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BULLETIN
and
SUMMARY OF RESEARCH PROGRESS
Fiscal Years 1967-69

U. S. ARMY, CORPS OF ENGINEERS

COASTAL ENGINEERING RESEARCH CENTER

FOREWORD

The U. S. Army Coastal Engineering Research Center, established in 1963 in accordance with Public Law 88-172, carries on most of the functions of the former Beach Erosion Board of the Corps of Engineers. The Center has basic research and development in coastal engineering as its primary function.

This Bulletin is a progress report on the Center's activities, and also provides a means for disseminating brief technical articles concerning shore processes and related matters which are of general interest. Contributions of a similar nature from individuals in field offices of the Corps of Engineers or other agencies are encouraged.

At the time of publication, Lieutenant Colonel Edward M. Willis was Director of the Center; Joseph M. Caldwell was Technical Director.

NOTE: Comments on this publication are invited. Discussion will be published in the next issue.

This Bulletin is published under authority of Public Law 166, 79th Congress, approved July 31, 1945, as supplemented by Public Law 172, 88th Congress, approved November 7, 1963.

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A SUMMARY OF

HURRICANE PROTECTION IN NEW ENGLAND HARBORS

by

Colonel Frank P. Bane
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History shows the southern coast of New England has been the target of 63 hurricanes since 1635. Of that number, 10 were major storms. Disastrous loss of life and property was suffered in 1954; three tropical storms took 68 lives and inflicted $300 million in damages. The September 1938 hurricane killed 500 persons and caused one-half of a billion dollars in property damage.

The southern New England coast was battered by Hurricane Carol on 31 August 1954. In specific harbors now protected by hurricane barriers built by the Corps of Engineers, losses totaled $70,890,000. Harbor losses at Providence, R. I. were $41 million; the toll at New Bedford, Mass., was $26.2 million; at Stamford, Conn., harbor losses reached $3.4 million.

Hurricane barriers now in place had first construction costs as follows:

Fox Point, R. I.	$16 million, including 30 percent non-federal contribution
New Bedford, Mass.	$18.1 million, with local contributions of $7 million
Stamford, Conn.	$11.7 million, including non-federal contributions of $4.4 million

The Stamford barrier has been operated 17 times to protect the harbor from tidal flooding, with an estimated loss prevention of $1.5 million. New Bedford barrier, in 16 operations, has prevented $500,000 in losses.

The Fox Point structure has yet to be tested in a major storm. However, in a recurrence of the August 1954 hurricane it would prevent $39.8 million in damages and $52.4 million in a repetition of the September 1938 storm.

In a recurrence of the August 1954 flood, the New Bedford barrier will prevent $33.9 million in losses; in a recurrence of the September 1938 hurricane the savings would be $40 million.

Stamford barrier will prevent $3.3 million in losses in a recurrence of the August 1954 flood and $6.7 million in a recurrence of the September 1938 storm.

NEW BEDFORD - FAIRHAVEN - ACUSHNET HURRICANE BARRIER
NEW BEDFORD, MASSACHUSETTS

by

Saul Cooper
Chief, Reservoir Control Center
and
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Project Engineer, New Bedford

The New Bedford-Fairhaven Barrier is comprised of three principal features: (1) the harbor and dike across the entrance to New Bedford and Fairhaven Harbor; (2) the Clark Cove Dike which protects a low area at the head of the cove and prevents high tidal surges from flanking the harbor barrier from the west; and (3) the Fairhaven Dike which protects a low area in Fairhaven and prevents tidal flanking from the east (Figure 1).

The harbor barrier extends from Fort Phoenix at the southwestern tip of Fairhaven, westward across the harbor to New Bedford, a distance of 4,600 feet. A dike extension at the western end of the barrier extends 4,500 feet south along the eastern shore of Clarks Point and returns to high ground. The top of the harbor barrier, where the volume of wave overtopping during the design storm does not seriously affect the harbor stages, is elevation 20 feet above mean sea level. Top of the dike along the shore is at elevation 22, which is 2 feet higher than the barrier in order to minimize wave overtopping into the land area.

The largest sector gates in the world, each weighing 400 tons (Figure 2), are the twin keys to total integrity of the project. Set in massive concrete abutments on the east and west edges of the main ship canal in New Bedford Harbor, the leaves will close off the 150-foot wide navigation opening. Each leaf is 93 feet in arc length, 59 feet high, and rotates on a vertical axis, moving on five huge trunnion pins. The leaves ride on six 27-inch diameter by 7-inch wide wheels, and move to closed position in 12 minutes (Figure 2).

A unique feature was built into the gate framing. The recesses must be dewatered for routine maintenance of the gates, thus the channel sides of the gates were fitted with stopgate guides. Thirty-five stopgates will be inserted for the dewatering operation; the gate framing will transfer the hydrostatic pressure to the abutments.

Construction of the abutment structures required 44,800 cubic yards of concrete; 72,000 barrels of cement; and 2 million pounds of reinforcing steel.

Figure 1. New Bedford-Fairhaven-Acushnet Hurricane Protection Barrier looking upstream. Small boats are using the 150-foot passage provided by the navigation opening.

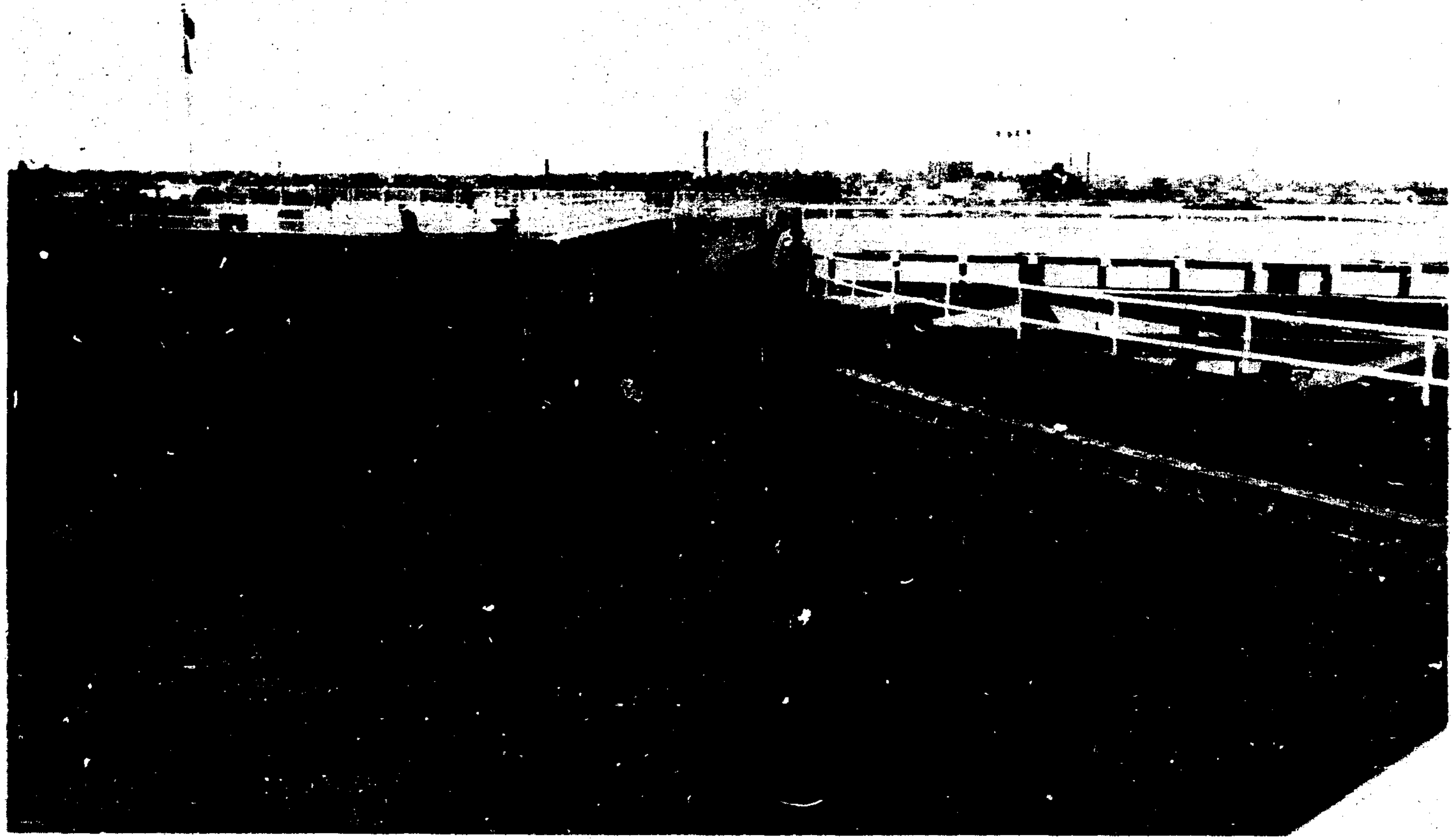

Figure 2. Leaves of the world's largest sector gates move to a closed position in 12 minutes.

To permit construction in the main ship lane, a by-pass channel was dredged to ensure uninterrupted marine traffic in New Bedford Harbor. More than 800,000 cubic yards of material were removed.

Two twin-barrel, gated conduits are provided in the barrier section to ensure circulation of water in the southwest corner of the harbor during normal tide cycles. Each conduit is 6 feet wide and 9 feet high. As this section of the project contains the navigation gates and appurtenances, both the harbor barrier and the gates are maintained and operated by the Corps of Engineers. Local interests contributed a lump sum payment to cover the Federal cost for this maintenance and operation, as required by law, as part of their obligations.

The shore dike has three, gated conduits - 48, 36, and 24 inches in diameter, and a street gate on Rodney French Boulevard East. The gate consists of two hinged sections, each 31 feet long and 14.5 feet high. The street and conduit gates are operated by community personnel.

The Clark Cove area is protected by a dike 5,800 feet long around the north and east sides of the cove, with short flanking dikes returning to high ground. The top of the dike across the head of the cove is elevation 22.0, while the flanking dike along the easterly side, more exposed to wind and waves, is elevation 23. Operational structures in this section of the project consist of two street gates, six gates on utility conduits, and a pumping station. The closure at Rodney French Boulevard West has two hinged swing gates, each 31 feet long and 12.5 feet high. The six utility gates range in size from a 24-inch diameter seawater intake conduit to a 96- by 84-inch main interceptor sewer. The opening on Cove Road at the west end of the dike is closed with two gate leaves, each 31 feet long and 13.5 feet high.

The pumping station is for the evacuation of interior storm runoff, overtopping and sewage. It contains four vertical propeller-type pumps driven by 350-horsepower motors. Each pump is designed to discharge 122 cubic feet per second against a head of 20 feet. A ponding area adjacent to the station provides about 10 acre-feet of storage for inflows that may exceed the pumping capacity. Before initiating pumping, the following operations are necessary: (1) closing a gravity outfall gate; (2) opening the intake gate to the sump; and (3) opening two gates to permit inflow from the ponding area. The Clark Cove Dike, street gates, pumping station and utility gates are maintained and operated by the City.

The southern portion of Fairhaven is protected by a dike 3,100 feet long, with the top of the dike at elevation 20. This portion of the project has a single 4 by 4-foot sluice gate in the drainage conduit that requires operation. There is sufficient ponding area for temporary storage of storm runoff during the few hours that gate closure is required. This dike and gate are maintained and operated by the Town of Fairhaven.

The Reservoir Control Center, a branch of the Engineering Division, New England Division, issues instructions to field personnel for operation of the navigation gates. The center maintains close liaison with the Weather Bureau and receives the bureau's advisories. Predicted times of gate closures are furnished to the Coast Guard.

To assist in acquiring operations data for the navigation gates, NED has developed a hydrologic radio network off the southern New England coast. Data concerning tidal surge, wind speed and direction, and barometric pressure are received. Readout from this network is received at the Corps' Cape Cod Canal office which operates 24 hours a day, and at division headquarters, Waltham, Massachusetts. Reception of coastal data does not supersede the need for Weather Bureau advisories on the position of storms as they move northward along the Atlantic coast, but supplement Bureau data and provide a reliable one- or two-hour advance notice of an approaching tidal surge. The coastal radio network is of great value in spotting abnormal tidal conditions from unpredicted coastal storms developing just south of New England.

Closing a navigation channel at the mouth of a harbor in anticipation of a tidal surge poses a problem. New Bedford-Fairhaven Harbor, covering 1,100 acres of water area, is used temporarily for storing the flow from the Acushnet River, overtopping and the local urban runoff. The total inflow could be considerable should the rainfall be heavy during a storm. To provide adequate storage for this inflow below damage stages (elevation plus 5 mean sea level), the navigation and sluice gates in the barrier are closed when the incoming astronomical tide, antecedent to the expected arrival of the hurricane tidal surge, rises to elevation plus 2.0 mean sea level. It takes about 12 minutes to either close or open the gates. As a hurricane moves north along the Atlantic Coast, the Coast Guard periodically radios to shipping interests the Corps of Engineers bulletins on estimated day and time of gate closure. Mariners must heed warnings if they wish to find shelter in the harbor; once the large sector gates have been closed, they will not be opened for mariners until the storm has passed and the tide has receded.

Operational experience to date has been confined to coastal storms. Since January 1966, the gates have been closed on 16 occasions and the barrier staffed for possible operation on more than 50 occasions. The following table lists pertinent data for closures. Closure criteria has undergone several revisions based on experience gained during the alerts and operations. Prior to completion of the New Bedford barrier, studies indicated that the navigation gates should be closed at elevation 3 feet mean sea level if it appeared that the tide might exceed 4 feet mean sea level.

NEW BEDFORD-FAIRHAVEN

HURRICANE GATE OPERATION

Date	Tide Elevation		Surge	Wind	
	Ocean	Harbor		Velocity (knots)	Direction
23 Jan 66	4.0	3.0	2	20	NE
23 Jan 66	3.5	3.0	2	20	NE
30 Jan 66	5.2	5.2	4.3	30	SW
15 Sep 66	4.2	3.5	0.9	20	N
29 Dec 66	6.0	5.2	3.5	50-60	SSE
11 May 67	4.0	3.5	1.9	40	SSE
3 Dec 67	4.7	4.1	1.6	20	SSE
3 Dec 67	4.2	3.6	2.1	20	SSE
23 Oct 68	4.5	4.0	1.2	18-26	SW
13 Nov 68	3.4	3.4	2.4	30-37	S
23 Dec 68	4.8	3.6	2.1	28	S
11 Dec 69	4.7	3.1	1.7	35-45	SSE
3 Feb 70	5.0	3.7	3.4	35-45	S
10 Feb 70	4.0	3.3	1.7	30	ESE
26 Mar 70	3.8	3.5	1.9	40-50	SSE
2 Apr 70	4.4	3.5	2.3	45-50	S

Gate inoperative 23 Jan 1966

After the east gate was stuck during an operation on January 23, 1966, the criteria for closure was re-evaluated. Conferences were held by the Corps with city and town officials, representatives of industrial concerns situated behind the barrier, representatives of the fishing industry, and maritime officials. Many meetings were held to consider the potential damage which would result from various closure elevations and the adverse effects of closures on fishing and boating interests. It was mutually agreed that for coastal storms, if the ocean elevation was expected to exceed 5.0 feet mean sea level, the gates would be closed at 4.0 feet.

This criterion, considered tentative and subject to change, was in effect in December 1966 when the ocean unexpectedly rose to 6.0 feet and the gates were closed when it reached 5.2 feet. The surge from this storm was not expected to cause abnormal tides requiring gate operation, and therefore, the gates were not closed at 4.0 feet. However, when the ocean level continued to rise when the predicted tide should have been cresting, the gates were closed. Damages of about $40,000 were sustained by property owners behind the barrier. Losses of some $160,000 were prevented.

Further conferences were held and local interests recommended a change in the closure criterion. Industrial concerns stated that they would have to move equipment and people thus incurring additional losses if the criterion remained unchanged.

Because of the strong southerly winds associated with the storm of 28 December 1966, the tide at the upper end of the harbor rose about 0.3 foot higher than at the barrier. New criteria for closure included wind speed and direction and the tide elevation at the barrier. This was modified recently to take into account potential damage on the Fairhaven side of the barrier from winds out of the west to southwest quadrant.

FOX POINT HURRICANE PROTECTION BARRIER
PROVIDENCE RIVER, PROVIDENCE, R. I.

by

Roy S. Martin
and
Lawrence S. Parente

Fox Point Hurricane Protection Barrier is located on the Providence River about one mile south of the heart of Providence, Rhode Island, the capital city, which was inundated to a depth of from 6 to 8 feet in the September 1938 hurricane tidal surge.

The project at Fox Point consists of six principal features: (1) a concrete barrier section; (2) a pumping station to discharge interior runoff; (3) three Tainter gates to close the river opening through the barrier; (4) a canal to provide cooling water to the Narragansett Electric Company; (5) street gates; and (6) utility gates.

The main barrier includes the concrete section, the pumping station, the three Tainter gates and the cooling-water canal intake. The pumping station (Figure 3) contains five 109-inch vertical propeller-type pumps, each driven by a 4,500-horsepower (3,600 kilowatt) motor. Each pump has a discharge capacity of 1,400 cubic feet per second against a 20-foot head for a total of 7,000 cubic feet per second. The pumps are manually controlled during an operation.

The three Tainter gates are each 40 feet wide and 40 feet high. Each sill is at elevation of 15 feet above mean sea level, and the top of the gate in closed position is +25 feet. The gates are normally in open position with the bottom of the gate at an elevation about +25 feet to permit passage of small boats and barges. The gates may be lowered at 1.5 feet per minute for a total elapsed time of about 25 minutes from fully open to closed position. They open at a rate of 6 inches per minute. It takes 30 minutes to lift the gates 15 feet to provide a full waterway opening. An additional 46 minutes is required to open the gates to an elevation of 23 feet.

The cooling-water canal was necessary to offset the detrimental effect of the tidal barrier on water temperatures. The canal is formed by a panel wall along the west side of the river. Cool circulating water is taken from downstream of the barrier through two gated openings in the pumping station at a rate of up to 1,000 cubic feet per second. The two control gates, each 10 feet wide and 15 feet high are normally open. They are partially or completely closed, as necessary, to control the rate of bay water inflow during a hurricane operation. The warm water from the power plant discharges in conduits through the canal into the river.

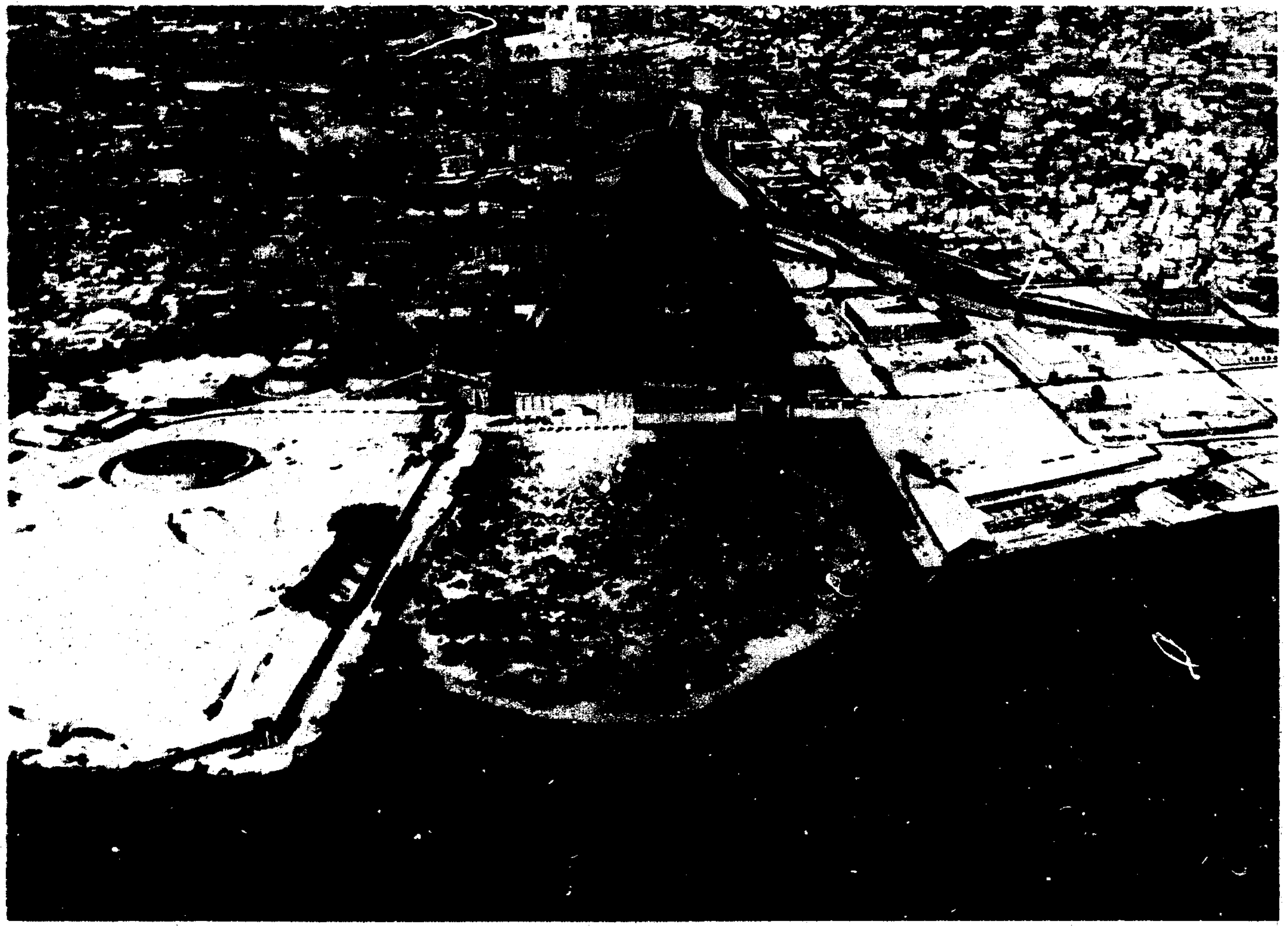

Figure 3. The Fox Point Barrier Pumping Station and Cooling Canal. Providence, Rhode Island, is in the background.

Three street gates require closure during a hurricane. The largest, located at Allens Avenue, consists of two leaves each 35 feet long and 12.5 feet high. A smaller gate in the west embankment is located within the yard of the Narragansett Electric Company. The third gate closes the South Main Street opening in the east embankment.

Seven motorized sewer gates operate during a storm. The two largest, each 72 x 60 inches, are located in a manhole in Allens Avenue and control tidal backwater in a 102-inch diameter trunk sewer.

POWER REQUIREMENTS FOR PUMPING STATION

Power requirements at the pumping station (Figure 4) (18,000 kilowatts for the 5 pumps) are so large that the Narragansett Electric Company requires advance notice as to the number of pumps to be used. This permits planning load distribution, and obtaining additional power from other sources in the New England power network if necessary.

The number of pumps required for an operation is related to the flow in the Providence River. An estimate of the flow is based on the discharge at an index gaging station on the Moshassuck River, located just upstream of tidewater and near the confluence with the Woonasquatucket River. An automatic telephone transmitter in the gaging station will permit frequent review of river stages and discharge data during the initial phases of a storm. This information is used for making a quick estimate or revision of the number of pumps required and the corresponding powerload.

COOLING WATER CANAL GATES

Cool water from the bay side of the barrier flows into the canal through two gated passageways in the pumping station (Figure 3). Normally, these gates are open and the passageways, with a total cross-section area of 300 square feet, produce a negligible head loss between the bay and the canal.

The canal panel wall, consisting of steel H-piles and timber stoplogs, is designed for a head differential of 2 feet between the canal and the river in either direction. To prevent excessive heads from developing, either by closure of the canal sluice gates or by too much inflow from the bay during a storm, eight flap gates, each 6 by 12 feet, are located in the wall. Four of them open when a head differential exceeds 1 foot in either direction. Although it is difficult to set the canal sluice gates to obtain a precise discharge, the automatic opening of the flap gates in the canal wall compensates for either excessive or insufficient flow through the sluice gates.

Figure 4. The five 4500 HP motors which run the pumps. 18,000 kilowatts are required for operation.

All cooling water entering the canal during an operational period is added to the interior runoff, and has to be pumped back into the bay. The canal sluice gates are closed to stop the canal inflow from the bay whenever three or more pumps are needed to evacuate the riverflow. During these periods, when the sluice gates are closed, water for cooling purposes enters the canal through the flap gates. The need for three pumps signifies a high rate of fresh-water inflow which should provide an adequate source of water for cooling purposes during an operational period.

The Reservoir Control Center has prepared a regulation manual for the Fox Point Project which is operated and supervised by the City of Providence. No significant operation experience has been acquired since completion of the project in 1966. However, the Center staff plans to monitor damage prevention operations to ensure that procedures in the manual are adequate for the efficient operation of the project.

STAMFORD HARBOR HURRICANE PROTECTION BARRIER

by

Lawrence S. Parente

The Stamford, Connecticut, Harbor Hurricane Barrier (Figure 5) is comprised of 11,700 feet of protective works extending from the east bank of the West Branch of Stamford Harbor, at the mouth of the Rippowam River, eastward across the mouth of the East Branch and across the base of Shippan Point, then turning northward along Westcott Cove to high ground behind Cummings Park recreational area.

The project is divided into three principal features: (1) the West Branch; (2) the East Branch; and (3) the Westcott Cove Barriers.

The West Branch barrier extends along the east bank of the West Branch of Stamford Harbor from the mouth of the Rippowam River to Dyke Park, a distance of 4,340 feet. This part of the barrier includes 700 feet of concrete-capped, steel-piling wall, 700 feet of gravity and pile supported concrete walls, and 2,100 feet of rockfaced earth dike as well as a reinforced concrete pumping station. The Dyke Lane pumping station contains two 30-inch and three 48-inch vertical propeller-type pumps. The 30-inch pumps, driven by 150 horsepower motors, have a total capacity of 100 cubic feet per second; the three 48-inch pumps, driven by 500 horsepower motors, have a total capacity of 416 cubic feet per second. During storm conditions, the pumping station is utilized to pump the runoff from 197 acres as well as the cooling water from the Hartford Electric Company plant.

The East Branch barrier extends easterly from the end of the West Branch barrier in Dyke Park across the East Branch of Stamford Harbor, connecting with the higher ground at Wallace Street. It includes 1,960 feet of rockfaced earth dike, a 90-foot navigation channel and the East Branch pumping station, located at the navigation gate structure. The top of the barrier is 17.0 feet above mean sea level.

When a storm is imminent, the navigation channel is closed by a steel flap-type navigation gate approximately 95 feet long, 36 feet high and 6 feet thick. The gate is one of the largest of its type in the world, and the first of its kind to be used as part of a hurricane barrier. The gate weighs approximately 220 tons, has a compartment-type interior and is free draining. The gate normally lies on the harbor bottom in a horizontal position (open) and can be raised to a vertical position (closed) in 20 minutes by means of two 16" diameter x 62 feet long lifting arms. The gate can be made watertight, disconnected from its hinges and the lifting arms, made buoyant by pumping water out of the interior, and then towed to a drydock for maintenance.

Figure 5. The Stamford Harbor Hurricane Barrier is 11,700 feet long. The Small Boat Harbor is shown in the foreground.

The East Branch pumping station, with a total capacity of 100 cubic feet per second, contains two 30-inch vertical propeller-type pumps, driven by 125 horsepower motors. This section of the project has a contributing interior drainage area of about 1,200 acres, and the harbor has a surface area behind the barrier between 60 and 80 acres. Therefore, storing 1 inch of runoff from the drainage area would cause a rise between 1.5 and 2.0 feet in the water level.

During navigation gate closure, most of the runoff will be stored in the East Branch with the limited pump capacity of 100 cubic feet per second serving only to supplement storage capacity. Therefore, special consideration is given in the development of gate closing schedules to allow for storage space in the East Branch for interior runoff. The combined pumping and storage capacity can handle a 10-year storm with the gate closed at elevation 2.0 feet mean sea level. However, for coastal storms, closure elevations will depend on the forecast of tides, together with antecedent rainfall.

The Westcott Cove barrier extends easterly from Homestead Lane to Auldwood Street, northeasterly to Iroquois Street, northerly to the head of Halloween Cove in Cummings Park and easterly to higher ground in Cummings Park. It includes 4,200 feet of rockfaced earth dike, access ramps and parking areas and the Wampanaw and Cummings pumping stations. The top elevation of the dike from the westerly end to the Wampanaw pumping station is 19.0 feet mean sea level, and for the remainder 18.0 feet mean sea level. Side slopes of the dike are 1 on 2 on the oceanside and 1 on 3 on the landside.

The Wampanaw pumping station is a reinforced concrete structure housing two 20-inch vertical propeller-type pumps, driven by 75 horsepower electric motors and having a capacity of 50 cubic feet per second to handle the interior runoff from 30 acres. The discharge is carried through the barrier by a 4-by 4-foot conduit.

The Cummings Park pumping station is a reinforced concrete structure housing three 30-inch vertical propeller-type pumps driven by 125 horsepower electric motors which have a capacity of 140 cubic feet per second to handle the interior runoff from 147 acres. The discharge is carried through the barrier by a 5-by 5-foot conduit.

One 18-inch sluice gate and two gate valves, 12 and 6 inches, are located on sanitary sewers that pass through the Westcott Cove barrier. These gates will be closed only in the event of hurricanes when the unprotected area served by the sewer would be evacuated. Sea-water flow through the sanitary sewer lines during normal coastal storms would be negligible.

All 17 closures to date have been associated with coastal storms. Because of the northeast-southwest orientation of Long Island Sound, the area is susceptible to tidal surges from "Northeasters". The most significant operation occurred on 12 November 1968 when the project was nearing completion. The tide reached 9.4 feet, mean sea level, and was exceeded only by the hurricanes of September 1938 and August 1954, and a coastal storm in November 1950 when the tide reached 9.5 feet mean sea level. It is estimated that this one operation prevented $750,000 in damages. Total benefits to date for this project amount to $1,500,000.

Details of the 17 closures are furnished in the following table.

STAMFORD HURRICANE GATE OPERATION

Date	Tide Elevation		Surge	Wind	
	Ocean	Harbor		Velocity (knots)	Direction
23 Oct 68	6.0	5.8	1.0	10-15	SW
25 Oct 68	6.1	5.6	1.0	9	WNW
12 Nov 68	9.35	5.6	5.6	15-25	NE
4 Dec 68	6.8	6.3	3.0	20-26	NE
20 Dec 68	6.2	5.0	1.0	6	N
23 Dec 68	6.5	4.7	2.3	8	SW
28 Jul 69	6.4	5.7	1.1	5	E
10 Nov 69	7.0	6.2	2.0	15	NNE
11 Nov 69	6.85	5.9	1.8	10-20	NNW
12 Nov 69	6.5	5.9	1.6	10	SW
14 Nov 69	6.7	5.8	2.4	12-16	NE
8 Dec 69	6.9	6.2	2.2	20-30	ENE
11 Dec 69	6.2	6.0	1.3	20-30	SW
26 Dec 69	7.8	5.8	4.5	15-25	NE
7 Jan 70	5.9	5.5	1.1	10-18	NE
8 Jan 70	6.35	6.1	1.5	6-8	NNE
11 Feb 70	6.3	5.7	2.1	15-30	ENE

A METHOD FOR DRIVING PIPE IN BEACH ROCK

by

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Beach profile studies and other types of coastal engineering operations often require the use of permanent structures on the beach or in the water for use as references for measuring sand movement.

Structures, such as pilings and groins, may be used for this purpose if they are available in the study area. However, if such a structure is not present, some other reference device is required small enough to be easily installed yet strong enough to withstand heavy surf and other adverse conditions. Even if pilings or groins are present, their size and shape may adversely affect sand movement in the vicinity causing erroneous readings, and another reference device may be desirable. Metal rods or pipes are ideal for this purpose. Low initial cost and ease of installation in most areas make them economical; and the relatively small diameter required for necessary strength has little effect on normal water flow and sand transport.

In sandy areas, pipes may easily be installed by a jet pump, or by driving to a sufficient depth with a sledgehammer. These methods cannot be used where rock is on or near the surface. Such rock was encountered by Florida Ocean Sciences Institute who had contracted to install several rows of beach profile markers at various points along the coast in the southeastern Florida area. These markers were to extend in a line from the winter berm of the beach out into the water to about a 5-foot depth. Underlying much of the beach in the study area are various types of rock ranging from a relatively soft quartzose worm rock to a harder, dense calcium carbonate which is cemented to underlying worm rock. Because this rock hampered pipe installations at almost every marker location, it was necessary to develop a method for penetrating the rock. This article describes the equipment and method used to overcome this problem.

Several means of rock penetration are currently in use and were examined for their applicability to this particular problem. Jetting was eliminated; the extremely high-pressure equipment needed for rock penetration would be cumbersome and expensive. Drilling was considered, but discarded for the same reasons. Blasting with shaped charges is relatively inexpensive, but proximity to waterfront dwellings made this method impractical. After considering all possibilities, driving was chosen as the most practical method.

Two problems were immediately recognized - the development of a means of driving the pipe, and the selection of pipe rigid enough to cut through the rock and to withstand the pounding of the driver. Early attempts to drive a one-inch pipe with a sledgehammer proved futile for the following reasons: To hit the top of a 15-foot pipe, the technician had to stand on a platform some 10 feet high. Although not too much trouble on land, this was virtually impossible in the water. Even on land, pounding with the sledge mushroomed the top of the pipe, and bent the pipe before much penetration was accomplished, thus making it impossible to attach another section of pipe to the first section for deeper driving.

After several futile attempts with 1-inch pipe and a sledge, the following became apparent: First, pipe with a minimum inside diameter of 2 inches would have to be used to provide sufficient cross-section area so that the pipe would cut rather than punch through the rock. Further, that some other means of driving the pipe be used and that the driving device be capable of being raised on a tripod or platform high enough to drive at least a 10-foot section of pipe. The device must also be able to be used both above and under water. A pneumatic jackhammer fits all these qualifications quite well, and is an item easily obtainable by rental.

The actual driving was accomplished with the following basic equipment: Pneumatic jackhammer, pipe driver adapter (Figure 1), pipe and couplings, snatch blocks, rope, and pipe wrenches. The first step in assembling the driving rig is to form a collapsible tripod by lashing together the ends of three pipes of equal length (as shown in Figure 2). A tripod with legs 20 feet long (1-inch pipe is stout enough) is tall enough to drive a 10.5-foot section of pipe. For easy transportation, these legs may be cut in half and rejoined by use of a pipe coupling. A snatch block is attached to the apex of the tripod with a short piece of line and a 1-inch manila rope is fed through the block to lift the jackhammer.

The tripod is raised by the following method:

1. With tripod flat (as in Figure 2), spread legs "A" and "B" until they form an angle of about 120 degrees.

2. Bury the ends of legs "A" and "B" to prevent them from slipping.

3. Lift tripod at lashings as high as possible, then use leg "C" to push legs "A" and "B" into an upright position, thereby forming a tripod, which can be walked from one pipe location to another without disassembly.

Note: The work crew must observe caution and coordinate its movements to permit a safe raising or transporting of the tripod since it is heavy and unwieldy.

Once the tripod is on location, the lifting rope is tied to the jackhammer and a trigger rope is attached (as shown in Figure 3) to operate the jackhammer when it is out of reach. The jackhammer, which is fitted with a driving shaft, is then lifted to the top of the tripod. The pipe (with a coupling on top to protect the pipe threads). and with the

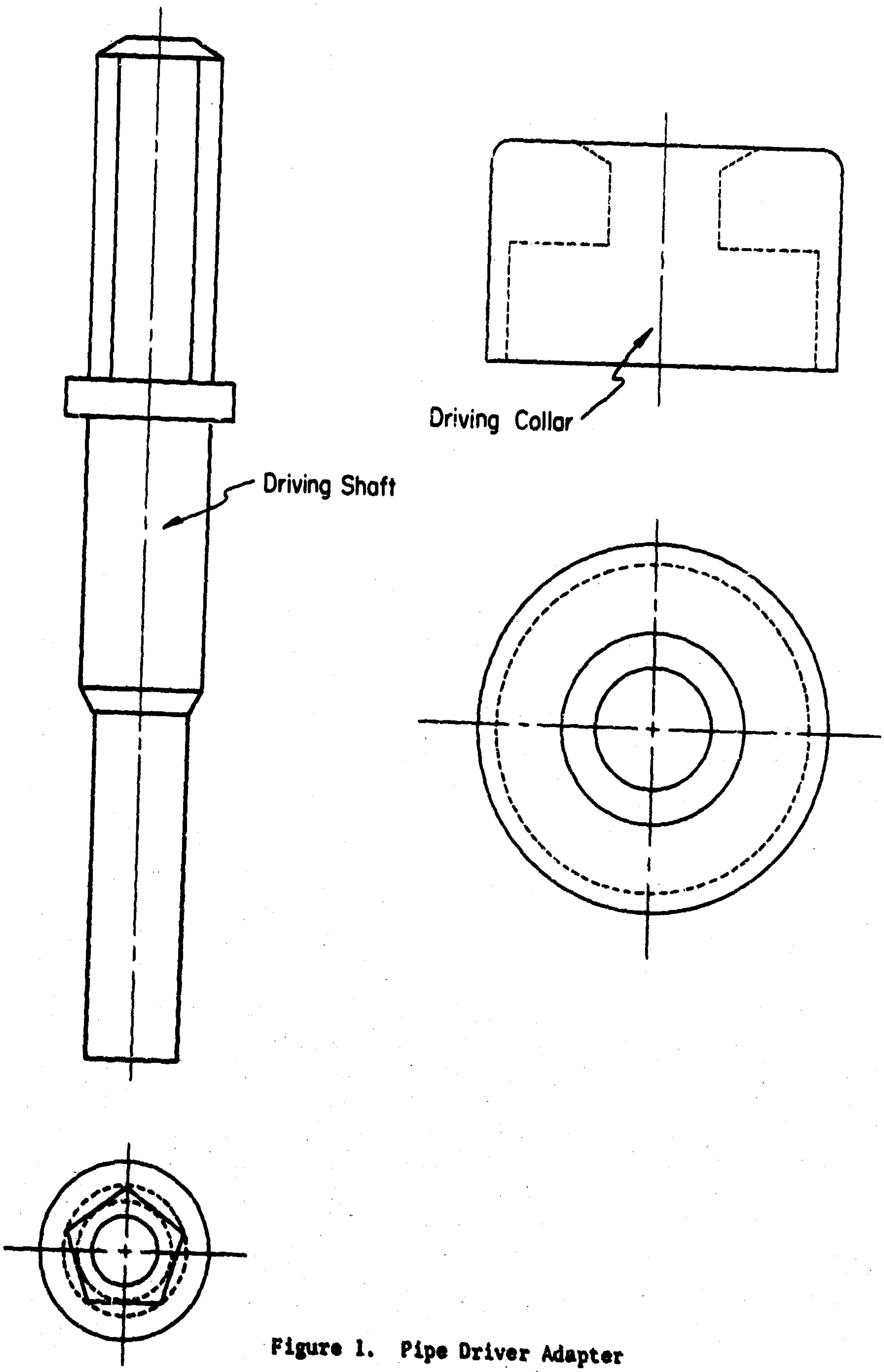

Figure 1. Pipe Driver Adapter

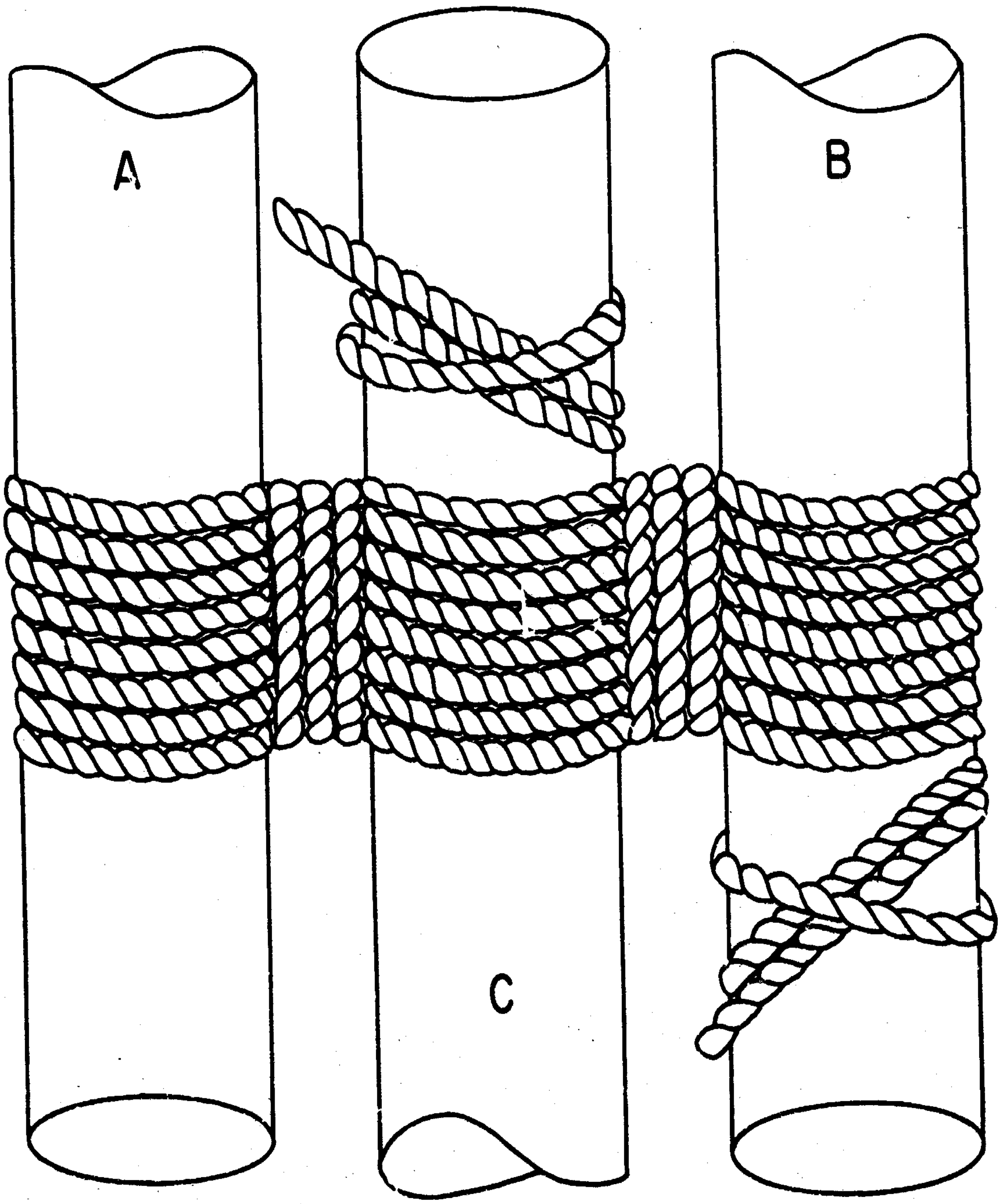

Figure 2. Collapsible Tripod

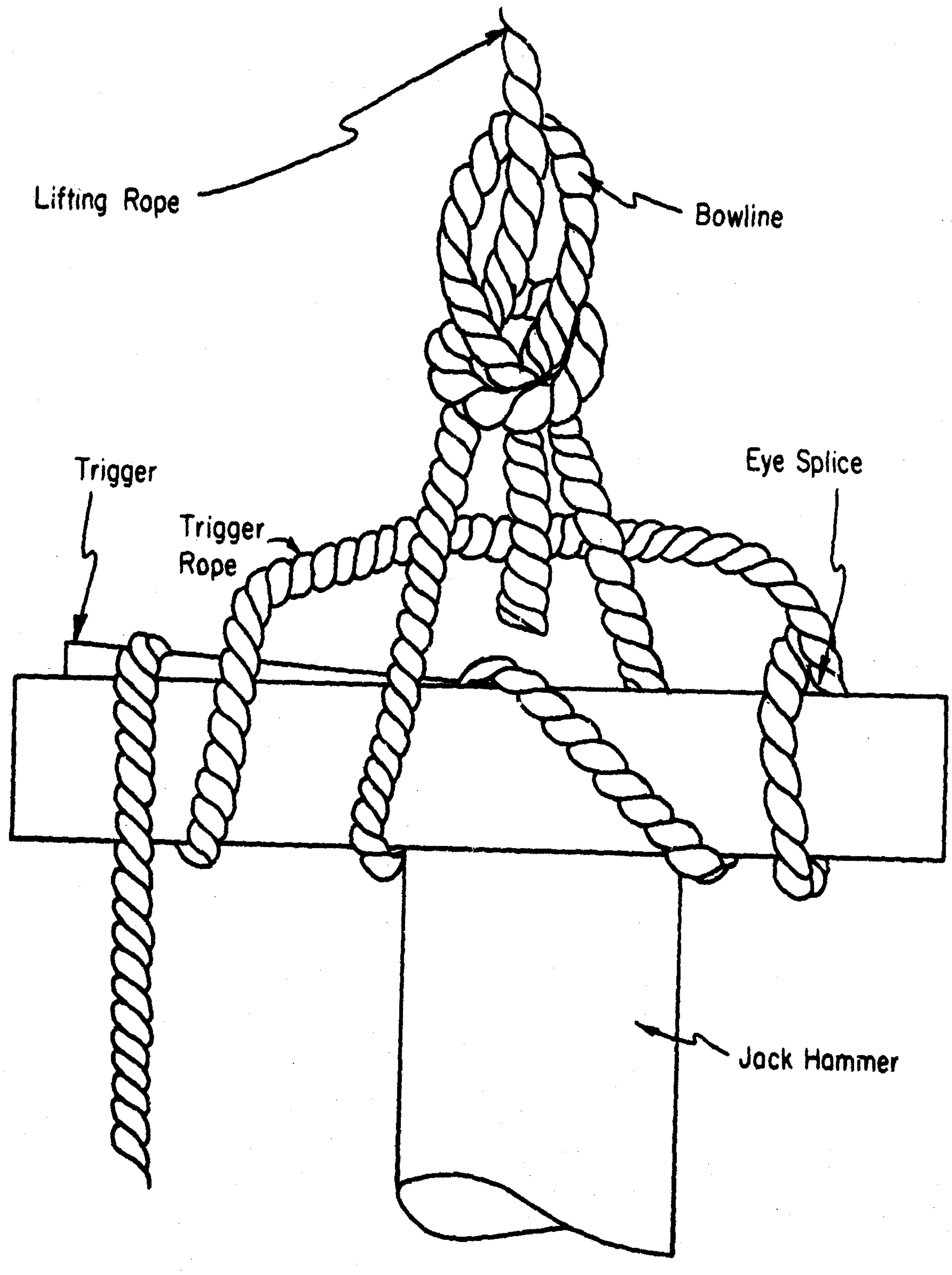

Figure 3. Trigger Rope Attached to Jackhammer

driving collar on top of the coupling, is manually raised to a vertical position and the driving shaft on the jackhammer is slipped into the driving collar. The bottom of the pipe is then placed in the desired position, the trigger rope is pulled and the pipe is driven to the desired depth, thereby completing the operation. The jackhammer is then untied and removed, and the tripod is walked to the next location.

This method may be used in shallow water as well as on the beach. Underwater driving requires some changes in the technique. Since a jackhammer has a minimum pressure differential at which it can efficiently operate (this varies with type of jackhammer), an increase in air pressure from the compressor is required as the depth of water in which the jackhammer is used increases.

Further, since man is almost weightless in water, a diver cannot exert a downward pull on the hammer-lifting rope, and some additional rigging is required to lift the hammer to the top of the tripod. In depth of 3 to 15 feet, another snatch block may easily be attached near the bottom of one leg of the tripod along with a foot peg for the diver to pull against to prevent lifting the tripod leg from its footing - Figure 4. Since it may be necessary to lift the top of the jackhammer above the surface of the water, an airlift bag cannot be used. When used under water, the jackhammer must be completely disassembled after each day of use and rinsed in diesel fuel to prevent corrosion.

The experience gained from driving 60 pipes has made subsequent operations easier. The most troublesome difficulty is that all couplings along the pipe being driven must be constantly tightened, since they loosen from impact. Driving on a loose coupling will cause the destruction of pipe and coupling threads.

When starting to drive pipe through rock not covered by sand, the jackhammer with a chisel bit may be used to make footings for the tripod legs. When using this chisel bit to make a hole, the jackhammer must be rocked back and forth to keep the bit from sticking in the hole, since most commercially available jackhammers will not back-hammer. If pipe is to be driven through hard rock, a drive shoe may be used to cut the rock. A drive shoe resembles a standard pipe coupling with threads in one end to fit the pipe and a cutting edge on the other end. They may be obtained from Clayton Mark & Company, 1900 Dempster Street, Evanston, Illinois.

In the future, it will be necessary to drive pipe in rock under water as deep as 30 feet. A boat or barge will be needed to float the large diesel air compressor. Instead of a tripod, airlift bags will be used to lift the jackhammer and to float it from one pipe location to the next (Figure 5). In shallower water where the lift bag will be subjected to surface waves, a combination of both bag and tripod will be used. The bag will be used to raise the jackhammer to the top of the tripod.

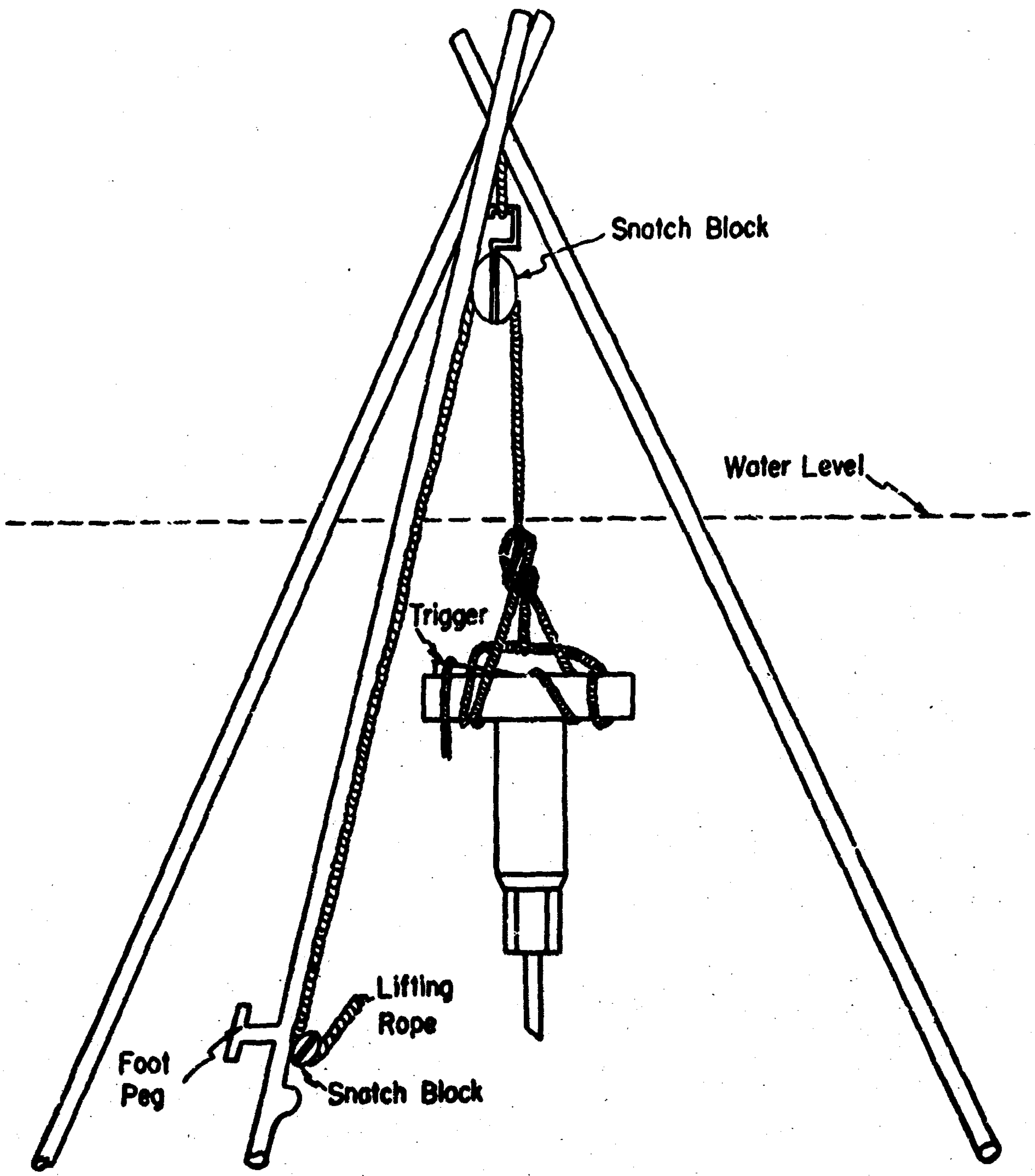

Figure 4. Snatch Block Attached to Leg of Tripod to Prevent Lifting from Footing

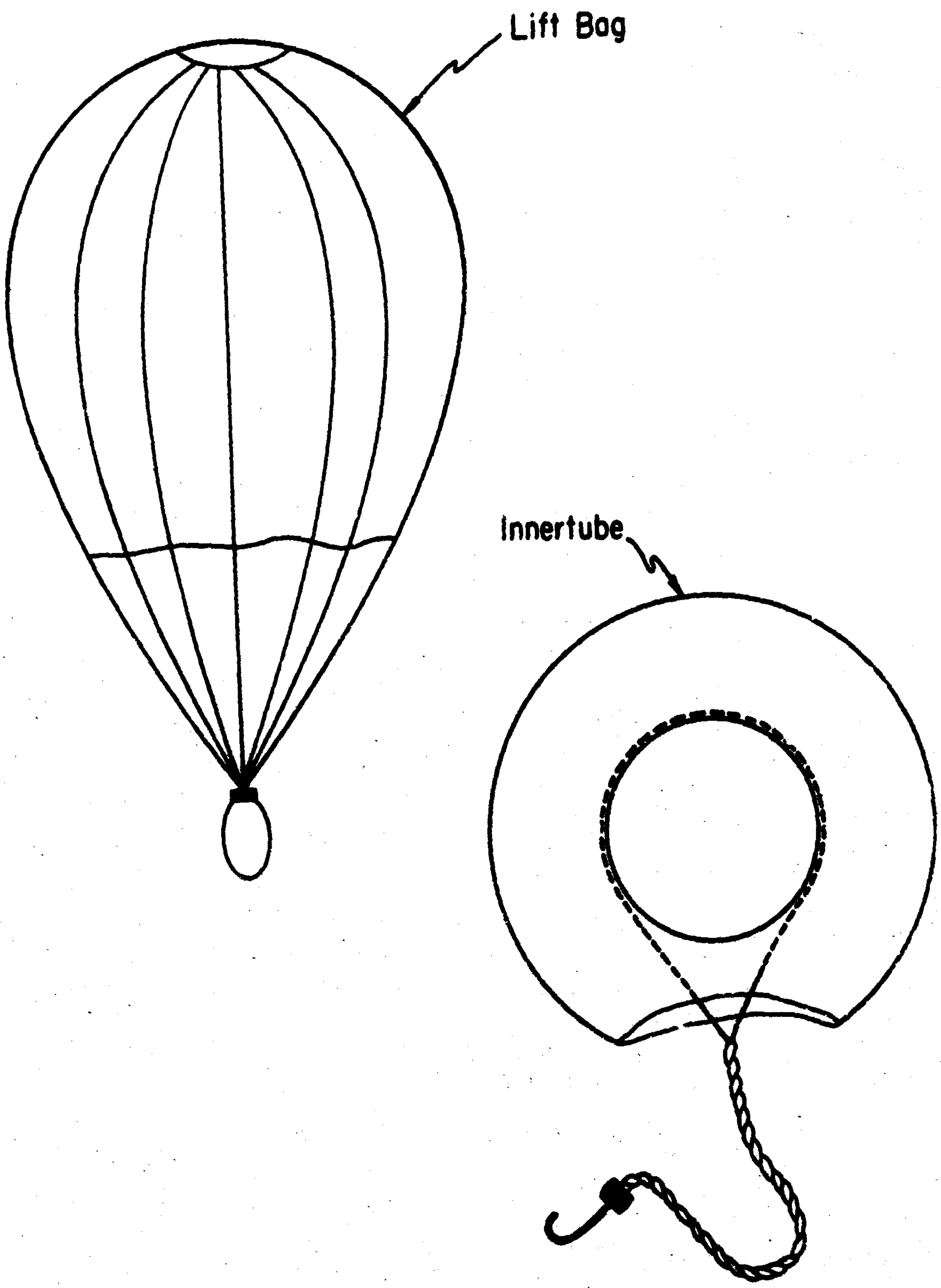

Figure 5. Airlift Bags to Float Jackhammer to New Location

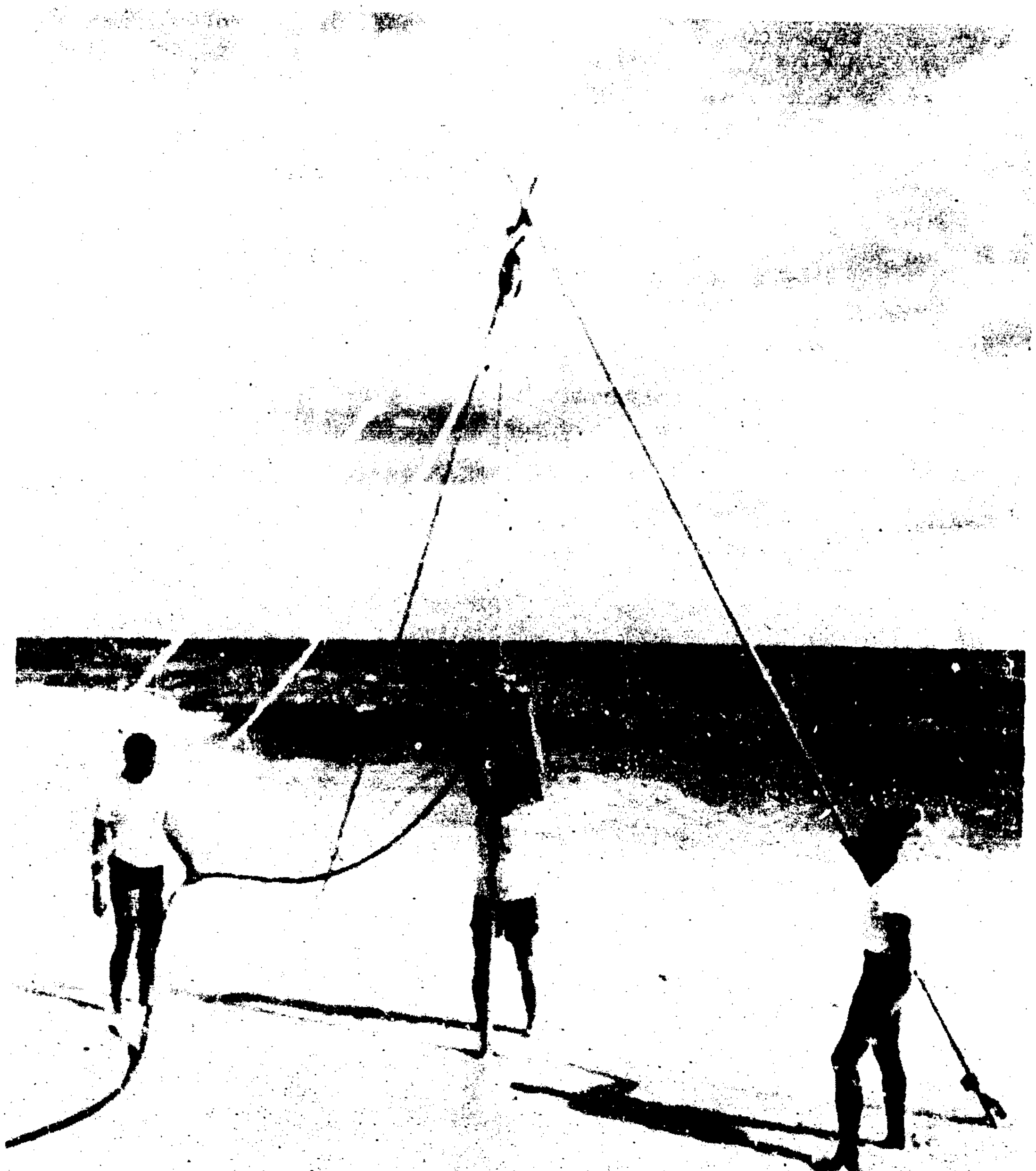

Figure 6. Driving Pipe with Jackhammer Supported by Tripod

This method of driving pipes for permanent reference markers has proven very successful. None of the many pipes driven have ever come loose or been lost. The tops of a few have come unscrewed resulting in temporary loss of the bottom section which was covered with sand. However, these were easily located, and new sections added as needed. Short pipes have been driven into underwater rock by the same method to serve as permanent anchors for other research purposes such as guying [illegible]sea instrument towers and anchoring cables. Although often under strain, these pipes have never loosened. It is hoped that the successful use of this method can be applied to many other types of underwater construction.

(NOTE: This paper was delivered at a meeting on "Beach Profiling: Purposes, Techniques, and Results" held 9 November 1969, in Atlantic City, N. J. The work described was done under CERC Contract DACW72-69-C-0018)

WAVE DIFFRACTION IN A LABORATORY MOVABLE-BED SETUP

by

John C. Fairchild

INTRODUCTION

In laboratory tests of longshore transport such wave effects as diffraction, refraction, reflection and interference can undesirably influence basic test results or may impose anomalous effects on the results which make interpretation difficult. Observations and analyses of laboratory tests of longshore transport in CERC's Shore Processes Test Basin (SPTB) have shown that each of the wave properties noted may cause undesirable or anomalous effects. However, at the present stage of the test, there is more positive evidence supporting the wave-diffraction effects. This paper presents data and analysis illustrating some of the wave-diffraction problems in laboratory tests of longshore transport on a sand beach. Recommendations are made concerning the mitigation of such effects.

WAVE-DIFFRACTION IN A LABORATORY SET-UP

In laboratory tests of longshore transport it is desirable to have the waves approach the test beach with a constant wave height and shape along the crest length. Waves of constant height and shape will produce a uniform distribution of energy along a test shore which, in turn, should cause a uniform rate of littoral transport. If such a uniform delivery of waves could be approximated in the laboratory, it would greatly enhance the testing and analysis of "wave energy-longshore transport" relationships. There are several properties of waves, however, which preclude a high degree of accuracy in the orderly delivery of uniformly sized waves; among these are wave reflection, refraction, and diffraction. Also, it is conceivable that basin-resonance phenomena might introduce spurious results. An important one of these properties is wave diffraction, that phenomenon whereby energy is transferred laterally along a wave crest. This lateral transfer of wave energy is most noticeable where an otherwise continuous train of waves is interrupted by a barrier such as a breakwater. In such a case, wave diffraction causes a change in wave height and direction along the crest in the lee of the breakwater. This effect has been described elsewhere in considerable detail (1, 2, 3)* and is usually illustrated in a series of diagrams of diffracted wave heights in relation to undiffracted wave heights (Figure 1).

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*1. CERC TR 4
2. Wiegel, R. L., Diffraction of Waves by Semi-Infinite BKW, Proceedings, ASCE, Vol 88, Hy 1, January 1962
3. Penney, W. G. and A. T. Price, Diffraction of Sea Waves by BKW Directorate of Miscellaneous Weapons Dev., Tech. History No. 26, Art. Harbors Section 3-D, 1944

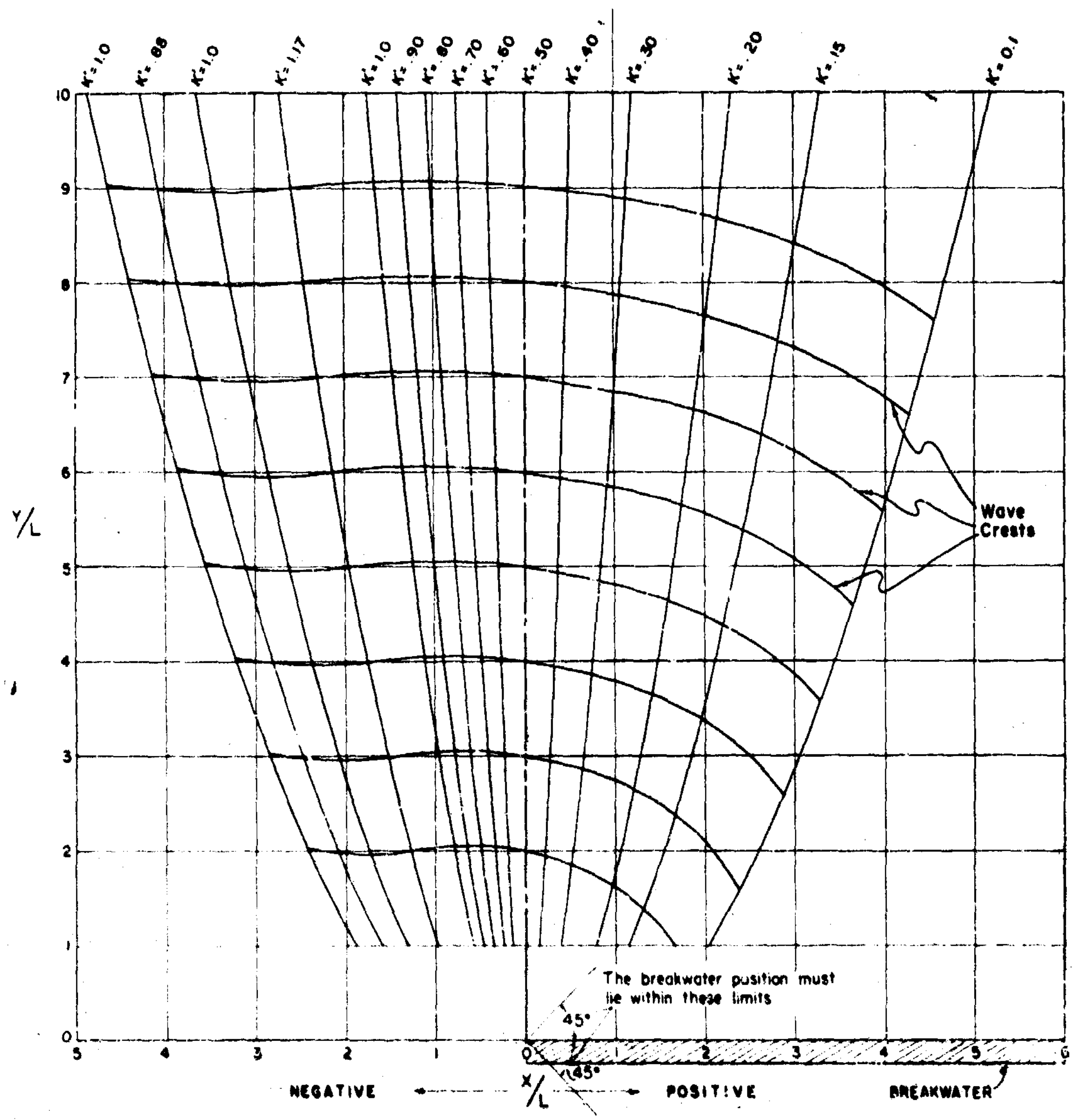


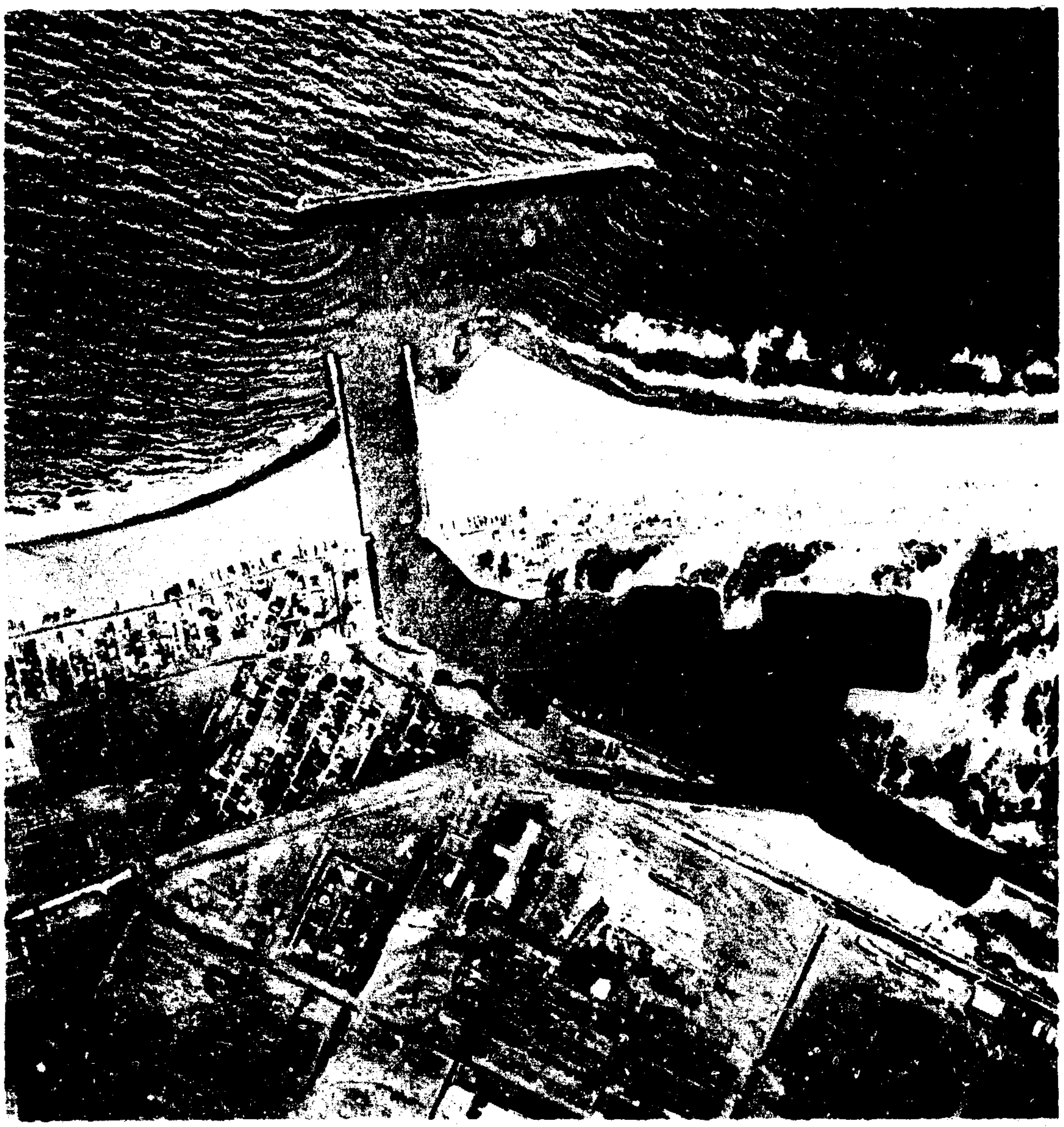
Figure 1. Generalized diffraction diagram.

Figure 1 is a diagram using isograms of the ratio between diffracted wave height and incident wave height (K'), and wave crest lines to show the pattern of diffracted waves in the area shoreward of a breakwater. The coordinate system in the figure is plotted in dimensionless units of x/L and y/L where x and y are the distances along the abscissa and ordinate from the origin (breakwater tip) and L is the wave length at the breakwater depth. The central K' line in Figure 1 which passes through the origin has a value of 0.5 meaning that the wave height along this line has been reduced by 1/2 its value just seaward of the breakwater. These wave-height reductions occurred through lateral flow of wave energy toward the lee or sheltered side of the breakwater. In Figure 2, an aerial photograph, wave diffraction behind a prototype breakwater is shown.

In a typical laboratory longshore-transport test, there are several structures which may interrupt the wave train and thus cause wave diffraction in the test area. These are guide walls, splitter walls, sand traps, ends of wave-generator bulkheads or the end sections of any wall or structure in the path of wave advance. The open basin setup in Figure 3 shows that wave diffraction can occur at the ends of wave generators 1 and 4. The area directly in front of the wave generators is analogous to that portion of the wave crests in Figure 1 which advances past the breakwater end and then diffracts to its lee. The area updrift and downdrift of the wave-generator ends is analogous to the lee or sheltered area in Figure 1. The more relevant part of the diagram in Figure 1 to wave-diffraction effects in the SPTB longshore transport tests is the half of the diagram to the left of the ordinate line which passes through the origin. That portion of the Figure 1 diagram has been superposed in Figures 4 through 7(by a scale adjustment) to show the relative diffraction effects from the ends of the wave generators. The range of K' values (diffracted height/incident height) to the left of the center ordinate in Figure 1 begin at 0.50 and go up to a maximum of 1.17. With this range of K' values it would be theoretically possible for the highest waves (K' = 1.17) to be 2.34 times higher than the lowest waves. Moreover, since the wave energy, $E \sim K H^2$(H is wave height), the highest waves (K' = 1.17) could apply 5 1/2 times as much wave energy per foot of beach as the lowest waves (K' = 0.50). This energy factor of 5 1/2 considers the wave diffraction from only one wave generator end. When the concentrated effects from both generator ends are considered to occur as shown in Figure 4, then it is theoretically possible that the energy factor of 5 1/2 may be doubled. If this is so, then some local segments of the shore receive 10 or more times the energy level of other shore segments.

WAVE DIFFRACTION EVIDENCE

A longshore-transport test in 1965 illustrates some problems caused by wave diffraction. This test (No. 1-65) was one of a continuing series of longshore transport tests in the SPTB. Figure 3 shows the relative position of the wave generators, the test beach, the sand trap, and the sand feeder. Previous tests were conducted with training walls at both ends of the generators to confine the wave energy as it approached the



June 1962

Figure 2. Wave diffraction behind breakwater at Channel Islands Harbor, California

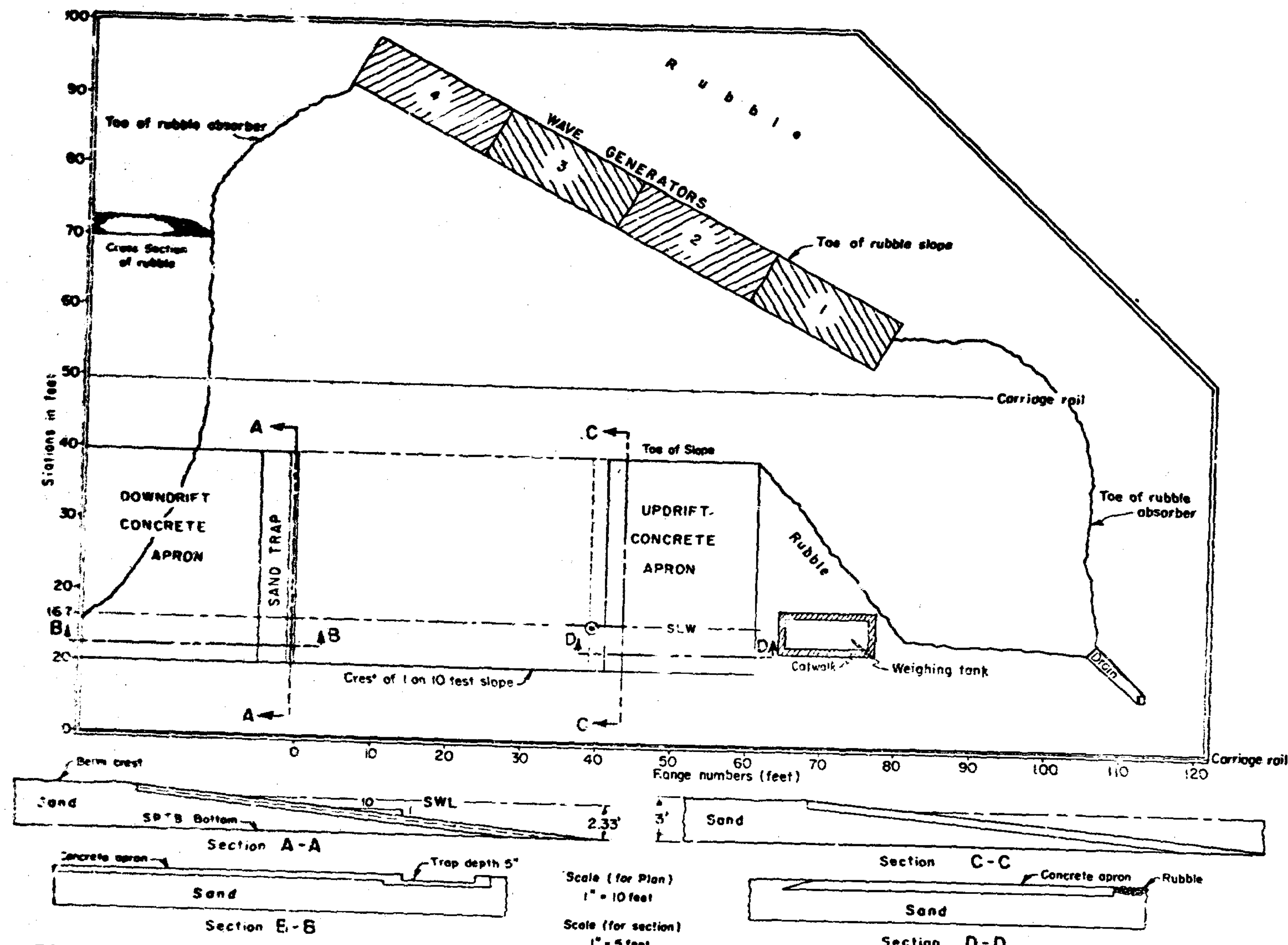


Figure 3. Longshore transport test layout in the north sector of the Shore Processes Test Basin (1965 and 1966).

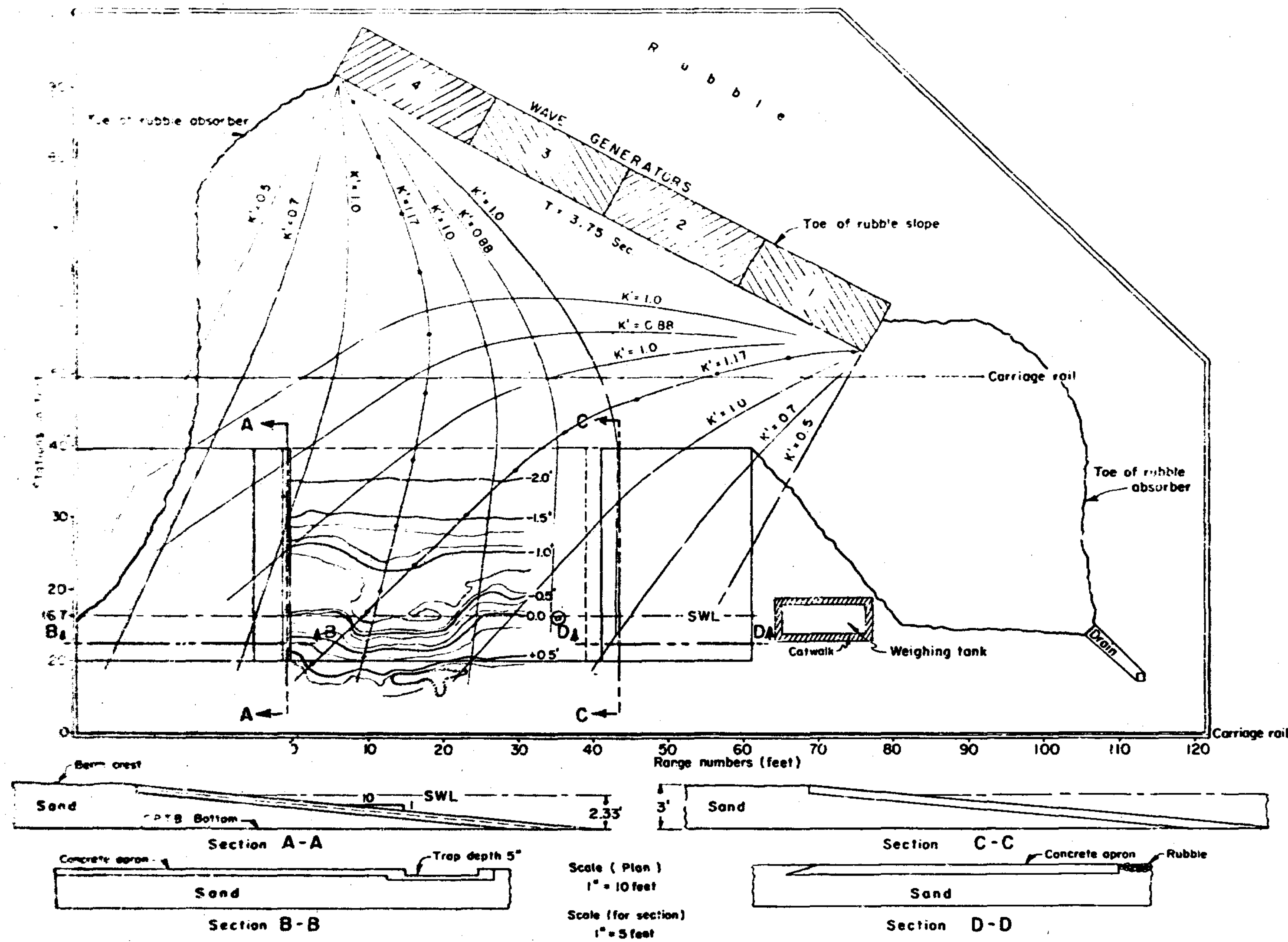


Figure 4. Relationship of wave diffraction K' lines and beach contours after 27 hours of wave action in longshore transport test No. 1-65.

beach. However, reflection from the walls or possibly resonance between the walls created undesirable wave conditions, and for this test the training walls were removed and a wider rubble absorber beach was installed. The new setup was arranged to permit generation of a wave crest approximately 80 feet long which approached the toe of the test slope at a 30-degree angle, shoaled and refracted, and finally impinged along a test shoreline 92 feet long. The downdrift part of the test beach, a concrete slab on a 1 on 10 slope, was about 30 feet long and included a recessed trap for catching sand drift transported by the wave action. The updrift part of the test beach was also a concrete slab on a 1 on 10 slope and 20 feel long. The sand beach area lying between was 42 feet long. The wave period for the test was 3.75 seconds; water depth was 2.33 feet; total test duration was 40 hours.

At the end of 27 hours of testing a large cut had eroded just updrift of the sand trap. Wave diffraction analysis at this stage of the testing showed that waves diffracting from the updrift and down drift ends of the wave generators concentrated wave energy in the erosion area. Figure 4, a plan drawing of the test area with beach contours, shows the beach profile adjustment after 27 hours of testing. Superposed diffraction diagrams show the diffracted wave patterns from the ends of the wave generators and their relation to the eroded area. (Dashed lines show the approximate displacement effect of the K' lines due to wave refraction.) In Figure 4 the maximum wave-diffraction (K') lines from the ends of the wave generators intersect in the eroded area near the SWL. The dashed lines show that wave-refraction effects shift the intersection of the K' lines about 5 feet updrift thus placing the concentrated diffraction effects nearer the center of the eroded area. Thus it appears that a concentration of energy, due primarily to wave diffraction, caused severe localized erosion of the test beach.

To further check on wave diffraction as the cause of erosion, the downdrift generator was turned off after 27 hours of testing time. This shifted the source of wave diffraction from one end of the generators about 25 feet updrift and thus provided a more even distribution of energy along the whole beach (Figure 5). As the test continued beyond 27 hours, the eroded area filled rapidly so that at the end of 35 hours the shoreline was essentially straight and uniform, and remained so during the remaining 5 hours of the test. This result supports the original hypothesis that the localized erosion was caused by the concentration of energy by wave diffraction from the two wave generator ends.

Further tests were made in 1966, and in Figures 6 and 7 respectively, diffraction diagrams for the 2.18- and a 1.25-second wave periods tested are superposed on a plan drawing of the 1966 SPTB longshore transport test layout. The 2.18-second test was run for a total of 25 hours while the 1.25-second test was run a total of 50 hours. The sand bottom and beach contour system that existed at completion of each of these tests is superposed on the respective plan drawings. These additional figures show diffraction patterns for shorter wave periods in the SPTB test depth (2.33 ft) while Figures 4 and 5 show patterns for a relatively long wave period,

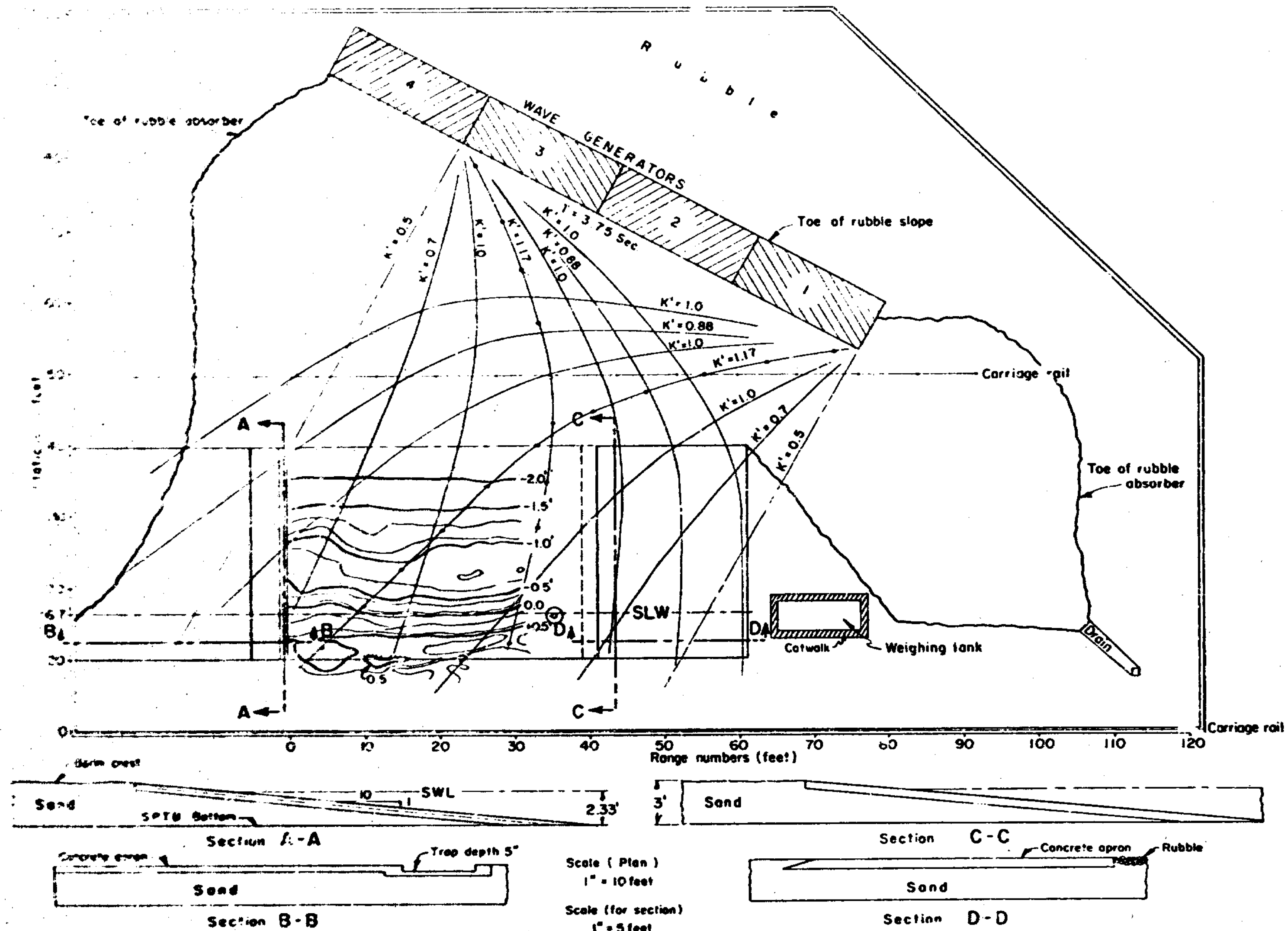


Figure 5. Relationship of wave diffraction K' lines and beach contours after 40 hours of wave action in longshore transport test No. 1-65.

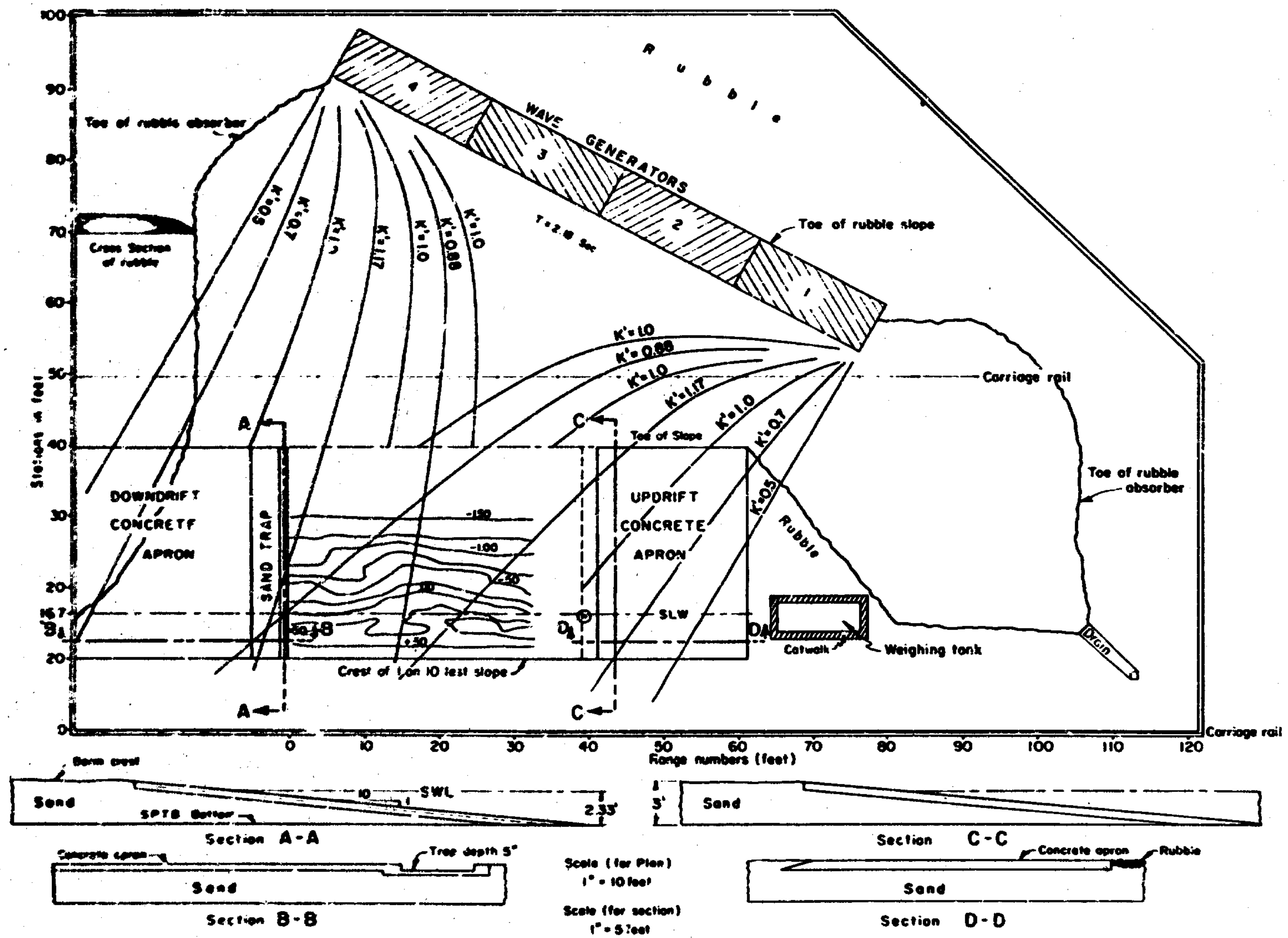


Figure 6. Wave diff-action K' lines and beach contours superposed on longshore transport test No. 2-66 layout.

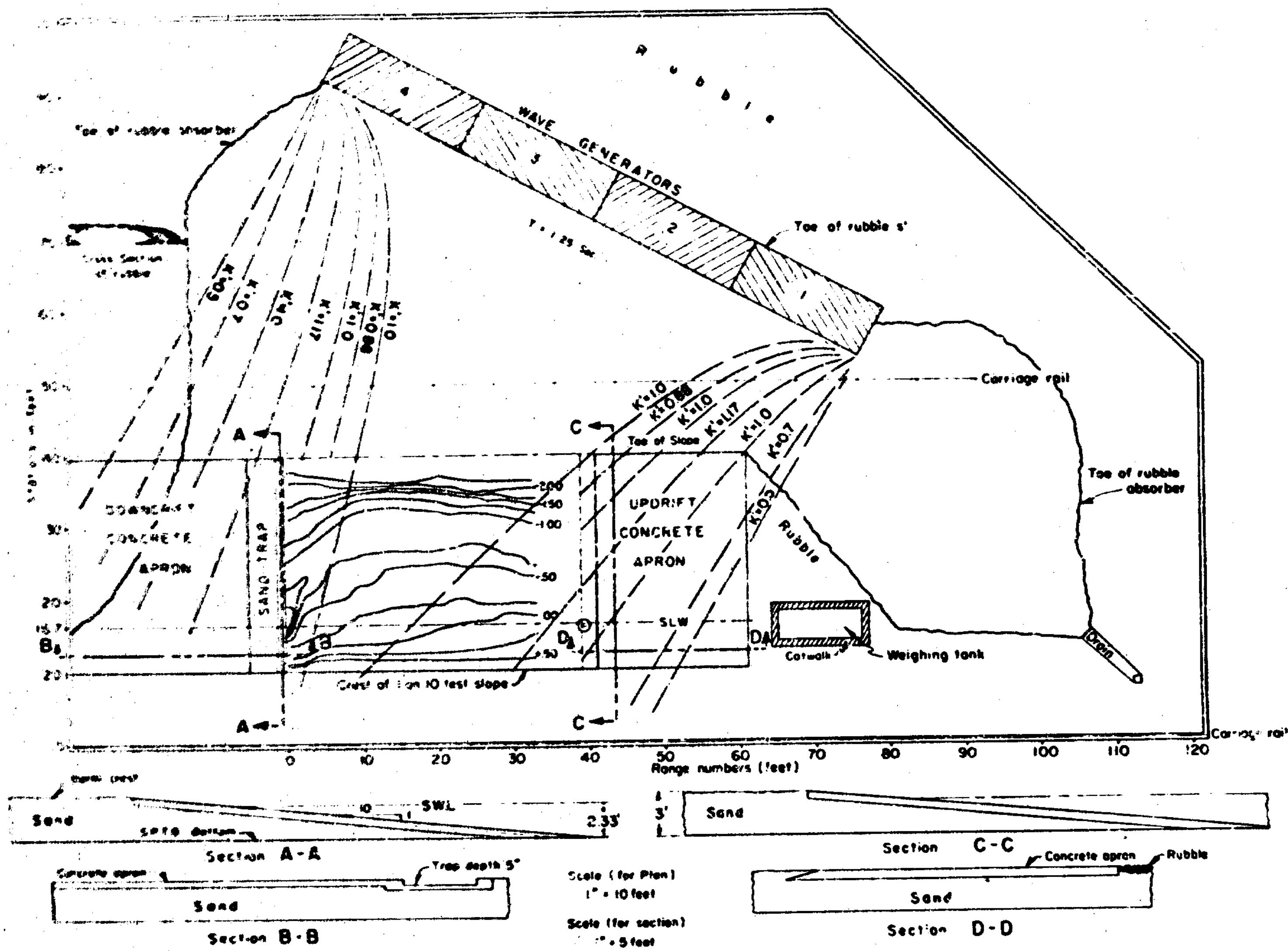


Figure 7. Wave diffraction K' lines and beach contours superposed on longshore transport test No. 3-66 layout.

3.75 seconds. An examination of the K' lines in Figures 6 and 7 in relation to the test beach area shows that considerably more of the test beach area is free of diffraction effects than is true in the longer wave-period diagrams. There also appears to be some correlation of wave diffraction and beach contours in the 1966 test as shown in Figure 6. Note the convex seaward "bulge" in the contour system which begins near the SWL at range 30 and extends downdrift to about range 10. The location of this bulge in the contours suggests a possible relation to the low-value K' lines from both ends of the generators. In Figure 7, there is no readily apparent effect of wave diffraction; however, most of the diffraction effects are outside the test beach area.

CONCLUSIONS AND RECOMMENDATIONS

Experience in testing of longshore transport in the SPTB has shown that while the problems of wave diffraction in a given test setup may not be completely eliminated, they can usually be significantly reduced. The reduction of diffraction depends upon varying the geometric relation of the various wall or structure ends, wave generator location and angle, and test beach location consistent with sound practice in accomplishing the test objective. In most instances the final test setup or configuration will include some degree of compromise.

For example, when training walls are extended orthogonally from the wave generator ends, wave diffraction from these generator ends is prevented. However, wave diffraction will still occur at the ends of the training walls along with other problem effects, such as cross basin reflections which may become resonant to some wave lengths. On the other hand, an open-basin test area without training or splitter walls provides no guarantee against spurious wave diffraction effects. Therefore, it appears that wave diffraction must unavoidably occur somewhere in the test area, whether it be an open or a wall-compartmented basin. Diffraction in an open basin has been experimentally demonstrated by the local erosion in test No. 1-65, discussed previously and shown in Figure 5.

Since wave diffraction will unavoidably occur in any practical and workable test set-up, the project engineer must plan carefully. Such plans must enable him to recognize problems of wave diffraction likely to arise, so that he may act to eliminate or overcome them. In the past, wave diffraction in many movable-bed, wave-action studies may have been overlooked or else the effects were judged to be of relatively minor significance to the test results.

Now that it has been shown what wave diffraction does cause spurious results in longshore transport tests, it is recommended that pretest analysis of any possible wave-diffraction effects be made in such tests.

Pretest analysis should evaluate each proposed test setup as to the seriousness of possible wave-diffraction effects, and make or recommend remedial setup changes only after the analysis has been completed. If this is done, the longshore transport data and analysis obtained in a given test will be less subject to unexplained or overlooked anomalous effects due to wave diffraction; and consequently will be more reliable since a more direct and confident interpretation and conclusion can be made from the test results.

SUMMARY OF RESEARCH PROGRESS FOR FISCAL YEARS 1967, 1968, 1969

Compiled by

John C. Fairchild
Research Division
Coastal Engineering Research Center

I. WIND-WAVE ACTION IN COASTAL WATERS

A. Studies at CERC

1. Wave Runup. Several hundred consecutive waves and corresponding runup were measured on the beach at Nags Head, North Carolina. Water level changes in the area shoreward of the surf zone (surf beat) were also measured to study the possible effect on runup. It appears that a statistical analysis is more meaningful than a direct wave-to-wave correlation.

2. Wave Setup. In this study, analysis of the data from 92 laboratory tests is continuing. The ratio of maximum setdown to breaker height averaged 0.02, 0.03 and 0.07 for slopes of 0.05, 0.10, and 0.20, respectively. At the original stillwater line, the ratio of setup to breaker height averages about 0.21 with no obvious dependence on beach slope or breaker height. Offshore of the breaker point, setdown is in approximate agreement with linear theory.

Experimental results, briefly described above, were presented as a talk at the 1967 annual meeting of the American Geophysical Union.

3. Wave-Breaker Study. A report "Breaker Type Classification on Three Laboratory Beaches" by C. J. Galvin, Jr., was published in the Journal of Geophysical Research, June 1968. Breaker types for waves on plane laboratory beaches were shown to be reasonably predicted by either of two dimensionless parameters. For waves on smooth concrete slopes, breaker type depends on beach slope, m, wave period, T, and either deep-water or breaker height, H_o or H_b. For 43 varied laboratory conditions, breaker type was sorted fairly well by either of two dimensionless combinations of these variables; an offshore parameter, $H_o/L_o m^2$, or an inshore parameter, H_b/gmT^2. As either of these parameters increases, breaker type changes from surging (or collapsing) to plunging or spilling. For the offshore and inshore parameters, respectively, the surge-plunge transition values are about 0.09 and 0.003. In another study, an approximate calculation suggests that the minimum horizontal distance traveled by the crest of a plunging wave from the breaker position to its touchdown point is about 2 times the breaker height. Laboratory experiments - on three plane beaches and on one composite beach - indicate that the average value of this distance ranges from about 2 to more than 4 times the breaker height on plane beaches as slope decreases from 1 on 5 to values flatter than 1 on 20. A talk on breaker travel (available as ASCE preprint 630,

"Horizontal Distance Traveled by a Plunging Wave") was delivered at the ASCE Transportation Meeting in February 1968, and an extended version of this preprint, "Breaker Travel and Choice of Design Wave Height" by C. J. Galvin, Jr., was published in the ASCE Journal of the Waterways and Harbors Division, May 1969. In this paper it was shown that structures sited in shallow water on moderate or steep slopes can be subject to breaking waves with heights significantly greater than would be predicted by accepted design practice. Computer programs have been developed which predict the breaking position and height at breaking of waves moving into shallow water over a slope.

4. Secondary Waves. Steep waves in shallow water have nonlinear properties similar to those exhibited by interacting "solitons", nonlinear dispersive wave entities that arise in solutions of the Korteweg-deVries (KdV) equation (Zabusky, N. J. and Kruskal, M. D., "Interaction of 'Solitons' in a Collisionless Plasma and Recurrence of Initial States", Physical Review Letter, Vol. 15, 1965, pp. 240-243). Finite-amplitude, initially sinusoidal waves generated by the periodic motion of a wave generator in shallow water, disperse into two or more waves or differing amplitudes and speeds, with speeds dependent on amplitudes. In both experimental and numerical solutions, the larger of the waves eventually overtakes, interacts with, and passes through the smaller secondary waves. During interaction the net crest elevation of the two superimposed waves is less than the elevation of the larger when isolated. The interacting secondary waves eventually assume a form nearly identical to the initial sinusoidally generated form. Conditions in the region $d/L \leq 0.09$, $H/d \geq 0.05$ produced interacting or secondary waves. The number of waves increased as d/L decreased, and was relatively independent of H/d. At lowest (H/d, d/L) conditions, interactions resulted in confused wave forms. Regular periodic waves were limited to the remaining region, largely $0.3 \leq d/L \geq 0.09$. Abstracts of two talks "Secondary Waves as Solitons" and "Shapes of Unbroken, Periodic, Gravity Water Waves", both by C. J. Galvin, Jr., were published in the Transactions of the American Geophysical Union, v. 49, March 1968. Digitized records of the 2-, 3-, 4-, and 5-wave conditions in the 96-foot tank were prepared for comparison with the numerical solution of the Korteweg-deVries equation developed at Bell Telephone Laboratories. Initial comparisons for the 3-wave case shows good agreement between predicted and observed wave forms.

Test results have shown that the secondary wave effect is primarily a function of the ratio of water depth to wave length, and not one produced directly by wave generator motions. Tests have also shown that secondary wave phenomenon occurs, regardless of scale, with tests in the large wave tank repeating results in the small wave flume. Spectral analyses have shown that the secondary wave represents a transfer or dispersal of energy from the primary to higher harmonics. Analyses have been broadened to include a study of all unbroken wave forms produced by a wave generator.

5. Wave Forces on Piling. A large volume of data on wave forces on piling was collected in CERC's large wave tank. This data consisted primarily of strain-gage measurements of wave force (longitudinal and

lateral), made synchronously with measurements of wave height and period. A total of 300 tests were made of different wave conditions, water depths, and exposures or arrangements of test structures. These data were analyzed, and a preliminary report was published. A more comprehensive analysis of these tests and a final report are still required.

Some reanalysis of this data is currently underway with two main objectives. One is to recheck and complete the earlier analysis and publish a final report. The other is to select typical tests covering the range of wave force and wave heights measured (including maximum), and digitize records of these tests so that they may be readily available for computer applications of wave forces on piling.

6. New Bern Stone Study. A porous marlstone from New Bern, North Carolina, has been tested in the tank for large waves at CERC and in the field. The tests were designed to determine the suitability of the New Bern stone for use as a cover layer in major coastal structures under wave attack. Rubble-mound breakwaters were tested in the large wave tank in 1965 to determine stability coefficients (K_D) for use in field designs. In addition, weight losses of selected stones were recorded. Tests show that the physical characteristics (density, wearing qualities, and water uptake) vary considerably from stone to stone. The results indicated that designs based on the K_D value recommended in CERC Technical Report No. 4, Shore Protection, Planning and Design, for similar conditions would be somewhat conservative.

As a result of the weight losses observed in the tank for large waves, field tests were conducted to determine if the stone dissolves in sea water. Testing was carried out on an existing jetty at Fort Macon, North Carolina, from November 1967 to June 1969 by personnel of the Wilmington District. Thirteen stones were placed in jetty voids in the wave action zone. The stones were periodically removed, weighed, and replaced. Rock weight losses were recorded. Results show that the stone from the New Bern quarry is variable with respect to surface texture, coloration, apparent durability, and water uptake, but failed to produce positive information on the mode or modes which caused stone weight losses. The use of New Bern stone in cover layers in ocean waves is not recommended. A final report is being prepared.

B. Contract Studies

1. Wave Forces on Coastal Structures - University of California, Contract DACW72-69-C-0001. Work continued at the University of California, particularly on the development of a computer program to determine forces and moments on piling of different diameters for different wave heights, periods, and water depths using different values of C_D and C_M. Work was also continued on the forces exerted on structures by "bores" or broken waves moving up a sloping beach. Laboratory work using a 6-inch pile with both regular and irregular waves has been completed and the results were

presented in a technical report, "Wave Forces on Circular Cylindrical Piles Used in Coastal Structures" by Juan Jen, issued as a University of California Technical Report HEL 9-11. Another report "Statistical Predictions of Wave-Induced Responses in Deep-Ocean Tower Structures" by Edward Foster, Jr., was also received. Work has continued on the statistical distribution function of wave forces on piling using prototype data, including basic studies to develop statistical methods of designing for the resistance of structures to wave-induced forces.

More recent contract work has emphasized development of methods for the application of wave spectra, as well as monochromatic waves, to the design of coastal structures, including pile-supported structures. Laboratory work was also initiated to measure the lift forces induced by eddies formed by wave action at piles. This work included rebuilding the transducers on the test piles to obtain greater sensitivity and reduced "pen drift". The reasons for the emphasis on the formation of eddies is discussed in an appendix to a paper, "Waves and Their Effects on Pile-Supported Structures" by R. L. Wiegel, issued as University of California Technical Report HEL 9-15. Under this contract, Professor Leon E. Borgman completed a year of residence at CERC and conducted seminars for CERC staff members on time-series analysis and on directional wave spectra including a "wrapped around" normal distribution spectrum model.

2. Wave Interaction with Coastal Structures - Massachusetts Institute of Technology, Contract DACW72-68-C-0032. Laboratory work continues on the characteristics of waves generated by waves overtopping a breakwater and on energy transmission through permeable breakwaters. Tests of two small models at different scales produced similar results. The shallow-water theory for reflection and transmission has been evaluated. An equation based on Lorentz' approximation of a square-law damping coefficient and his condition of equivalent work is being programmed to obtain a computer solution for the reflection and transmission coefficients for a variety of rectangular, homogeneous breakwaters under different wave conditions. Solutions for sloping-faced and multi-layered structures will be attempted if the validity of the existing solution is established.

3. Evaluation and Development of Water-Wave Theories for Engineering Application University of Florida, Contract DACW72-67-C-0009. A paper, "Relative Validities of Water-Wave Theories", was prepared and presented by Robert G. Dean at the ASCE Specialty Conference on "Civil Engineering the Oceans" in San Francisco, September 1967. Based on this paper the results of this contract effort are summarized as follows: The fits to the two free-surface boundary conditions (kinematic and dynamic) were calculated for seven analytical wave theories and one numerical wave theory. Of these eight theories, only the second-order solitary theory and the stream-function theory (the fifth order was examined) provided perfect fits to the kinematic free-surface boundary conditions. Of the eight theories tested, none provided exact fits to the dynamic free-surface boundary condition. It was found that the stream-function theory provided the best fit to this boundary condition for d/T^2 greater than 0.08. For ratios below this value the first-order cnoidal

wave theory and the Airy wave theory provided the best fits. This result is surprising because the Airy wave theory does not predict many of the known characteristics of shallow-water waves.

To provide an indication of the correspondence between the fits to the dynamic boundary condition and the accuracy of important engineering quantities, total drag forces were calculated for an intermediate-depth wave and a shallow-water wave for several wave theories. Drag force calculations for high-order stream-function solutions were used as a base in assessing the accuracy of the other calculated drag forces. It was found that in intermediate-depth water, there was reasonable correspondence between the differences in dynamic boundary condition fits and those in calculated drag forces. In shallow water, however, it was found that the drag forces differed by a factor of 4 when calculated by the two theories providing best fits to the boundary condition. An unexpected result was that some of the lower-order theories fit the boundary conditions better than their higher-order counterparts for some shallow-water conditions. This was true for the Stokes theory for all wave heights, and for the cnoidal theory for the larger wave heights.

Recent work under this contract has concentrated on the limiting wave height using the stream-function theory in which the kinematic breaking criteria is $H/d_b = 1.0$, rather than the usually accepted value of 0.78. Work continues on the development of accurate numerical solutions for standing-wave systems. A general theoretical development was made using a perturbation technique for solving the finite-amplitude standing waves up to the third order, and a computer program prepared which can extend the method to any order and tabulate a number of wave properties useful in coastal engineering design. Several cases of various relative depths and breaking indices are being investigated to check the range of applicability of the theory and determine the order of solution needed for the best fit of the boundary conditions.

4. Wave Diffraction and Refraction - University of California, Contract DACW72-68-C-0016. Laboratory and theoretical work continue, particularly on the diffraction of waves having true spectral attributes. Laboratory and theoretical studies of the diffraction of waves through a rock reef continue. An asymptotic computer-solution of the diffraction of waves through a breakwater gap has been found promising. The diffraction of long waves is being studied in regard to harbor surges and tsunamis. Theoretical studies indicate that a simplification can be made in the statistical analysis of directional wave spectra. The simplification assumes a "circular normal distribution". If this is the case, and if the waves are being generated within ±90 degrees of the mean wind direction, then the mean wave direction and the circular equivalent of "σ" (standard deviation) can be obtained from the phase angle and coherence, which can be calculated from the spectra. A report of this work, "Diffraction of Water Waves over Bottom Discontinuities", by Nabil Hilaly, was received.

The effect of the length of record on the confidence limits in wave-spectra estimates was determined. Two interim reports, both dated August

1968, were received under this contract: "Determination of Approximate Directional Spectra for Surface Waves" by Yoshima Suzuki, and "Diffraction of Wind Waves" by Shou-Shan Fan. Computer simulation techniques are being used to study further the effect of the length of wind-wave records, number and spacing of gages, and the time between samples of digitized records, on the confidence limits of calculated directional wave spectra. A paper on this subject "The Precision of Computations Based on Ocean Wave Spectral Relations" by Leon E. Borgman was presented at the 1969 spring meeting of the American Geographical Union. A related paper, "Directional Spectra Models for Design Use" by Leon E. Borgman, was presented at the Offshore Technology Conference in Houston, Texas, May 1969.

5. Refraction Analysis of a Spectral Sea - Science Engineering Associates - Contract DACW72-68-C-0022. In a report "Refraction Studies on a Continuous Directional Spectrum", by T. Karlsson, estimates of the governing equation are derived for the distribution of a continuous directional wave spectrum in water of any depth. Assuming steady conditions and using normal relationships from monochromatic refraction methods - such as the rate of change of propagation direction and constant frequency of the wave elements - the equation describing the refraction of the continuous wave spectrum is obtained. Using finite-difference techniques, a computer program was prepared for numerical evaluation of the refraction equation. This program is decribed in the report, including the necessary input and output. The conditions under which the finite-difference scheme is stable are investigated in detail, and a stability criterion established. Finally, a few numerical examples are presented including some generalized coastline configurations and a specific area of the Atlantic Coast.

6. Ship Waves in Navigable Water - University of California, Contract DACW72-68-C-0034. Prototype measurements have indicated that major ship-generated waves in navigation channels are more likely to be caused by high-speed small craft than by larger commercial craft, which generally move at relatively low speeds. Laboratory tests were made with six ship models to determine the height of ship waves for each model at varying distances from the ship track. Relationships were developed between the maximum wave height in the ship-generated wave train, water depth, ship draft, ship length, and the distance from the ship path. These relationships allow prediction of the maximum waves in a group of ship-generated waves where the characteristics of the ship and the water depth are known. It is also possible to compute the maximum speed which a ship of given characteristics should be allowed to travel if the wave height at a particular point is to be less than a specific height.

Models of a mariner-type cargo ship and a barge were used to study ship waves in shoaling water. The models were towed at various speeds and water depths, and wave heights and patterns were measured by wave recorders and stereophotos. Motion pictures were taken with a view to creating a demonstration movie. Motion pictures were also taken in a pilot model study in a ripple tank to determine conditions of the ship speed and beach slope at which waves are absorbed or reflected from the channel boundary.

Analysis of the tests was completed, and three reports were written. The first was on the maximum recorded wave heights in the tests, the second on ship waves at recreational beaches. The third, a summary paper, "Ship Waves in Shoaling Waters" by J. W. Johnson, was presented at the Eleventh Conference on Coastal Engineering in London, September 1968. A report on the characteristics of ship waves in shallow water and in shoaling water, "Mapping of Ship Waves Breaking on a Beach" by F. H. Moffitt was issued as University of California Hydraulic Laboratory Technical Report HEL 12-8, December 1968.

II. SHORE PROCESSES

A. Studies at CERC

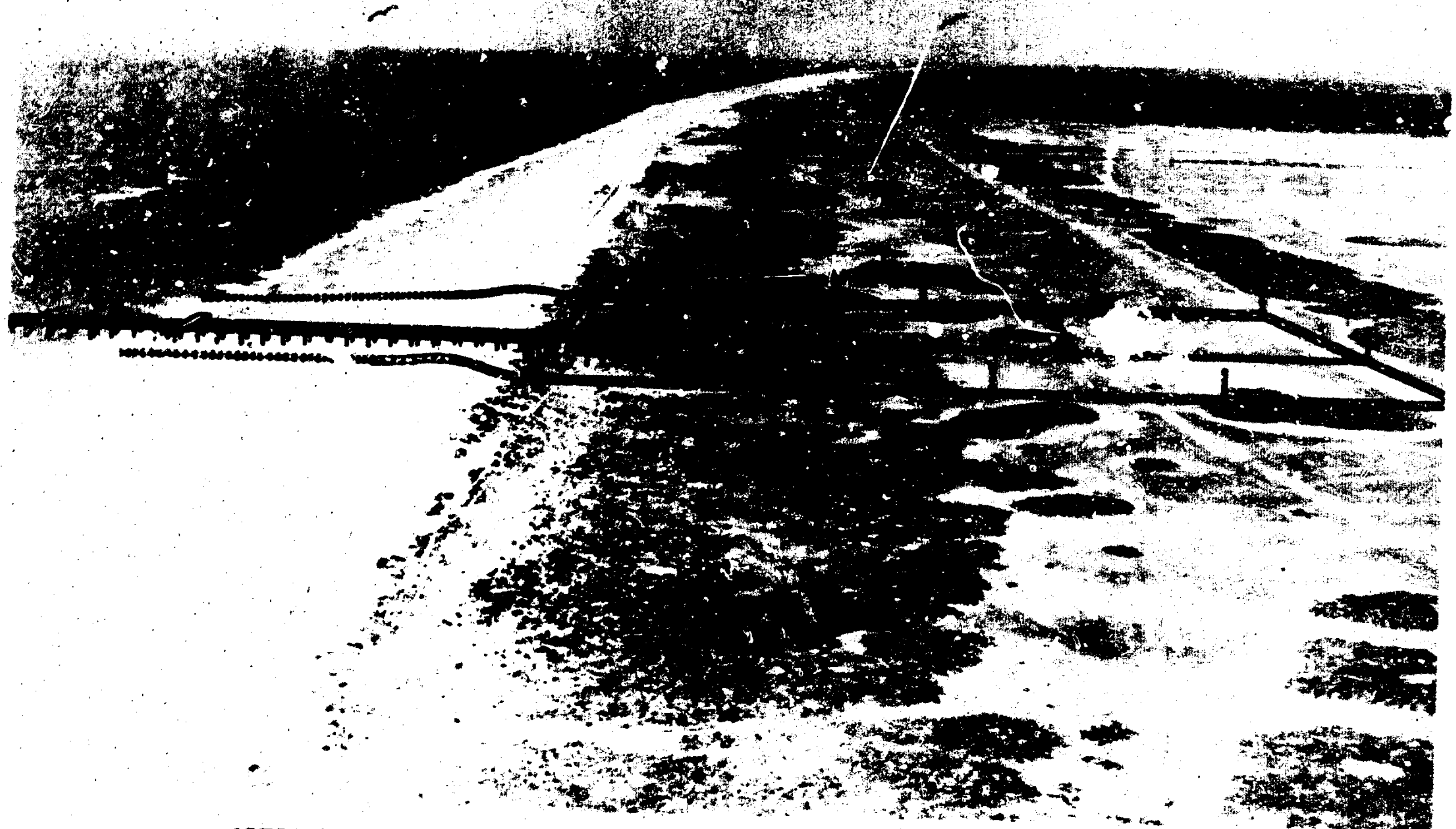
1. Laboratory Tests of Alongshore Transport. Laboratory testing in the Shore Processes Test Basin (SPTB), for determination of the relationship between incident wave characteristics and the rate of alongshore transport, was discontinued in October 1966. The main reason for discontinuing the SPTB tests was the difficulty experienced with wave-height variation both in time and location. The most recent laboratory summary report of these tests is for the period 1964-1966 and is available on loan from the CERC Library. Similar summaries of laboratory tests made before 1964 are available at CERC on a limited distribution basis. A data presentation report of these tests is being prepared and should be published in late 1970.

2. Longshore Current. The review of longshore current studies was published in Reviews of Geophysics, Volume 5, August 1967, and the compilation of longshore current data was published as CERC Miscellaneous Paper 2-67, *A Compilation of Longshore Current Data*, by C. J. Galvin, Jr. and R. A. Nelson, March 1967. Other results of longshore current measurements made by CERC at Virginia Beach, Virginia, in September 1966, were included in a paper, "Empirical Equation for Longshore Current Velocity", Journal of Geophysical Research, Vol. 73, November 1968.

3. Field Research Pier. Plans and specifications are under review for the CERC Research Pier to be built on Assateague Island. The Assateague Island location was chosen for several reasons: 1) a fairly straight, unbroken, open coastline extends some 15 miles both north and south of the pier site; 2) the absence of shoreline structures; and 3) it is a 4-hour drive from the CERC Laboratory. Field studies planned include: wave runup, wave setup, littoral current relationships between breaker height, depth, and travel distance, and also tests of the amount of sediment in suspension.

4. Measurement of Suspended Material. Analyses of suspended sand samples collected at Nags Head, North Carolina (1964) and Ventnor, New Jersey (1965) have been completed, and reports of the field sampling and analyses are under review. Study of the glass beads used as a sand simulant in the laboratory also included collection (by pumping) of suspended sediment samples. Preliminary analysis shows that sediment concentrations for both prototype (large wave tank tests) and the model tests (glass beads used as a sand simulant) lie within the range of 1 to 40 parts per thousand by weight. The prototype wave had a period of 11.33 seconds and was 5 to 5.5 feet high.

5. Sand Simulant Tests. Study continues in an attempt to define scale characteristics for beach sediment under wave action, using glass beads with a mean diameter of 115 microns as a sand simulant. A study was made to compare model beach profiles obtained using the sand simulant with those obtained using quartz sand with a mean diameter of 200 microns and



ARTIST'S CONCEPTION OF PROPOSED RESEARCH PIER ON ASSATEAGUE ISLAND waves up to 5.5 feet in height in the large wave tank. A 1 to 10 scale model beach was built in one of the small wave tanks. Different wave conditions were run, and periodic beach profiles made to determine the response of the beach to wave attack. Time lapse movies were taken to show the development of various beach features. Analysis is yet to be completed, but the simulant did not compare well with equilibrium beach profiles in the large wave tank.

6. Beach Evaluation Program. In a program of beach evaluation, repetitive profiles are taken at selected beach areas. Storm-wave action between surveys is analyzed, and correlations between the wave action and observed profile changes are established as a beach-vulnerability criteria. In September 1962, repetitive profile lines were established at nine locations between Delaware Bay and Cape Cod. Profiles at these locations were originally resurveyed at weekly and biweekly intervals. The current survey interval is about 6 weeks. Storm waves are measured by CERC ocean wave gages and storm surges by USC&GS tide gages at locations as near to wave gages as possible.

In 1968, repeat surveys were made at eight beaches from southern New Jersey to Rhode Island on the profiles previously established. A total of 751 profile surveys were made - an increase of 61 percent over the total in 1967. A report "CERC Beach Evaluation Program: 1962-1968, Part I: Background" was prepared by C. J. Galvin, Jr. to include general information on the study. From January through March 1968, weekly surveys and daily wave observations were made at four of the beaches for 10 consecutive weeks, and the results of this work formed the basis for a talk "Winter Profile Fluctuations on Four Atlantic Coast Beaches" given by C. J. Galvin, Jr. at the annual meeting of the Geological Society of America in Mexico City in November 1968. For this 10-week period, these data showed that four beaches over a 200-mile segment of the Atlantic coast had varied responses to wave conditions and that wave heights were slightly correlated to the degree of erosion on two of the beaches.

On each of five beaches (two on Long Island, three in southern New Jersey), rows of pipe were placed along two selected profiles, and the sand levels at the pipes were observed weekly. These pipe profiles may be an economical substitute for conventional surveys on some beaches. Observations of sand levels on the pipes show: 1) maximum vertical change was greater on Long Island than on southern New Jersey; 2) during the interval January through March 1968, the maximum weekly change at any one pipe was about 5 feet; 3) during the same three months, the maximum weekly change at any pipe usually exceeded the net three-month change; and 4) the maximum vertical change in sand level occurred at positions where initial elevations were relatively high. A laboratory report, *Pipe Profile Data and Wave Observations from the CERC Beach Evaluation Program - January-March 1968*, by H. D. Urban and C. J. Galvin, Jr. was published as CERC Miscellaneous Paper 3-69, September 1969.

Automation of the data continues; the digitizing, plotting, and computing of profile changes have been completed for the entire backlog of surveys. Optical-scanning forms for data acquisition are now in use to record survey data, wave observations, and sand elevations on pipe profiles. All Pipe-Profile and Wave-Observation Forms have been digitized, and initial analysis has begun. Maximum values of sand level change for 1-week intervals at the pipe profiles on the eight original beaches, during the third quarter of fiscal year 1969, was similar to the values obtained for the same period in fiscal year 1968. However, net 3-month changes showed significantly more erosion in fiscal year 1969 than in fiscal year 1968. Monthly averages of wave height for five New York and New Jersey beaches during fiscal year 1969 (based upon approximately 500 wave observation reports) were higher, on the average, than results obtained from 60,000 visual observations made by the Coast Guard from September 1954 through December 1965 at five stations in the same area.

7. Wave Height Variability in the Shore Processes Test Basin. For this experiment, two parallel wave flumes were built in the Shore Processes Test Basin (SPTB), and equipped with a special carriage for instruments and personnel. Each flume was 10 feet wide; one had a sandy beach with a 1 on 10 slope, the other a concrete "beach" with a 1 on 10 slope. Similar parallel wave flumes each 3 feet wide were constructed in the SPTB in 1967. Twelve tests were run in the 3-foot and 10-foot flumes in 1968 (compared with four tests in 1967) to obtain additional information on wave height variability and changes in reflection caused by deformation of sand beaches. Data reduction through 1968 is complete, and methods of analyzing reflection are being compared. During the 1969 test season, tests were made in the 3-foot flumes and the 96-foot indoor wave tank, comparing wave reflections from the stable concrete slope with those from the changing sand beach to determine how wave reflection was altered by the changing sand profile. Work has begun to determine how accurately the results from the half-scale Froude model in the 96-foot tank match the test data obtained from the 3-foot flume in the SPTB.

8. Experimental Study of Dune Building with Sand Fences and Vegetation. In the North Carolina dune-study area, periodic surveys were continued throughout the 3-year period, and wind records were obtained during most of the time. Some new fencing has been installed and new grasses planted; the older sections are being maintained and observed. CERC Technical Memorandum 22, *Dune Stabilization with Vegetation on the Outer Banks of North Carolina*, discusses types of grasses and fertilizers used. At the London Conference on Coastal Engineering, September 1968, a report was presented which summarized important results of the study and included information on work being carried out by North Carolina State University and the National Park Service on Ocracoke Island. Consultative advice was furnished the Galveston District, and plans were developed for an experimental program along the Texas Coast. A program was developed for installation of experimental grass plantings and fence sections in the Cape Cod area and forwarded to the New England Division for their comments.

Results from the North Carolina dune study have shown, at least in this area, that the way in which sand fences are used is important in the economy of barrier dune construction with sand fences. Two arrangements were tested and one of these, the installation of fences 1/3 of the way up the accumulation of the previous fence, proved to be superior to the other. In addition, American beachgrass was shown to be very effective in catching and holding wind-transported sand. A strip of American beachgrass 25 to 50 feet wide planted on an initially level beach will catch and hold all of the sand being moved by the wind in the area (from 3 to 6 cubic yards of sand per linear foot of beach each year). The cost of planting and fertilizing such a strip of beach grass is minimal (about 60¢ per linear foot of beach for planting and very little for fertilizing). The building of dunes with grasses has pronounced advantage over dune construction with fences in that grasses grow a "stabilized" dune, since the grasses grow upward as the wind-blown sand accumulates and keeps a protective vegetative cover on the dune as it increases in height. While sand fences are effective in creating barrier dunes, the dunes must later be planted with some type of vegetation to stabilize them against further movement by the wind.

Tests on the use of fabrics as sand fences have shown that they will trap and hold sand, and that their effectiveness increases as their porosity decreases. However, the most effective fabric, having a porosity of about 40 percent, was not quite as effective as slat-type snow fencing with a porosity of 50 percent.

B. Contract Studies

1. Study of Beach Processes in the Inshore and Foreshore Zones - Massachusetts Institute of Technology, Contract DACW72-68-C-0012. "A Drag Gage for Measuring Particle Orbital Velocity in Water Waves", a laboratory report by C. H. Horstman and P. S. Eagleson, was received. A series of tests were carried out to reduce errors and compare relative performance of both the hot-film and drag-type velocity gages. Laboratory measurements were made of the change in wave shape, particle velocities, and forces. Theoretical work on the shoaling wave also continued, and a report, "On the Propagation of Periodic Water Waves over Beaches of Small Slopes", was prepared by C. C. Mei with contributions by G. A. Tlapa, P. S. Eagleson and M. M. Kolpak. An attempt is being made to connect a linearized deepwater theory and the nonlinear shallow-water theory to give a correct theoretical description of the wave motion as the wave propagates from deep water up to the breaking point. Higher order approximations are being included so that the effect of finite amplitude in the shoaling zone may be taken into account.

One set of velocity measurements was made in which the speed of the orbital currents was measured at depth increments from the stillwater level to about 1/2 inch above the tank bottom. These measurements showed a depth-dependent phase difference in the velocity minimum and maximum. A report based on earlier work on the contract for studying wave shoaling by second-order theory, "An Asymptotic Theory of Water Waves on Beaches of Mild Slope", by G. A. Tlapa, C. C. Mei and P. S. Eagleson, was published in the Journal of Geophysical Research, July 1968. Experimental work continued through measurement of water particle velocities using a hot-film probe and a vane-type direction sensor in conjunction with a set of wave gages. Photographic measurements of flow velocities were also taken for comparison, using a 16 mm movie camera and neutrally buoyant oil droplets in the water. A technical report, describing the instruments, their calibration, use, and data illustrating their accuracy, is in press. A summary report of the theoretical results is being prepared.

2. Nearshore Sediment Movement - University of California - Contract DACW72-67-C-0015. The segmental oscillating flume constructed to conduct suspended sediment studies under this contract has been modified by shaping the flume bottom into a circular arc. Two-dimensional roughness elements of particular shape have been cast from plastic material and will be used in future tests. Work in this oscillating flume involved considerable effort to perfect a hot-wire velocity probe, an optical sediment-concentration meter, and a magnetic tape data acquisition system. Some tests were made in the swinging flume using black plastic particles (density = 1.19) to achieve greater accuracy in the results and the amount of scatter present. In these tests, where concentration values were about 15 grams per liter, the amount of light being transmitted was reduced to such a low level that the present equipment could not sense or record it. A report is in progress concerning recent measurements of suspended sediment using the optical concentration meter for the black plastic particles.

Data obtained for the measurement of turbulence in an oscillatory flow by means of a single hot-film probe have been reduced and a draft report is nearly complete. In addition, a report on "Recent Sediments on Monterey Bay, California by T. E. Yancey, was received.

3. Mechanics of Sediment Transport by Waves and Currents - University of California - Scripps, Contract DA-49-055-CIVENG-66-1. This particular study of the relation between sand transport and wave action is well advanced. The transport relation appears to be in good agreement with previous work, indicating a proportionality of transport rate to the longshore wave energy flux and also with the Inman-Bagnold model. The model gives a transport rate that is proportional to the product of wave stress and the longshore current. The contractor is using the root-mean-squared wave height to compute the wave energy rather than the significant wave height which has been used to compute wave energy with much of the previous data. The energy of several wave spectra has been determined and compared with the energy obtained by using the significant wave height. A comparison of the two concepts for widely varying wave conditions indicates that when all, or nearly all, of the wave energy is contained in a narrow frequency band, both concepts give essentially the same result. However, when the energy is distributed over a wide frequency range, the energies calculated by the two concepts diverge considerably. The study has included field measurements of: (1) nearshore circulation and mixing; (2) wave setup and runup; (3) longshore transport of sand; and (4) comparison of various methods of estimating the energy of real waves. Progress in

these areas was reported in the following three papers presented at the American Geophysical Union (AGU) meeting in Washington, D. C., April 1969; (1) "Dispersion of Water in and Near the Surf Zone" by D. L. Inman, R. J. Tait, and C. E. Nordstrom; (2) "Low Frequency Resonance in the Surf Zone" by R. J. Tait and D. L. Inman; and (3) "The Longshore Transport of Sand on Beaches" by P. D. Komar and D. L. Inman. Reports are in preparation on the subject research of the first and second AGU presentations noted above, and a doctorate dissertation, "The Alongshore Transport of Sand on Beaches" by P. D. Komar has been completed.

4. Development of an Optical Sensor for Measurement of Mean and Statistical Properties of Suspended Sediment Concentration, University of Iowa, Contract DACW72-68-C-0009. The following synopsis of a paper, "An Electro-Optical Probe for Measurement of Suspended Sediment Concentration", presented at the 1969 International Association for Hydraulic Research, describes the progress:

> "A new concept and instrument are introduced for measurement of suspended sediment concentration *in situ* in rivers, estuaries, and shoaling waves. The transducer for the probe consists of a gallium arsenide diode as a light source and a silicon planar diode as a light sensor. The light from the source detected by the sensor is modulated by the suspended-sediment concentration in the gap between the source and sensor. The probe circuitry and appurtenant instruments are described in some detail. The frequency response of the instrument is adequate to permit measurement of turbulent fluctuations and/or periodic variations of sediment concentration. The calibration and response characteristics of the instrument are described, and some data obtain in steady uniform flow and in breaking waves are summarized."

5. Movable Bed Scale Model Technology, TETRA TECH, Inc., Contract DACW72-68-C-0020 The following tasks have been carried out under this contract:

a. A review of the history and development of movable bed scale model technology.

b. A critical analysis of the uses, limitations and basic requirements of a scale model study.

c. An extensive survey of the literature on coastal and estuarine sedimentation processes, including artificial sediments. Results of the survey have been summarized, with particular emphasis on limits and deficiences of the present status of knowledge, and uncertainties involved in various proposed formulae.

d. Establishment of similitude relationships based upon important governing parameters. Relationships proposed by previous investigators have been critically reviewed. Four basic unknowns - horizontal scale, vertical scale (or distortion), sediment size, and specific weight - can be determined from established similitude relations.

e. Establishment of a standard procedure which includes data acquisition, data correlation, choice of scales, artificial sediments, preliminary wave-tank tests, adjustment of the model, determination of time scale of bed evolution and interpretation of the results, including scale effects.

The foregoing results will be useful in evaluating and using movable bed models as a tool in solving coastal engineering problems. A final report was submitted and is being considered for publication.

6. An Analytical and Experimental Study of Bed Ripples Under Water Waves - Georgia Institute of Technology, Contract DA-49-055-CIVENG-65-1. Work was done in the U-tube test facility on the evolution of a ripple bed from an initially flat bed under oscillatory flow, on the criteria for the initiation of sediment movement under oscillatory flow, and on the energy dissipated over a flat bed compared with that dissipated over a rippled bed. Theoretical models were developed for use in analyzing the data taken in the oscillating flow tests.

It was found that a critical sediment number which defines the point at which a particular sediment will begin to move under the influence of fluid flow is a function of the wave period on unrippled sand beds, but is independent of wave period on sediment beds already containing sand ripples Observations of the manner in which an initially flat bed evolves into a rippled bed show that a ripple system forms spontaneously over a flat bed if the maximum velocity is greater than the critical velocity. A ripple system can be induced to form with lesser velocities by a disturbance which creates a non-uniform flow just above the bed. The geometry of equilibrium ripples was also investigated. The ratio of ripple amplitude to mean particle diameter and the ratio of ripple amplitude to ripple wave length were found to be unique functions of a single variable - ratio of water motion amplitude to mean particle diameter. Determination of the added energy dissipation with oscillatory flow over a rippled bed, as compared with oscillatory flow over a smooth flat bed, produced drag coefficients which appeared to be abnormally large in comparison with drag coefficients determined in other flow situations. The apparent discrepancy was traced to the existence of an essentially uniform velocity distribution above the bed in oscillatory flow, in contrast to the non-uniform velocity distribution above the bed in unidirectional flow. The conclusion reached is that a velocity in the vicinity of the bed deformation is a more rational reference than a velocity far from the bed such as the mean velocity in unidirectional flow.

7. Localized Scour Around Piling Subjected to First Order Stokian Water Waves, Georgia Institute of Technology, Contract DACW72-67-C-0017. A final report on this work was received with the following conclusions:

A method was proposed for computing terminal depth of local scour and determining time variation of scour depth around a vertical circular pile structure in a sand bed under the action of first-order Stokian water waves. The conclusions, limited to a homogeneous level bed consisting of sand of fairly uniform size, are:

a. With sediment transport into the scour hole from surrounding mobile flat bed, the terminal depth of scour is independent of magnitudes of approach velocity and sediment size. Terminal scour depth is 2.4 times the diameter of the cylinder when the ratio of the diameter of pile structure to the diameter of sand is reasonably great.

b. Dunes influence localized scour around a vertical cylinder. The terminal scour depth decreases with the decreasing values of a/D_g in the region of three-dimensional dunes. When the dune system is two-dimentional, the scour hole will join the dune system with scour hole acting as part of the dune system.

c. No terminal scour depth is predicted in absence of the sediment transport into the scour hole. For an immobile flat bed around the pile, the similarity relationship should be used within the range of the data (S/D less than unity).

d. Localized scour around a pile is not expected to be critical in the design of pilings of relatively small diameter. The generalized scour, or lowering of the bed load transport due to increased velocities during the storms, would be greater than the localized scour due to the pile structure alone. In the design of hydraulic structures as wide as cofferdams, knowledge is essential concerning the maximum depth of the localized scour and how this depth is related to different flow conditions.

8. The Use of Grasses for Dune Growth and Stabilization Along the Gulf Coast with Initial Emphasis on the Texas Coast, Gulf Universities Research Corporation (GURC), Contract DACW72-69-C-0012. Work under this contract included selection and preparation of a nursery site on Padre Island, Texas. A general ecology study and reconnaissance of Padre Island was made to find the potentially most useful plant species for the study objectives. Limited amounts of several seed types including sea oats and *spartina patens* were collected for trial propagation. A conference was held between representatives of GURC, the Galveston District, the Padre Island National Seashore, and CERC, January and July 1969, to coordinate and plan the future conduct of the study. Similar conferences are now planned at 6-month intervals. Transplanting efforts were emphasized on South Padre Island, but were hampered by the storm surge of 14 February 1969. On North Padre Island, two areas (about 27,000 plants) of *Uniola paniculata* were transplanted to the nursery. Other work on North Padre Island included two experimental plantings on the beach and the collection of soil samples from the nursery and beach areas for particle size and nutrient analysis.

9. Dune Stabilization, Outer Banks, North Carolina, North Carolina State University, Contract DACW72-69-C-0021. Progress during the initial report period (December 16, 1968 to March 31, 1969) included installation of 2,800 feet of single sand fencing on all the "fence only" sections of both dune experiments on Ocracoke Island. Double lines of sand fences were installed across all storm-induced breaks in the sections (about 3,000 feet). Much of this had to be reinstalled due to unusually high water during the winter. Additional replantings were made on March 24, 25, and 26, 1969 and included approximately 6,500 hills of American beachgrass, 4,000 hills of sea oats and 7,000 hills of *Spartina patens*.

10. Basic Research to Analyze Time Sequence Changes in Beach Configuration in Response to Wind and Wave Conditions in Coastal Regions, Florida Ocean Sciences Institute, Contract DACW72-69-C-0018. Three Atlantic coast beaches - Hollywood, Jupiter, and Boca Raton - were the initial work sites. The second report from the contractor describes pipe emplacements along profile lines as being complete at Hollywood and Boca Raton and almost complete at Jupiter. A profile change with time graph and monthly summaries of the littoral environmental observations (wind, wave, littoral drift) were included in the report. A summation of data reports had no apparent correlation between the three sites nor any predictable change with time at any one site.

III. LONG PERIOD WAVES AND SURGES

A. Studies at CERC

Activities at CERC consisted of supervision of the contract work, and furnishing advice and aid to Districts of the Corps of Engineers in the analysis of harbor surging problems.

B. Contract Studies

1. Engineering Damage from the Tsunami of the Alaskan Earthquake of 1964, Science Engineering Associates, Contract DA-49-055-CIVENG-66-6. Work was completed and resulted in a report published as CERC Technical Memorandum No. 25, *The Tsunami of the Alaskan Earthquake, 1964: Engineering Evaluation.* Parts of the contractor's report will also be used by the National Academy of Sciences in its comprehensive report on the Alaskan tsunami. The contractor studied the generation, propagation, runup, the damage caused by the tsunamis, and the effects in many areas. The inferred heights and periods tend to confirm empirical-statistical results permitting approximate prediction in terms of given earthquake magnitude. The report also presents a theory of greater probability of earthquakes during syzygy. The tsunami appeared to propagate as a system of modulated waves, and, during propagation over the Continental Shelf, there was an apparent transfer of energy to higher harmonics. Particular coastlines, inlets, and bays are resonators for certain harmonics and amplify their effects (a possible explanation of high susceptibility to damage of Crescent City, California). Intensive study of the type of damage was made and reported on, and recommendations for consideration in determining design criteria for tsunami protection were presented.

2. Tsunamis, University of California Contract DACW72-67-C-0002. Study continued on the effect of the velocity of horizontal motion in a slip fault on tsunami generation, primarily through laboratory studies on the waves generated by a slip fault through a submerged scarp. The study, theoretical and experimental, is important to such areas as San Francisco Bay where faults run through the north end of the bay. The speed at which the slip occurs is of primary importance. A computer program has been written for the problem of waves generated by an initial displacement of the water surface, and theoretical results were compared with earlier laboratory studies. In addition, theoretical studies of the analysis of nonstationary time series were completed. Emphasis was on the analysis of tsunami records. The studies resulted in a report, "Methods for the Analysis of Non-Stationary Time Series with Application to Oceanography", by Lloyd J. Brown (1967). Another report "Non-Linear Wave Effects on Tide Gages" by Ralph H. Cross (1967), was also received.

In 1969, considerable headway was made under this contract toward solving the problem of the prediction of wave-time histories from a model "rockfall into a reservoir". The model was a box dropped vertically down the end wall of a wave flume onto the stillwater surface. A surface-amplitude equation for the idealized linear problem of the falling box is developed with basic assumption that the flow of fluid at x=0 caused by the falling box is horizontal and uniform (i.e., independent of the vertical) and thus only a function of time. Analytical solutions of the surface amplitude from falling box experiments, compare favorably. Actually, the flow pattern underneath the box or underneath a rockfall would be quite complex, so that in reality the assumption was an oversimplification, yet it checked favorably with experimental results.

3. Modification of Two-dimensional Long Waves over Variable Bottom Topography - Texas A&M University, Contract DACW72-67-C-0003. Numerical procedures for evaluation of amplitude phase of long waves around an island have been applied to the condition of the Wake Island model previously investigated in the laboratory by Van Dorn. Taking into account the complex nature of the tsunami-island interaction problem (particularly with the longer periods), the numerical evaluation was considered satisfactory.

a. Tests have been carried out using an incident wave of Gaussian form (nearly impulsive input) for the case of an island for which analytical results are available. The spectrum of tsunamis, with particular attention to the leading waves, is also being studied. Because of the influence of the earth's rotation, there is a theoretical basis for a maximum period in the vicinity of the leading waves of a tsunami. Preliminary analysis indicates some evidence for this in the observed data. This is presently being investigated quantitatively through spectral analysis and filtering techniques as applied to actual tsunami records at islands in the Pacific. A numerical study of the natural modes of oscillation within a rotating circular basin with axially symmetric topography was also completed.

b. Previous work on the normal modes for long waves in a rotating circular basin is being prepared for publication. The results of this study can be summarized in a relatively simple dimensionless form consistent with studies by Lamb and Stokes. Using a new method somewhat similar to the stream-function theory proposed by R. Dean, an evaluation has been made of the characteristics of gravity waves of permanent form.

c. This investigation has been extended to include a greater range of relative depths and a greater resolution of relative depth and relative wave heights. Computations pertinent to this problem were made at the Navy Postgraduate School, Monterey, California. The results indicate that a practical lower limit of relative depth exists (for given wave steepness) for which the method of computation is convergent.

d. Further work was done on the revision of the present computing scheme for the tsunami-scattering problem to reduce distortion of the shorter waves, particularly in the far field regions of the model. Interest has developed in the excitation T-phase (compressional waves in water) associated with the earth shocks, which also cause tsunamis. Measurements of the T-phase, being carried out at the University of Hawaii, may possibly give definitive information about the subsequent tsunami signature. The T-phase is a dispersive wave, but - unlike gravitational waves - the higher frequencies have a greater group velocity which approaches the speed of sound in water. The spectrum of T-phase received at different points could give information concerning the duration of the initiating shocks as well as their magnitude and location.

e. Analysis of the characteristics of the wave signature near the wave front is continuing. Analysis by narrow band-pass filtering for the Wake Island record of 1957 reveals that the part of the spectrum near 50 minutes is strongest near the wave front, although the dominant or apparent period near the front is close to 10 minutes. There is some question whether the long period part was directly related to the tsunami or whether it is part of the ocean background.

4. A Digital Representation of Electric Network Long-Wave Analogs, Westinghouse Electric Corporation, Contract DACW72-68-0013. Major effort under the contract has been devoted to: 1) completing the programmer's problem-definition write-ups, 2) continuing with the error analysis, and 3) working out details of a general method for programming the network equations for long-wave propagation in a body of water of arbitrary configuration.

5. Analysis of Tide Gage Records to Determine Tsunami Signatures, Contract DACW-72-69-C-0017 with Dr. Basil W. Wilson. This work is on a more objective analysis of tide-gage records to determine tsunami signatures, to identify the effects of basin and shelf resonance, and to define a basis for separating the two effects.

IV. SHORE PROTECTION IN BAYS, LAKES AND RESERVOIRS

A. Studies at CERC

1. Reservoir Slope Protection. The cost of slope protection (such as riprap) to protect the upstream embankment slope against the wave action of a reservoir pool accounts for a high percentage of the total cost of the embankment. CERC is testing methods for better design of riprap to reduce initial and maintenance costs. Slope-protection tests using different riprap designs of representative rock types and gradations are now in progress at CERC in the Tank for Large Waves. Riprap designs are placed against various model slopes in the wave tank, and subjected to bursts of waves simulating the actual wind-induced waves in reservoirs. Two tests are complete; one is in progress to extend the work on riprap stability to waves of longer period. The completed tests had wave periods of 8.5 and 11.5 seconds, and each had an armor-layer thickness of 1.5 feet. Another separate test which followed had a wave period of 8.5 seconds and an armor-layer thickness of only 1.0 foot.

2. Surveys in Chesapeake Bay. A study continues at a location in Chesapeake Bay to relate erosion rates of different types of shore characteristics (bluff, beach, and dune) to incident wave and water level conditions. Periodic surveys are obtained. It is hoped to extend the study to other areas with different soil types.

B. Contract Studies

No contract studies were made during the report period.

V. BIOLOGICAL-ECOLOGICAL STUDIES

A. Studies at CERC

CERC has initiated an ecological research program. To date, planning has concentrated on the biological-ecological effects of Engineering activities, particularly in the use of offshore sources of sand for beach nourishment, and the creation and maintenance of navigation channels by dredging bottom materials and redeposition of the resultant "spoil". The research projects developed from this planning effort will incorporate a positive approach to the modification of Corps of Engineers activities to achieve "beneficial" ecological changes.

For instance, research will be directed toward the ecological significance of bottom topography and bottom type with the thought of conducting offshore sand mining in a manner to uncover or create "reefs" or other types of bottom conditions favorable to the growth and concentration of fish and shellfish population. Research will be conducted on the ecological characteristics of salt marshes and other types of estuarine habitats in an effort to "create" favorable habitats for fish and shellfish by modification of dredging and spoiling techniques, and by encouragement of certain specialized types of marine and estuarine plant life.

Other approaches to coastal biological-ecological problems are being investigated.

B. Contract Research

No contract research was made during the report period.

C. Research Funded by Others

CERC is supervising a study of the effects on water characteristics and marine organisms by waste materials being dumped in the ocean off New York Harbor. Funding for this study was furnished by the Office, Chief of Engineers.

VI. ENVIRONMENTAL DATA COLLECTION

A. Studies at CERC

1. Sand Inventory. Field work on sand inventory surveys to locate offshore deposits usable in shore restoration and maintenance projects was completed in the following areas:

(a) All of New England south of Portland, Maine, and including the north shore of Long Island, New York (completed January 1968).

(b) South Shore of Long Island (completed September 1968).

(c) Chesapeake Bay entrance and the Virginia coast from Cape Henry to Sand Bridge (completed August 1968).

CERC Technical Memorandum No. 29, *Geomorphology and Sediments of the Nearshore Continental Shelf, Miami to Palm Beach, Florida,* was published in 1969. Sand inventory reports for east central Florida, Chesapeake Bay Entrance, and New Jersey are in preparation. Data analysis and reduction for sand inventory surveys of north Florida, New England, and Long Island are partly complete.

Preliminary "quick-look" analysis reports for six areas covered by the New England survey have been furnished the New England Division, Corps of Engineers, for use in operational planning. After CERC completed the sand inventory studies, procedures were worked out to transfer sand-inventory cores and geophysical records to Federal repositories where they will be made available to researchers for further study.

Five technical papers resulting from CERC sand inventory studies were authored or co-authored by staff members during the report period.

2. Radioisotopic Sand Tracer (RIST) Study. Using radionuclide tagged native sand as a tracer, the initial phases of a detailed study of sand movement in the littoral zone were completed during the report period. Problems relating to tagging, instrumentation and control were resolved by systems tests and the first full-scale study of areas near Point Conception, California, has been finished. This phase of the study is described in a preliminary report covering RIST activities from July 1966 to June 1968; it was published as CERC Miscellaneous Paper 2-69. One technical paper, Field Tracing Xenon-Irradiated Sand in the Nearshore Environment, was presented at the Geological Society of America meeting at Mexico City, Mexico, in November 1968, by David B. Duane. Additional field projects are scheduled in which the RIST system will enable alongshore transport application to problems in sand movement.

3. Review of Methods of Wave Analysis. The meaning and usefulness of instrumental wave records have been studied critically during the past 3 years. The standard form of wave record analysis in coastal engineering is to assign a "significant wave height and period" to each record.

Two nominal definitions of the significant wave height are widely used. According to the earliest usage, "significant wave height" is defined as the average height of the one-third highest waves. Some later researchers defined the significant wave height as four times the root mean square departure of the water surface from its mean position, that is to say, the significant wave height is taken as four times the standard deviation of the wave record.

Several definitions have been employed for the "significant period". The earliest applied to wave records identifies the "significant wave period" as the average period of the one-third highest waves. Other definitions: 1) the average period of the more prominent waves in the record; 2) the average period at which the water surface rises from below to above the mean water level. Another possibility is the average period between wave crests omitting reference to the height of individual waves.

None of the calculations implied by the above definitions can be carried out quickly or accurately from the usual form of pen-and-ink records, and so simpler approximations are employed in practical work. Since 1963, it has been customary at CERC to select the most prominent period in the record as the significant period, and to identify the height of the Nth largest wave in the record as the significant wave height, where N is an empirically determined function of the period, and is approximately equal to the analyses interval, divided by six times the selected wave period.

For visual observations of waves, the most common definition of the significant period is the average period of 10 successive waves. Sometimes a number other than 10 is used. When wave spectra are computed, the most common choice is the period corresponding to the peak on the spectrum graph. Several forms of displaying the wave spectra are widely used, and the period corresponding to the peak of the curve depends on the form of display used.

Wave records, obtained about 1/2 mile from shore at the end of the Atlantic City Steel Pier, have been used to test the reproducibility and the equivalence of several of these definitions of significant wave height and period. One test was based on two independent analyses of two different instrumental records from the step-resistance relay wave gage, for the period 1 August 1966 to 31 July 1967. The working definition of significant height and period was the same for both analyses. The two analyses for significant height agreed within 1.0 foot 63 percent of the time; within 2.0 feet 92 percent, and disagreed by 4.0 feet, or more, only 1 percent of the time. When two records of gages of different design located about 2 feet apart were compared for the period 1 July 1967 to 31 December 1967, the percentage of agreement within 1.0 foot dropped to 54. The other figures were unchanged.

When the periods were compared, two analyses of the records from the step-resistance gage agreed within 1 second 35 percent of the time, within

4 seconds 92 percent of the time, and disagreed by 7 seconds, or more, only 1 percent of the time. When two gages were considered, the figures were in agreement within 1 second 31 percent of the time, 6 seconds 92 percent of the time, and within 12 seconds 99 percent of the time. It should be noted that 77 percent of all waves recorded from this location for the 5-year period beginning in 1963 were between 4 and 11 seconds, and 60 percent of all periods were between 5 and 10 seconds.

The lowest correlation between height definitions, obtained from applying this program to the wave records for December 1966, was 0.94. The root-mean-square value appeared to be the best standard. The correlation coefficients between period definitions varied from -0.03 to +0.95, with a median value below 0.50.

The difficulty in determining a characteristic period for a wave observation, results from the fact that a natural sea always contains waves of many periods.

It is often assumed that the actual surface of the sea may be considered as the sum of a large number of simple waves. The relation between wave amplitude and period is usually given in the form of an energy spectrum. It is also often assumed that this spectrum has a universal form whenever the waves and winds are in equilibrium, and there is much empirical data to support this assumption. If the spectrum were truly universal all of the time, a simple relation would exist between all rational definitions of the significant wave period or wave height.

This equilibrium state, however, is rare in the spectra CERC has examined. Most of the coastal records appear to show the effects of swell from one or more distant storms as well as from locally generated waves. Thus, waves having two or more distinct periods often appear to be about equally prominent in the records. When this happens, the human analyst may choose any of the prominent periods, or he may take the average of them all. The records examined to date show a bias in favor of the shorter periods. This is the best choice for only a few engineering problems. The average of all prominent periods is generally the poorest choice for future application.

A consideration of the ways in which period data are used in engineering shows that the optimum definition depends on the use which is to be made of the data.

A shift in emphasis from a simply defined "significant period" to spectrum considerations is clearly in order. Such a shift is now taking place in most coastal engineering laboratories. Nevertheless, the records of significant wave and period collected by this laboratory over the last 15 years do have some practical value. It is planned to publish most of these data in 1970.

4. Wave Sensors and Wave Data Collection. The program for collecting wave data using recording type wave sensors was expanded during fiscal years 1967-68-69. All east coast wave recording stations were adapted to telemeter the wave data from each station to the CERC laboratory on a full-time basis by leased telephone lines. Data from each station is recorded on pen-and-ink strip-chart recorders and on digital magnetic tape. Wave gage development during this period included microwave, infrared, and laser-type sensors. The laser-type wave sensor shows good promise for wave height measurement; one such gage is under test at Atlantic City, New Jersey.

During this period several schemes for obtaining wave direction measurement were considered. These devices included a flowmeter, Rayleigh disc, Banwell force gage, and several surface gages used in a spaced array. Further research on wave direction measurement techniques is in progress.

A test for determining the pressure-depth wave-height correlation was started at Atlantic City, New Jersey. This study uses three wave gages mounted in a vertical column. The waves recorded by two pressure sensors at different depths are being compared to waves on a Baylor surface wave staff. This test is continuing.

5. U. S. Coast Guard-CERC Cooperative Surf Observation Program. This program continues with visual estimates of surf contributed from 14 active Coast Guard Stations. In fiscal years 1967-68-69, three stations (Hampton Beach, New Hampshire, Virginia Beach, Virginia, and Oak Island, North Carolina) have discontinued observations, and two stations (Piedras Blancas, California, and Yakutat, Alaska) have begun observations. Analysis of the data collected from 1954 through 1965 has resulted in two laboratory reports describing general features of the data and tabulating yearly and monthly averages of wave height and period. The data show marked differences in wave characteristics between the east and west coasts of the United States, and usually good agreement between data from adjacent stations with similar exposure on the same coast. Wave heights and periods average about 1.5 feet and 6 seconds on the east coast, and 3 feet and 12 seconds on the west coast. Wave period shows little change from month to month, but wave height exhibits noticeable seasonal change. Analysis of data is continuing.

B. Contract Studies

During the report period, contracts for the study of coastal geomorphology, sedimentation, and littoral processes were in effect with the University of Massachusetts, North Carolina State University, University of Georgia Marine Institute, Old Dominion University Research Foundation, the Virginia Institute of Marine Sciences, and University of California at Los Angeles. Preliminary results were received in quarterly progress reports.

Non-funded studies of micro-organisms in the bottom sediment cores, obtained through the sand inventory program, were made at the University of Pittsburgh, University of Virginia and Amherst College. Significant data bearing on sedimentation rates in Chesapeake Bay Entrance, Block Island Sound, and eastern Cape Cod Bay may result from these studies. A brief summary of each contract follows:

1. Environmental Aspects of Beach Processes, Sediment and Erosion in the Coastal Region of North Carolina, North Carolina State University, Contract DACW-72-68-C-0017 This is a study of beach processes and sedimentation in response to ambient environmental conditions at Pea Island on the Outer Banks of North Carolina. Beach samples, profiles and environmental data have been collected for 2 years, and will be collected in 1970. Preliminary analysis of collected data shows that many short-period changes occur in the beach morphology in response to transient environmental influences.

2. Investigation of the Potential Source, Transport and Depositional Patterns of Clastic Sediments in a Portion of the Georgia Coast - University of Georgia Marine Institute Contract DACW-72-68-C-0030. Under this contract, investigations of the potential sources, transport and depositional processes of coastal and shelf sediments have been in process for more than a year. A large volume of processed data was collected and compiled into an initial report. The data includes beach profiles of Sapelo Island, Georgia, sediment analysis of samples taken along the beach profiles, including carbonate and heavy-mineral data, and meteorogical data.

3. Genesis of the Nearshore Modern Sand Prism on a Barrier-Island-Spit-Headland Coast - Institute of Oceanography, Old Dominion College Contract DACW-72-69-C-0016. This contract covers a study designed to distinguish between nearshore "modern" sands and offshore relict sediments. Detailed surveys of the prime study area off False Cape, Virginia, and a reconnaissance survey between Cape Henry and Cape Hatteras have been completed. Ten diving stations have been established, and marked for relocation purposes. These will be revisited on a periodic basis to observe and sample offshore bottom conditions.

4. Storm Wave and Storm Surge Modification of Virginia's Ocean Coast - Virginia Institute of Marine Science Contract DACW-72-69-C-0031. This contract provides for the systematic collection of beach profile data on a repetitive basis before and after winter storms and hurricanes. This data will be related to storm surge data for the same time to provide techniques of predicting expected wave and storm surge modifications of the Virginia Coast.

5. Modal Analysis Research of Coastal Sediments - University of California at Los Angeles, Contract DACW-72-69-C-0025. Research under this contract involves the application to coastal sedimentation problems of a new technique of statistical analysis of sediment characteristics. Using the twofold approach of modal analysis of the total sample population under study, and of the frequency distribution of its component mineralogic population, an attempt will be made to relate these parameters to sediment origin, movement and modifications resulting from addition to and losses of the original material during transportation.

6. <u>Laser Wave Gage Feasibility Study, Electro-Optical Systems, Contract DACW-72-67-C-0014</u>. The laser wave gage uses a ruby red continuous-wave laser light source and associated optics that produce a spot of light about 6 inches in diameter on the water surface from a distance of about 20 feet above the water. The spot of light on the water surface is reflected back towards the light source and through another set of optics which is sensed by a photocell. The laser beam is amplitude-modulated by an optical type modulator. A portion of the transmitted laser signal is fed directly to the receiving photocell which is applied to one input of a phase comparator; the reflected signal from the water is applied to the other input of the phase comparator. Output of the phase comparator will then be a measure of the time between the transmitted laser signal and the reflected laser signal. The signal from the phase comparator is processed by suitable electronics to provide a change of 0 to 5 volts d.c. representing a dynamic change in elevation of 0 to 25 feet of the spot of light on the water surface. The gage was field tested at Atlantic City, New Jersey.

VII. PUBLICATIONS

A. CERC Publications, Fiscal Years 1967-68-69

A list of Technical Memoranda, Miscellaneous Papers, and Reprints follows:

Technical Memoranda

18 - Correlation of Littoral Transport with Wave Energy Along Shores of York and New Jersey, November 1966, by John C. Fairchild.

19 - Budget of Littoral Sands in the Vicinity of Point Arguello, California, December 1966, by A. J. Bowen and D. L. Inman.

20 - Behavior of Beach Fill and Borrow Area at Sherwood Island State Park, Westport, Connecticut, May 1967, by W. H. Vesper.

21 - A Multi-Purpose Data Acquisition System for Instrumentation of the Nearshore Environment, August 1967, by W. A. Koontz and D. L. Inman.

22 - Dune Stabilization with Vegetation on the Outer Banks of North Carolina, August 1967, by W. W. Woodhouse, Jr. and R. E. Hanes.

23 - A Model Study of the Entrance Channel, Depoe Bay, Oregon, September 1967, by John P. Ahrens.

24 - Tables of the Statistical Distribution of Ocean Wave Forces and Methods of Estimating Drag and Mass Coefficients, October 1967, by L. E. Borgman and L. J. Brown.

25 - The Tsunami of the Alaskan Earthquake, 1964: Engineering Evaluation, May 1968, by Basil W. Wilson and Alf Tørum.

26 - Hurricane Surge Frequency Estimated for the Gulf Coast of Texas, February 1969, by B. R. Bodine.

27 - Corrosion and Protection of Steel Piling in Seawater, May 1969, by Laverne L. Watkins.

28 - Bed Forms Generated in the Laboratory Under an Oscillatory Flow: Analytical and Experimental Study, June 1969, by M. R. Carstens, F. M. Neilson, and H. D. Altinbilek.

Miscellaneous Papers

1-67 - The Wave Record Program at CERC, January 1967, by J. M. Darling and D. G. Durm.

2-67 - A Compilation of Longshore Current Data, March 1967, by C. J. Galvin, Jr. and R. A. Nelson.

3-67 - A Feasibility Study of a Wave-Powered Device for Moving Sand, June 1967, by Frederick F. Monroe.

1-68 - Annotated Bibliography of BEB and CERC Publications, July 1968, compiled by R. H. Allen and E. L. Spooner.

1-69 - Oolitic Aragonite and Quartz Sand; Laboratory Comparison Under Wave Action, April 1969, by Frederick F. Monroe.

2-69 - Radioisotopic Sand Tracer Study, Point Conception, California, May 1969, by D. B. Duane and C. W. Judge.

Reprints

R. 1-67 Coastal Processes and Beach Erosion, January 1967, by J. M. Caldwell.
R. 2-67 Wave Tests of Revetment Using Machine-Produced Interlocking Blocks, August 1967, by Jay V. Hall, Jr.
R. 3-67 Rock Movement in Large Scale Tests of Riprap Stability under Wave Action, August 1967, by Thorndike Saville, Jr.
R. 4-67 Variations in Groin Design, September 1967, by Dennis W. Berg and George M. Watts.
R. 1-68 Surf Observations Along the United States Coasts, February 1968, by J. M. Darling.
R. 2-68 Longshore Current Velocity; A Review of Theory and Data, August 1967, by Cyril J. Galvin, Jr.
R. 3-68 Breaker Type Classification on Three Laboratory Beaches, June 1968, by Cyril J. Galvin, Jr.

B. Technical Report No. 4

Technical Report No. 4, Shore Protection, Planning and Design, is a comprehensive report for engineers concerned with designing coastal structures for shore stabilization or improvement of navigation. The 3rd Edition was printed in the summer of 1966 and went on sale at the Superintendent of Documents in November 1966. Early in 1969, the original printing of 5,000 had been sold, and a new printing of 3,000 was ordered. Several minor typographic errors were corrected, and a subject index was added as Appendix F.

This version may be ordered from the Superintendent of Documents, U. S. Government Printing Office, Washington, D. C. 20402. The price, postpaid, is $3.00 in the United States and its possessions, Canada, Mexico, and some Central and South American countries. For South American countries not having a postal agreement, and for other countries, the postpaid price is $3.75.

An extensively revised and reorganized version is now in preparation; publication will probably be late in 1971.

C. The Clearinghouse

The Clearinghouse for Federal Scientific and Technical Information was expanded in the Department of Commerce, National Bureau of Standards, in 1964. It now serves as a focal point for the collection, announcement, and dissemination of unclassified U. S. Government-sponsored research and development reports.

All Beach Erosion Board and CERC publications have now been cataloged in the Clearinghouse system, and are available to the general public. A list of BEB and CERC publications with Clearinghouse accession numbers and ordering instructions is available from CERC.

VIII. MISCELLANEOUS TESTS

A. Riprap Test - Missouri River Division

Starting in November 1965, many laboratory tests were run at CERC to determine the stability of riprap to wave action. These tests were run for the Missouri River Division, U. S. Army Corps of Engineers, and were motivated by a number of failures and potential failures of riprap protecting earth dams and embankments on some of the large reservoirs in the Missouri River Basin. In many areas of the Missouri Basin, design is complicated by the shortage of high quality riprap stone, and overdesign can be extremely expensive.

About 30 slopes protected by riprap were constructed and tested until failure in the large 635-foot wave tank. About the same number were constructed and tested in the 85-foot medium wave tank, and about 70 in the 72-foot small wave tank. The armor material used for riprap included quarry stone (both limestone and quartzite), granitic fieldstone boulders, and concrete and leadite tribars. The median weight of the armor stones tested ranged from as light as 0.03 pound in the small wave tank up to as heavy as 390 pounds in the large wave tank; the weight of tribars tested ranged from 0.12 to 81 pounds. All of the tests conducted in the large and medium tanks had embankment slopes of 1 on 2, 1 on 3, or 1 on 5. In the small-tank tests, these same embankment slopes were tested, plus two additional ones of 1 on 7 and 1 on 10. The zero-damage wave heights in the large tank ranged upward to over 5 feet in a number of tests. The most frequently used wave-period in the large wave tank was 3.67 seconds with a water depth of 15 feet; this gives a depth to wave length ratio, d/L, of 0.24. A d/L ratio of 0.24 was also used extensively in the smaller scale tests, but some tests were run using both longer and shorter waves in all tanks.

A detailed report, being prepared by the Missouri River Division, covers most of the riprap tests conducted for them by CERC. This report will probably be published by CERC late in 1970.

B. Sand-Filled Nylon Bag Tests.

Testing in the Large Wave Tank was completed in 1968 on a study to provide data on: (a) the stability of nylon bags filled with 2 tons of sand and placed to form a submerged breakwater; and (b) the wave height attenuation effect of the structure. The filling and operation of the nylon bags followed construction procedures and techniques very similar to those which could be used at actual field coastal sites. Reduction and analysis of the data are not yet complete.

C. Synthetic Seaweed

Testing in the Large Wave Tank on Olefern (plastic seaweed frond) was completed in 1968. The tests were at prototype scale to determine wave-induced loading on the front, and frond reaction to the passage of waves of various heights and periods. Measurements were made by attaching a small wire from the end of a frond to a load cell which in turn sensed the change in load for a given wave cycle and the different load levels or intensities. Reduction and analysis of the data continue

D. Techniques of Coastal Construction

A continuing program is in progress to compile data and information on special techniques used in the construction of coastal works. The data and information will include not only the structural materials and their ordinary handling, but also the use of special or nonconventional equipment in the construction procedure.

Training films will be made up from the edited print of movie film pertaining to the construction of coastal works along the Southern California Coast. An answer print of the first film entitled, "Building Breakwaters - California Style" was completed for the Los Angeles District by contract, and CERC has received a copy for review. Preparation of a second film entitled "Coastal Engineering and Investigation Methods" is in progress. Other films made under this program are: (1) a film produced by the Avisun Corporation showing features of a synthetic seaweed and describing a placement operation at Wallops Island, Virginia, was received and a copy obtained for CERC; and (2) a film copy showing shore conditions at a recently completed Beach Erosion Control project in the Charleston, South Carolina, District, and a copy was received. Arrangements are now complete for procuring a movie film depicting conditions before, during, and after construction of a Beach Erosion Control project in Florida.

E. Corrosion Test of Steel Piling in Sea Water

Test piles were installed cooperatively by the National Bureau of Standards and CERC during 1967 to study corrosion and protection of steel piling installed in a marine environment. A total of ninety-six 8-inch H and 8-inch cylindrical piles were installed in the Atlantic Ocean at Dam Neck, Virginia, just south of Virginia Beach. The piles are installed in sets of three and include bare piles, coated piles, and coated or bare piles with sacrificial anodes. Data will be obtained on the piles by pulling one of each set of three at 5-year intervals for detailed analysis of corrosion and abrasion damage. Data is also collected annually by visual inspection, photographs and electrical measurements. The first report of findings from this investigation is expected to be available in 1973. The information obtained from this installation should aid in estimating the life of steel piling with and without various forms of protection in similar marine environments. A paper was prepared for presentation at the American Society of Civil Engineers National Meeting on Structural Engineering in Portland, Oregon, by L. L. Watkins, Engineering Development Division.

F. Sand Slurry Abrasion Test of Coatings for Steel Pilings

Coated steel panels were mounted in front of a submerged jet from which sand slurry under pressure was ejected. Data includes coating descriptions and thicknesses, slurry temperatures and the time required for given areas of steel exposure. Although the tests must be repeated in order to determine if results will be duplicated, initial results indicate considerable variation in the abrasion resistance of protective coatings. The test also showed that one coating armored with aluminum oxide required 9 times as long to expose the same area of steel as the coating unarmored. A report, which described equipment, test procedures and test results, aimed at determining the relative abrasion resistance of various protective coatings for steel to sand suspended in water, was prepared by L. L. Watkins for presentation at the National Meeting on Structural Engineering, ASCE, in Portland, Oregon.

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13. ABSTRACT

This Bulletin consists of 3 articles and a summary of CERC research for FY's 1967-69. The lead article describes hurricane protection in New England harbors. Operation of the gates is described; and based on records of storms since 1935, prevention of losses is projected. A Method for Driving Pipe in Beach Rock describes placing pipe economically to obtain beach profiles without surveying. Wave Diffraction in a Laboratory Movable-Bed Setup discusses how diffraction causes deleterious results.

Summary of Research:
Wind-Wave Action in Coastal Waters - Studies in coastal areas and wave tanks reported. Shore Processes - CERC studies cover alongshore transport, suspended material, sand simulants, beach profiles and dune building. Long-Period Waves and Surges - Analyzes harbor surging, tsunamis, 2-dimensional long waves. Shore Protection in Bays, Lakes and Reservoirs - Study in Chesapeake Bay by CERC. Biological-Ecological Studies - Program to use offshore sources of sand to nourish beaches; create bottom conditions favorable to marine life; research the effects of waste dumped in the N. Y. Harbor. Environmental Data Collection - CERC studies on sand inventory, sand tracers, wave analysis, wave sensors and surf observation. A laser wave gage was tested at Atlantic City, N. J. Publications - A list of publications for FY's 67-68-69. Miscellaneous Tests - Riprap in reservoirs of the Missouri River Division, sand-filled nylon bags; synthe[illegible]eaweed; special techniques for coastal construction; corrosion of steel piling [illegible]ea water; and sand slurry abrasion of coated steel pilings.

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Beach nourishment						