975
RANGE	1.68302		
Q3-Q1	0.249388		
MODE	-0.114386		

EXTREMES

LOWEST	HIGHEST
-0.949478	0.646617
-0.883261	0.657434
-0.597889	0.706511
-0.519264	0.73018
-0.483567	0.733545

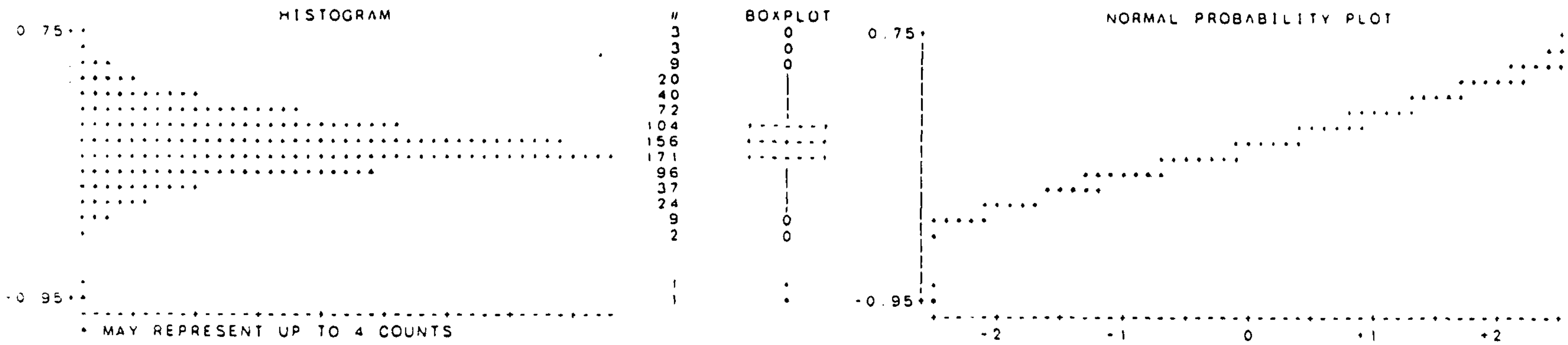

Figure 49. Summary statistics for measured-predicted Hmo

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VARIABLE=TDIFF OBSERVED - PREDICTED PERIOD

MOMENTS

N	748	SUM WGTS	748
MEAN	0.0302034	SUM	22.5922
STD DEV	0.250489	VARIANCE	0.0627448
SKEWNESS	-0.1148	KURTOSIS	0.369374
USS	47.5527	CSS	46.8704
CV	829.34	STD MEAN	0.00915879
T:MEAN=0	3.29775	PROB>|T|	0.00102074
SGN RANK	19765	PROB>|S|	.000827571
NUM ^= 0	748		
D:NORMAL	0.0279234	PROB>D	>.15

QUANTILES(DEF=4)

100% MAX	0.891453	99%	0.57325
75% Q3	0.192852	95%	0.437608
50% MED	0.0226553	90%	0.355148
25% Q1	-0.120806	10%	-0.288897
0% MIN	-0.977579	5%	-0.381672
		1%	-0.630007
RANGE	1.86903		
Q3-Q1	0.313658		
MODE	-0.165281		

EXTREMES

LOWEST	HIGHEST
-0.977579	0.646762
-0.801202	0.655499
-0.754775	0.692916
-0.691143	0.724591
-0.677071	0.891453

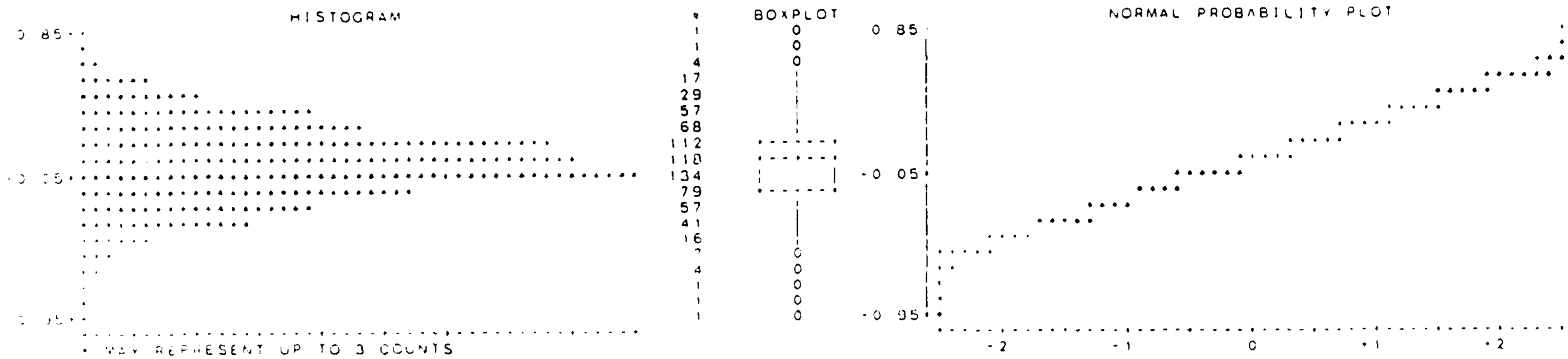

Figure 50. Summary statistics for measured-predicted wave period

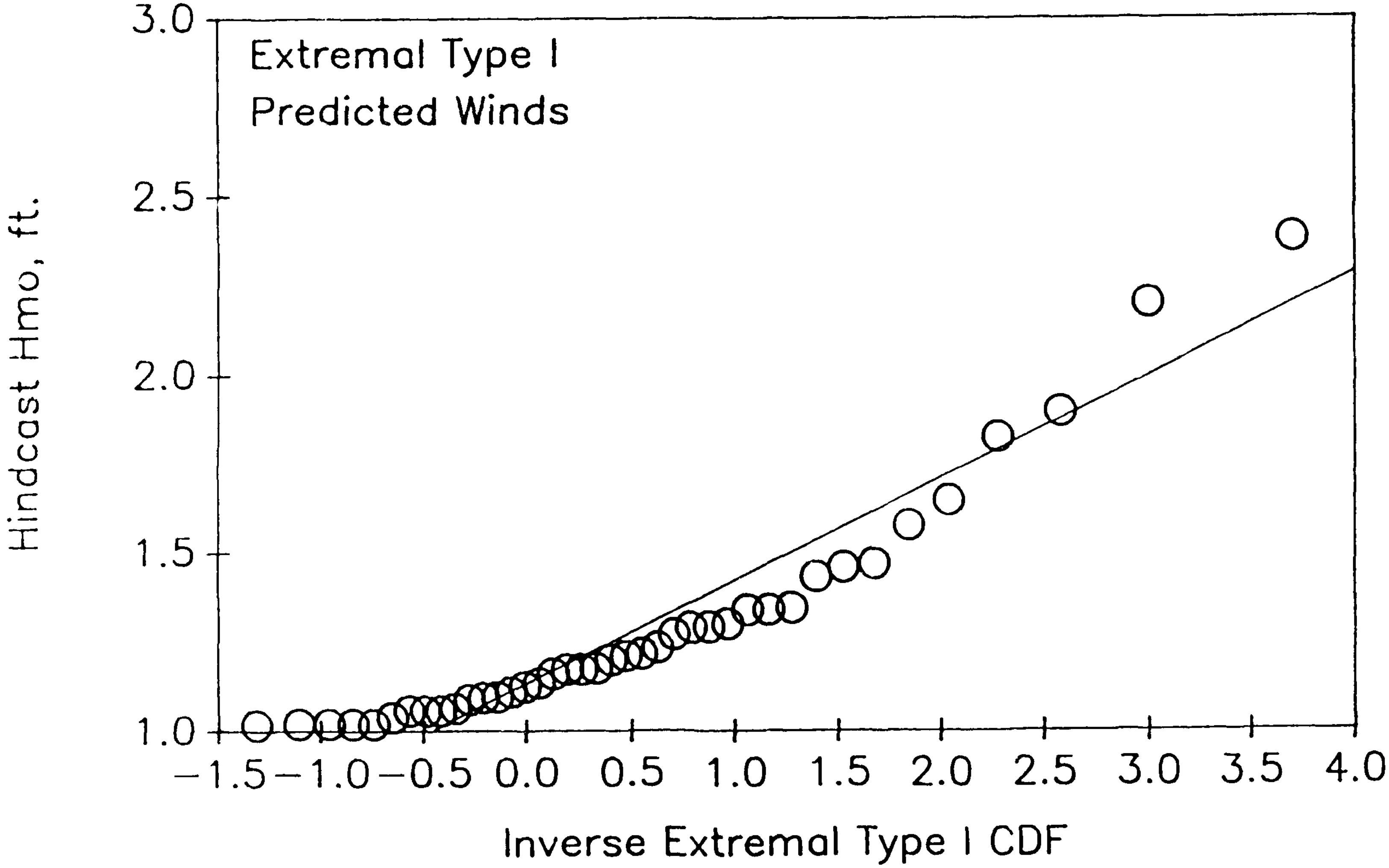

Figure 51. Extremal Type I predicted winds

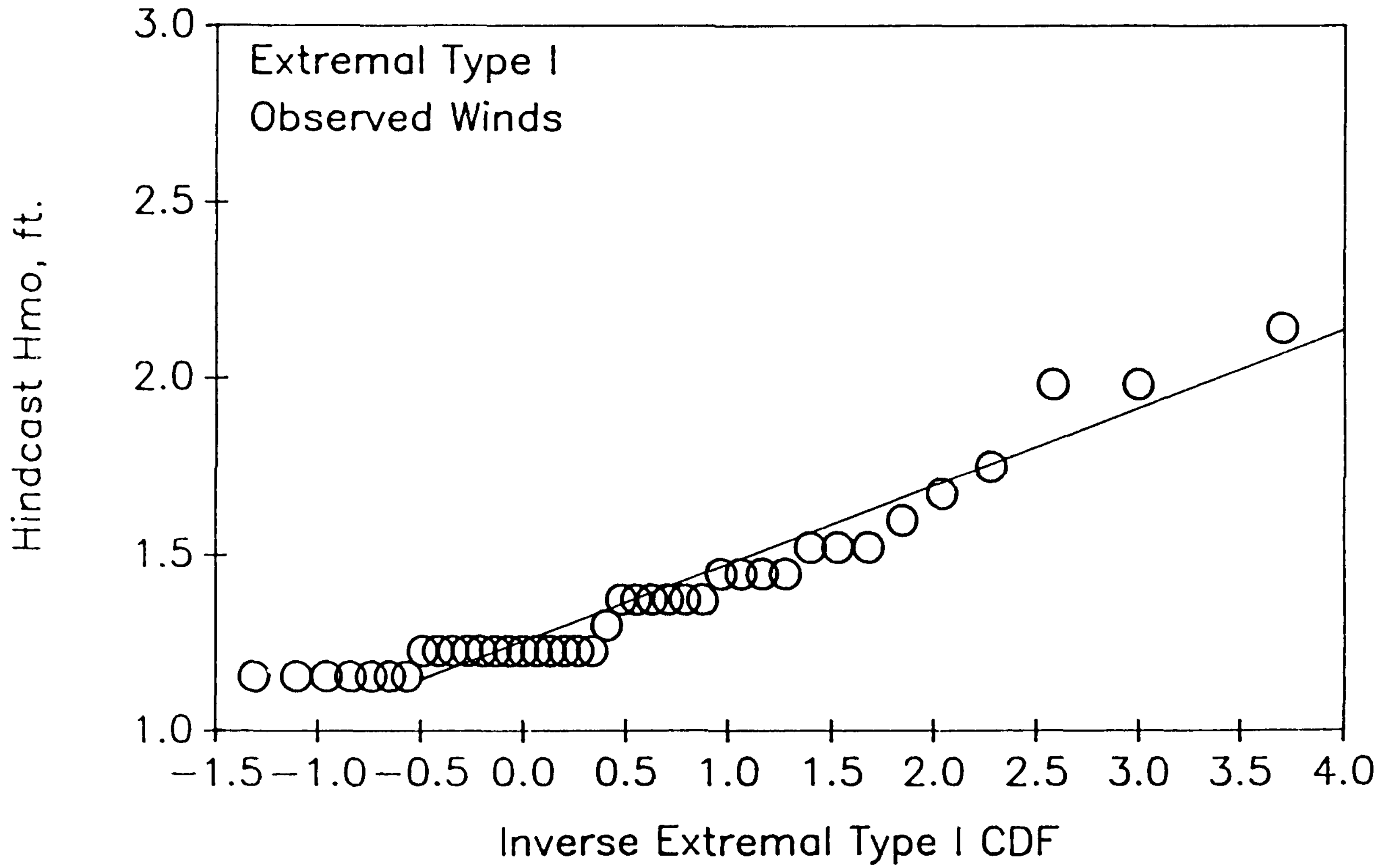

Figure 52. Extremal Type I observed winds

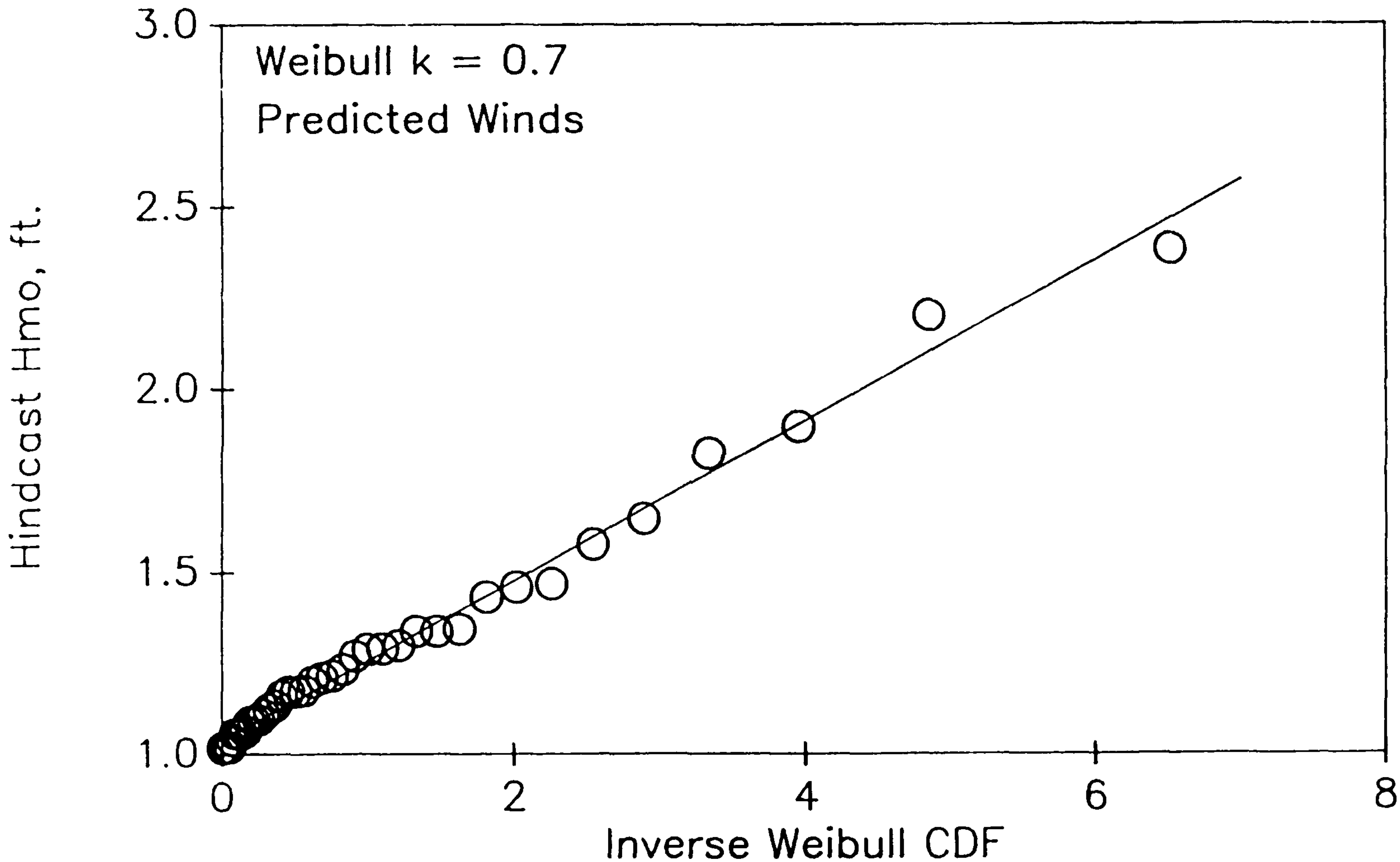

Figure 53. Weibull predicted winds

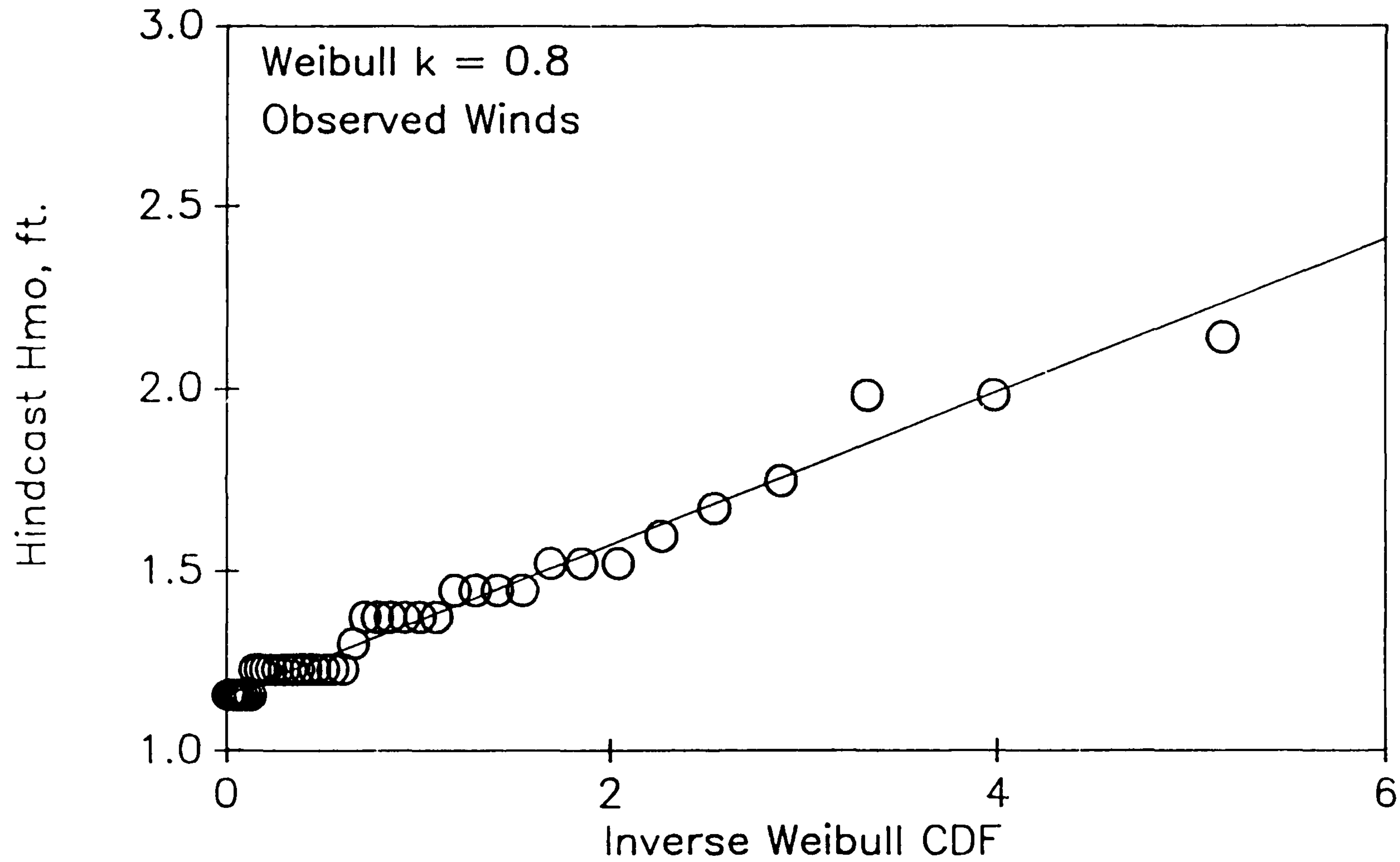

Figure 54. Weibull observed winds

Waterways Experiment Station Cataloging-in-Publication Data

LeBlanc, Catherine J.
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