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Dredging Operations Technical Support Program
Monitoring Completed Navigation Projects Program

Guidance and Lessons Learned from Monitoring Completed Navigation Projects

Compiled by Lyndell Z. Hales and Donna L. Richey

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Final report

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ABSTRACT: The purpose of this report is to provide comprehensive site-specific and generic lessons learned from intensive monitoring of 12 different project features at each of 38 navigation projects located in 16 U.S. Army Corps of Engineers Districts around the continental United States, Alaska, Hawaii, and other Pacific islands. Generic lessons learned from seven geographic regions (Hawaii and the Pacific Islands, Alaska, Pacific coast of the U. S. mainland, Gulf of Mexico, Atlantic coast of the U. S. mainland, the Great Lakes, and inland navigation sites) have been deduced from the site-specific lessons learned for each of these seven geographic regions. From these generic lessons learned after several years of monitoring and/or periodic inspection, data collection, and data analyses at each of the 38 navigation projects, guidance has been developed for planning and design of 12 navigation project features evaluated by the MCNP program to the present time. The 12 navigation project features for which guidance has been developed include: (1) breakwaters, (2) floating breakwaters, (3) beach nourishment and sediment transport, (4) jetties, (5) jetty spurs, (6) weir-jetties, (7) inlets, (8) wave transformation, (9) harbors, (10) confined aquatic disposal (CAD) cells, (11) breakwater stone deterioration, and (12) inland navigation dam submersible gates.

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Conversion Factors, Non-SI to SI Units of Measurement

Non-SI unites of measurement used in this report can be converted to SI units as follows.

Multiply	By	To Obtain
cubic yards	0.7645549	cubic meters
cubic yards per year	0.7645549	cubic meters per year
cubic yards per yard	0.836	cubic meters per meter
cubic feet	0.3048781	meters
feet	0.3048781	meters
inch	2.54	centimeters
miles (U.S. statute)	1.609347	kilometers
miles per hour	1.609347	knots
tons (2,000 pounds, mass)	907.1847	kilograms

Preface

The studies reported herein were conducted as part of the Monitoring Completed Navigation Projects (MCNP) program. Overall management of the MCNP program is provided by Headquarters, U.S. Army Corps of Engineers (HQUSACE). Funding for this publication was provided by the Dredging Operations Technical Support (DOTS) Program, Dr. Douglas Clarke, Program Manager, U.S. Army Engineer Research and Development Center (ERDC), Environmental Laboratory (EL), Vicksburg, MS. Acting Director of EL was Dr. Elizabeth Fleming. Coastal and Hydraulics Laboratory (CHL), ERDC, Vicksburg, MS, is responsible for technical and data management and support for HQUSACE review and technology transfer.

These studies were performed under the general supervision of Mr. Thomas W. Richardson, Director, CHL. Program monitors for the MCNP program are Messrs. Barry W. Holliday, Charles B. Chesnutt, and David B. Wingerd, HQUSACE. CHL/MCNP Program Managers during the conduct of these studies were Ms. Carolyn M. Holmes and Messrs. J. Michael Hemsley, E. Clark McNair, and Robert R. Bottin, Jr. This report was compiled by Dr. Lyndell Z. Hales and Ms. Donna L. Richey, CHL.

The purpose of this report is to provide comprehensive site-specific and generic lessons learned from intensive monitoring of 12 different project features at each of 38 navigation projects located in 16 Corps of Engineers Districts around the continental United States, Alaska, Hawaii, and other Pacific islands. Generic lessons learned from seven geographic regions (Hawaii and the Pacific Islands, Alaska, Pacific coast of the U. S. mainland, Gulf of Mexico, Atlantic coast of the U. S. mainland, the Great Lakes, and inland navigation sites) have been deduced from the site-specific lessons learned for each of these seven geographic regions. From these generic lessons learned after several years of monitoring and/or periodic inspection, data collection, and data analyses at each of the 38 navigation projects, guidance has been developed for planning and design of 12 navigation project features evaluated by the MCNP program to the present time. The navigation project features for which guidance has been developed include (1) breakwaters, (2) floating breakwaters, (3) beach nourishment and sediment transport, (4) jetties, (5) jetty spurs, (6) weir-jetties, (7) inlets, (8) wave transformation, (9) harbors, (10) confined aquatic disposal (CAD) cells, (11) breakwater stone deterioration, and (12) inland navigation dam submersible gates.

The following District MCNP Team Members contributed significantly to these MCNP investigations: Honolulu District: Messrs. S. J. Bok, Jr., D. T. Meyers, and M. S. Okihiro; Alaska District: Messrs. K. J. Eisses and A. C. Jeffries; Seattle District: Mr. E. E. Nelson; Portland District: Ms. H. P. Moritz and Messrs. K. R. Soderlind, S. A. Chesser, H. D. Herndon, H. R. Moritz, and M. D. Siipola; San Francisco District: Ms. E. Tatami and Messrs. T. R. Kendall, W. S. Appleton, R. Lou, K. Guy, D. Thuet, and J. Hawkins; Los Angeles District: Mr. A. T. Shak; Galveston District: Mr. E. A. Reindl, Jr.; Mobile District: Mr. L. K. Robinson; Wilmington District: Mr. J. T. Jarrett; Baltimore District: Mr. G. P. Bass; Philadelphia District: Ms. M. A. Chasten and Messrs. K. D. Watson, J. A. Gebert, and W. F. Rothert; New England District: Mr. C. O. LaBlanc and Dr. T. J. Fredette; Buffalo District: Ms. D. Rowan and Messrs. M. C. Mohr, D. W. Marcus, and D. Clark; Detroit District: Messrs R. L. Erickson and C. Thompson; Chicago District: Ms. H. P. Moritz and Messrs. E. W. Mathews, M. K. Tibbets, M. M. Baig, and K. Richards; and Rock Island District: Messrs. D. Wehrley and J. W. Bartek.

COL James R. Rowan, EN, was Commander and Executive Director of ERDC. Dr. James R. Houston was Director.

1 Introduction

Monitoring Completed Navigation Projects (MCNP) Program

The goal of the Monitoring Completed Navigation Projects (MCNP) program (formerly, Monitoring Completed Coastal Projects) is the advancement of coastal and hydraulic engineering technology. The program is designed to determine how well projects are accomplishing their purposes and are resisting attacks by their physical environment. These determinations, combined with concepts and understanding already available, will lead to: (a) the creation of more accurate and economical engineering solutions to coastal and hydraulic problems; (b) strengthening and improving design criteria and methodology; (c) improving construction practices and cost-effectiveness; and (d) improving operation and maintenance techniques. Additionally, the monitoring program will identify where current technology is inadequate or where additional research is required.

To develop direction for the program, the U.S. Army Corps of Engineers (USACE) established an ad hoc committee of engineers and scientists. The committee formulated the objectives of the program, developed its operation philosophy, recommended funding levels, and established criteria and procedures for project selection. A significant result of their efforts was a prioritized listing of problem areas to be addressed. This is essentially a listing of the areas of interest of the program.

Corps offices are invited to nominate projects for inclusion in the monitoring program as funds become available. The MCNP program is governed by Engineer Regulation 1110-2-8151 (Headquarters, U.S. Army Corps of Engineers (HQUSACE) 1997)). A selection committee reviews and prioritizes the nominated projects based on criteria established in the regulation. The prioritized list is reviewed by the Program Monitors at HQUSACE. Final selection is based on this prioritized list, national priorities, and the availability of funding.

The overall monitoring program is under the management of the Coastal and Hydraulics Laboratory (CHL), U.S. Army Engineer Research and Development Center (ERDC), with guidance from HQUSACE. An individual monitoring project is a cooperative effort between the submitting District and/or Division office and CHL. Development of monitoring plans and conduct of data collection and analyses are dependent upon the combined resources of CHL and the District and/or Division.

Background

In the late 1970s, the Field Review Group (FRG) members of the Corps' Coastal Program (CP) recommended to HQUSACE, Directorate of Research and Development (DRD), that selected coastal structures be intensely monitored for a finite length of time, beginning when construction is completed, to determine if the structures are indeed performing as intended during the design stages. Many aspects of the exceedingly complex interactions among environmental factors such as waves, currents, tides, and sediments with breakwaters, jetties, inlets, and harbors are not well understood even today. Hence, it was desired to learn definitively if elements used in the design of these structures were appropriate for the conditions under which they were applied. If not, the knowledge gained would then provide guidance for the development of new technologies that would be more applicable to the conditions existing at a particular site. It was recommended that monitoring be conducted for not only new structures but also those that were undergoing extensive rehabilitation as the result of unacceptable performance or structure failure because of adverse environmental conditions. This monitoring would logically be conducted with Corps Operational and Maintenance (O&M) funds, since the knowledge gained would enhance the Corps' ability to design and install more effective and cost-saving elements into future new work, and also would contribute to rehabilitation and modification of existing structures.

HQUSACE DRD concurred in the recommendations of the Coastal Program FRG members and established a Monitoring of Completed Coastal Projects (MCCP) program in 1978. (This program was expanded in the late 1990s to include inland navigation projects and became known as the Monitoring of Completed Navigation Projects (MCNP) program.) Because of limited O&M funding availability for this monitoring and analysis effort, it would be impossible to monitor every structure in the Corps' inventory. Thus, DRD directed each Corps coastal and Great Lakes district to submit nominations from their respective districts to the FRG pertaining to structures within that district which they believed would yield pertinent knowledge regarding uncertainties existing at that time with respect to coastal design and construction. These nominations would be discussed and considered by the FRG, and a prioritized ranking would be developed based on the FRG's insight into the inherent problems which should be better understood for reducing Corps O&M costs. This prioritized ranking would be based on such factors as the degree of uncertainty of physical process interactions existing at a site-specific location and also on the capability of extrapolating this new knowledge and understanding gained from that site-specific location to a regional basis.

The knowledge gained from monitoring and analysis of a finite small number of specific structures should provide not only site-specific lessons learned but should also result in generic lessons learned that would be applicable to a significant region of coastline that experiences essentially the same range of environmental factors and other parameters. In effect, it was the considered opinion of the FRG members that knowledge gained from structures exposed to large waves and high tide ranges (such as the Pacific coast) could not be extrapolated to regions with dissimilar conditions such as the Great Lakes. Likewise, knowledge

obtained from structures experiencing ice conditions in Alaska may not be appropriate for coastlines along the Gulf of Mexico.

Accordingly, a large number of nominations for monitoring of completed coastal structures were received by DRD. After extensive review and stringent criteria had been established and applied to each nomination, the first structure selected for monitoring and analysis was the University of Washington Oceanographic Laboratory, Port of Friday Harbor, floating breakwater in the Puget Sound of Washington. Subsequently, five other floating breakwaters in the Puget Sound were monitored and analyzed during the early- to mid-1980s. Several more monitoring efforts were initiated during this time period, including structures consisting of breakwaters, jetties, inlets, and harbors. Other aspects studied and analyzed included wave transformation over reef areas, beach nourishment, and sand transport along coastlines and around structures. These phenomena were not at all well understood, and knowledge gained from these monitoring efforts has proven exceedingly valuable for calibrating and verifying numerical simulation models that describe these events.

The time period over which monitoring efforts take place is typically around 3 to 5 years. Most of the data collection and analysis has been performed by ERDC (formerly the Waterways Experiment Station), CHL (formerly Coastal Engineering Research Center), although occasionally some monitoring/analysis has been conducted by the respective Corps Districts or private consulting engineering firms.

Beginning in 1995, it was determined that a significant amount of information could be gained by routine periodic inspections of some specific structural elements intermittently over a large number of years, instead of a continuous intensive monitoring effort over a relatively short time period. Many of the phenomena of interest result from small perturbations occurring over very long time periods. Also, some structures which had earlier been monitored extensively for a relatively short time period (few years) had begun to reveal exposure aspects not previously detectable. Based on this philosophy, the Periodic Inspections work unit of the MCNP program was initiated.

Structures Monitored

Since inception of the MCCP (subsequently the MCNP) program, 38 navigation projects have been monitored extensively and/or periodically inspected in 16 Districts. Twelve (12) different project features have been monitored and evaluated at one or more projects, including (a) beach nourishment and sand transport, (b) wave transformation, (c) weir-jetties, (d) jetties, (e) breakwaters, (f) jetty spurs, (g) inlets, (h) harbors, (i) floating breakwaters, (j) confined aquatic disposal (CAD) cells, (k) breakwater stone, and (l) inland navigation dam submersible gates. The 38 projects monitored and their features evaluated are shown by Corps District in Table 1. One or more knowledgeable individuals from each respective District assumed responsibility of working with ERDC as District MCNP Team Members in developing and executing the appropriate monitoring program for a particular structure.

Table 1
Corps of Engineer Districts, Coastal Projects Monitored, and Project Features Evaluated

Districts	Location	Features Evaluated
Honolulu	Nawiliwili, HI	Breakwater
	Kahului, HI	Breakwaters
	Laupahoehoe, HI	Breakwater
	Barbers Point, HI	Harbor
	Agat, Guam	Wave transformation
	Ofu, American Samoa	Breakwater
Alaska	St. Paul Harbor, AK	Breakwater,
		Harbor
Seattle	Friday Harbor Marina, WA	Floating breakwater
	UW Friday Harbor Laboratory, WA	Floating breakwater
	East Bay Marina, WA	Floating breakwater
	Zittle's Marina, WA	Floating breakwater
	Brownville Marina, WA	Floating breakwater
	Semiahmoo Marina, WA	Floating breakwater
Portland	Columbia River Mouth, WA/OR	Wave transformation, Beach nourishment and sediment transport
	Yaquina, OR	Jetty
	Siuslaw, OR	Jetty spurs
	Umpqua, OR	Jetty Inlet
San Francisco	Crescent City, CA	Breakwater
	Humboldt Bay, CA	Jetties
	Spud Point, CA	Breakwater
	Fisherman's Wharf, CA	Breakwater, Harbor
Los Angeles	Morro Bay, CA	Breakwaters, Sediment transport, Harbor, Inlet
	Redondo Beach, CA	Wave transformation
Galveston	Colorado River Mouth, TX	Weir-jetty
Mobile	East Pass, FL	Weir-jetty
Wilmington	Carolina Beach, NC	Beach nourishment and sediment transport
Baltimore	Ocean City, MD	Inlet
Philadelphia	Barnegat Inlet, NJ	Jetty, Inlet
	Manasquan, NJ	Jetties, Inlet
New England	Oakland Beach, RI	Beach nourishment and sediment transport
	Boston Harbor, MA	Confined aquatic disposal cells
Buffalo	Cattaraugus Creek, NY	Beach nourishment and sediment transport, Jetties, Breakwater, Inlet
	Cleveland Harbor, OH	Breakwater, Breakwater stone
Detroit	St. Joseph, MI	Beach nourishment and sediment transport
Chicago	Burns Harbor, IN	Breakwater, Harbor, Breakwater stone
	Calumet Harbor, IL	Breakwater stone
	Chicago Harbor, IL	Breakwater stone
Rock Island	Marseilles Dam, IL	Submersible gates

Because of the regional nature of the environmental factors acting on each specific structure, generic lessons learned are deduced from those projects located in seven unique geographic regions, including (a) Hawaii and the Pacific Islands, (b) Pacific Coasts of the U.S. Mainland, (c) Atlantic Coast of the U.S. mainland, (d) Gulf of Mexico, (e) Great Lakes, (f) Alaska, and (g) inland navigation sites. Features evaluated by site-specific projects to develop generic lessons learned are shown by geographic regions in Table 2.

Table 2
Geographic Regions, Project Features Evaluated, and Coastal Projects Monitored

Region	Project	Location
Hawaii and the Pacific Islands	Wave transformation	Agat Harbor, Guam
	Breakwaters	Ofu Harbor, American Samoa Nawiliwili Harbor, Kauai, HI Kahului Harbor, Maui, HI Laupahoehoe, HI
	Harbor	Barbers Point, Oahu, HI
Alaska	Harbor	St. Paul Harbor, AK
	Breakwater	St. Paul Harbor, AK
Pacific Coast of the U.S. Mainland	Breakwaters	Fisherman's Wharf, San Francisco, CA Spud Point, Bodega Bay, CA Crescent City, CA Morro Bay, CA
	Floating breakwaters	Friday Harbor Marina, WA UW Friday Harbor Laboratory, WA East Bay Marina, WA Zittle's Marina, WA Brownville Marina, WA Semiahmoo Marina, WA
	Jetties	Yaquina Bay, OR Humboldt Bay, CA
	Jetty spurs	Siuslaw River, OR
	Wave transformation	Redondo Beach, CA
	Harbor	Fisherman's Wharf, San Francisco, CA Morro Bay, CA
	Inlet	Morro Bay, CA
Atlantic Coast of the U.S. Mainland	Jetties	Manasquan Inlet, NJ Ocean City, MD
	Inlets	Manasquan Inlet, NJ Ocean City, MD
	Beach nourishment and sediment transport	Oakland Beach, RI Carolina Beach, NC
Gulf of Mexico	Weir-jetties	Colorado River Mouth, TX East Pass Inlet, Destin, FL
Great Lakes	Breakwaters	Cattaraugus Creek Harbor, NY Burns Harbor, IN Cleveland Harbor, OH
	Breakwater stone	Cleveland Harbor, OH Burns Harbor, IN Calumet Harbor, IN Chicago Harbor, IL
	Beach nourishment and sediment transport	Cattaraugus Creek Harbor, NY St. Joseph, MI
	Inlet	Cattaraugus Creek Harbor, NY
Inland navigation sites	Submersible gates	Marseilles Dam, IL

2 Site-Specific Lessons Learned, Hawaii and the Pacific Islands

Nawiliwili Harbor, Kauai, Hawaii

Nawiliwili Harbor (Figure 1) is located on the southeast coast of the island of Kauai, approximately 185 km (115 miles) northwest of Honolulu, Oahu, HI. The harbor is protected by a 625-m- (2,050-ft-) long rubble-mound breakwater. The breakwater protects the inner breakwater of the small boat harbor, the commercial harbor, and major industries along its waterfront.

Figure 1. Nawiliwili Harbor, Kauai, HI (after Bottin and Meyers 2002a)

Item monitored

Breakwater.

Period monitored

October 1995 and October 2001 periodic inspections.

Reason(s) for monitoring

Base conditions for future periodic inspections were determined in October 1995, and the first periodic inspection was conducted in October 2001. Purposes of the periodic inspections are to: (a) develop methods using limited land-based surveying, aerial photography, and photogrammetric analysis to assess the long-term stability response of the concrete armor units on the Nawiliwili breakwater; and (b) conduct land surveys, broken armor unit inspections, aerial photography, and photogrammetric analyses to test and improve developed methodologies and accurately define armor unit movement above the waterline (Bottin and Boc 1996; Bottin and Meyers 2002a; Bottin 2003a).

Results of the October 1995 periodic inspection

Details of the inspection are:

a. The Nawiliwili Harbor breakwater has been repeatedly subjected to major storm events, including three hurricanes, during its 70-year history. As a result, extensive breakwater damage has occurred. Major rehabilitations were completed in 1959, 1977, and 1987. The structure was originally armored with keyed-and-fitted stone but now has several sizes of tribar and dolos concrete armor units. The Nawi1iwili breakwater is one of the most complex rubb1e-mound structures the Corps of Engineers has constructed. No sound, quantifiable data relative to the movement or positions of the concrete armor units had been obtained for the structure prior to this study.

b. Under the Periodic Inspections work unit, data from limited ground-based surveys, aerial photography, and photogrammetric analysis have been obtained to establish very precise base level conditions for the Nawiliwili Harbor breakwater. Accuracy of the photogrammetric analysis was validated and defined through comparison of ground and aerial survey data on control points and targets established on the structure. A method of high-resolution, stereo-aerial photographs, a stereoplotter, and AutoCad-based software has been developed to analyze the entire above-water armor unit fields and quantify armor positions and subsequent movement. A detailed broken armor unit survey conducted during the current effort has resulted in a well-documented data set that can be compared to subsequent survey data.

c. Now that base (control) conditions have been defined at a point in time and methodology has been developed to closely compare subsequent years of high-resolution data for the Nawiliwili Harbor breakwater, the site will be revisited in the future under the Periodic Inspections work unit to gather data by which assessments can be made on the long-term response of the structure to its environment. The insight gathered from

these efforts will allow engineering decisions to be made, based on sound data, as to whether or not closer surveillance and/or repair of the structure might be required to reduce its chances of failing catastrophically. Also, the periodic inspection methods developed and validated for these structures may be used to gain insight into other Corps structures.

Results of the October 2001 periodic inspection

Details of the inspection are:

a. Similar data were obtained during 2001 and compared with the 1995 data. An analysis of these data indicated negligible movement of the concrete armor unit on the breakwater. Maximum movement of the targets established on the concrete armor units in the horizontal and vertical directions, respectively, were 0.01 m (0.42 ft) and 0.14 m (0.45 ft); and the average movement of all horizontal and vertical targets was 0.03 m (0.1 ft) and 0.05 m (0.15 ft). Maximum movement of the targeted armor unit centroids was 0.1 m (0.34 ft) and 0.1 m (0.37 ft) in the horizontal and vertical directions, respectively, while average movements were 0.03 m (0.1 ft) and 0.04 m (0.14 ft) in the horizontal and vertical directions.

b. A total of 70 broken/cracked concrete armor units were identified in the 1995 survey, and 77 broken/cracked units were identified in 2001. However, high-wave action during the 1995 waking inspection prevented a close examination of armor units at the water's edge. Of the seven additional broken units in 2001, six were located along the water's edge and may have been broken in 1995 as the result of the excessive wave action. Therefore, it appears that minimal armor unit breakage occurred between 1995 and 2001.

Kahului Harbor, Maui, Hawaii

Kahului Harbor (Figure 2) is the only deep-draft harbor on the island of Maui, the second largest of the Hawaiian Islands. The harbor is approximately 150 km (94 miles) southeast of Honolulu, and is centrally located on Maui's north shore.

Item monitored

Breakwaters.

Period monitored

April 1993 and October 2001 periodic inspections.

Figure 2. Kahului Harbor, Maui, HI (after Bottin and Meyers 2002b)

Reason(s) for monitoring

Base conditions for future periodic inspections were determined in April 1993, and the first periodic inspection was completed in October 2001. Purposes of the periodic inspections are to: (a) develop methods using limited land-based surveying, aerial photography, and photogrammetric analysis to assess long-term stability response of armor unit layers and concrete rib caps on the Kahului breakwaters; and (b) conduct initial land surveys, armor unit breakage inspections, aerial photography, and photogrammetric analyses to test and improve developed methodologies and accurately define armor unit movement over the entire above-water armor unit fields (Markle and Boc 1994; Bottin and Meyers 2002b; Bottin 2003a).

Results of the April 1993 periodic inspection

Details of the inspection are:

a. The Kahului harbor complex got its start when the armor stone east breakwater was constructed in 1900. The west breakwater was constructed in 1919. In 1931, the east and west breakwaters were extended to their current lengths of 845 m (2,766 ft) and 705 m (2,315 ft), respectively. All original construction used a single layer of keyed and fitted 7,265-kg (8-ton) armor stone. Subsequent storms and rehabilitations have occurred since 1931. In 1966, both breakwater heads were armored with two layers of 31,780-kg (35-ton) tribars. A concrete rib cap was placed on the east breakwater. In 1969, a concrete rib cap and 260 reinforced tribars weighing 17,250 kg (19 tons) each were placed on the west

breakwater. An inspection in 1973 revealed that 29,965-kg (33-ton) tetrapods on the sea side of both heads had sustained considerable damage and they, along with the 7,265-kg (8-ton) stone areas on both trunks, were in need of repair.

b. The most recent repairs were completed in 1984. This rehabilitation was carried out to eliminate the need for future "piecemeal" repairs. A total of 540 tribars weighing 5,900 kg (6.5 tons) each, 755 tribars weighing 8,170 kg (9 tons) each, and 10 tribars weighing 22,700 kg (25 tons) each, were placed during this rehabilitation.

c. By means of limited land surveys, low-level helicopter inspections with 35-mm photography, aerial photography, and photogrammetric analysis, base conditions have been established for the Kahului breakwaters. Accuracy of the photogrammetric analysis techniques has been checked through comparison of ground and aerial survey data on armor units that had been specifically targeted and surveyed for this purpose. A method using high-resolution, stereo-pair aerial photographs, a stereoplotter and AutoCAD files has been developed and tested to analyze the entire above-water armor unit fields to quantify armor unit movement that exceeds a threshold value of 0.2 m (0.5 ft).

d. During testing of the method, it was observed that very little change, in regard to armor unit movement, occurred between 1990 and 1993, but this should be anticipated, as the wave climate to which the structures were exposed was very mild during this time period. Low-level helicopter surveys of concrete armor units revealed only minimal amounts of breakage on the Kahului structures. A walking inspection of the Kahului breakwaters conducted under another research study revealed higher levels of armor breakage than found by aerial studies. However, the level of breakage is still minimal, but the area at the confluence of the sea side of the head and trunk of the west breakwater is beginning to show a slight concentration, or cluster, of breakage, and this area should be monitored more closely than other areas. Also, the land-based breakage survey revealed that the accuracy of aerial breakage inspections can be questionable and that for more accurate armor unit breakage counts, detailed walking inspections should be conducted over the armor unit fields.

Results of the October 2001 periodic inspection

Details of the inspection are:

a. Similar data were obtained during 2001 and compared with the 1993 data. An analysis of these data indicated some armor unit movements on the Kahului breakwaters (particularly the east breakwater). One target moved about 0.9 m (3 ft) horizontally and one moved almost 1.5 m (5 ft) vertically. Both these units were located around the seaward head of the structure. The average movements of the targets, however, were on the order of about 0.15 m (0.5 ft). An evaluation of nontargeted units indicated several units had changed horizontal positions (on the order of 0.3

to 0.9 m (1 to 3 ft)) also around the seaward quadrant of the head of the east breakwater. These units are intact, however, and continue to be functional.

b. For the west breakwater, however, comparisons of target coordinates showed relatively close agreement with those obtained in 1993. The average movement of all targets in both the horizontal and vertical directions was less than 0.02 m (0.5 ft). Considering the movements of targeted armor units' centroids, average movements in both the horizontal and vertical directions were less than 0.2 m (0.6 ft) for the east breakwater, and less than 0.1 m (0.4 ft) for the west breakwater.

c. A total of 29 broken/cracked armor units on the east breakwater and 58 on the west breakwater were identified during the 2001 survey. These data establish a base from which to evaluate future breakage in subsequent surveys. The areas of concentrated breakage on the Kahului east and west breakwaters should be inspected annually to monitor any increase in breakage and thus reduction in stability.

Laupahoehoe Boat Launching Facility, Hawaii

Laupahoehoe (Figure 3) is located on the north coast of the Island of Hawaii, approximately 40 km (25 miles) north-northwest of Hilo.

Item monitored

Breakwater.

Period monitored

April 1993 and October 2001 periodic inspections.

Reason(s) for monitoring

Base conditions for future periodic inspections were determined in April 1993, and the first reinspection was completed in October 2001. Purposes of the periodic inspections are to: (a) develop methods using limited land-based surveying, aerial photography, and photogrammetric analysis to assess long-term stability response of armor unit layers and concrete rib caps on the Laupahoehoe breakwater; and (b) conduct initial land surveys, armor unit breakage inspections, aerial photography, and photogrammetric analyses to test and improve developed methodologies and accurately define armor unit movement over the entire above-water armor unit fields (Markle and Boc 1994; Bottin and Meyers 2002b; Bottin 2003a).

Figure 3. Laupahoehoe Boat Launching Facility, HI (after Bottin and Meyers 2002b)

Results of the April 1993 periodic inspection

Details of the inspection are:

a. The initial design of the 76-m- (250-ft-) long Laupahoehoe rubble-mound breakwater called for the vertical placement of core stone to be armored with a 27,240-kg (30-ton) dolos, and with the crest to be stabilized with a concrete rib cap. The rib cap increases crest stability, reduces wave overtopping, provides buttressing for crest armor units, allows ease of access for maintenance, and is less reflective than the solid concrete cap. The toe of the dolos was keyed into the hard basalt bottom by means of a trench excavated around the perimeter of the breakwater. However, the breakwater stability model study noted that the stone beneath the rib cap showed some displacement and consolidation during testing. The constructability review of the plans also noted that the vertical placement of the breakwater core stone would be a formidable task in the area's year-round rough ocean conditions.

b. A stable breakwater core was achieved through the innovative design of a reinforced concrete pipe rib cage. Because of the interior geometry of the structure, cylindrical reinforced concrete pipes were stood on end and backfilled to provide a stable support for the rib cap. This unique design feature, along with the trenched toe for the dolos, appears to be performing well structurally, and periodic photogrammetric surveys will provide a basis for a long-term structural assessment of the project and its possible application at other sites.

c. By means of limited land surveys, low-level helicopter inspections with 35-mm photography, aerial photography, and photogrammetric analysis, base conditions have been established for the Laupahoehoe Boat Launching Facility breakwater. Accuracy of the photogrammetric analysis techniques has been determined through comparison of ground and aerial survey data on armor units that had been specifically targeted and surveyed for this purpose. A method using high-resolution, stereo-pair aerial photographs, a stereoplotter, and AutoCAD files has been developed and tested to analyze the entire above-water armor unit fields to quantify armor unit movement that exceeds a threshold value of 0.2 m (0.5 ft).

d. During testing of the method, it was observed that very little change, in regard to armor unit movement, occurred during the 1991-1993 monitoring period, but this should be anticipated, as the wave climate to which the structures were exposed was very mild during this time. Low-level helicopter surveys of concrete armor units revealed no breakage on the Laupahoehoe breakwater.

Results of the October 2001 periodic inspection

Details of the inspection are:

a. Similar data were obtained during 2001 and compared with the 1993 data. An analysis of these data indicated negligible movement on the breakwater.

b. Average target movement in both the horizontal and vertical directions was less than 0.1 m (0.2 ft). Considering the movements of targeted armor units' centroids, average movements in both the horizontal and vertical directions were around 0.03 m (0.1 ft) for the Laupahoehoe breakwater.

c. No broken/cracked armor units were found on the Laupahoehoe breakwater.

Barbers Point, Oahu, Hawaii

Barbers Point Harbor (Figure 4) is located on the southwest coast of the island of Oahu. It is approximately 3 km (2 miles) upcoast of the southwestern corner of the island and 24 km (15 miles) west of Honolulu Harbor. The project

Figure 4. Barbers Point Harbor, Oahu, HI (after Lillycrop et al. 1993)

area is about 32 km (20 miles) from downtown Honololu and lies within the Barbers Point Industrial Park.

Item monitored

Harbor.

Period monitored

July 1986 – March 1990 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period July 1986 – March 1990 to: (a) evaluate and validate results of model studies conducted for the harbor design; (b) perform wave gauging to measure wave climates in deep water and nearshore areas, and long-period oscillations of the harbor; (c) relate the conditions outside the harbor to surge found inside the harbor; (d) evaluate the effectiveness of the wave absorber; and (e) compare the measured data to the predictions of state-of-the-art physical and numerical model studies (Lillycrop et al. 1993).

Results of the July 1986 – March 1990 monitoring

Details of the inspection are:

a. Results of the numerical model study show that the model did well in predicting the resonant modes of oscillation that were measured in the prototype harbor. Differences between the results are (1) the numerical model resonant peaks occur at slightly offset periods from the prototype, which could result from differences in dimensions of the grided harbor and the prototype configuration; and (2) the numerical model magnitudes of amplification are larger than the prototype measurements that are expected since the model neglected dissipative effects.

b. The 1967 hydraulic model and prototype results for short period waves did not exceed the desired maximum criteria of 0.8 m (2.5 ft) or the maximum tolerable criteria of 1.4 m (4.5 ft) in the deep-draft harbor. Generally, the model wave heights are larger than the prototype in the north and east corners; however, the prototype wave heights are larger in the south corner. The configuration of the small boat harbor tested for short waves was not constructed in the prototype; therefore, data are not available for comparison.

c. The 1985 hydraulic model study to evaluate various configurations of the small boat harbor using long waves as input determined that harbor oscillations would occur at periods between 100 and 150 sec in the small boat harbor. These resonant modes are consistent with the prototype data measuring oscillations occurring at approximately 110, 125, and 132 sec.

d. A comparison of the sea-swell significant wave heights from the deep-water buoy and the slope array determined a correlation of 0.95; therefore, the sea-swell conditions in the nearshore at Barbers Point can be accurately estimated with data from the offshore buoy.

e. Long-period modes of harbor oscillation were identified both prior to and after inclusion of the small boat harbor. The resonant peaks prior to inclusion of the harbor occurred at approximately 910, 132, 110, 70, 60, and 47 sec. After inclusion of the small boat harbor, resonant peaks occur at approximately 1,024, 630, 200, 167, 132, 125, 110, 85, and 57 sec. The 1,024-sec peak is the Helmholtz mode of the deep-draft harbor, and the 630-sec peak is the Helmholtz mode of the small boat harbor.

f. Comparison of the infragravity significant wave heights measured inside the harbor with those measured at the slope array shows a high correlation between significant wave height inside and outside the harbor. It can be concluded that an increase in harbor seiche is associated with an increase in swell energy outside the harbor. Therefore, nonlinear processes that transfer energy from swell waves to infragravity waves outside the harbor are clearly an important mechanism for harbor resonance forcing at this location.

g. The high correlation between the harbor seiche and sea-swell wave heights rules out free long waves generated from distant sources as an important forcing mechanism at Barbers Point, since these free waves are not necessarily coincident with energetic sea and swell.

h. The rubble-mound wave absorber effectively reduces the wave energy inside the harbor for wind-wave periods of 20 sec or less. The wave absorber is less effective in decreasing wave energy for longer waves with periods of 50 sec or greater.

i. Removing the wave absorber will increase wave heights at some locations inside the harbor by an estimated 125 percent. Analysis indicates that the wave absorber decreases the reflection coefficients up to 50 percent.

j. Overall, the comparison is good between the prototype measurements and the numerical and physical model predictions of the resonant modes of oscillation. The numerical model, which was simulated both prior to and after inclusion of the small boat harbor, was consistent with the prototype measurements in predicting the shift of the Helmholtz mode and the appearance of additional peaks with the inclusion of the small boat harbor. The physical model did not resolve the long-period modes because of the length of simulations; however, the model accurately predicted the remaining resonant modes occurring in the harbor. Numerical model magnitudes of amplification were consistent with the prototype amplifications since the model was calibrated to the measurements using bottom friction. The physical model magnitudes varied from the prototype depending on the wave period.

k. Numerical model strengths include: (1) ease of model setup and modifications; (2) availability of data throughout the modeled harbor grid that permits visualization of the wave response over the entire gridded region; (3) quick response time; and (4) less cost to run the model. Limitations include simulation with unidirectional regular waves without directional spreading effects, neglect of nonlinear effects, and lack of good reflection coefficient and bottom friction data for accurately calibrating the model.

l. Physical model strengths include the ability to simulate: (1) directional wave spectra; (2) nonlinear wave transformation as waves travel into harbors; (3) reflection, transmission, and overtopping of structures; (4) dissipation because of bottom friction within scale and depth limitations; (5) currents; and (6) navigation studies with model ships. Limitations are mainly the result of the cost to construct and modify models and to collect data.

m. Long-period modes (resonance) cannot be effectively damped out once a harbor is constructed. A model investigation of resonant modes should be carried out before final project planning to ensure that the constructed harbor does not have unacceptable resonant modes of oscillation.

Agat Harbor, Guam

Agat Harbor (Figure 5) is located on the western side of the island of Guam. Agat is fringed by coral reefs characterized by a broad, shallow flat with a near-uniform depth of 0.3 m (1 ft) mean lower low water (mllw) (nearly exposed at low tide) that extends about 1 km (0.6 mile) offshore. The face of the reef is live

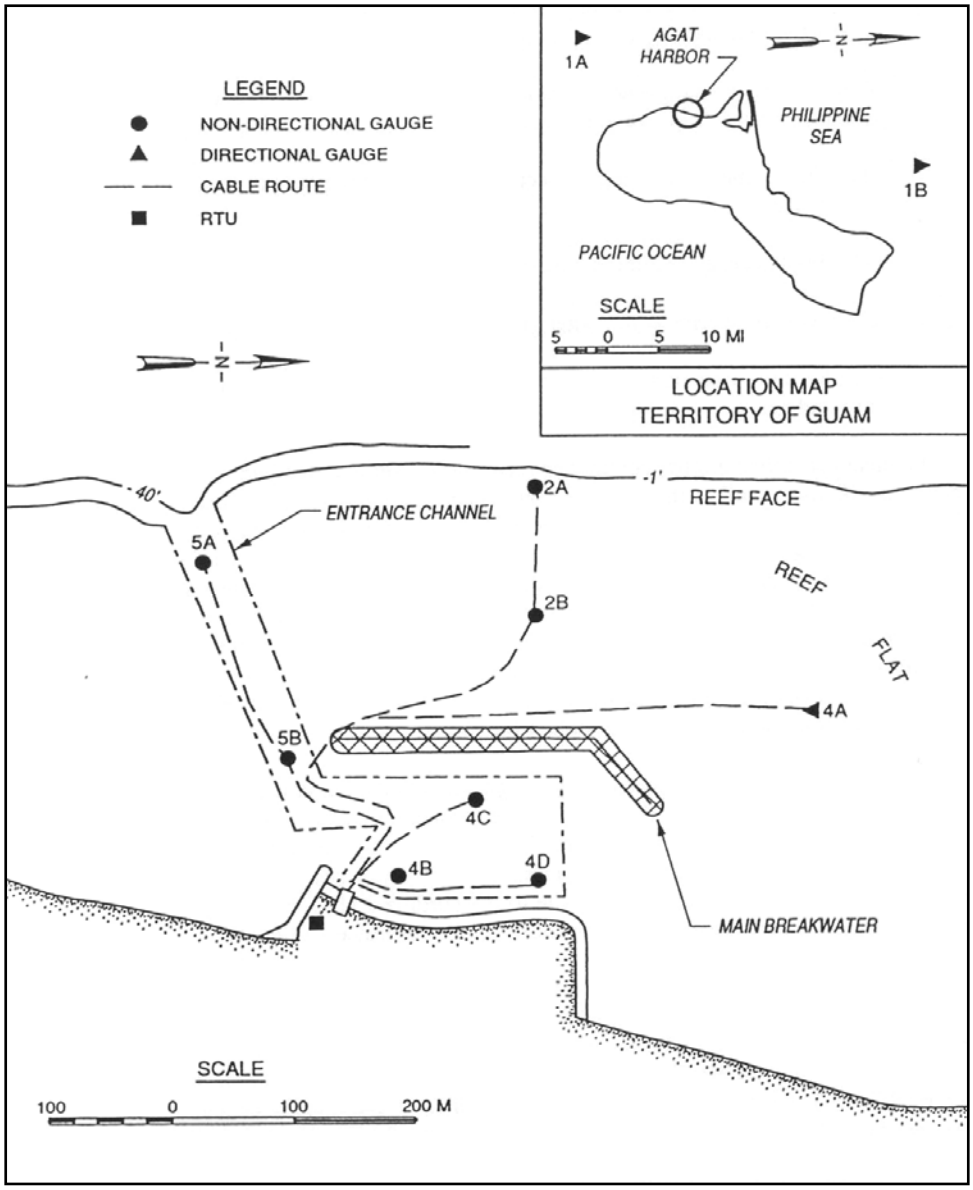

Figure 5. Agat Harbor, Guam (after McGehee and Boc 1997)

coral with a near vertical slope down to approximately -6 m (-20 ft). The face is an excellent dissipater of wave energy.

Item monitored

Wave transformation.

Period monitored

February 1991 – April 1994 monitoring.

Reason(s) for monitoring

Monitoring was performed during the time period February 1991 – April 1994 to determine wave transformation across coral reefs, wave and surge levels behind coral reefs, wave transformation down steep-sided channels, wave-induced circulation on a flat reef, response of the project and adjacent shorelines, and to validate the Harbor Shallow Water (HARBS) model. Little engineering data exist relative to design guidance for wave characteristics and surge levels on coral reefs (McGehee and Boc 1997).

Results of the February 1991 – April 1994 monitoring

Details of the monitoring are:

a. Most hydrodynamic data obtained during the monitoring effort represented mild conditions. Therefore, some of the quantitative objectives of the study were not met because of the lack of data during the rare high-energy events.

b. Wind waves dissipate most of their energy in breaking at the reef face. Wave energy propagates across reef flats as bores, moving water shoreward, that returns seaward through breaks in the reef face. Agat Harbor and its entrance channel provided such a pathway.

c. Wave heights on the reef flat do not increase appreciably as wave height offshore increases, but the amplitude of seiche of the entire reef is affected by incident energy. Wave groups (surf beats) with periods near the principal seiche modes of a reef flat may induce harmonic coupling.

d. The combination of seiche, return flow from wave setup, and mass transport of bore-like waves can result in large currents running parallel to shore. For structures located on the reef flat, forces from the resulting currents may be of larger magnitude than forces from the wind waves themselves.

e. Insufficient wave data were obtained from the sensors inside the harbor to validate the HARBS model. Peak period on the reef flat bears little resemblance to the incident wave period. Long-period waves dominated the signal. The model was conducted for wind waves in the 8- to 20-sec range; however, wave periods measured in the harbor were much longer (100 to 200 sec).

f. Insufficient wave data were obtained to determine wave transformation down steep-sided channels. During simultaneous operation of both sensors in the channel (for correlation), wave conditions were always low.

g. The detached breakwater design promotes flushing of the harbor but can result in a significant influx of sediment during high-current events.

h. There is no indication that wind waves on a reef flat will exceed the depth limited breaking criteria used for sloping beaches. The highest wave height to water-depth ratio is about 0.72, slightly lower than the

0.78 breaking wave criteria used in design. However, this energy-based significant wave height includes all of the low-frequency energy as well and is really associated with the seiche amplitudes. The height of the highest wind waves on the reef flat, a figure needed in calculating stone stability, will probably not even exceed one-half the water depth, as long as the water depths are shallow. However, as the water depth increases as a result of surge, the breaking wave height limit will increase. Without verification of a lower breaking limit under typhoon conditions, the standard depth-limited criteria should be retained for design.

i. No measurement of surge levels during typhoons approaching from the west was obtained that exceeded the initial design estimate of 1.4 m (4.6 ft). Estimates of surge from measurements or models of planar beaches are unlikely to apply. Some information on the wave-induced setup is available from laboratory studies. Data from a two-dimensional (2-D) physical model study of a reef-type profile are compared to numerical predictions of wave height and water level behind the reef. Setup on the reef flat on the order of 10 percent of the incident wave height was predicted for cases typified by the prototype measurements. (Though the reef profile modeled is described as representing Agat Harbor, the bathymetry is dissimilar enough from the prototype that detailed comparisons with data in this report are not likely to be productive.) For the 9.8-m (32-ft) incident waves measured during Typhoon Russ, wave setup of about 1 m is likely, in addition to atmospheric effects. In any case, wind waves propagating shoreward are not the only, and maybe not even the predominant, environmental loading for structures on reef flats. The physical model simulated the low-frequency energy observed on the reef flat, and predicted heights on the order of one-fourth to one-half the incident wind wave height. Forces on structures resulting from the currents associated with these long waves should be considered as well as wave forces.

j. The shortest path (hydraulically) for the return flow to take is toward the ends of the reef flat, where breaking and setup are not occurring. Since the harbor is connected to deep water by the entrance channel, the low water level is brought conveniently close (from the return flow's perspective). If just one-third of the return flow takes this shortcut through the harbor and entrance channel back to sea, velocities across the l00-m-(330-ft-) wide opening would be on the order of 1 m per sec (3.3 ft per sec). This is sufficient to balance the out-of-phase flow from the seiche, and double the in-phase flow, resulting in a pulsing flow of up to around 4 knots (2.5 miles per hr). This is a little less than observed, but no allowance has been made for the setup return flow. Highest velocities would occur where the gradient is steepest, which is near the shoreward side of the harbor basin. This pattern could explain the displacement of the toe stone at the northwest corner of the basin, an area exposed to the highest velocity currents flowing into the harbor.

k. The sediment that entered the harbor came from the veneer of sand that is evident in many places overlaying the old coral on the reef flat. It was transported there by the currents flowing through the harbor that acts as an effective settling basin. Given the evidence of significant offshore

sediment transport through the natural pathways, this process will continue for the current harbor configuration. Since the transport is episodic, it is impossible to predict the short-term rate of influx. If it is a persistent problem, alternative geometries that would reduce influx of sediment while maintaining the desirable flushing characteristics could be investigated.

Ofu Harbor, American Samoa

Ofu Harbor (Figure 6) is located on Ofu Island in American Samoa, a group of seven islands (five volcanic islands and two coral atolls) located in the South Pacific Ocean. They are located about 6,700 km (4,150 miles) southwest of San Francisco, CA, and about 3,700 km (2,300 miles) south-southwest of Hawaii.

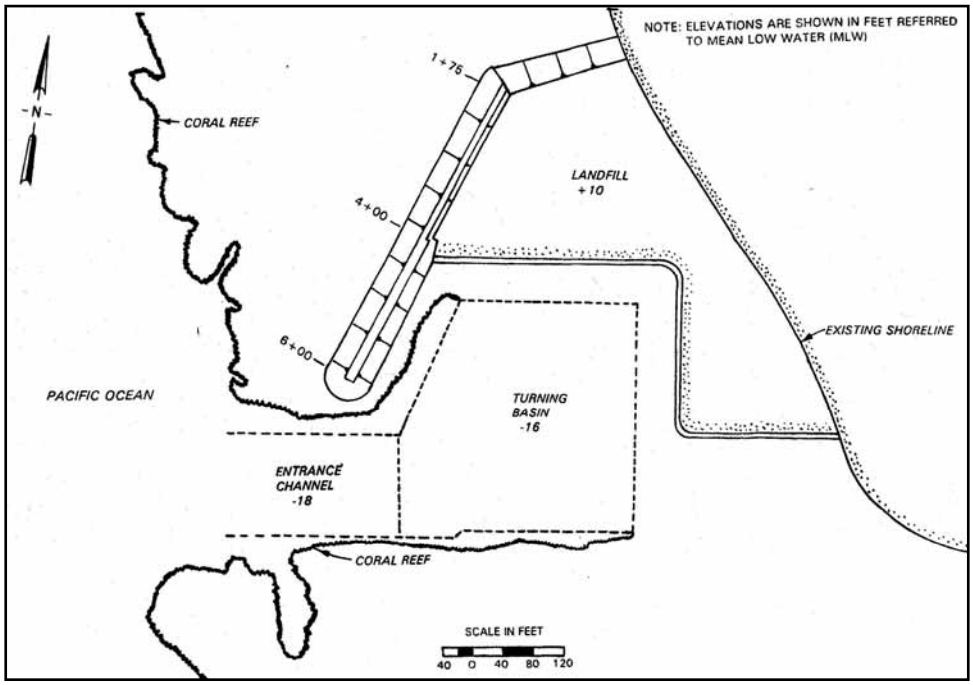

Figure 6. Ofu Harbor, American Samoa (after Bottin and Boc 1997)

Item monitored

Breakwater.

Period monitored

June 1997 and August 2002 periodic inspections.

Reason(s) for monitoring

Base conditions for future periodic inspections were determined in June 1997, and the first periodic inspection was conducted in August 2002. The purposes of the periodic inspections are to: (a) develop methods using limited land-based surveying, aerial photography, and photogrammetric analysis to assess the long-term stability response of the concrete armor units on the Ofu Harbor breakwater; and (b) conduct land surveys, armor unit inspection, aerial photography, and photogrammetric analyses to test and improve developed methodologies and accurately define armor unit movement above the waterline (Bottin and Boc 1997; Bottin and Meyers 2003; Bottin 2003a).

Results of the June 1997 periodic inspection

Details of the inspection are:

a. The breakwater was constructed in 1994 by using various-sized concrete units for construction material instead of basalt stone. Unique concrete underlayer units consisting of 1,634-kg (1.8-ton) units with 0.4-m-(1.3-ft-) diam holes to dissipate wave energy were used. Concrete underlayer units weighing 454 and 2,270 kg (0.5 and 2.5 tons) were also formed by pumping high-strength fine-aggregate concrete into geotextile fabric bags. The breakwater armor consisted of a single layer of uniformly placed 4,086-kg (4.5-ton) concrete tribar units. To improve the stability of the tribars, work included the construction of a toe trench to stabilize the armor unit toe and a concrete rib cap system on the breakwater crest to stabilize and buttress tribars at the upper sea-side and harbor-side slopes. The rib cap forms were fabricated and concrete poured into the top section of the tribars.

b. Ofu Harbor is subjected to severe storm conditions in the South Pacific, including tropical storms, hurricanes, and cyclones. The original revetment and mole used for harbor protection was damaged several times, and in 1991, was almost completely destroyed. As a result, a new breakwater was constructed in 1994 which included the use of 4,080-kg (4.5-ton) concrete tribar armor units. Various concrete underlayer units were also used in the structure, since local stone was not available. No sound, quantifiable data relative to the movement or positions of the concrete armor units had been obtained for the structure prior to this study.

c. Under the Periodic Inspections work unit, data from limited ground-based surveys, aerial photography, and photogrammetric analysis were obtained to establish base level conditions for the Ofu Harbor breakwater. Logistical problems were encountered attempting to obtain low-altitude aerial photography in this remote location. The planned low-altitude photography was not obtained; however, oblique images taken from a fixed-wing aircraft were analyzed using convergent photogrammetric techniques, which proved to be acceptable. Accuracy of the photogrammetric analysis was validated and defined through comparison of ground and aerial survey data on control points and targets established

on the structure. The procedure utilized the oblique images, a stereoplotter, and Intergraph-based software to analyze the entire above-water armor field and quantify armor positions. A detailed walking survey of the structure conducted during the effort resulted in a well-documented data set that can be compared to subsequent surveys.

d. Now that base (control) conditions have been defined at a point in time and a methodology has been developed to closely compare subsequent years of data for the Ofu Harbor breakwater, the site will be revisited in the future under the Periodic Inspections work unit to gather data by which assessments can be made on the long-term response of the structure to its environment. The insight gathered from these efforts will allow engineers to decide, based on sound data, whether or not closer surveillance and/or repair of the structure might be required to reduce its chances of failing catastrophically. Also, the periodic inspection methods developed and validated for this structure may be used to gain insight into other Corps structures.

Results of the August 2002 periodic inspection

Details of the inspection are:

a. Low-altitude photography was obtained and the accuracy of the photogrammetric analysis was validated and defined through comparison with ground survey data on control points and targets established on the structure. A procedure using high-resolution, stereo-aerial photographs, a stereoplotter, and MICROSTATION-based software was developed to analyze the entire above-water armor unit fields and quantify armor positions. A detailed walking survey of the structure conducted during the effort also results in a well-documented data set that can be compared to previous and subsequent surveys.

b. Aerial survey data were compared with the June 1997 ground data. An analysis of these data indicates negligible movement of the concrete armor units on Ofu Harbor breakwater. Maximum movement of the targets established on the tribar armor units in the horizontal and vertical direction, respectively, were 0.14 m (0.45 ft) and 0.11 m (0.35 ft); and the average movement of all horizontal and vertical targets were 0.01 m (0.04 ft) and 0.02 m (0.07) ft. Maximum movement of the targeted armor unit centroids were 0.09 m (0.29 ft) and 0.07 m (0.24 ft) in the horizontal and vertical directions, respectively, while average movements were 0.01 m (0.04 ft) and 0.02 m (0.08 ft) in the horizontal and vertical directions.

c. The current walking inspection revealed more widespread separations between the “cheese block” concrete underlayer units relative to the 1997 inspection; however, most of the separations were less than 0.09 m (0.3 ft). No armor unit breakage was noted. Overall, the structure appeared to be in excellent condition.

3 Site-Specific Lessons Learned, Alaska

St. Paul Harbor, Alaska

St. Paul Harbor (Figure 7) is located in a cove on the southern tip of St. Paul Island, and is the island's only settlement. St. Paul Island is the northernmost and largest island of the Pribilofs in the eastern Bering Sea.

Figure 7. St. Paul Harbor, Alaska (after Bottin and Jeffries 2001)

Items monitored

Harbor and breakwater.

Period monitored

July 1993 – June 1996 monitoring, and August 2000 periodic inspection.

Reason(s) for monitoring

Monitoring was conducted during the period July 1993 – June 1996 to determine if the harbor and its breakwater structure were performing (both functionally and structurally) as predicted by model studies used for the project design, and to develop base conditions for future periodic inspections. The first periodic inspection was conducted during August 2000 (Bottin and Eisses 1997; Bottin and Jeffries 2001; Bottin 2003b).

Results of the July 1993 – June 1996 monitoring

Details of the monitoring are:

a. When working in high-energy wave environments at remote locations, extra precautions must be taken to ensure that wave data are collected. The loss of two-directional wave gauges outside the harbor significantly reduced the value of some of the other data obtained. The wave data were required for correlation with other monitoring elements. Devices hard-wired to shore (to obtain real-time data) and/or other appropriate measures to improve the probability of success should be included in project budgets. In the future, in-depth research of conditions should be conducted to ensure success.

b. Photogrammetric analysis of the main breakwater proved to be an excellent tool in mapping the above-water portion of the structure and quantifying changes in elevation. Results revealed most of the breakwater is below its design elevation. Almost one-third of the structure adjacent to the harbor roadway is at least 0.6 m (2 ft) below its design elevation of +l1.3 m (+37 ft). Analysis also indicated essentially no change in elevation of the breakwater during the monitoring period.

c. When monitoring projects in remote areas, logistical problems may be experienced. Delivery dates and/or availability of equipment, supplies, materials, etc. are uncertain, and shipping costs are significantly higher. In most cases, equipment and supplies required are not available locally, and must be shipped from the mainland. These problems should be considered during the development of future monitoring plans in remote locations. Additional time and costs associated with these problems also should be considered.

d. Failure to obtain incident wave data outside the harbor had a negative impact on analysis of some of the other data collected during the monitoring effort. Incident wave data were required for correlation with wave data obtained inside the harbor, wave runup, and wave overtopping data to validate design methods and procedures.

e. Wave height data obtained inside the harbor appeared to validate the three-dimensional (3-D) model study. Maximum significant wave heights measured in the immediate lee of the main breakwater during storm wave events were in agreement with those predicted during the physical model study.

f. The videotape analysis used to obtain wave runup data along the face of the St. Paul Harbor main breakwater was successful, except during periods of low visibility. The technique is relatively low cost, logistically simple, and provides relatively accurate measurements.

g. Trends in wave hindcast data obtained outside the harbor (to define incident wave conditions) correlated reasonably well with runup data in a qualitative sense (i.e., larger wave heights correlated with higher runup and smaller wave heights with low runup). The absolute values of the hindcast significant wave heights, however, appeared to be substantially lower than the waves experienced in the prototype based on runup values measured, overtopping observed, and local forecasts.

h. Since construction of breakwater improvements, a scour hole has formed at the head of the main breakwater extension, sediment has accumulated north of and adjacent to the detached breakwater (forming an underwater spit that is migrating toward the entrance channel), and sediment has moved into the harbor between the detached breakwater and the shoreline. To this point, the scour hole has not impacted the structure's stability, nor has the underwater spit interfered with navigation. Accretion inside the harbor has not occurred in the Federal channel or mooring areas. Sediment patterns in the harbor, as predicted by the 3-D model, were validated by the prototype data.

i. The St. Paul Harbor main breakwater is currently functioning in an acceptable manner and is in good condition structurally; however, the armor stone continues to degrade. The number of broken/cracked armor stones on the 320-m-long (1,050-ft-long) breakwater extension increased from 73 in July 1993 to 230 in June 1996. A geologic assessment indicated that about 25 percent of the original stone placed was geologically unacceptable, and a significant amount of the stone on the structure was blast damaged. Continued deterioration is predicted because of the freeze-thaw and wet-dry cycles, as well large waves and sea-ice action. The structure should be monitored very closely, since the rate of deterioration is expected to increase. Inspection of 100 percent of shot stone for near-invisible hairline blast fractures also should be conducted by skilled personnel. In future construction, the highest grade of geologically acceptable stone should be placed above the waterline in this extremely harsh environment.

Results of the August 2000 periodic inspection

Details of the inspection are:

a. To minimize further breakwater damage and reduce overtopping of the main breakwater, the construction of submerged reef breakwaters seaward of the structure was initiated during the calendar year 2000 construction season. The current monitoring was conducted to determine changes in the armor unit field since the previous study and establish new base conditions since construction of the reef breakwaters.

b. The current monitoring entailed reestablishing targets and conducting limited ground-based surveys, aerial photography, and photogrammetric analysis of the St. Paul Harbor main breakwater for comparison against conditions obtained in 1996. The entire above-water armor unit field was analyzed and quantified through the use of high-resolution, aerial stereo-pair photographs, a stereoplotter, and Intergraph-based software. A detailed broken armor unit survey also was conducted during the current effort and compared to previous survey data.

c. Results of this periodic inspection indicated essentially no change in the overall breakwater crest elevation and shape of the structure since the 1996 survey. Although still below design elevation, the structure has not, in general, settled or subsided to any great extent. There are localized areas in the breakwater, however, where voids have occurred (likely because of the displacement of armor stones). Voids were noted on both slopes of the structure as well as the breakwater crest.

d. A total of 221 broken armor stones was documented during the year 2000 survey versus 230 in 1996. Analysis indicated that 33 broken stones, documented in the 1996 survey, could not be found during 200, suggesting they may have been moved away by wave and/or ice action. The rate of stone breakage appears to have declined. Only 24 new broken armor stones occurred in the past 4-year period versus 157 broken stones that occurred during the 1993 – 1996 time frame. Voids resulting from displaced stones were visually observed in localized areas of the breakwater during the broken stone inventory.

e. The main breakwater is currently functioning in an acceptable manner, with the exception of the excessive overtopping, and is considered to be in good condition structurally. Construction of the three offshore sub-merged reefs seaward of the breakwater should provide additional protection from further wave-induced damage and reduce overtopping. Subsequent inspection should be conducted to analyze the performance of the improved project. It is recommended that additional armor stone be placed in some of the apparent voids in the breakwater along the reef construction, particularly the large void between stations 8+80 and 9+70, where core stone is exposed.

4 Site-Specific Lessons Learned, Pacific Coast of the U.S. Mainland

Port of Friday Harbor Marina – Puget Sound, Friday Harbor, Washington

The 580-boat marina at Friday Harbor (Figure 8) is located on the eastern shore of San Juan Island on the inland waters of northwestern Washington, about 50 km (32 miles) east of Victoria, British Columbia, and 110 km (69 miles) north of Seattle, WA. The 488-m- (1,600-ft-) long floating breakwater was constructed and installed by the Corps of Engineers in 1984.

Figure 8. Port of Friday Harbor, San Juan Islands, Washington (after Nelson and Hemsley 1988)

Item monitored

Floating breakwater.

Period monitored

January 1984 – July 1986 monitoring.

Reason(s) for monitoring

Onsite data were obtained during the time period January 1984 – July 1986 pertaining to the performance and durability of the floating breakwater. Operational experiences such as recreational use, transient moorage difficulties/ preferences, and wave/wake transmission, diffraction, and reflection problems were documented (Nelson and Hemsley 1988).

Results of the January 1984 – July 1986 monitoring

Details of the monitoring are:

a. Tides at Friday Harbor are typical of those along the Pacific coast of North America, ranging from the lowest ever recorded at -1.2 m (-4 ft) mllw to +3.4 m (+11 ft) mllw. Water depth at the site varies between 12 and 15 m (40 and 50 ft). Maximum current velocities are northerly at less than 0.5 m per sec (1.5 ft per sec) during spring ebb tide. Currents are less than 0.3 m per sec (1.0 ft per sec) during flood tide and are southerly. Winter storms can produce winds in excess of 80 knots (50 miles per hr) from the northeast. Design wave conditions exhibit a significant wave height H_s of 1.0 m (3.2 ft) and period T of 3.2 sec from the northeast, and H_s of 0.8 m (2.7 ft) and T of 2.6 sec from the southeast.

b. The breakwater consists of five rectangular concrete pontoons, three of which are 100 m (330 ft) long by 6.4 m (21 ft) wide by 1.8 m (6 ft) high. Two pontoons are 4.9 m (16 ft) wide by 1.7 m (5.5 ft) high. Breakwater anchors are 52 steel H-piles embedded their full length. Anchor lines consist of 3.5-cm- (1-3/8-in.-) diam galvanized bridge rope with 9.1 m (30 ft) of 3.2-cm (1-1/4-in.) stud-link chain at the upper end. Anchor-line lengths were sized to provide a scope of 4:1 to 5:1. A 908-kg (2,000-lb) concrete clump weight is attached approximately 15 m (50 ft) from the upper end of each anchor line. Anchor-line initial tension is approximately 4,540 kg (10,000 lb). Three large aluminum anodes were attached to each anchor line to prevent corrosion.

c. The only damage to the breakwater itself occurred shortly after completion of construction as a result of a collision with a large 33-m (110-ft) steel pleasure boat. A small piece of concrete on a corner of the C-float was broken off, exposing some of the structure's reinforcing steel. The crude epoxy patch that was used to cover the damaged area remains intact and appears to be successfully protecting the underlying steel.

d. During original planning, it was determined that no vessels should be moored on the seaward side of the breakwater. Loads for the original breakwater and anchor system design included no allowance for

additional loading because of vessels moored on the seaward side of the breakwater.

e. After project completion, the Port found that there was a considerable demand for moorage on the seaward side, particularly for large 23-m (75-ft) vessels. Additional loads that could be generated if large vessels were moored on the seaward side of the breakwater were calculated and were well within the allowable design criteria. However, no adequate tie-up facilities are available on the seaward side of the breakwater, which has proven to be a popular fishing pier.

f. Maximum longitudinal motion (north-south) was about 0.8 m (2.5 ft), and maximum lateral motion (east-west) was about 15 cm (6 in.). North-south motion was probably increased by the sail effect of larger vessels temporarily moored to the floats. Several minor storms from the south did not produce particularly large waves but did have winds in excess of 40 mph and may have been responsible for the maximum north-south excursions.

g. An underwater inspection of a portion of the anchor lines was made to assure that the anticorrosion system was working properly. All the fittings, cable, and chain appeared to be in excellent condition. Surface corrosion of the aluminum anodes had begun as expected. One construction discrepancy was discovered at the joint between the two large 6.4-m- (21-ft-) wide floats. Lengths of the seaward and landward anchor lines that cross under the float had been adjusted during construction. Here, both the landward and seaward anchor lines from the two floats join a common anchor line forming a “y.” The adjustment resulted in the anchor lines rubbing against each other. While damage to the anchor lines did not appear to be particularly serious, a delay of remedial action would have resulted in continual wear and eventually in failure of one or both anchor lines. The repair involved releasing the seaward anchor line from the breakwater, lowering it under and around the landward anchor line, then reconnecting and retensioning it.

h. After 2.5 years of operation, the Port has experienced several persistent maintenance problems. Access and interfloat ramps are only 1.2 m (4 ft) wide. This width precludes access to the breakwater by electrically powered vehicles. The Port would like to use such vehicles to reduce travel time for the 0.8-km (0.5-mile) round trip to the end of the breakwater.

i. Stanchions, located on the breakwater to supply electrical service to transient boats, are relatively tall. Their height, combined with their placement near the edge of the breakwater, makes them vulnerable to being knocked over by bowsprits of docking boats. This particular stanchion design is also prone to being pulled over by boaters who neglect to unplug their shore power lines before departing.

j. Electrical junction boxes present another problem. The boxes are mounted flush with the deck, so they fill with water unless access plates are carefully sealed. Much of the hardware that provides mechanical support for the electrical wiring was not designed specifically for use in a

marine environment; consequently this hardware is now badly corroded and eventually will have to be replaced.

k. A relatively minor but persistent problem involves the bull rail. Blocks supporting the bull rail are held in place with only one bolt. Some of the blocks have rotated and present a hazard to boats tied up alongside. Finally, the Port is aware of at least one incident in which a person fell off the breakwater and had to swim to an adjacent dock because he was unable to pull himself up over the bull rail and onto the deck of the breakwater. Plans are underway to install life rings and possibly add safety ladders at various locations along the breakwater.

l. Virtually no wear or damage was noted on the fenders separating the floats. All readily measurable dimensions were unchanged, and, except for minor corrosion, all hardware and fasteners were in excellent condition.

University of Washington Oceanographic Laboratory – Puget Sound, Friday Harbor, Washington

The floating breakwater at the University of Washington Oceanographic Laboratory (Figure 9) is about 1 km (0.5 miles) north of the Port of Friday Harbor. The site has an open fetch to the east of about 6.5 km (4 miles).

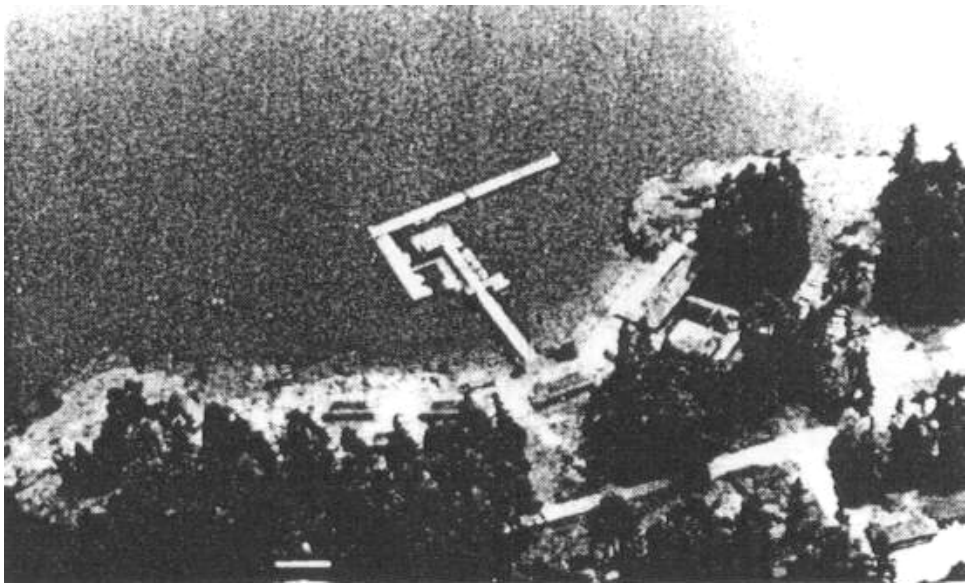

Figure 9. University of Washington Oceanographic Laboratory, Friday Harbor, San Juan Islands, Washington (after Nelson and Hemsley 1988)

Item monitored

Floating breakwater.

Period monitored

1979 – 1985 monitoring.

Reason(s) for monitoring

Onsite data were obtained during the time period 1979 – 1985 pertaining to the performance and durability of the floating breakwater. Operational experiences such as recreational use, transient moorage difficulties/preferences, and wave/wake transmission, diffraction, and reflection problems were documented (Nelson and Hemsley 1988).

Results of the 1979 – 1985 monitoring

Details of the monitoring are:

a. Tide conditions are the same as for the floating breakwater at Friday Harbor, but the site is more exposed to the east. Design parameters were a 46-knot- (28-mph-) wind fetch-limited significant wave height of 0.8 m (3.0 ft), a period of 3.5 sec, and a current of 1.5 knots (0.9 mph). Boat wakes up to 0.6 m (2 ft) are common. Water depth varies between 3 and 18 m (10 and 60 ft).

b. Installed in 1979, the breakwater is a reinforced concrete caisson cast over a polystyrene foam core with a cross section of 1.4 by 4.6 m (4.5 by 15 ft), and a design freeboard of 0.5 m (1.5 ft). It is L-shaped with two 40-m (130-ft) sections on the long leg parallel to the east-west shore and a third 40-m (130-ft) section on the short north-south leg. The anchor system is laid out to maintain a 1.8-m (6-ft) space between the sections to avoid linkage and impact problems. Short gangways provide access between units. The breakwaters are used as staging areas for handling nets and other gear, as well as to provide a protected mooring area.

c. Each float is independently anchored by 2.54-cm- (1-in.-) diam stud-link chain anchor lines that are attached to the four corners of each section. Each corner line is oriented at a 45-deg angle to the breakwater. Clump weights (2,721 kg (3 tons)) are attached to the anchor lines except the landward line on the north-south leg. Because bottom conditions at the site consist of a shallow covering of sand over bedrock, only gravity anchors were considered. The main anchors are 2.4- by 2.4- by 1.8-m (8- by 8- by 6-ft) concrete blocks.

d. On 11 February 1985, a southwesterly storm with winds estimated at 55 to 65 knot (35 to 40 mph) caused one of the landward anchor lines to part. The last link of the chain broke at the upper connection point.

Inspection revealed, additionally, that undersized shackles had been used to connect the stud-link chain to the breakwater connection flange, and severe pitting of the 7-year-old chain was evident, particularly in the upper 3 m (10 ft). The broken line was the shortest of all the anchor lines, and, because it was located in an area that became very shallow at low tide, no clump weight was attached. All anchor lines were replaced in the spring of 1985.

e. Zinc anodes were attached at various places along the new anchor chains in an attempt to reduce the rate of corrosion. An unrelated but interesting note is that numerous large blocks at 180-kg (400-lb) displacement) of styrofoam are fastened under the breakwater because it initially had insufficient freeboard. Apparently, the concrete thickness tolerances were exceeded during the breakwater construction, resulting in excessive structure weight.

East Bay Marina – Puget Sound, Olympia, Washington

The floating breakwater at East Bay Marina, Olympia, WA (Figure 10), is located at the southernmost terminus of Puget Sound, approximately 145 km (90 miles) south of Seattle.

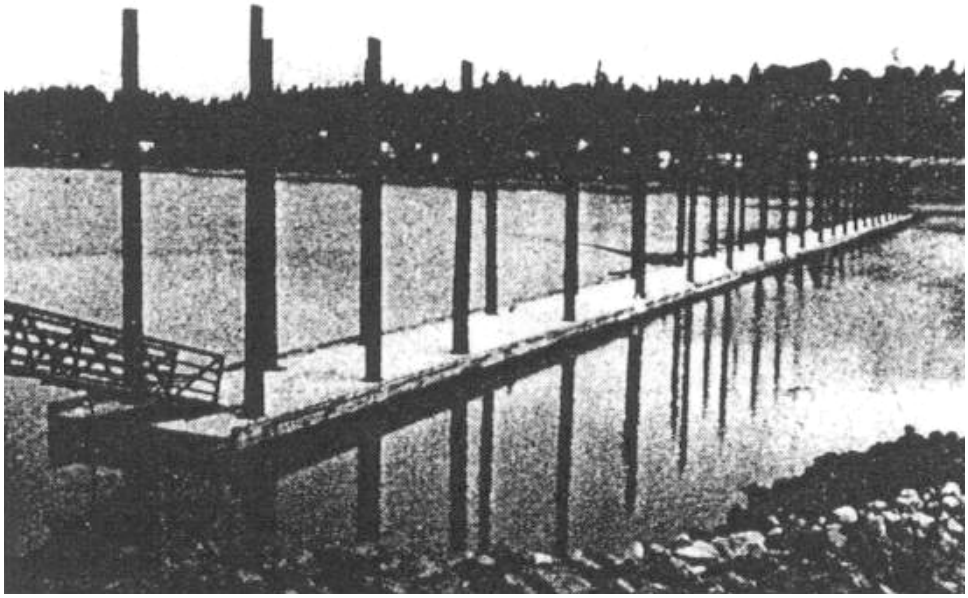

Figure 10. East Bay Marina, Puget Sound, Olympia, WA (after Nelson and Hemsley 1988)

Item monitored

Floating breakwater.

Period monitored

1981 – 1984 monitoring.

Reason(s) for monitoring

Onsite data were obtained during the time period 1981 – 1984 pertaining to the performance and durability of the floating breakwater. Operational experiences such as recreational use, transient moorage difficulties/preferences, wave/wake transmission, diffraction, and reflection problems were documented (Nelson and Hemsley 1988).

Results of the 1981 – 1984 monitoring

Details of the monitoring are:

a. Tidal range here varies from a lowest recorded -1.5 m (-5 ft) mllw to a highest recorded +5.5 m (+18 ft) mllw. The marina site is exposed to wind waves generated from the northwest through northeast directions. Design wave height at the breakwater is a 0.6-m (2.0-ft) significant wave with a period of 2.8 sec from the north-northwest.

b. The breakwater consists of seven rectangular concrete modules, 30 m (100 ft) long by 4.9 m (16 ft) wide by 1.7 m (5.5 ft) deep. Module walls are 12.7 cm (5.0 in.) thick with welded wire reinforcing, and each module is longitudinally posttensioned. The breakwater is held in place by timber anchor piles driven 6.1 m (20 ft) into the medium-dense sands below the bay muds. Modules are connected by large rubber fenders bolted between adjacent units. Dredging was required under the breakwater to a depth of -3.7 m (-12 ft) mllw to prevent the structure from striking bottom at extreme low tides and to provide keel clearance for boats at or near the breakwater.

c. No northerly winds of any significance occurred at the East Bay site between construction and monitoring, and no damage had been noted on the breakwater after 3 years of operation. A potential problem, pointed out by the concessionaire operating the marina for the Port of Olympia, was that the holes through which the pilings passed were large enough to allow a child to fall between the piling and the float. As a temporary solution, plywood rings were placed over the pilings. Another problem which did not affect the breakwater but did affect all of the access floats within the marina was that during one period of extreme cold, numerous waterlines ruptured because of either differential expansion between the floats and polyvinyl-chloride waterlines or the freezing of trapped water. Waterlines on the breakwater were enclosed within the float and were not damaged.

Zittle's Marina – Puget Sound, Johnson Point, Washington

Zittle's Marina (Figure 11) is located at Johnson Point, near the southern end of the Puget Sound, Washington.

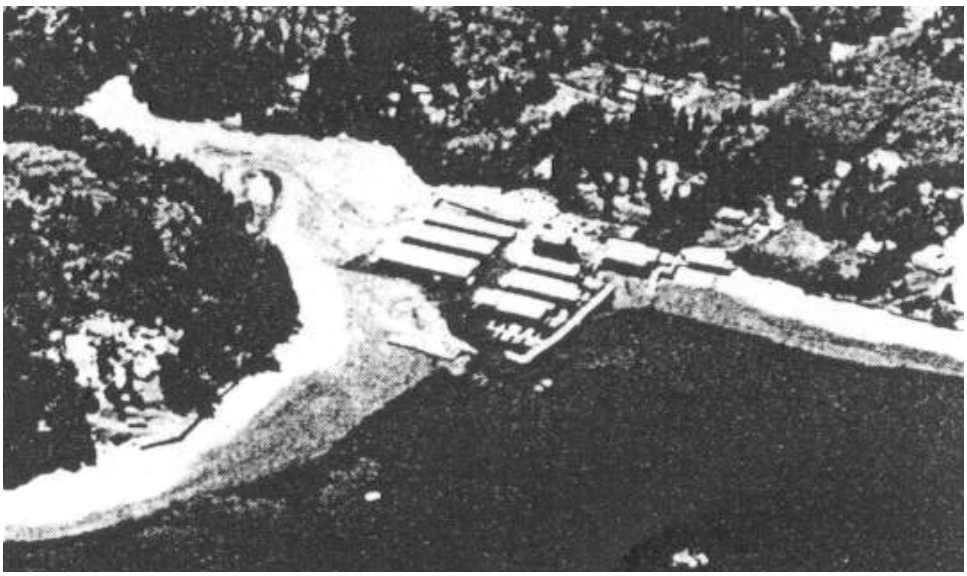

Figure 11. Zittle's Marina, Puget Sound, Johnson Point, Washington (after Nelson and Hemsley 1988)

Item monitored

Floating breakwater.

Period monitored

1983 – 1986 monitoring.

Reason(s) for monitoring

Onsite data were obtained during the time period 1983 – 1986 pertaining to the performance and durability of the floating breakwater. Operational experiences such as recreational use, transient moorage difficulties/preferences, and wave/wake transmission, diffraction, and reflection problems were documented (Nelson and Hemsley 1988).

Results of the 1983 – 1986 monitoring

Details of the monitoring are:

a. The pipe-tire breakwater at Zittle's Marina is a matrix of 40-cm- (16-in.-) diam pipes and truck tires held together with conveyor belting. It was constructed by the Seattle District as part of the Floating Breakwater Prototype Test Program. The breakwater was damaged as a result of faulty welds during the test and was surplused at the end of the test program. A local marina operator salvaged the breakwater, towed it to the marina, and repaired it. The marina site is approximately 24 km (15 miles) south of the East Bay Marina, and tides at this location are essentially the same as those given for East Bay. It is completely protected from all directions except an open area to the north with a fetch of about 3.2 km (2 miles). No estimate of wave heights at the site has been made; but because of the limited exposure, wave heights probably do not exceed 0.9 m (3 ft).

b. Since its installation at Johnson Point in 1983, the breakwater has sustained no damage; however, it has not been subjected to significant wave action (i.e., over 0.6 m (2 ft)). Even in this relatively mild environment, the marina operator feels that the breakwater performs a necessary function of providing protection from wave "chop" and boat wakes. The operator has made progress in his attempt to refurbish the pipe-tire breakwater by repairing the damaged portions and adding several sections. Flotation of the breakwater is about the same as when it was turned over to him in November 1983.

c. An attempt was made to remove and inspect foam flotation from approximately 10 tires, but the matrix of tires was so tightly bound that only one piece of foam was recovered. This piece was badly worn, weighing only 113 gm (4 oz) compared to an average of 540 gm (19 oz) initially. Of the tires that were inspected, about 50 percent had no foam at all, 25 percent had badly worn foam similar to the one that was recovered, and 25 percent had intact foam flotation. These results raise a question concerning the necessity for foam in the original design.

d. Observations made during the Prototype Test Program indicated that unfoamed tires tended to sink. These same tires now appear to have adequate flotation and are indistinguishable from the foam-filled tires. Several factors may contribute to this apparent contradiction. First, tidal currents were as high as 2 knots (1.2 mph) at the Prototype Test Program site. Resultant drag forces tended to pull the breakwater under and, once submerged, the tires may have lost their entrapped air. Since tidal currents are very low at Johnson Point, these forces are no longer at work. Second, the mild wave climate at Johnson Point probably leaves the trapped air undisturbed for longer periods of time, while the large waves at the test sites may have deformed the tires enough to allow loss of some trapped air. Third, during the summer, the breakwater is moored in shallow water where it goes aground at low tide, and the trapped air is replaced with the rising tide. There is little wave action at lower stages of tide, the bottom material is sandy with a little mud, and the sidewalls of the tires are high; therefore, very little sediment is trapped in the tires, and little, if any, weight is added as they sit on the bottom. Although the tires still float at approximately the same level as they did originally,

their ability to resist being submerged is considerably less than when originally constructed.

e. During the final inspection in November 1986, the tires between pipes would no longer support a person's weight. Apparently, without foam, the trapped air compresses as the tires are submerged, resulting in decreased buoyancy. If marginally buoyant tires were submerged deeply enough, they could become negatively buoyant. Therefore, in areas where tidal currents are high or wave heights are greater than about 0.5 m (1.5 ft), the necessity for including some type of incompressible flotation remains a requirement of conservative design.

f. On the two sections the marina operator added to the breakwater, creosote-treated logs were used in place of foam-filled steel pipes, and steel cable was used instead of conveyor belting to bind the tires together. None of the tires in the new sections had any foam flotation, but they were floating at about the same height as the older section.

Port of Brownsville Marina – Puget Sound, Brownsville, Washington

The Brownsville Marina (Figure 12) is located on the Kitsap Peninsula on the western margin of Puget Sound approximately 22 km (14 miles) west of Seattle, WA.

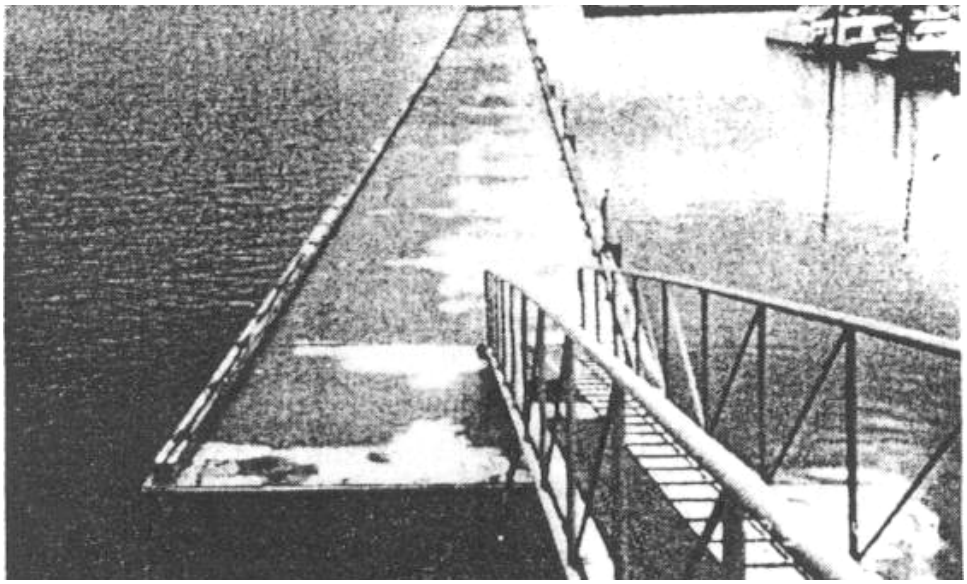

Figure 12. Brownsville Marina, Puget Sound, Brownsville, WA (after Nelson and Hemsley 1988)

Item monitored

Floating breakwater.

Period monitored

1981 – 1983 monitoring.

Reason(s) for monitoring

Onsite data were obtained during the time period 1981 – 1983 pertaining to the performance and durability of the floating breakwater. Operational experiences such as recreational use, transient moorage difficulties/preferences, and wave/wake transmission, diffraction, and reflection problems were documented (Nelson and Hemsley 1988).

Results of the 1981 – 1983 monitoring

Details of the monitoring are:

a. The maximum tide range at Brownsville Marina is about 6 m (19.5 ft). The breakwater, which provides protection from northerly waves (estimated H = 1.0 m (3.2 ft), T = 3.4 sec), was installed in 1981. It is a rectangular concrete pontoon 5.5 m (18 ft) wide and 1.5 m (5 ft) high, and is composed of 24 units, each 4.6 m (15 ft) long. Units are post-tensioned together to form a single 110-m- (360-ft-) long float. This float is moored in 3 to 6 m (10 to 20 ft) of water (at a 0.0-m (0.0-ft) tide) by stake piles, each attached to a 3.8-cm- (1.5-in.-) diam stud-link chain anchor line. No clump weights are attached to the anchor lines, but the oversized chain serves essentially the same purpose as clump weights. A north-south leg of the breakwater is exposed to much smaller waves from the south-east. It is composed of a series of 27 surplus U.S. Navy submarine net floats, each 3.7 m (12 ft) long and 1.8 m (6 ft) in diameter, and a 48-m- (157-ft-) long by 7-m- (23-ft-) wide landing craft ballasted to a 4.9-m (16-ft) draft. Floats and landing craft are ballasted with seawater. This makeshift portion of the breakwater is held in place by 7.6-cm- (3-in.-) diam nylon rope attached to the timber piles.

b. No damage or significant change occurred at the Brownsville Marina during the 2-year monitoring period. Like the Friday Harbor structure, this breakwater has become a popular fishing platform.

Semiahmoo Marina – Puget Sound, Drayton Harbor, Blaine, Washington

Semiahoo Marina, Drayton Harbor (Figure 13), is located at Blaine, WA, at the U.S./Canadian border and the northwestern tip of the continental United States.

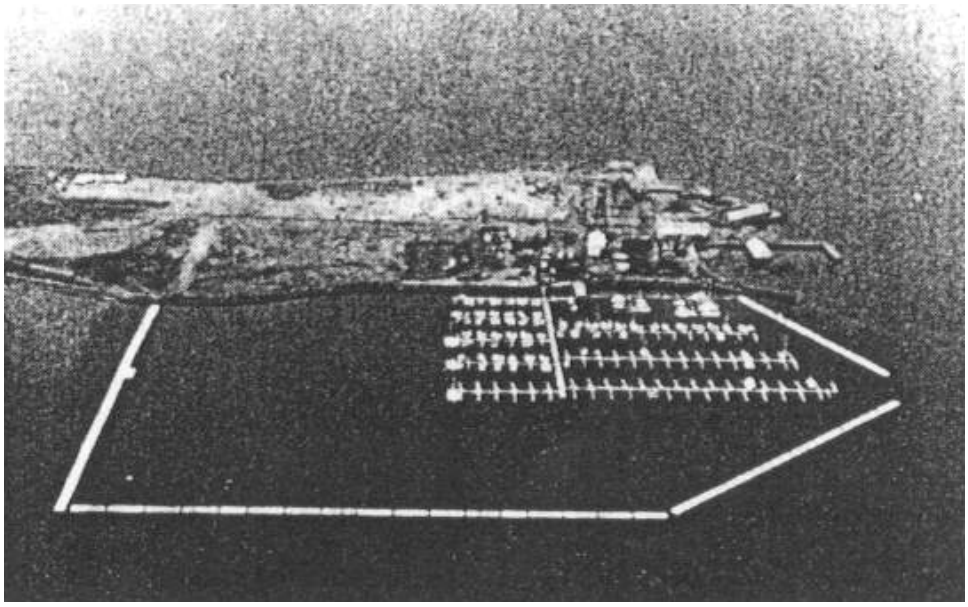

Figure 13. Semiahoo Marina, Drayton Harbor, Blaine, WA (after Nelson and Hemsley 1988)

Item monitored

Floating breakwater.

Period monitored

1981 – 1986 monitoring.

Reason(s) for monitoring

Onsite data were obtained during the time period 1981 – 1986 pertaining to the performance and durability of the floating breakwater. Operational experiences such as recreational use, transient moorage difficulties/preferences, and wave/wake transmission, diffraction, and reflection problems were documented (Nelson and Hemsley 1988).

Results of the 1981 – 1986 monitoring

Details of the monitoring are:

a. Since Drayton Harbor is shallow, the marina site had to be dredged to 3 m (-10 ft) mllw. It is exposed only to the southerly quadrant with a fetch of 2.7 km (1.7 miles) to the south and 3.7 km (2.3 miles) to the southeast. Mean tide range is 1.7 m (5.7 ft), diurnal range is 2.9 m (9.5 ft), and maximum range is 5.2 m (17 ft). Wind waves used for design are not available but are probably in the 0.6- to 0.9-m (2- to 3-ft)

range. Exposure to the south and southeast is likely to allow winds of over 40-knot (25-mph) speeds every winter, with 50-knot (30-mph) speeds on occasion.

b. The breakwater, constructed in 1981, is of the concrete caisson type. It was cast in 1.4- by 4.6- by 4.6-m (4.5- by 15- by 15-ft) units using polystyrene foam blocks as interior formwork and for positive flotation. The design draft was 0.9 m (3 ft). The total length of the breakwater, arranged in a U-shape, is approximately 1,065 m (3,500 ft). The marina eventually will have 840 slips for pleasure craft and fishing boats.

c. Each basic unit was truck-hauled to the site where four units were post-tensioned together to form 18.3-m (60-ft) modules. Next, the 18.3-m- (60-ft-) long modules were coupled by a chain-rubber fender connector. The anchor system uses clump weights on the anchor line consisting of a successive length of 2.54-cm- (1-in.-) diam nylon rope and stud-link chain to timber piles with a set of lines at each module connection.

d. This breakwater was inspected in July 1986. Maintenance problems were relatively minor over the monitoring period. Considerable effort was required to adjust anchor-line tensions and clump-weight placement to align the breakwater units. Shortly after the breakwater was installed, a severe storm destroyed a large number of the inter-float connections between the 18.3-m- (60-ft-) long units. The connectors were redesigned using large cylindrical rubber fenders. These connectors have required no maintenance since their installation. The 2.54-cm- (1-in.-) diam stud-link chain in the anchor line was scheduled for partial replacement in October 1986. Because the service life of some of the chain was shorter than expected, a corrosion protection system is being considered for inclusion in the replacement plan. The main portion of this breakwater is detached from shore, and no boats moor to the breakwater itself. It has, therefore, become an excellent habitat for sea birds and seals.

Columbia River Mouth, Washington/Oregon

The Mouth of the Columbia River (MCR) (Figure 14) is the western terminus of the Columbia River where the River enters the Pacific Ocean as the boundary between the states of Washington and Oregon.

Items monitored

Wave transformation, and Beach nourishment and sediment transport.

Period monitored

October 1994 – September 1999 monitoring.

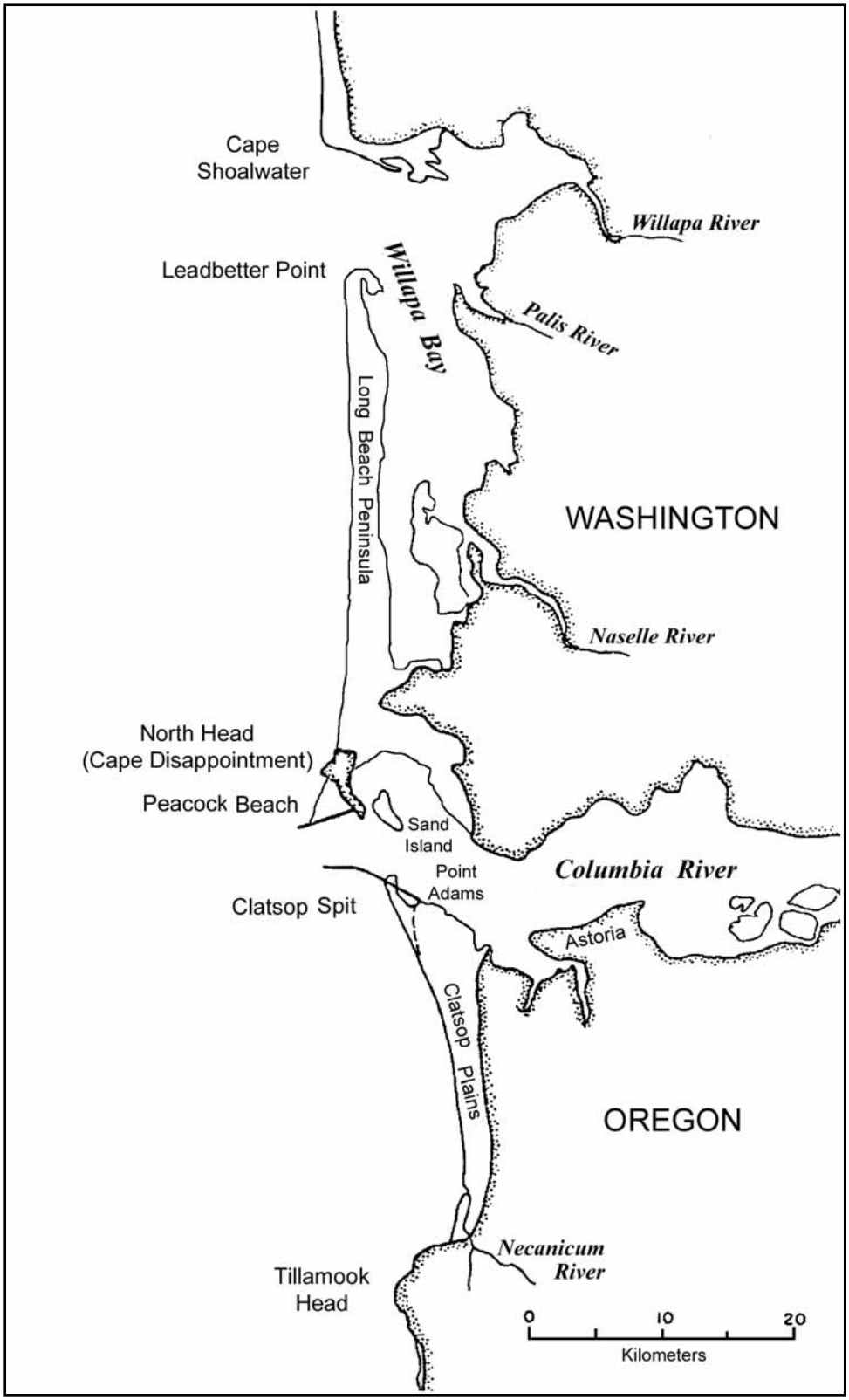

Figure 14. Mouth of the Columbia River, Washington/Oregon (after Gailani et al. 2003)

Reason(s) for monitoring

Monitoring was conducted during the time period October 1994 – September 1999 at the MCR to investigate dangerous wave transformation and the ability of numerical models to predict sediment transport from ocean dredged material disposal sites (ODMDS) onto nearby beaches. The entrance of the MCR requires annual dredging of 3 to 5 million cu m (3.9 to 6.5 million cu yd) of fine-to-medium sand to maintain the navigation channel at the authorized depth. The sandy dredged material is placed in Environmental Protection Agency (EPA) approved ODMDS. Dredging at the MCR is performed by hopper dredge. The use of ODMDS for disposal of material dredged from the MCR became regular after 1945 and continues to the present time. Since 1986, dredged material placed within the designated ODMDS has accumulated at a rate much faster than the U.S. Army Engineer District, Portland, had anticipated when the disposal sites were formally designated. ODMDS, which are intended to be moderately dispersive and have a 20-year life cycle, reached capacity within 10 years of initial operation. Exceedence of ODMDS capacity at the MCR creates two operational problems for the District: (a) The overall footprint of disposed dredged material extends beyond the existing ODMDS formally permitted boundaries by as much as 915 m (3,000 ft) in some cases; and (b) Dredged material within the ODMDS has accumulated to such an areal and vertical extent that adverse sea conditions are created. In some cases, mounds rise 18.3 to 21.3 m (60 to 70 ft) above the surrounding bathymetry. Mariners report that the ODMDS mounds cause waves to transform and steepen and/or break in the vicinity of the sites. This wave transformation is exceedingly hazardous to navigation.

The objectives of monitoring at the MCR were to: (a) analyze existing data to document historic bathymetric response at the MCR entrance and the ODMDS due to environmental conditions; (b) monitor selected MCR ODMDS locations to observe bathymetric response with respect to dredging disposal operations and the forcing environment; (c) explain qualitatively and quantitatively the rates of sediment dispersion at the MCR ODMDS; and (d) assess the suitability of new USACE Dredging Research Program sediment fate models including Short-Term FATE (STFATE), Long-Term FATE (LTFATE), and Multiple-Dump FATE (MDFATE), Regional Coastal Processes WAVE (RCPWAVE) model, and synthetically generated input data from Height Period Direction PREliminary (HPDPRE) wave model, Height Period Direction SIMulation (HPDSIM) wave model, and Advanced CIRCulation (ADCIRC) hydrodynamic circulation model for predicting sediment dispersion in the environment of the MCR. The study approach consisted of the execution of four fundamental tasks: a regional coastal processes analysis; oceanographic field data collection and analysis; state-of-the-art numerical modeling; and a comprehensive analysis of sediment transport processes (Gailani et al. 2003).

Results of the October 1994 – September 1999 monitoring

Details of the monitoring are:

a. Given the dynamics of the area, it is suggested ODMDS E be utilized whenever possible to add sand to the littoral system. Although beaches to

the north of the entrance have been experiencing accretion throughout the period of record, a 17-km (10.6-mile) length of coast north of this accretion zone has been expanding to the south with time. The problem is chronic and would be best mitigated with sediment added to the system. Assuming ODMDS E is not overfilled, it would seem cost-effective to dispose of sandy sediment at this site to nourish beaches to the north. Furthermore, because erosion along beaches of Clatsop Spit can be associated with blocking of sediment from the river by the south entrance jetty, it would be reasonable to establish a disposal site in this area to fortify beaches. Assuming the operation to be cost-effective relative to other sites, this disposal practice could reduce the need for ODMDS A.

b. The numerical simulation wave model RCPWAVE was used to predict behavior of waves as they are shoaled, refracted, and diffracted by the bathymetry that the waves pass over. RCPWAVE was used to compare the MCR wave climate due to the present (1994) ODMDS bathymetry with the wave climate due to past (1985) bathymetry before prominent mounds were formed at the ODMDS. The existing dredged material mounds at ODMDS A and B increased the height of incident waves within or in proximity to the ODMDS by 30 percent for 6-sec waves, 60 percent for 10-sec waves, and 80 percent for 16-sec waves, compared to 1985. A 10-percent increase in wave height due to shoaling could cause a wave to break. The areas most affected by dredged material mounds at ODMDS A and B are located immediately north and south of the MCR entrance.

c. Presently, the safest ocean approach to the MCR entrance channel is directly in line with ODMDS F. The present wave condition at the MCR requires that strict site management measures be implemented to: (1) prevent additional mounding at ODMDS A and B; and (2) prevent the formation of new mounds at ODMDS F which could adversely affect incoming waves to the MCR.

d. The numerical simulation modeling objectives at the MCR were all accomplished and included: (1) verifying the applicability of the DRP numerical models for the evaluation of ODMDS; (2) assessing the data collection needs for site evaluation by the DRP models; (3) identifying the capabilities and limitations of the DRP models; and (4) developing a systematic methodology for the application of the DRP models at other Corps districts.

e. Predictive techniques for determining environmental conditions and sediment transport processes under both waves and currents were developed to assess the movement of disposed material at the MCR ODMDS B and E. These techniques assist in determining crucial information for the management of dredged materials at navigation channels and harbors, with implications pertaining to mound dispersal, channel infilling, and protective cap erosion. The potential transport climate at proposed ODMDS M was also analyzed. The data indicate that transport processes at ODMDS E are more active that at ODMDS B, which supports observations from surveys indicating ODMDS E is more dispersive.

f. Three sediment transport methods were applied to simulate the time periods of data collection. The methods applied to simulate sediment transport by both waves and currents were those of: (1) van Rijn; (2) Wikramanayake and Madsen; and (3) Ackers and White. All methods performed reasonably well under most conditions. Environmental conditions at the MCR vary significantly both seasonally and annually. To estimate the long-term sediment transport climate, a 12-year synthetic database of wave and current conditions was developed from combined field measurements and numerical modeling. The sediment transport methods were then applied to the 12-year period of the developed database. The estimated sediment transport indicated significant variability in annual transport and a predominant transport direction to the north at ODMDS B and E.

g. A technique for using helicopters to deploy and retrieve oceanographic instrumentation platforms for wave and other data collection under severe wave conditions was developed. Depending on the length of the desired measurement, the platform can be immediately withdrawn and repositioned, or released and subsequently recovered with the helicopter. This technique is exceedingly useful where safe navigation of a vessel and over-the-side research vessel operations for deploying instruments is not possible under severe wave climates.

Yaquina Bay, Newport, Oregon

Yaquina Bay is located at the western terminus of the Yaquina River where the River enters the Pacific Ocean at Newport, OR (Figure 15), about 90 km (55 miles) west of Corvallis, OR.

Item monitored

North jetty.

Period monitored

October 1988 – September 1994 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period October 1988 – September 1994 to determine the likely cause for chronic damage to the Yaquina Bay north jetty. This monitoring also offered the potential for increasing understanding of failure mechanisms associated with rubble-mound structures and for improving methods of monitoring coastal structure performance in similar hostile wave and current environments (Hughes et al. 1995).

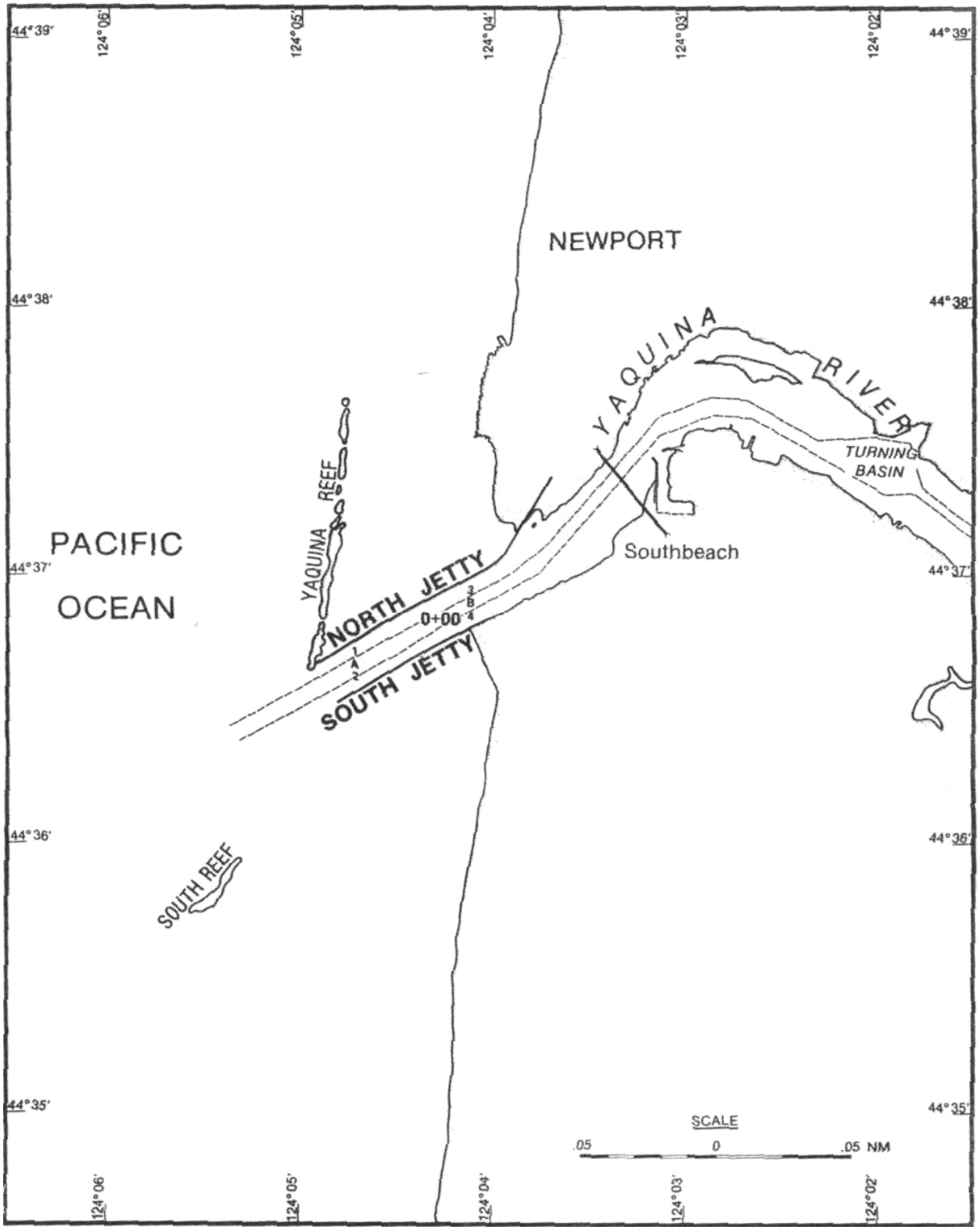

Figure 15. Yaquina Bay, Newport, OR (after Hughes et al. 1995)

Results of the October 1988 – September 1994 monitoring

Details of the monitoring are:

a. Wave height data occurring over the 6-year duration of the monitoring period aided in providing wave statistics characterizing the site. The jetty was exposed to wave heights up to about 8 m (26.2 ft). Even when reproducing the most severe wave conditions in a fixed-bed physical model of the site, jetty damage was not reproduced. It was concluded that structure damage was the result of more than just severe wave attack.

b. A geophysical survey provided detailed bathymetry, maps of seafloor features, charts depicting depth of bedrock and sediment thickness, and geological profiles. A sandy bottom in the vicinity of the damaged area of the jetty was discovered that had the potential to scour during storm events. This finding prompted a moveable-bed modeling effort to determine if scour would lead to armor layer instability.

c. Analysis of side-scan sonar images, collected as part of the geophysical survey, was instrumental in determining the underwater configuration of the jetty toe and its relationship to the Yaquina Reef and surrounding sandy bottom. SEABAT track lines provided sufficient data to detail the Jetty's underwater configuration. Armor stone displacement and migration downslope have resulted in underwater slopes of 1V:4H to 1V:10H along the damaged area.

d. Data obtained from photogrammetric analysis of the north jetty included contour maps of the structure, jetty cross sections, and contours showing changes from one flight to the next. These data were used to estimate volumetric changes resulting from armor stone loss in and around the damaged areas and from plot individual armor stone movement. Gradual deterioration indicated that armor displacement is continually occurring during severe storm conditions and most likely is not associated with liquefaction of the jetty foundation.

e. Through a semi-quantitative physical model which featured a moveable-bed section, it was determined that waves alone did not cause armor instability. Oblique approaching waves modified by seaward flowing currents along the jetty and the hard-bottom reef at the structure tip caused waves to break directly onto the structure, resulting in extensive damage and ultimately eroding the jetty to below the still-water level. Damage to the model test section was believed to be a legitimate representation of what occurred in the prototype.

f. Currents acquired in the prototype with an Acoustic Doppler Current Profiler in the vicinity of the north jetty indicated that, even in very mild wave conditions, the jetty redirects longshore-flowing currents to produce moderate seaward flowing currents adjacent to the structure. This finding lends credence to the wave/current damage hypothesis.

Siuslaw River Mouth, Florence, Oregon

The Siuslaw River enters the Pacific Ocean at Florence, OR (Figure 16), about 95 km (60 miles) west of Eugene, OR.

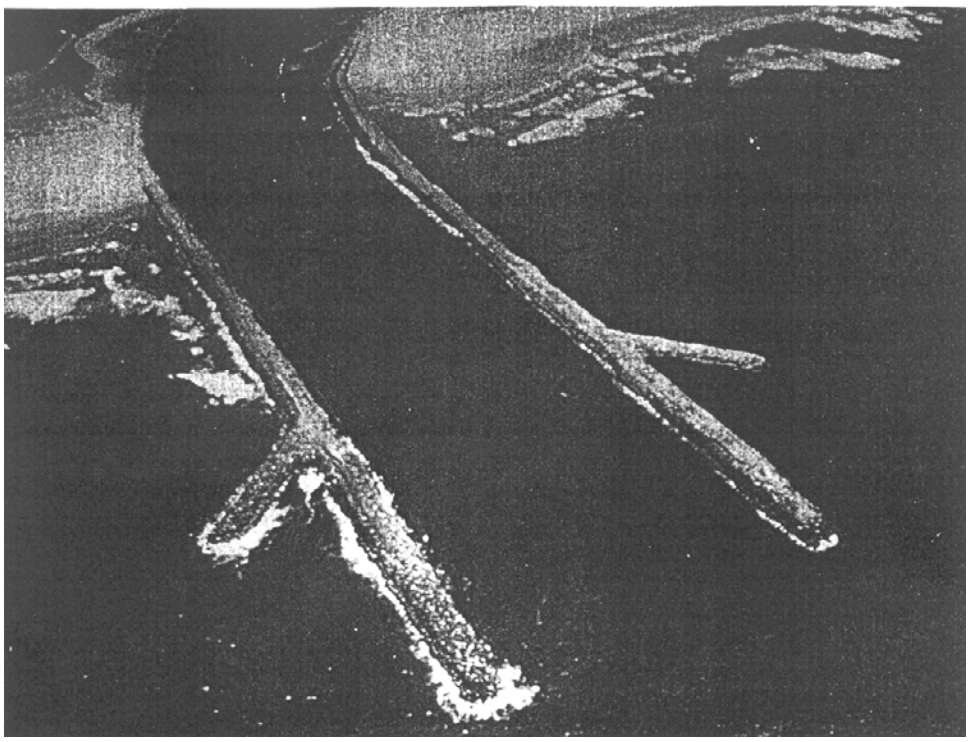

Figure 16. Siuslaw River mouth, Florence, OR (after Pollock et al. 1995)

Item monitored

Jetty spurs.

Period monitored

1987 – 1990 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period 1987 – 1990 to identify shoaling and current patterns and to determine the effectiveness of jetty spurs in reducing maintenance dredging (Pollock et al. 1995).

Results of the 1987 – 1990 monitoring

Details of the monitoring are:

a. Bathymetric data obtained during the monitoring effort revealed that the jetty spurs effectively deflected sediment away from the entrance channel. Sediment either circulated back toward shore, where it was reintroduced into the littoral system or was carried offshore away from the jetty by a jet of water parallel to the spur.

b. Drogues, dye studies, and aerial photographs were initially used to determine current patterns in the area but were not adequate in delineating bottom currents. An Airborne Coastal Current Measurement (ACCM) system was developed through the MCNP program to measure and establish bottom current patterns in the area. The system proved to be a very effective method for obtaining bottom currents in hostile wave environments where boat operation is dangerous or where quick mobility is necessary. Current patterns obtained correlated well with depositional patterns identified through bathymetric data obtained.

c. The Helicopter-Borne Nearshore Survey System, initially developed by Portland District, proved to be effective in measuring seabed bathymetry at Siuslaw in hazardous regions where other survey vessels cannot operate safely. Soundings were taken quickly and proved to be accurate and repeatable.

d. Current patterns and sediment depositional patterns obtained through the monitoring efforts parallel predictions and verify 3-D physical model laboratory experiments of spur jetties at the Siuslaw River site.

e. Navigation conditions at the jettied entrance have improved as supported by analysis of shoaling and sediment volume accumulation in the channel and by inspection of bathymetric data. Accumulation of material has shifted offshore into deeper water as opposed to in the entrance channel. Prior to jetty improvements, navigation was limited to high tide conditions during the summer months, and fishing operations had to be moved to other harbors in the winter months. Vessels are now able to navigate the entrance year-round, barring storm events, and are not confined to periods of high tide.

f. Shoreline change north and south of the jetties is most prevalent immediately adjacent to the structures where fillets have developed. This process is more pronounced to the north. These changes were predicted reasonably well with a numerical model using a simple wave energy littoral transport equation and an equilibrium shoreline concept.

g. Overall, the jetty improvements were a success. The construction cost of the spur system was estimated to be approximately $5 million less than the original design cost estimate for jetty extensions alone, and annual maintenance dredging costs have been reduced by approximately 133,800 cu m (175,000 cu yd). Results of the monitoring provide strong support for the effectiveness of spur jetties at this site and their potential use at other sites.

Umpqua River Mouth, Reedsport, Oregon

The Umpqua River entrance (Figure 17) is located on the southern Oregon coast at Reedsport, OR, approximately 285 km (180 miles) south of the Columbia River and 650 km (405 miles) north of San Francisco, CA.

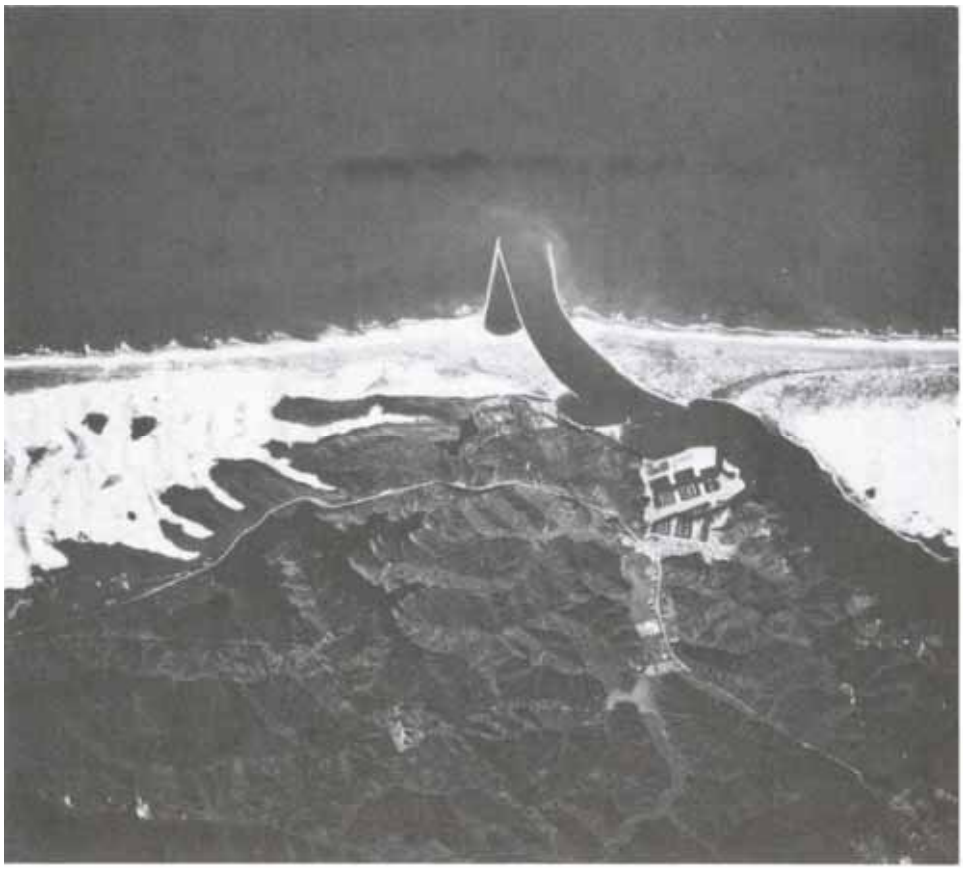

Figure 17. Umpqua River mouth, Reedsport, OR (after Herndon et al. 1992)

Items monitored

South jetty and inlet.

Period monitored

May 1983 – May 1984 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period May 1983 – May 1984 to determine the effects that an extension to a third jetty constructed inside previously completed arrowhead jetties at the mouth of the Umpqua River would have on the inlet. The arrowhead jetties, constructed in 1938, were not

satisfactory in eliminating shoaling of the entrance channel. A third, or training jetty was constructed in 1951 on the south side of the entrance channel. This training jetty was 1,295 m (4,240 ft) long, and generally paralleled the entrance channel. The seaward terminus was about 0.8 km (0.5 mile) landward of the outer end of the old south arrowhead jetty. The training jetty might have caused a slight increase in channel shoaling and a possible increase in wave activity in the entrance. A 790-m (2,600-ft) extension to the training jetty was recommended and completed in 1980 so that the training jetty now terminated at the same location as the old south arrowhead jetty (Herndon et al. 1992).

Results of the May 1983 – May 1984 monitoring

Details of the monitoring are:

a. The channel has improved in terms of depth and, to a somewhat lesser degree, width, as anticipated.

b. There were no significant deleterious impacts to adjacent shorelines, tidal or salinity regimes, or current patterns.

c. The physical hydraulic model proved to be an excellent predictive tool for hydrodynamics and salinity changes. Increased channel shoaling predicted by the qualitative shoaling studies has not manifested itself.

d. Regime theory or appropriate inlet stability analysis is important in tidal inlets on sandy coasts where maintenance dredging may be needed.

e. Jetties at tidal entrances should be constructed parallel to each other and to the navigation channel. Converging or arrowhead jetties often fail to provide for stable entrances and safe navigation.

Crescent City Harbor, Crescent City, California

Crescent City Harbor is located at Crescent City, CA, on the northern California coastline, approximately 27 km (17 miles) south of the Oregon/ California border. Rehabilitation of the breakwater in 1986 used 38,135-kg (42-ton) dolosse, 20 of which had been instrumented for response to the ocean environment, and which should be monitored and analyzed. A cross section of the rehabilitated breakwater is shown in Figure 18.

Item monitored

Breakwater.

Periods monitored

1986 – 1989 monitoring and October 1993 periodic inspection.

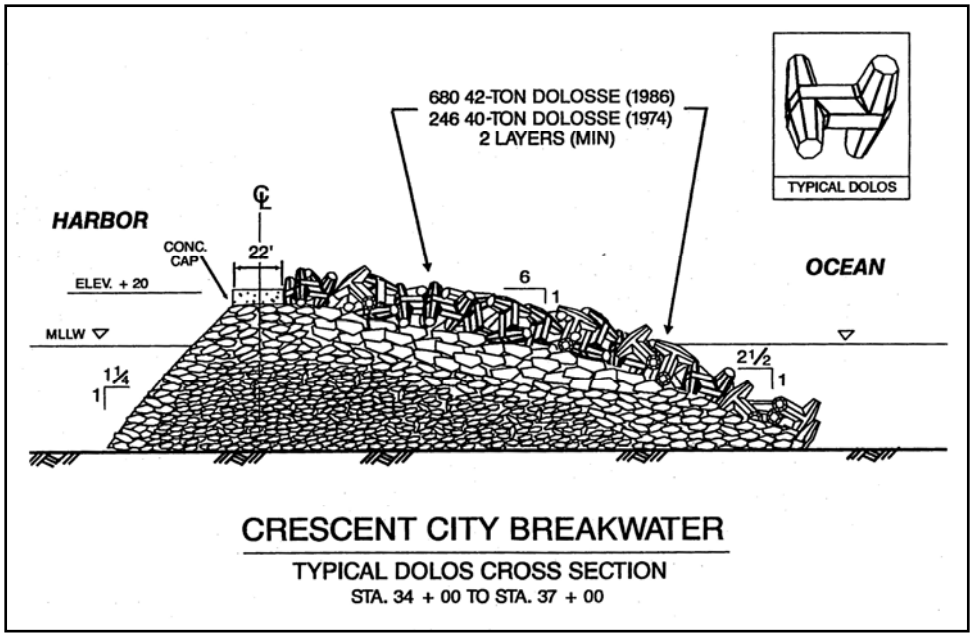

Figure 18. Crescent City breakwater cross section, Crescent City, CA (after Markle et al. 1995)

Reason(s) for monitoring

Monitoring was conducted during the time period 1986 – 1989 to define long-term trends in dolos movement, breakage, and static stresses at the Crescent City breakwater (so these data could be used to further improve the structural dolos design procedure) and to observe the long-term response of the dolos portion of the Crescent City breakwater to its incident environment. The breakwater had been rehabilitated in 1986 using 38,135-kg (42-ton) dolosse. During the rehabilitation, 20 dolosse were instrumented to measure loading and armor unit motion. This monitoring was carried out as part of the Crescent City Prototype Dolos Study (CCPDS). Near the end of the CCPDS in 1989, it was noted that dolos movement was subsiding, but static loads were still showing increases. For this reason, additional monitoring data obtained during the period November 1989 through October 1993 after conclusion of the CCPDS were analyzed as a periodic inspection (Markle et al. 1995).

Results of the 1986 – 1989 monitoring

Details of the monitoring are:

a. Storms that occurred early during the first postconstruction winter season have produced the largest dolos movements to date. Reduced movement during subsequent storms indicates that the dolosse have consolidated and nested into a more stable matrix.

b. Surges in dolos movement, where evident, have tended to follow peaks in the wave power record.

c. During nesting, the greatest movement of dolosse was on the upper slope of the centrally located dolos test section and in the vicinity of the waterline. The movement on the upper slope is thought to have resulted from the existence of a slight contour dip or trough in this region of the breakwater and because many of the dolosse placed there had initial boundary conditions that did not inhibit sliding.

d. Since initial nesting, dolos movement has slowed but continues to occur primarily near the waterline as well as on the upper slope just north of the centrally located dolos test section.

e. Spatially averaged movement within the dolos test section has been comparable to that found outside of the test section; however, the region of high movement within the test section has been generally located upslope.

f. The dominant direction of dolos movement has been upslope with slight settling plus rotation about the vertical axis (yaw). Upslope movement (i.e., a wave runup dominated movement) is thought to result, at least in part, from the breakwater's mild slope.

g. Breakage, while typically associated with some amount of movement, has not necessarily been associated with significant movement, and vice versa. For the large dolosse at Crescent City (which can have little residual strength), the extent to which movement causes a detrimental shift in boundary conditions has appeared more important than the absolute magnitude of the movement itself.

h. One of the primary findings from the field monitoring is that the most significant structural design parameter for these large dolosse is static stress. Subtle movement in the dolos matrix can cause shifts in dolos boundary conditions which, in turn, produce a change in dolos static stress. Field data on dolos movement, static stress, and breakage should continue to be collected in order to better understand the long-term nesting behavior of large dolosse.

Results of the October 1993 periodic inspection

Details of the inspection are:

a. Aerial photography and subsequent photogrammetric analysis can provide very accurate data on movement of armor units located above the waterline. The methods require only minimal ground truthing to ensure accuracy of the data. Low-altitude helicopter surveys result in significant improvements in data accuracy and photo image resolution when compared to higher altitude, fixed-wing surveys.

b. From the winter of 1990 through the spring of 1993, very little significant movement occurred in the visible dolos field; thus, no patterns of movement could be established in the manner they were defined during the CCPDS. Strain gauges positioned inside instrumented dolosse revealed that static stress loads in some of the units were reaching levels that left little residual strength for pulsating wave loads and impact loads.

It was determined that the most significant structural design parameter for large dolosse is static stress.

c. Low-level helicopter inspections and 35-mm photography provide a good first indication of levels of armor-unit breakage and give a basis for determining if an on-the-ground inspection is needed to gain more precision regarding armor-unit breakage that is not captured by the aerial inspection.

d. As of the spring of 1993, dolos breakage seemed to have subsided and was at a level that was not a major concern, although 47 broken dolosse were identified on the structure. However, with the question of rising dolos static stresses, close inspections following significant storm events are recommended.

e. It is recommended that, at some future date, the Crescent City breakwater be revisited as part of periodic inspections to ascertain if dolos movement, breakage, and static stress levels have changed so that additional insight can be gained into the long-term response of the structure to its environment.

Humboldt Bay, Eureka, California

Humboldt Bay is located on the Pacific coast of northern California at Eureka, CA (Figure 19). It is located approximately 135 km (85 miles) south of the Oregon/California border.

Item monitored

Jetties.

Period monitored

December 1996 periodic inspection.

Reason(s) for monitoring

Base conditions for future periodic inspections were determined during December 1996. Purposes of the periodic inspections are to: (a) develop methods using limited land-based surveying, aerial photography, and photogrammetric analysis to assess the long-term stability response of the concrete dolos-armored units on the heads of the Humboldt Bay jetties; and (b) conduct land surveys, broken armor-unit inspections, aerial photography, and photogrammetric analyses to test and improve developed methodologies and accurately define armor unit movement above the water line (Bottin and Appleton 1997).

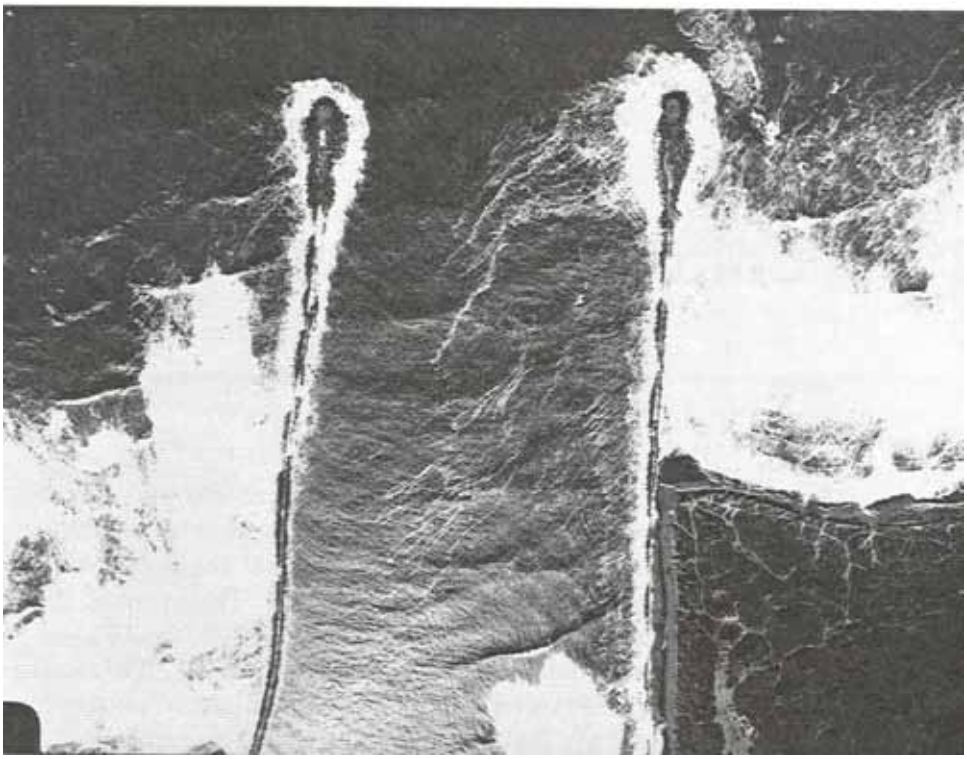

Figure 19. Humboldt Bay entrance, Eureka, CA (after Bottin and Appleton 1997)

Results of the December 1996 periodic inspection

Details of the inspection are:

a. The Humboldt Bay jetties have experienced a long history of damage and subsequent repairs since original construction was completed in 1899. Rehabilitations were completed in 1911, 1927, 1932, 1939, 1950, 1957, 1963, 1971, 1988, and 1995. These rehabilitations consisted of the construction and/or installation of concrete monoliths, parapet walls, mass concrete, stone, concrete blocks, tetrapods, and dolosse. Since the dolos rehabilitation of the heads of the jetties in 1971, damages have been primarily along the trunk (stone) reaches of the jetties. No extensive work has been required along the dolos fields since their construction. Prior to this study, no sound, quantifiable data relative to the movement or positions of the dolos concrete-armored units had been obtained for the jetties.

b. Under the current Periodic Inspections work unit, data from limited ground-based surveys. aerial photography, and photogrammetric analysis have been obtained to establish very precise base-level conditions for the seaward dolos-covered portions of the Humboldt Bay jetties. Accuracy of the photogrammetric analysis was validated and defined through comparison of ground and aerial survey data on control points and targets established on the structures. A method using high-resolution, stereo-aerial photographs, a stereoplotter, and Intergraph-based software has been developed to analyze the entire above-water concrete armor unit

fields and quantify armor positions and subsequent movement. A detailed broken armor unit survey has resulted in a well-documented data set that can be compared to subsequent survey data.

c. Now that base (control) conditions have been defined at a point in time, and methodology has been developed to closely compare subsequent years of high-resolution data for the Humboldt Bay jetties, the site will be revisited in the future under the Periodic Inspections work unit to gather data by which assessments can be made on the long-term response of the structure to its environment. The insight gathered from these efforts will allow engineers to decide, based on sound data, whether or not closer surveillance and/or repair of the structures might be required to reduce their chances of failing catastrophically. Also, the periodic inspection methods developed and validated for these structures may be used to gain insight into other USACE structures.

Spud Point, Bodega Bay, California

Spud Point Marina breakwater (Figure 20) is located in the northwestern part of Bodega Harbor, an enclosed bay on the California coastline about 95 km (60 miles) north of San Francisco. Bodega Harbor serves light-draft vessels and is the only harbor of refuge in the 240-km (150-miles) stretch of coastline between San Francisco Bay and Noyo Harbor.

Item monitored

Breakwater.

Period monitored

August 1985 – March 1988 monitoring.

Reason(s) for monitoring

The concrete pile-supported breakwater structure was selected for monitoring during the time period August 1985 – March 1988 because of its unusual baffled design. Openings in the breakwater below the mean lower low tide level permit relatively unimpeded marina flushing. The baffle panel submergence depth was chosen using theoretical wave height transmission results. A field study of wave transmission was conducted using boat wakes and pressure sensors to measure the generated waves. Soundings of potential scour zones and a side-scan sonar survey were made. Circulation through the breakwater and marina were measured, and the breakwater was examined for structural integrity (Lott 1991).

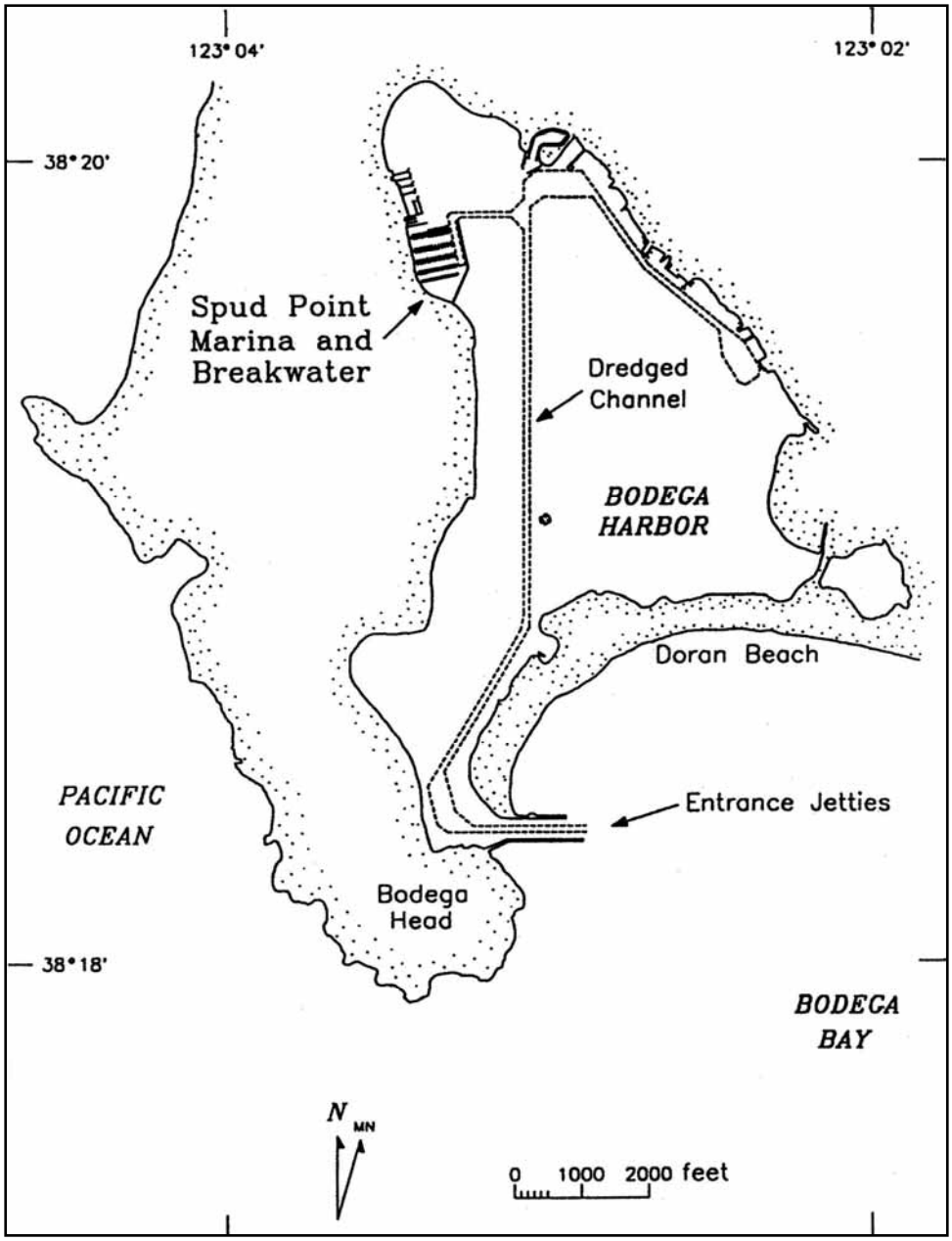

Figure 20. Spud Point breakwater, Bodega Bay, California (after Lott 1991)

Results of the August 1985 – March 1988 monitoring

Details of the monitoring are:

a. Vessel-generated waves may be the controlling design wave in small bodies of water. Predictions for vessel-generated waves are needed in addition to predictions for wind-generated waves.

b. In designing openings in breakwaters to allow water circulation, consider natural circulation patterns. Openings (culverts or gaps) that are aligned parallel to the normal flow will be more effective. Thus, the openings for

circulation will be placed in breakwater segments that are angled across the flow patterns.

c. Cast concrete breakwater caps may develop hairline “shrinkage” cracks. While small cracks may not affect structural integrity in warmer climates, expansion of freezing water can cause spalling of concrete in colder climates.

d. Current monitoring satisfied the objective of determining the flushing characteristics of the breakwater, given the limited scope of the field effort. Volumetric flushing estimates have not been presented, since a more quantitative description of the flushing characteristics (such as that required for a circulation model study) would require greater density of data for the northern and southern ends of the marina, over a wider range of tide and wind conditions. The type of recording electromagnetic current meter used for this study would be excellent for a more comprehensive study if enough were deployed to cover the boundaries and internal points of interest.

e. Monitoring efforts for structural integrity were adequate, given the constraints of the study. If side-scan surveys are performed in the future, an improvement would be to conduct them at extreme high tides to permit complete breakwater coverage and to lessen the risk of tow fish damage. Future baffle opening surveys could be compared with the data presented herein to obtain quantitative scour information if a side-scan survey indicated development of scour problems. Given the small tide and wind-induced current velocities, scour development is unlikely, except possibly during a prolonged high-wave event (from standing wave-induced bottom velocities along the outside of the wall and through the openings). An improvement over spot checks of cap elevation would be to install brass disks in the cap and survey their 3-D position as part of a periodic inspection program, and after any major earthquake activity. Future visual inspections should pay careful attention to the hairline cracks in the cap and should be documented photographically according to a retrievable position identification system.

f. The monitoring effort did not produce quantitative information on transmitted waves. In hindsight, the planning process relied too heavily on anecdotal information such as operator estimates of boat wake generation and the scope of the project did not allow enough flexibility or pretesting of the procedure before the full-scale commitment of personnel and equipment. However, the experiment yielded some good qualitative information and the knowledge gained can be used to make the approach more workable for a future application where boat wakes are the primary concern. At this site, higher resolution pressure sensors or a different type of wave sensor (to measure water surface variation directly or even look at particle velocities instead) coupled with a more sophisticated data analysis and careful timing with respect to tides may be more successful.

g. Spud Point breakwater meets its performance criteria with respect to currents, since currents through the breakwater were measurable and exchange was clearly taking place. The reported prebreakwater dominant

pattern of north-to-south flow apparently has been preserved. A baffled design should be considered for lower energy environments where good circulation is critical to acceptability of the proposed structure.

h. The breakwater has been successful in meeting the structural integrity performance criteria, assuming that the hairline cracking observed along the top of the cap at the bents is superficial and is not progressing or resulting in corrosion of the underlying reinforcement.

i. The breakwater appears to be meeting its performance criteria for wave transmission. Although this monitoring effort did not quantify the wave attenuation, the difficulty experienced in producing transmitted waves is evidence that the breakwater is providing significant wave attenuation. The marina operator reports that the breakwater has been very satisfactory and that only the largest tenant boat (the 19.8-m- (65-ft-) long Sea Angler) produces a wake large enough to penetrate the breakwater and cause rolling of the docked boats. Even with overtopping, the water inside the marina remained calm.

j. Although the transmitted wave heights were small during the experiment, the significant rolling of some of the boats, caused by the largest wakes, suggests that parameters other than wave height may be of interest for wake or wave transmission criteria. It is unknown whether wind waves of similar height would have caused the rolling. The boats were docked so that they were broadside to the breakwater. One explanation may be that the baffled type of breakwater reduces vertical water particle motions and surface disturbances, yet allows enough horizontal motion to pass through the breakwater in the lower part of the water column to cause lateral motion in docked vessels. This emphasizes that for some protected breakwaters, protection against wakes may govern the design more than protection against wind waves.

Fisherman’s Wharf, San Francisco Bay, California

The area of San Francisco traditionally known as Fisherman’s Wharf (Figure 21) is located along the north-facing waterfront of the city opposite Alcatraz Island. The Fisherman’s Wharf study area is bordered on the west by the Golden Gate National Recreation Area. To the east is a succession of piers and waterfront structures. Located only about 5 km (3 miles) east of the narrowest part of the Golden Gate (the connecting entrance between the Pacific Ocean and San Francisco Bay), the site is subject to both ocean and bay influences.

Items monitored

Breakwater and harbor.

Period monitored

December 1982 – December 1989 monitoring.

Figure 21. Fisherman's Wharf, San Francisco Bay, California (after Lott 1994)

Reason(s) for monitoring

Monitoring was performed during the time period December 1982 – December 1989 to evaluate performance of the breakwater and to determine its impact on the surrounding harbor area. Specific objectives included (a) documenting wave attenuation of the structure compared to model studies and design criteria; (b) evaluating effect of the structure on surge within the harbor complex; (c) determining effect of structure on water circulation within the harbor and surrounding areas and currents, especially at the entrance; (d) determining actual scour, measuring scour, evaluating cause and comparing with predicted scour; (e) evaluating effect of the structure on littoral processes, including shoreline response and deposition within the harbor; and (f) determining integrity of the structure, investigating spalling, cracking, and settlement of the wall (Lott 1994).

Results of the December 1982 – December 1989 monitoring

Details of monitoring are:

a. An impermeable vertical-wall detached breakwater structure forms the main element of the breakwater system. This 460-m- (1,509-ft-) long structure was built using driven prestressed/precast interlocking sheet piles. A cast-in-place reinforced cap beam ties the piles together. For most of its length, the detached breakwater is oriented approximately in the shore-parallel west-southwest to east-northeast direction. This alignment intercepts waves from the northwest, yet is essentially parallel to the prevailing tidal currents.

b. Measured wave data for the postbreakwater period show that significant wave heights (wind waves and swell) within the protected harbor did not exceed performance criteria. The breakwater is performing its primary purpose--to provide wave protection for the traditional Fisherman's Wharf sma1l-craft berthing area for the fleet of historic ships berthed at Hyde Street pier and for the planned berthing expansion area between Hyde Street pier and Pier 45.

c. These data show that the qualitative performance criterion for surge (no increase) was met for the central part of the harbor (Hyde Street pier and the berthing expansion area) but was violated at the end of Pier 45 and the inner-most reaches of the traditional berthing area. Primarily, the breakwater appears to have caused some shifting of surge energy among preexisting resonant peaks. Changes in surge were not large; some locations showed decreases. The breakwater has not caused surge to become a new problem in the harbor.

d. Measurements of entrance channel currents during relatively extreme tidal conditions found peak current speeds to be well below the performance criterion limit of 2 knots (1.2 mph). Direct comparisons between pre- and postbreakwater prototype current data (as a means of evaluating the breakwater's effects) were not feasible. However, results from postbreakwater prototype measurements of general circulation (current patterns and speeds) compare reasonably well with numerical model predictions.

e. Although the available bathymetric data do not permit evaluation of breakwater impacts on deposition rates, the analysis suggests that significant deposition has taken place along the landward side of the breakwater since construction. No maintenance dredging has been required, however, so the performance criterion is still being met at this time. It was not possible to establish outer boundaries of zones where circulation and deposition have been influenced by the breakwater from the available measured data.

f. Lead-line data show that the nominal design scour depth of 3 m (10 ft) along the wall has definitely been violated at the westward end of the detached breakwater. This performance criterion has also been approached closely at the point of junction between the curvilinear and straight sections of the detached breakwater. Other locations may have been at or beyond the 3-m (10-ft) scour limit for unknown periods of time. Scour effects of the breakwater appear to be limited to the immediate vicinity of the structure. It is emphasized that measured scour relative to the specified performance criterion may not be a reliable measure of structural stability or instability.

g. There is no evidence that the breakwater caused increased beach erosion at Aquatic Park. Insufficient quantitative data were available to rigorously determine beach changes, or the boundaries or sediment budget of the Aquatic Park littoral system. A quantitative analysis of beach changes due to the breakwater would be difficult, if not impossible, because of beach management practices (importation and movement of

sand by mechanized equipment). From onsite visual observations, the present-day beach matches the prebreakwater description.

h. The breakwater structures have not moved or deteriorated significantly, despite the effects of the October 1989 Lorna Prieta earthquake that caused extensive damages in the immediate vicinity. No cracking or spalling of original concrete has occurred. The function of the breakwater has not been impacted at all by the very small changes in vertical and horizontal alignment.

i. Measured wave heights met performance criteria and the breakwater has not been damaged by waves. Therefore, design waves (selected primarily using Shore Protection Manual (SPM) methods (HQUSACE 1984)) have not been shown to be incorrect or inappropriate. Because the directions of incident waves were not measured, it is unclear whether the site has experienced conditions similar to those used in design. Incident wave data measured during the study do not provide a better basis for statistical projection of extreme waves than the incident data measured prior to breakwater construction.

j. In future monitoring at sites like Fisherman's Wharf that are subject to simultaneous ocean-generated and locally generated waves, some modifications to standard open-coast wave data processing and analysis procedures should be considered. Specifically, analysis should avoid overlapping frequency coverage between surge, ocean-generated (swell), and locally generated waves. Sampling rates (both frequency of gauge polling and frequency of pressure sampling within bursts) should be specifically tailored to the frequency regimes present. Sampling rate considerations and decisions about hard-wired versus self-recording gage technology should also include examination of how fast wave conditions might change at the site. The directionality of incident wave conditions should definitely be obtained. Wave monitoring should be planned and initiated as early as possible in the design process to allow definition of baseline conditions.

Morro Bay Harbor, Morro Bay, California

Morro Bay Harbor is located in a natural embayment on the central coast of California, about midway between Los Angeles and San Francisco. It serves as the only all-weather small craft commercial/recreational harbor between Santa Barbara and Monterey. Morro Bay extends inland and parallels the shore for a distance of about 6.4 km (4 miles) south of its entrance at Morro Rock (Figure 22).

Items monitored

Breakwaters, inlet, and sediment transport.

Figure 22. Morro Bay Harbor, Morro Bay, California (after Thompson et al. 2002)

Period monitored

January 1998 – August 2001 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period January 1998 – August 2001 to evaluate the effects of channel deepening and widening at the harbor entrance on the breakwaters and the inlet. Prior to the latest improvements at Morro Bay Harbor entrance in December 1995, Morro Bay Harbor was known as one of the most dangerous harbors in the United States. Deaths and vessel damage resulted from steep and breaking wave conditions in the harbor entrance. Breaking waves occurred when incident wave heights exceeded 3 m (10 ft). Hazardous conditions occurred as 2.4- to 3-m (8- to 10-ft) waves tended to steepen sharply when they reached the shallower harbor entrance, particularly during ebb tide conditions. Improvements in 1995 consisted of construction of a deepened, expanded entrance channel. The new channel doglegs westerly from the old entrance channel and flares open to a width of 290 m (950 ft). The authorized depth of the channel extension is -9.1 m (-30 ft). However, the plan provides for advanced maintenance by deepening the new channel to -12.2 m (-40 ft) and dredging an additional sand trap to a depth of -9.1 m (-30 ft) within the harbor entrance structures on the north of the head of the south breakwater.

Hypotheses to be tested by the monitoring plan included: (a) improvements would result in significant improved navigation conditions in the harbor entrance; (b) improvements would have no negative impact on the existing structures; (c) improvements could be effectively maintained with a 3-year dredging interval in the entrance; (d) model investigations would accurately quantify wave conditions in the entrance, and correctly define sediment patterns and deposition in a qualitative sense; and (e) methodology used in determining sedimentation rates in the harbor entrance would be valid based on field data, model predictions, and sound engineering judgment (Thompson et al. 2002).

Results of the January 1998 – August 2001 monitoring

Details of monitoring are:

a. The monitoring did not include quantitative data collection to verify that the improvements at the entrance would result in significantly improved navigation conditions. However, the Morro Bay harbor master's office reports that hazardous breaking wave conditions in the deepened entrance occur significantly less often than in the preproject condition. This information, coupled with survey data showing that a deepened entrance was maintained during the monitoring period, lead to the conclusion that the improvements have resulted in significantly improved navigation conditions at the harbor entrance.

b. Existing structures in the vicinity of the modified entrance include the south and north breakwaters. Photogrammetric surveys of the above-water portion of the south breakwater show no significant changes over a 2-year monitoring period (1998 through 2000). The north breakwater was predicted to be unaffected by the project, so it was not subjected to detailed monitoring. No significant changes to the north breakwater were reported or observed during the monitoring period. No episodic storms occurred during monitoring; thus, the hypothesis that the improvements will have no negative impact on existing structures was supported but not conclusively proven by this monitoring study.

c. Dredging intervals during the monitoring study were considerably shorter than the 3-year desired design interval that the improvements were expected to maintain. The longest interval was 15 months, and all others were less than 10 months. The volume predicted by the Los Angeles District for removal during initial dredging was 684,300 cu m (895,000 cu yd), and 752,980 cu m (984,800 cu yd) were actually dredged. None of the subsequent dredging episodes reestablished design advance depth in the entrance channel and sediment trap. The stored volume generally increased between successive surveys except when dredging occurred. The annual sedimentation rate predicted by the Los Angeles District was 15,300 cu m per month (20,000 cu yd per month), and the advanced maintenance and sand trap storage capacity designed for a 3-year maintenance cycle was 12,500 cu m per month (16,400 cu yd per month). Overall, in comparison to shoaling volumes and rates from survey data, the District-predicted shoaling over a 3-year time period appears consistent with project experience. Infilling could be

more rapid during unusually stormy winters. The monitoring study indicates that the 3-year dredging interval is a maximum.

d. Numerical and physical modeling studies quantified transformation of various incident wave conditions in the preproject and with-project harbor entrances. Wave gauge data collected outside the entrance gap and in the main channel were sufficient to show that physical model wave data through the entrance are accurate representations of the prototype. Prototype shoaling patterns are typically qualitatively similar to those observed in the physical model for 250-deg wave directions, which is a fairly typical direction although mean wave direction is 265 deg. However, the qualitative deposition patterns predicted in physical modeling of 250-deg wave directions should be appropriate for prototype conditions, and they are well supported by prototype data.

e. Numerical model HARBor Deep (HARBD) was run with monochromatic waves and does not include wave breaking (Bottin and Thompson 2002). Numerical model Conjugate Gradient WAVE (CGWAVE) was run with spectral waves and includes wave breaking. CGWAVE results compare much more favorably than HARBD results with physical model data obtained within the harbor. This is partly attributable to CGWAVE's being a more comprehensive model and partly to CGWAVE's being expressly configured to match physical model test conditions. CGWAVE also matches the inner harbor prototype gauges remarkably well.

f. Prediction of sedimentation rates in the harbor entrance was a difficult but crucial element of project design. Based on incident wave information available for design, the net potential longshore transport was strongly southward. Prototype incident wave data yield the same conclusion. However, northward transport was recognized by the District as dominant in the process of harbor entrance shoaling. Prototype sedimentation data also support this conclusion. For design, the U.S. Army Engineer District, Los Angeles, used an effective southward transport of 54,300 cu m per year (71,000 cu yd per year), and northward transport of 305,800 cu m per year (400,000 cu yd per year). Recognizing that calculations involve assumptions and approximations, monitoring study data do not appear to support previous calculations of potential longshore sediment transport. The design methodology resulted in a predicted sedimentation rate that is very reasonable in comparison to shoaling rates observed during monitoring.

Redondo Beach, California

Redondo Beach King Harbor (formerly Redondo Beach Harbor), California (Figure 23), is a small-craft harbor located on the Pacific coast at the southern end of Santa Monica Bay. It lies within the City of Redondo Beach, about 27 km (17 miles) southwest of the business center of Los Angeles. The Harbor is entirely man-made and serves as a port of call for visiting craft from the entire Pacific coast.

Figure 23. Redondo Beach King Harbor, Redondo Beach, California (after Bottin and Mize 1990)

Item monitored

Wave transformation.

Period monitored

October 1992 – June 1994 monitoring.

Reason(s) for monitoring

Monitoring was performed during the time period October 1992 – June 1994 to compare observed wave transformation (as measured in the prototype) with theoretical wave propagation models for this area of steep, complex bathymetry. Field data measurements were compared with results from the Regional Coastal Processes Transformation Model (RCPWAVE) and from a spectral refraction model (STWAVE) which treats the propagation of spectral waves rather than monochromatic waves as in RCPWAVE (Sabol 1996; Rhee and Corson 1998).

Results of the October 1992 – June 1994 monitoring

Details of monitoring are:

a. Modeling wave transformation over a variable sea bottom remains a difficult task in most cases. Analytical solutions limit themselves only to

simple geometry, and numerical treatments base their predictions on the fundamental assumption of slowly varying sea depth. The difficulty in modeling the Redondo Beach site is heightened by the presence of a deep submarine canyon offshore that affects waves from the predominant direction of attack.

b. Computations from both RCPWAVE and STWAVE are in poor agreement (low correlation coefficients) with field measurements when swell heights are greater than 1.5 m (5 ft).

c. In general, RCPWAVE tends to overestimate wave heights. Depending on the location, computed wave height ranged from 19 to 62 percent greater than those observed.

d. STWAVE wave heights appear to be more accurate than RCPWAVE, but in general, they were underestimated. Computed wave height ranged from 12 to 13 percent less than those observed.

e. Both field measurements and model computations indicate no significant tidal influence on wave transformation.

5 Site-Specific Lessons Learned, Gulf of Mexico

Colorado River Mouth, Matagorda, Texas

The mouth of the Colorado River (Figure 24) is located on the Texas coastline near the town of Matagorda and runs through the Matagorda Peninsula into the Gulf of Mexico. It is located approximately midway between the ports of Galveston and Corpus Christi.

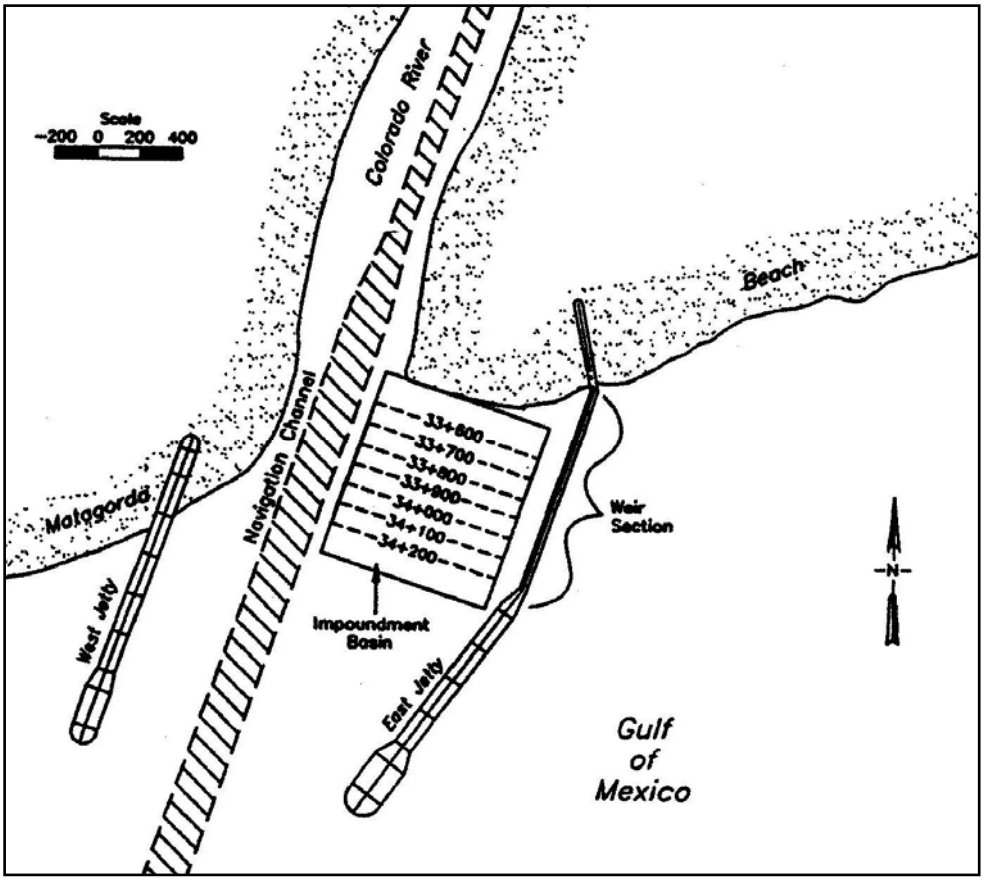

Figure 24. Colorado River mouth, Matagorda, Texas (after King and Prickett 1998)

Item monitored

Weir-jetty.

Period monitored

May 1990 – September 1992 monitoring.

Reason(s) for monitoring

Monitoring was performed during the time period May 1990 – September 1992 to evaluate the design and efficiency of a weir-jetty and adjacent impoundment basin at the mouth of the river (King and Pickett 1998).

Results of the May 1990 – September 1992 monitoring

Details of monitoring are:

a. The weir-jetty system at the mouth of the Colorado River has had minimal impacts on adjacent beaches.

b. The weir is in the proper cross-shore location, is at the correct elevation, and is the proper length.

c. The longshore transport rate was substantially underestimated during the design of the weir-jetty system. The impoundment basin was designed for a littoral drift transport rate of 230,000 cu m (300,800 cu yd); however, data indicate a net transport rate (in the direction of the weir) on the order of 510,000 cu m (667,000 cu yd). This resulted in the impoundment basin and entrance channel shoaling substantially more rapidly than expected following construction. The creation of a safe, navigable inlet was the primary purpose of the construction, and the shoaling of the inlet mouth adversely impacts navigation.

d. Good, reliable estimates of the longshore transport rate are needed prior to jetty and impoundment basin design. The current recommended method is to compute the longshore transport rate from at least 2 years of onsite wave data. Failure to do this will lead to uncertainties in anticipated dredging costs and may lead to poor choices in jetty and impoundment basin design.

e. Consideration should be given to enlarging the impoundment basin at Colorado River. Unfortunately, available area between the jetties is limited. Future project designs should have flexibility to allow for modifications of the size and shape of the impoundment basin based on operational experience.

f. Data obtained indicated that as the impoundment basin filled, it became less efficient at retaining sediments. This may occur because the bottom is subjected to increased wave and current forces as it fills.

g. Prior weir-jetty systems have been located at inlets that typically have minimal amounts of inland-derived sediments. Data obtained at the mouth of the Colorado River suggest significant volumes of riverine materials depositing in the entrance channel and impoundment basin. In future weir-jetty designs at river mouths that carry large sediment loads, both beach and river sediments must be taken into consideration. If the riverborne sediments are expected to pass through the system without creating substantial shoaling problems, care should be taken to situate the impoundment basin so that minimal trapping of the riverborne sediments occurs. This could be done through the use of retaining dykes, by physically separating the basin from the river mouth, or by other creative approaches.

h. The principal management problems at the mouth of the Colorado River are caused by the inadequate size of the impoundment basin and its inefficiency in retaining sediments. The solution to date has been to increase the frequency of the dredging schedule to approximately yearly. This is an effective strategy, but other strategies may be more cost-effective and should be considered.

i. Total dredging costs may be decreased if the impoundment basin is enlarged. There is some, but not a great deal of, area available between the two jetties to enlarge the surface area of the impoundment basin. One possibility would be to widen the basin to include the navigation channel within it. Surveys show that the impoundment basin fills from the weir side first, so the channel portion should be the last to fill. When the basin fills, it would shoal the navigation channel, but that happens now. The larger basin volume would delay the shoaling time. Surveys show that in the present configuration, the channel occasionally migrates and runs through the impoundment basin.

j. Another possibility is to extend the basin landward so that it encroaches on the portion of the beach that is inside the northeast jetty. This is a logical location to increase the impoundment basin size because the basin filled somewhat more rapidly at its shoreward end. However, there are safety concerns for swimmers associated with locating a deep dropoff next to shore. There is also a little room to enlarge the basin along its seaward edge.

k. The impoundment basin could also be somewhat deepened. This would not only provide greater storage capacity, but should also improve its efficiency at retaining sediments, at least shortly after dredging. Doubling the depth would only increase the volume by about 75 percent because of the effect of the 1:5 side slopes, so the best alternative may be to increase both the depth and the surface area.

l. At this time, it is probably too early to seriously consider more drastic solutions such as a redesign of the jetties. However, this may need to be considered as part of long-range plans. Such a redesign should allow for the impoundment basin to be enlarged in size and physically isolated from the riverborne sediments.

m. The area should continue to be monitored. A regular program of beach profile measurements should be taken once every 1 to 2 years for the

next decade or so, to determine if long-term changes are occurring near the mouth. The offshore bathymetry near the mouth should also be occasionally monitored to determine the ebb shoal bar response to the jetties.

n. It is important for the project design to have flexibility to allow for modifications of the size and shape of the impoundment basin based on operational experience.

East Pass Inlet, Destin, Florida

East Pass Inlet (Figure 25) is located at Destin, FL, and connects the Gulf of Mexico with the Choctawhatchee Bay.

Figure 25. East Pass Inlet, Destin, FL (after Morang 1992)

Item monitored

Weir-jetty.

Period monitored

1983 – 1991 monitoring.

Reason(s) for monitoring

Monitoring of waves, currents, tidal elevations, bathymetry, and shoreline changes at East Pass Inlet was conducted during the time period 1983 – 1991 to better understand the inlet's behavior during the past 120 years: (a) pre-1928 – spit development and breaching, covering the period when the inlet was oriented northwest-southeast between Choctawhatchee Bay and the Gulf of Mexico; (b) 1928-1968 – stable throat position but main ebb channel that migrated over a developing ebb-tidal shoal; covers the time after the inlet breached Santa Rosa Island in a north-south direction and then migrated eastward; and (c) after rubble-mound arrowhead jetties with sand bypassing weir-jetty in west jetty were built – the throat and main ebb channel were stabilized while the ebb-tidal shoal grew. Because of uncertainties regarding its effectiveness, the weir was closed in 1986 (Morang 1992).

Results of the 1983 – 1991 monitoring

Details of monitoring are:

a. Based on historical review and on analyses of data collected during this monitoring project, the project has performed as the original designers intended. Navigation through the inlet has been enhanced. The mouth of the inlet has been stabilized for the past 22 years, and the jetties have (at least temporarily) stopped the inlet's eastward migration. The structural design of the jetties was sound, and they have suffered only minor damage (the original sheet-pile weir failed and the spur jetty, built later, has partly slumped). The weir did allow littoral drift to enter the deposition basin. Maintaining the 3.7-m- (12-ft-) deep channel has required annual dredging of 74,000 cu m (97,000 cu yd), within the predicted range.

b. Geological evidence suggests that the jetties have reduced the amount of sand entering the inlet. The sand in littoral transport is now bypassing the mouth of the inlet. Some of this sand may be accumulating on the ebb-tidal shoal, but since the beaches to the east and west of the shoal are not eroding, it is reasonable to assume that a significant proportion of the sand bypasses. The arrowhead configuration of the jetties may result in flow fields that are unable to carry much sand into the inlet.

c. The following evidence supports the hypothesis that physical processes are still attempting to force the inlet east: (1) Norriego Point is eroding; (2) the thalweg migrated east within the inlet after the jetties were built and now hugs the east shoreline from the spur jetty north for about 610 m (2,000 ft).

d. The driving forces of the eastward migration of the channel are believed to be: (1) wave forces—predominant wave direction from 1987 to 1990 was from the southwest while the shoreline trends approximately east-west; and (2) currents within the inlet—geometry of the flood-tidal shoal and its associated channels cause the currents south of the highway bridge to flow northwest-southeast, and they must turn in the region between the jetties and the highway bridge. The inlet's east shore (Norriego Point), being the outer side of this turn, is eroded by the tremendous amount of water flowing against it.

e. The former weir has been one of the most contentious parts of the East Pass project. It was located on the correct side of the inlet. It allowed littoral drift to enter the inlet and settle into a deposition basin. After the weir was closed in 1986, the beach west of the west jetty grew seaward, confirming that eastward-flowing littoral currents carry a significant amount of sand.

f. The original sheet-pile weir was incorrectly designed and collapsed within a few months after construction. The repair with a rubble-mound structure similar to the main jetties was entirely successful.

g. The long-term functioning of the weir as a mechanism to allow sand to be bypassed by dredge to the other side of the inlet is unknown because the deposition basin was dredged only from 1968 to 1972. The reasons for discontinuing basin dredging are obscure. During the first few years after construction ended, the entire inlet system was adjusting to the new jetties, and the weir's performance during this period may not have been representative of the longer term. A project should be maintained as designed unless long-term or overwhelming evidence indicates that changes are needed. If maintenance practices are frequently adjusted, it is almost impossible to determine how successful the project has performed and what lessons can be learned to improve future projects.

h. One of the primary objectives of this monitoring was to evaluate how the stability of the jetty system could be improved. Realignment of the navigation channel may reduce the maintenance dredging but will not affect scour at the jetties nor reduce the eastward migration of the thalweg.

i. Pertaining to overall stability, East Pass could be rerouted to follow the Old Pass Lagoon Channel. This route had been stable for over 55 years before the 1928 breakthrough. Even today, the currents measured south of the highway bridge flow in directions similar to the orientation of Old Pass Lagoon.

j. If the existing East Pass Inlet is to be maintained, the following practices might reduce Norriego Point erosion: (1) the shoreline facing the inlet, from the northern tip of Norriego Point to the north end of the east jetty (1,525 m (5,000 ft)) could be armored. An alternative might be a sheet-pile wall with a scour apron; (2) a guide wall or series of walls could possible be built to deflect currents away from Norriego Point; and (3) a dredge could be kept onsite to dredge the Old Pass Lagoon entrance channel whenever necessary, and renourish Norriego Point.

k. The following might prevent scour at the jetties: (1) the spur jetty can be rebuilt with extensive toe protection to prevent collapse. The scour hole near the tip of the spur would have to be filled and then armored to prevent future scour. While the use of concrete and rubble fill in the past provided only temporary relief, an engineered approach employing precisely placed armor units might be more successful. A design using graded-stone layers might also be successful; and (2) the scour hole at the tip of the west jetty should also be filled and capped with armor stone to prevent damage to the jetty.

l. It must be emphasized that a comprehensive engineering study would be necessary before any of these or other alternatives could be implemented. It would also be necessary to evaluate how construction or modification in one part of the inlet might affect processes in another part.

6 Site-Specific Lessons Learned, Atlantic Coast of the U.S. Mainland

Carolina Beach, North Carolina

Carolina Beach, North Carolina, is located approximately 24 km (15 miles) south-southeast of Wilmington, NC, on a peninsula which separates the lower Cape Fear River Estuary from the Atlantic Ocean (Figure 26).

Item monitored

Beach nourishment and sediment transport.

Period monitored

April 1981 – September 1984 monitoring.

Reason(s) for monitoring

The objectives of monitoring the beach fill and inlet sediment trap during the time period April 1981 – September 1984 were to determine the adequacy of the trap to serve as a primary source of beach nourishment material for the project, and to assess the impact of the trap on sediment transport at the inlet's ebb tide channel and delta (Jarrett and Hemsley 1988).

Results of the April 1981 – September 1984 monitoring

Details of monitoring are:

a. The Carolina Beach project performed generally as expected, but experienced slightly higher volume losses than originally anticipated. Until the material in the construction berm along the southern 3,050 m (10,000 ft) of the project moves to the lower portion of the active profile, volume losses along this segment will probably continue to be high.

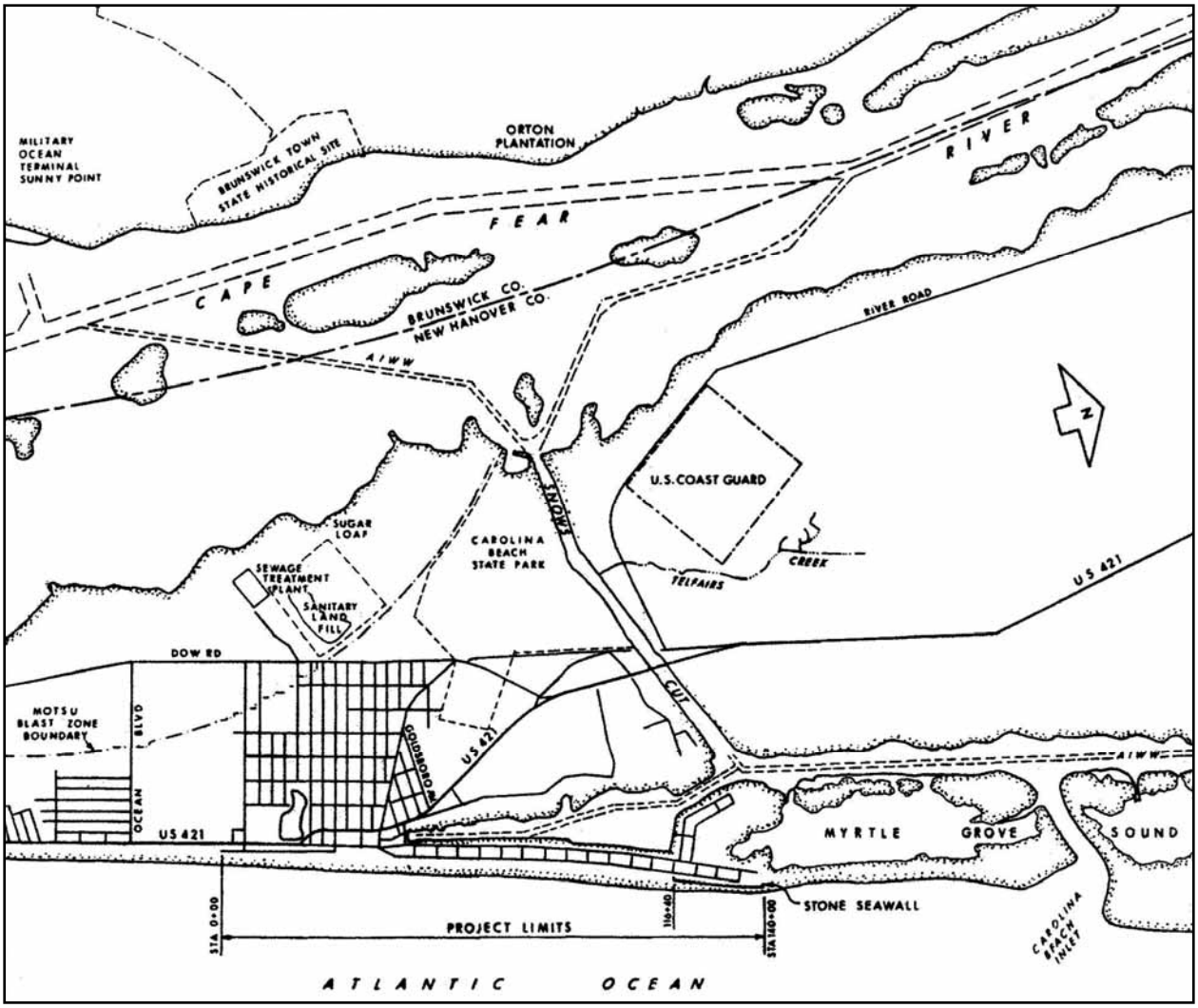

Figure 26. Carolina Beach, North Carolina (after Jarrett and Hemsley 1988)

Eventually, the southern 3,050-m (10,000-ft) segment is expected to attain some degree of stability. Along the northern 1,220 m (4,000 ft), the sediment deficit caused by Carolina Beach Inlet remains a problem and will require a continuous renourishment program with materials obtained from the Carolina Beach Inlet sediment trap.

b. The sediment trap in Carolina Beach Inlet functioned fairly well but was located too close to the main flow through the inlet to be completely effective. Relocation of the sediment trap seaward and away from the main flow should greatly enhance its overall sand trapping ability. Dredging of the channel through the ebb tide delta eliminated the opportunity to evaluate the trap's affect on the delta and ebb tide channel.

c. In computing the volume of material required to construct a beach fill having a certain width, the designer must assume that the improved beach profile will parallel the existing beach profile down to some depth

of closure. For example, at Carolina Beach, profile slopes seaward of the 7.6-m (25-ft) depth are relatively flat and generally outside the normal influence of littoral forces. Therefore, in this instance, design volumes were computed assuming that the improved beach profile would parallel the existing bottom out to -7.6 m (-25 ft).

d. Once the design volume is determined, the only practical way to construct the fill is to place the required quantity on the beach in the form of a sacrificial construction berm. The crest elevation of the construction berm should be equal to the natural berm elevation in the area. The width of the construction berm will depend on the slopes that the material assumes during placement and the volume of material to be placed. Since this slope is not generally known beforehand, surveys should be conducted during placement to ensure that the correct volume of material is distributed along the beach. Once in place, the construction berm material will be displaced to the deeper portions of the active profile by wave action.

e. Beach fills should be designed with adequate transitions from the artificial beach back to the natural beach. If the transition is too sharp, material will be eroded from the ends of the fill at a rapid rate and could be transported out of the project area.

f. Sediment traps in tidal inlets should be located in areas removed from the concentrated tidal flows. For example, an ideal location for a sediment trap would be in the area of an existing interior shoal that is fed with littoral material moving off the inlet shoulders. In the case of Carolina Beach Inlet, much of the trap was located in the area of concentrated tidal flows and, as a result, the trap only filled to about 66 percent of its dredged capacity. The trap should also be dredged as deep as possible but not deep enough to create problems with sloughing of the adjacent shorelines into the trap.

Ocean City Inlet, Ocean City, Maryland

Ocean City, Maryland, located on Fenwick Island, is part of the central Delaware-Maryland-Virginia (Delmarva) barrier island chain (Figure 27). Ocean City is situated about 55 km (35 miles) south of the entrance to Delaware Bay and about 170 km (105 miles) north of the Virginia Capes.

Item monitored

Inlet.

Period monitored

October 1986 – January 1989 monitoring.

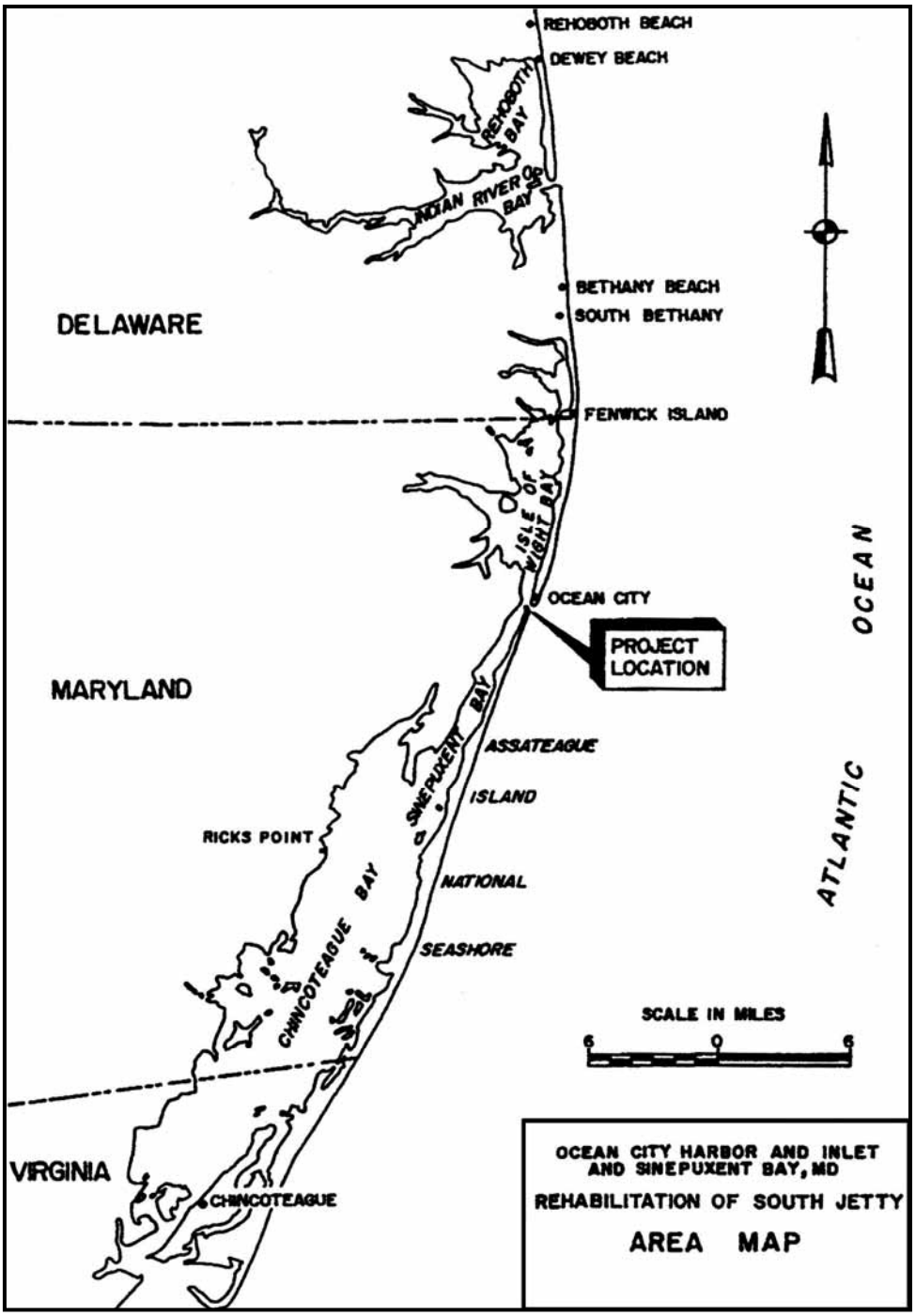

Figure 27. Ocean City Inlet, Ocean City, Maryland (after Bass et al. 1994)

Reason(s) for monitoring

Monitoring was conducted at the inlet during the time period October 1986 – January 1989 to: (a) verify studies relating to the cause of the problem shoal; (b) evaluate the effectiveness of the rehabilitated jetty cross section as a littoral barrier; (c) evaluate the effectiveness of the shoreline stabilization on the

northern shoreline of Assateague Island; (d) verify/calibrate the Shore Protection Manual Longshore Transport formula; (e) examine the distribution of longshore transport across the surf zone; (f) analyze the shoreline and profile response following rehabilitation of the jetty; (g) evaluate the ebb shoal equilibrium and northern Assateague Island growth; and (h) evaluate scour hole stabilization (Bass et al. 1994).

Results of the October 1986 – January 1989 monitoring

Details of monitoring are:

a. Construction of jetties caused establishment of a new equilibrium for the inlet ebb tidal delta system. Bathymetric measurements over the shoals and surveys along adjacent shorelines are required over an extended time period to establish the new equilibrium. When an equilibrium state is reached, natural bypassing may resume via the ebb-tidal delta.

b. Sealing the south jetty to prevent passage of sand was effective in preventing shoaling in the inlet.

c. Sealing of a jetty can result in erosion of a shoreline inside a jettied inlet, when that shoreline was previously nourished by sand passing through the jetty. Protective measures may be required for a shoreline inside a jettied inlet concurrent with sealing a jetty.

d. The sand-tightened cross section of the jetty has functioned as an effective littoral barrier and has trapped a high percentage of longshore transported material in this area, as evidenced by the accretion south of the jetty. Prior to reconstruction this material would have been permitted to pass through the jetty into the problem area. As a result of the elevation of this sediment accretion relative to the crest elevation of the south jetty, some transport of sand due to wind action is occurring.

e. The shoreline south of the jetty sometimes advanced and sometimes retreated after the sealing of the jetty. This could result from any one or a combination of the following effects: (1) shifts may occur in predominant wave direction and associated net littoral transport direction over longer periods of time; (2) slugs of material may bypass via the ebb-tidal delta causing cyclic shoreline movement, with accretion (shoreline advance) as they weld to the shoreline, and apparent erosion (shoreline retreat) as the material moves farther along the shoreline, consistent with the August 1986 to December 1987 record of shoreline position; and/or (3) the beach may steepen and flatten under different wave climates (i.e., there may be seasonal beaches). Records should be studied for extended time periods if there is any evidence of possible reversals. Single-year, or even multi-year, analysis is not sufficient.

f. Average shoreline configuration between segmented “headland” breakwaters used to prevent shoreline erosion can be generally predicted based on empirical understandings, considering the combined effects of tidal currents and variations in wave conditions.

g. The headland breakwaters constructed along the northern shoreline of Assateague Island have effectively stabilized the shoreline. The crenulated embayments between the breakwater segments have, in general, evolved as expected following breakwater construction. The response of the shoreline in this area confirms the preconstruction concerns regarding the potential erosion of the shoreline following the south jetty rehabilitation. Without the headland breakwaters, it is probable that significant erosion of the northern Assateague shoreline would have occurred.

h. Filling a scour hole at the end of a jetty and covering the area with a layer of armor stone is effective in preventing further scouring.

i. Following rehabilitation, the south jetty performed as expected, confirming the conclusions of preconstruction studies regarding the source of the finger shoal material. The rehabilitated jetty successfully acted to eliminate the source of material to the problem shoal area, as demonstrated by the significant accretion of sand on the south side of the jetty and stabilization of northern Assateague Island. As a result, the problem finger shoal was eliminated.

j. The distribution of longshore transport across the surf zone, as represented by the distribution of volumetric accretion on the south side of the jetty, showed trends similar to previous investigations. However, the lack of directional wave data precluded any in-depth analysis of the distribution.

k. The response of Assateague's northern shoreline and the profiles south of the jetty were generally as expected, particularly the oceanward advancement of the shoreline resulting from the enhanced sand-trapping ability of the rehabilitated jetty. Accretion from the sand tightening caused initial steepening of the profiles near the jetty. Later, offshore transport of sand occurred with the subsequent flattening of the profiles.

l. The volume of sediment contained in Ocean City Inlet's ebb-tidal delta increased steadily from 1934 to 1967. Since 1967, the rate of shoal growth has decreased markedly. The averaged growth rates were shown to be 270,000 cu m per year (353,000 cu yd per year) and 30,00 cu m per year (39,000 cu yd per year), respectively. These data show that the volume of the ebb-tidal delta increased rapidly after jetty construction but has gradually tapered to present day rates as a state of equilibrium is approached. Northern Assateague Island has exhibited a decrease in shoreline erosion since the late 1960s, corresponding to the slower rate of shoal growth since 1967. As the system moves toward equilibrium, more and more sediment may be bypassed, resulting in less severe erosion of the northern Assateague shoreline.

m. Results from this study of ebb shoal evolution suggest that natural bypassing of sand around Ocean City Inlet is an important process influencing shoreline change in the region. Further investigations are planned to fully document sediment bypassing at Ocean City Inlet. These include (1) updating shoreline change measurements, (2) adding and updating bathymetry data, and (3) correlating process data (storms, waves, etc.) with measured shoal growth since 1967. This information

should help improve shoreline erosion prediction estimates for northern Assateague Island. Preliminary examination of bathymetric shoal data, along with shoreline change data, support predictions of reduced erosion rates along Northern Assateague Island, suggesting that natural sediment bypassing is presently occurring.

Barnegat Inlet, New Jersey

Barnegat Inlet, New Jersey (Figure 28) is a stabilized inlet centrally located on the Atlantic coast approximately 80 km (50 miles) south of Sandy Hook, New Jersey, and 112 km (70 miles) northeast of Cape May, New Jersey. The overall orientation of the New Jersey coastline changes in this area from northeast to north-northeast. The inlet separates Island Beach State Park (to the north) from Long Beach Island (to the south), and serves as the primary link between the Atlantic Ocean and Barnegat Bay. There are no major rivers contributing to the Bay system.

Items monitored

South jetty and inlet.

Period monitored

October 1992 – September 1997 monitoring.

Reason(s) for monitoring

Evaluation of the effectiveness of a new south jetty at Barnegat Inlet, New Jersey, (completed in June 1991) on the inlet system needed to be ascertained to provide improved inlet and jetty system design guidance, to enhance construction of rubble-mound jetties, and to develop better maintenance techniques for tidal inlets. Monitoring was conducted during the time period October 1992 – September 1997. The new jetty was constructed parallel to the north jetty, and replaced an existing southern arrowhead jetty. The project performance was assessed with regard to providing a stable navigation channel and a stable rubble-mound jetty structure, and was then compared with project design, physical model predictions, and other design criteria.

The monitoring plan evaluated four fundamental hypotheses of the project design objectives: (a) the new south jetty and new channel alignment will not adversely affect tidal hydraulic response or high tide level in the inlet bay system (i.e., no flooding problem), and prototype hydraulic response will be as predicted by the physical model evaluation; (b) the new south jetty realignment will improve navigation safety by stabilizing the navigation channel location and depth between the jetties and over the outer bar (ebb-tidal shoal), and will eliminate dredging in these regions; (c) the new south jetty will be structurally

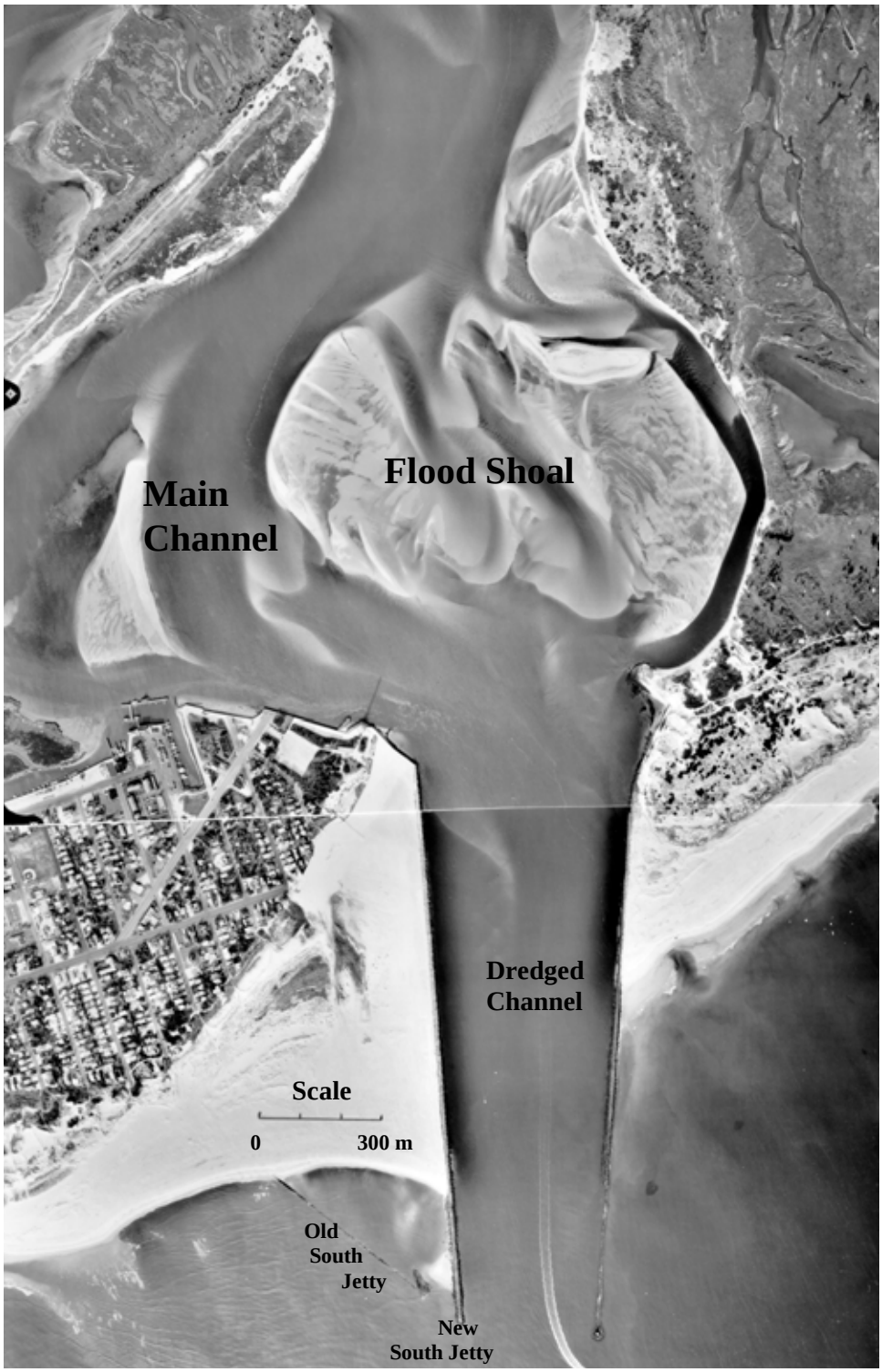

Figure 28. Barnegat Inlet, New Jersey (after Seabergh et al. 2003)

stable; and (d) the jetty system realignment will not adversely affect upcoast or downcoast beaches (Seabergh et al. 2003).

Results of the October 1992 – September 1997 monitoring

Details of monitoring are:

a. The hydraulic condition indicates a small bay tide range of 0.1 to 0.2 m (0.3 to 0.7 ft) relative to the ocean tide, because of the large bay size in relation to the inlet's cross-sectional area at the throat. Tidal prisms based on velocity measurements indicate the inlet has returned to prism magnitudes measured in preproject years. The oceanward side of the inlet gorge between the jetties is much deeper for the high parallel jetty system (keeping sediment out and promoting channel efficiency through higher velocities) than for the low elevation arrowhead jetty system. There has been no adverse impact by the new south jetty on either of the velocities in the inlet or tidal prism elevations.

b. Today's inlet, which has effectively adjusted to the new parallel south jetty, allows a more stable channel to exist along the north jetty as a result of the restriction of sediment input into the navigation channel. The more stable channel results in improved navigation conditions, although dredging is still required to remove shoal material which accumulates between the jetties.

c. The new south jetty clearly exhibits some degree of imperfection. On average, the structure has a Condition Index (CI) of at least 70, indicating the condition level is very good, even though minor deterioration and defects are evident. Although slight imperfections may exist locally, there exists no significant defects that would indicate imminent failure of the structure. All things considered, the new south jetty appears fundamentally sound and is serving the functional purpose for which it was developed.

d. The amount of beach profile change as determined by full profile surveys from the berm out to -9.1 m (-30 ft) mllw on both sides of the inlet are well within the realm of natural beach processes as approximated from net potential longshore sediment transport computation. The construction of the new south jetty entirely within the confines of the previously existing arrowhead jetty precludes the new south jetty from having any effect on either the upcoast or downcoast beaches.

Manasquan Inlet, New Jersey

Manasquan Inlet (Figure 29) is located at Point Pleasant, NJ, on the Atlantic coast of New Jersey, approximately 42 km (26 miles) south of Sandy Hook and 37 km (43 miles) north of Barnegat Inlet. The Inlet provides the northernmost connection between the ocean and the New Jersey Intracoastal Waterway. On a number of occasions prior to jetty completion in 1931, the Inlet closed completely.

Figure 29. Manasquan Inlet, Point Pleasant, NJ (after Gebert and Hemsley 1991)

Items monitored

Jetties and inlet.

Periods monitored

June 1982 – October 1984 monitoring, August 1994 periodic inspection, and October 1998 periodic inspection.

Reason(s) for monitoring

Monitoring was conducted from June 1982 – October 1984 to: (a) evaluate the performance of the dolos-armored units in maintaining structural stability of the jetties; (b) determine potential effects of the rehabilitated jetties on longshore sediment movement at the inlet; and (c) determine the effectiveness of the rehabilitated jetties in maintaining a stable inlet cross section. A periodic inspection was conducted in August 1994 to reexamine the dolos portions of the Manasquan Inlet jetties and determine changes that have occurred since prior monitoring ended in 1984. A second periodic inspection was conducted in October 1998 to determine dolos changes that might have occurred since the last inspection in 1994 (Gebert and Hemsley 1991; Bottin and Gebert 1995; Bottin and Rothert 1999).

Results of the June 1982 – October 1984 monitoring

Details of monitoring are:

a. Even though the jetty structures have experienced a near-design storm, they have continued to perform successfully and have not required even the low level of maintenance anticipated by the designers. This overall excellent performance of the jetties and, in particular, the low percentage of broken dolosse during the March 1984 storm serve to verify the design and construction procedures used in the rehabilitation.

b. There was a threshold of breakage of a dolos-armored structure beyond which the structure was likely to fail. The jetties at Manasquan are below this threshold and have remained stable even through a near-design storm.

c. Use of photogrammetric mapping of the jetties allowed a detailed evaluation of the motion of the armor units. This technique was found to be cost-effective and accurate, providing accuracy comparable with standard leveling techniques. Comparison of the results of this study with preliminary results from Crescent City, CA, seems to verify an hypothesis that dolos-armored units on flatter slopes tend to be forced up the slope by forces associated with wave runup, while those on steeper slopes, such as at Manasquan, will be moved downslope by wave rundown.

d. While photogrammetric mapping has been applied to dolos armor in this study, it is equally applicable to structures with any type of natural or man-made armor. The accuracy of photogrammetry is more than adequate to evaluate armor unit movement. Periodic mapping of a coastal structure would permit detection of incipient or progressive failure along any visible portion of the structure before such a problem was readily detected by other means. This detection would allow for early assessment and possible correction of the problem.

e. Photogrammetry offers several advantages over conventional land surveying techniques. First, it is possible to map armor units at or near the waterline of the structure, units that would be inaccessible or too hazardous to reach on foot. Second, photogrammetry is flexible in that all the information needed to perform the mapping can be obtained almost instantaneously, permanently, and at fixed cost with one aerial photographic flight. The mapping can then be performed at any time thereafter or not at all, depending on available resources, need for information, etc. In contrast, land survey methods capable of obtaining the location, orientation, and elevation data for mapping every visible armor unit are labor-intensive and would require more time and expense than photogrammetry. Had both base maps been prepared at the same time at Manasquan, the total cost of the initial, and most detailed, mapping of the jetties would have been about $6,000. For that amount, a map was produced of all visible dolosse with the positions of several points established on each of the 754 dolosse. The cost of leveling a total of 160 dolosse was estimated to be about $3,000. That cost is half of the photogrammetry but produced elevations only on less than 21 percent of

the visible dolosse. With the wider use of total stations, it is now possible to rapidly obtain position data using what has become standard surveying methods, but it is unlikely that improvements in survey techniques will reduce costs enough to challenge the cost-effectiveness of photogrammetry.

f. A final advantage of photogrammetry is that the product is graphical. It is, therefore, more readily interpreted with respect to location and magnitude of armor unit displacements.

g. Periodically, on the order of every 5 years, the jetties should be photogrammetrically mapped. This mapping will provide additional useful information on the long-term stability of dolosse.

h. It is apparent that the dolosse at Manasquan Inlet have benefited from the use of steel reinforcement. Even those units that have cracked have been kept whole by their reinforcement. There are signs that the reinforcing steel may be rusting. This can be seen only on a relatively small number of units, so it too early to speculate on the fate of the dolosse. Reinforcing escalates the cost of casting dolosse, so the decision whether to reinforce the units is still one of cost/benefit, although EM 1110-2-2904 (HQUSACE 1986) provided a rule of thumb for reinforcement. The work being done at Crescent City, CA, will provide some insight into what size dolos should be reinforced. At present, the largest dolosse are often designed for no impacts. However, the use of much of their unreinforced tensile strength is for supporting static loads. Smaller units will certainly move around and could benefit the most from reinforcement. The decision to reinforce dolos-armored units will continue to be based on engineering judgment until more information is acquired concerning the long-term effects of rust, the benefits associated with units maintaining their integrity even though cracked, and a better understanding of the relationship between impact load, static load, pulsating wave load, and dolos breakage.

i. Reinforcement of dolosse remains a matter of engineering judgment. Additional information is needed before guidelines can be developed for the use of reinforcing steel in dolosse. The USAED, Philadelphia, is encouraged to continue to evaluate the condition of the dolosse at Manasquan during site visits. Emphasis should be placed on indications that the reinforcing steel is rusting, including rust stains and spalled concrete.

j. Through the monitoring program, the value of sand-tightening the jetties was demonstrated. The jetties, particularly the south jetty. were quite porous, allowing considerable sand through the structure into the channel. There has been no maintenance dredging in Manasquan Inlet since the rehabilitation, another testimony to the design and the concept of sand-tight structures. In situations where porous structures contribute to shoaling of a channel, the economics of rehabilitating the structures should be investigated. The monitoring has shown that the sand-tight structures have had little apparent effect on the tidal prism.

k. Any monitoring program is at the whim of nature. Such an effort must have a finite life, during which it is hoped that there will be a significant

test of the structures. At Manasquan, there was such a test. The rehabilitation survived a near-design storm in late March 1984. A particular success of the monitoring was the collection of an excellent storm data set, one of the most complete ever collected in the United States.

l. Based on its success at Cleveland, OH, side-scan sonar was used at Manasquan to evaluate the condition of the underwater portions of the structures. It is recommended for inspection quality control during underwater placement of armor or for identifying problem areas after construction.

m. Use of the equation relating critical inlet cross-sectional area and tidal prism is appropriate for inlets that have exhibited historic stability.

n. Based on the studies that have been performed on sand transport in the Manasquan Inlet area, the use of Wave Information Studies (WIS) data seems to have the most potential for predicting sand transport with reasonable accuracy. Littoral Environmental Observations (LEO) data (formerly, but no longer collected), should be used for calculating sand transport with caution, because of the inherent inaccuracies involved in making the observations.

o. The data set from the March 1984 northeaster is one of the most complete data sets available in the United States. Researchers are encouraged to make use of these data and those collected at the Coastal Engineering Research Center Field Research Facility during Hurricane Gloria.

p. The monitoring effort at Manasquan Inlet has been quite successful. Data obtained have verified the excellent performance of the rehabilitated jetties, even in a near-design event; photogrammetry has been shown as a viable technique for monitoring the stability of coastal structures; and additional information has been gathered concerning design techniques used by coastal engineers.

Results of the August 1994 periodic inspection

Details of the periodic inspection are:

a. Results of the periodic inspection during August 1994, through photogrammetric analysis, indicate that dolosse on both jetties have been dynamic since their placement. Horizontal movement has ranged up to 2 m (6.6 ft), and vertical displacement (subsidence) as much as 1.6 m (5.3 ft). Most movements in both directions, however, have been less than 0.3 m (1.0 ft).

b. Horizontal movement for the majority of the dolosse has been relatively uniform (the entire unit moved in the same direction as opposed to rotating). Vertical motions revealed that most dolosse have subsided slightly. The downslope portions of the armor units, in general, tended to subside more than the upslope portions.

c. Photogrammetric maps also revealed missing dolosse at the waterline along the head of the north jetty on its channel side. Seventeen broken

armor units were identified in 1994 as opposed to five in 1984. The only area of concern was at the head of the south jetty, where a broken dolos resulted in exposure of core stone under the jetty cap.

d. To maintain the design cross-sectional stability of the structure, additional armor units are required in the void along the inside head of the north jetty and at the tip of the south jetty where core stone is exposed. Otherwise, the jetties appear to be in good structural condition. (Note: Repairs of the jetty voids with CORE-LOC™ armor units are scheduled during the late summer/early fall 1997 time frame).

Results of the October 1998 periodic inspection

Details of the periodic inspection are:

a. The periodic inspection of October 1998 entailed reestablishing targets and conducting limited ground-based surveys, aerial photography, photogrammetric analysis, and a broken armor unit survey for comparison with data obtained in 1994. Results of the monitoring effort indicated that dolos movement was less dynamic during the period 1994-l 998 as opposed to previous survey periods. Maximum horizontal movement detected was 0.7 m (2.3 ft), and maximum vertical displacement was 0.3 m (1.2 ft). In general, however, most movements in both the horizontal and vertical directions were less that 0.06 m (0.2 ft).

b. Horizontal movement for the majority of the dolos was relatively uniform (the entire unit moved in the same direction as opposed to rotating). Of the units that rotated, however, the majority on the north jetty rotated in a clockwise direction, while those on the south jetty rotated in a counterclockwise direction. Vertical motions revealed that some units moved upward slightly and some subsided slightly. Average vertical movements were on the order of about 0.06 m (0.2 ft). Even though major storms occurred during the period, the 14,528-kg (16-ton) dolosse appeared to have settled into the structure and became relatively stable.

c. During October 1997, void areas in both jetties (identified in previous surveys) were rehabilitated with 17,235-kg (19-ton) CORE-LOC armor units. Twenty-nine (29) CORE-LOCs were placed on the north jetty and 16 were placed on the south jetty interlocking with the existing dolosse. Some dolosse were repositioned to improve interlocking and provide space for the new CORE-LOCs into the overall protection scheme. Several broken dolosse also were removed from the structures. The new CORE-LOC armor units were targeted, and base data relative to their positions were established.

d. Eight broken dolos-armored units, four on each structure, were documented during the 1998 survey. Two units were newly broken since the 1994 survey. A total of 17 broken units were observed in 1994, but many were removed during the 1997 CORE-LOC rehabilitation. The jetties appear to be in excellent condition.

Oakland Beach, Warwick, Rhode Island

Oakland Beach (Figure 30) is located in Warwick, RI, approximately 16 km (10 miles) south of Providence, RI, and 24 km (15 miles) north of Newport, RI. The beach is in the upper portion of Narragansett Bay at the southern extremity of a point of lank known as Horse Neck. It faces Greenwich Bay to the south and is bordered by Warwick Neck to the east and brush Neck to the west.

Figure 30. Oakland Beach, Warwick, RI (after LeBlanc and Bottin 1992)

Item monitored

Beach nourishment and sediment transport.

Period monitored

April 1982 – April 1985 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period April 1982 – April 1985 to perform an assessment of the Beach Erosion Control Project at Oakland Beach with respect to beach nourishment and sediment transport. Monitoring included hydrographic and topographic surveys of the beach and nearshore area, aerial and ground photographs, wind data, littoral environmental observations, and sediment sampling. Littoral transport, structure (groin) stability, and wind and wave data were evaluated (LeBlanc and Bottin 1992).

Results of the April 1982 – April 1985 monitoring

Details of monitoring are:

a. There is a general trend of erosion (offshore movement) during storm conditions, and accretion (onshore movement) during mild wave conditions, similar to what is found on the open ocean coast.

b. Careful placement of profile lines is required for shorelines that are scalloped (e.g., where sand accumulates at groins). Linear interpolation between survey lines can give a misleading picture of the 3-D beachface.

c. Use of fill material coarser than the native material worked well at Oakland Beach, and appeared to result in better retention of the beachfill. This technique should be considered in the future in areas where a low wave climate exists and where the coarser material would be acceptable to the users of a recreational beach.

d. Presence of foreign material (e.g., glass fragments) on the beach can bias grain size analysis.

e. Material at Oakland Beach will eventually be lost around the terminal groin unless the beach is periodically reshaped.

f. Beaches that have winter ice cover may be protected from erosion during the winter storm season.

g. The Shore Protection Manual (SPM) method (HQUSACE 1984) for adjusting winds measured over land to a site on the coast should be used with care in areas not similar to the Great Lakes regimen where it was developed. Otherwise, winds at the coast may be over- or under-predicted. At Oakland Beach, because of the different nature of the site, the adjustment would have produced information noticeably different from that measured at the site.

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

i. The sand fill material placed at the site has been resistant to offshore loss.

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

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

l. All structures (groins) remained stable and in good condition throughout the monitoring period.

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

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

Boston Harbor, Boston, Massachusetts

The Boston Harbor Navigation Improvement Project (BHNIP), Boston, MA, (Figure 31) involves deepening of the main ship channel and three tributary channels to the Inner Harbor, and associated berthing areas. Lack of an upland disposal site and resource agency denial of permission to place and cap the contaminated sediments at an open-water site resulted in the decision to use in-channel Confined Aquatic Disposal (CAD) cells (Figure 32) for placement of contaminated material that would be dredged with an environmentally sensitive clamshell bucket.

Item monitored

Confined Aquatic Disposal (CAD) cells.

Period monitored

October 1998 – September 2001 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period October 1998 – September 2001 to determine the effectiveness of using CAD cells in Boston Harbor. The monitoring plan was composed of three primary activities: (a) water quality monitoring of suspended solids near the operation of two environmentally sensitive clamshell dredges and a normal clamshell to document the benefits of the special clamshell buckets; (b) monitoring contaminated dredged material consolidation and strength prior to and after placing the sand cap; and (c) calculating cap erosion predictions from both tidal currents and ship propeller wash to characterize the likely amount of cap damage to be expected from either source (compiled by Hales 2001).

Results of the October 1998 – September 2001 monitoring

Details of monitoring are:

a. Sediment resuspension and loading characteristics were studied under near-similar operating and environmental conditions for three clamshell dredge buckets: (1) Great Lakes Dredge and Dock (GLDD) Conventional (open-faced); (2) GLDD Enclosed; and (3) CableArm™. The Enclosed bucket had the lowest overall turbidity, with substantially less in the middle of the water column. However, the District expressed concern that the enclosed buckets were adding additional water to the already soft and weak sediments, possibly causing a further reduction of the bearing capacity of the sediments.

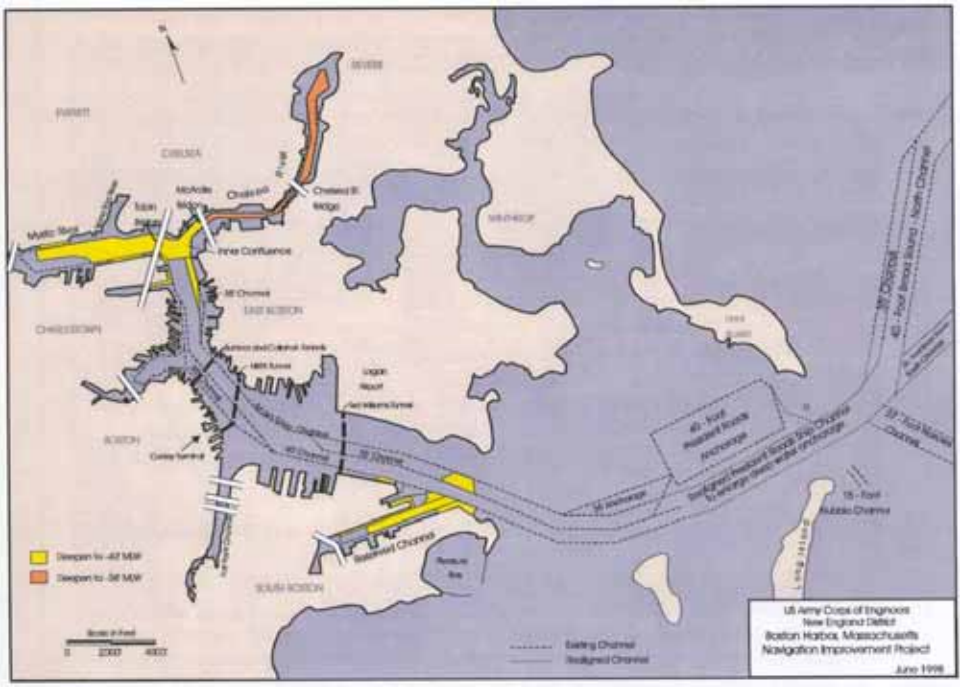

Figure 31. Boston Harbor Navigation Improvement Project, Boston, MA (after Hales 2001)

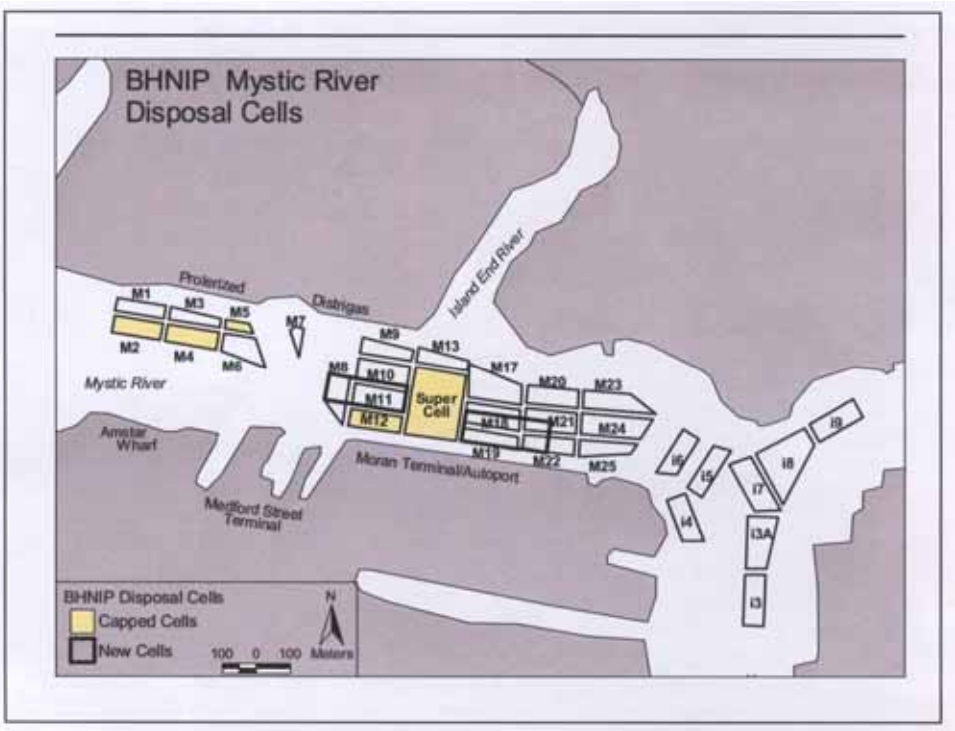

Figure 32. Confined Aquatic Disposal (CAD) cells being capped, Boston Harbor Navigation Improvement Project, Boston, MA (after Hales 2001)

b. Grab samples and core samples indicated that: (1) natural cohesion and strength of the sediments were altered by the dredging process, resulting in sediments in the CAD cells that were unstable because of high water content and low shear strength; (2) excess pore water was released not only through the cap but also was vented through diapir structures that served to breach the caps in discrete areas; and (3) future projects should include an evaluation of the in situ strength of the material to be capped and the porosity and permeability of the CAD cell sediments.

c. Laboratory modeling of the subaqueous sand capping process was conducted to allow a comparison to field performance. Simulations indicated the sand cap was stable when placed on top of clay material having undrained shear strengths greater than 17 psf (0.8 kPa) and water contents below 100 percent.

d. Underway measurements were obtained of temperature, salinity, turbidity, currents, and acoustic backscatter intensity within the water column. Data were acquired behind the 274-m- (90-ft-) long liquid natural gas (LNG) carrier *MV Matthew*. The track of the LNG carrier passed over uncapped CAD Cell M8/M11 and the capped Supercell. The 10.7-m (35-ft) draft of this vessel was approximately 88 percent of the water depth in the navigable channel. The volume of sediments resuspended from capped and uncapped CAD cells was very small (well less than 1 cu m (1.3 cu yd)) for each vessel passing, and settled to the seafloor within 1 hr of resuspension.

e. The erosion rates of two reconstituted sediments from Boston Harbor were determined as a function of density and shear stress: one from the CAD Cell M8/M11, and one from an area near the CAD cell called the Mid-Channel. Sediment cores were eroded to determine erosion rates as a function of density and shear stress. The erosion patterns were numerically simulated assuming both no-deposition scenarios and deposition scenarios.

f. The maximum depth of erosion assuming no-deposition for Scenario 1 (ship speed = 1.3 m per sec (4.3 ft per sec), water level elevation = +1.7 m (+5.6 ft) mllw) for the Mid-Channel sediments was 86 cm (2.82 ft). The same Scenario 1 with Open Cell sediments resulted in 45 cm (1.14 ft) maximum depth of erosion. The maximum depth of erosion assuming no-deposition for Scenario 2 (ship speed = 1.5 m per sec (4.9 ft per sec), water level elevation = +3.4 m (+1.0 ft) mllw) for the Mid-Channel sediments was 34 cm (1.12 ft). Comparison of the Mid-Channel and Open Cell results indicates that the Mid-Channel sediments are more erosive.

g. For the deposition scenarios, maximum depth of erosion was approximately 15 cm (5.9 in.) for the Mid-Channel sediments for Scenario 1. After redeposition, the 15-cm (5.9-in.) erosion had been reduced to approximately 11 cm (4.3 in.). For Scenario 1 Open Cell, maximum erosion was 12 cm (4.7 in.), and reduced to 8 cm (3.1 in.) after 700 sec. Erosion was near-zero approximately 20 m (65 ft) from the propeller. Maximum change in Scenario 2 Mid-Channel elevation was approximately 11 cm (4.3 in.), and this was reduced to 8 cm (3.1 in.) due to

deposition. For Scenario 2 Open Cell, maximum depth of erosion was approximately 8 cm (3.1 in.) and reduced to 5.5 cm (2.2 in.) after deposition. Erosion was near-zero approximately 25 m (80 ft) away from the propeller.

7 Site-Specific Lessons Learned, Great Lakes

Cattaraugus Creek Harbor, New York

Cattaraugus Creek Harbor is located at the mouth of Cattaraugus Creek (Figure 33) where the Creek begins in the state of New York. The Creek enters Lake Erie approximately 40 km (24 miles) southwest of Buffalo, NY, and 85 km (54 miles) northeast of Erie, PA. The town of Hanover, NY, is on the southern side of the Harbor, and the town of Brant, NY, is on the north side of the Harbor. The Cattaraugus Indian Reservation of the Seneca Nation, New York Indians, occupies the entire northern side of the Creek within the project area.

Figure 33. Cattaraugus Creek Harbor, New York (after Hemsley et al. 1991)

Items monitored

Beach nourishment and sediment transport, jetties, breakwater, and inlet.

Period monitored

May 1983 – December 1985 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period May 1983 – December 1985 to evaluate waves, jetties and breakwater structure stability, beach nourishment and sediment transport, inlet, channel stability, and ice-jam problems resulting from the construction of the project (Hemsley et al. 1991; HQUSACE 1984).

Results of the May 1983 – December 1985 monitoring

Details of monitoring are:

a. Although waves experienced at the project site have been lower than the design wave, localized damage has occurred at the south breakwater head. The primary cause of the damage appears to be stone cracking. The loss of a few stones by shattering causes adjacent stones to collapse into the void, resulting in a steepening of the structure slope. The problem has been recognized as causing damage at other structures. At Cleveland Harbor, Ohio, it was found that the majority of cracked stones were located at or above the waterline.

b. Very little stone movement occurred during the monitoring period, as might be expected since the design conditions were not experienced. Armor on the lakeside of the south breakwater moved perpendicular to the breakwater center line -0.03 to +0.05 m (-0.11 to +0.15 ft) with an average movement inward (landward) of +0.003 m (+0.01 ft). The stones rotated about the target -1.33 to +0.2 deg for an average clockwise movement of 0.23 deg. Vertical stone movement was -0.03 to +0.06 m (-0.1 to +0.2 ft) with an average upheaval of 0.01 m (0.05 ft). Ice had negligible effect upon the structure except for minor damage to one hand rail at the south breakwater head, indicating that stone sizes based on the local wave climate were sufficient for ice conditions as well.

c. Precautions applied during design to protect the structure toe have had the desired effect. Based on experience with structures located on erodible material, USAED, Buffalo, often used additional toe protection, a technique that has repeatedly produced a stable toe. Use of toe protection in this case where the lake bottom is susceptible to erosion has prevented structure failure and should be used in all similar cases.

d. Sediment transport patterns were predicted reasonably well during the design process, but the magnitude of the transport was underestimated. Predicted transport was 17,850 cu m per year (23,350 cu yd per year), while the calculated average transport over the entire area was 73,170 cu m per year (95,700 cu yd per year).

e. When selecting the method for evaluating sediment transport, the designer must be careful. While none appear perfect for any situation, some appear much less sophisticated than others. The selection of a technique must be tempered with experience and specific existing conditions.

f. It was found that the sediment generally got coarser only offshore of the creek entrance because of improved transport of coarse material out of the entrance by the streamflows. Another indication of the improved flows was the appearance of debris on the beach north of the creek mouth. The coarse material and debris were no longer being trapped on a bar at the creek mouth.

g. The shoreline to the south of the structures has responded as expected. Erosion immediately to the north has not occurred. Instead, there has been accretion for a considerable distance to the north, much farther than anticipated.

h. The scour hole that appeared off the south breakwater head, probably resulting from local wave effects and increased currents near the head, later filled. The reason it filled in is not completely understood but may be as a result of the natural bypassing of material around the south breakwater as the fillet grew combined with the transport associated with lower lake levels.

i. The potential for scour near the head of coastal structures and the appearance of such a scour hole at Cattaraugus Creek is additional justification for adequate toe protection. In this case, the scour hole did not have adverse effects, although additional toe protection in the form of a small stone or gravel mat would have prevented the hole from developing. It should be mentioned that a much deeper scour hole recently developed at the head of the Irondequoit west breakwater in Lake Ontario and required filling with large stone. The prevalence of these phenomena, the factors that cause their development, and recommended design criteria should be investigated.

j. While the maintenance dredging had been anticipated, it has not been required until now. The need for dredging now exists more as a result of channel migration near the mouth than of shoaling. With regard to dredging, the project has performed well, although a modification in the channel alignment is being considered. This monitoring study was the catalyst for this recommendation.

k. The use of filter fabric was successful, at least to date. There were obvious indications of transport through the south breakwater lakeward of where the fabric had been installed. There are indications that sediment has been prevented from penetrating the structure and reaching the channel where filter fabric has been used.

l. Recognizing placement limitations such as where large waves are anticipated and an unknown life expectancy, the use of filter fabric to help make a structure impermeable may be a good idea. So far, it has prevented sediment transport through the structure at a relatively small cost. Periodic inspections should note its continued performance.

m. As expected, there was deposition on the inside of bends in the channel and scour on the outside of those bends. The natural relocation of the channel at Cattaraugus Creek is further evidence that consideration should be given to accommodating natural scour at the outside of bends when designing a channel alignment. It is possible that dredging requirement can be reduced.

n. The inability to model lake ice prevented the reproduction of flooding when lake ice stopped ice flows from the stream. This did not allow the problem associated with the north berm to be modeled.

o. The use of ice-breaking equipment to break up harbor ice helped prevent flooding. Buffalo's ice breaker vessel was used for this purpose. As a part of the design process, consideration should be given to whether, and under what conditions, ice-breaking equipment could be used to advantage.

p. The physical model did an excellent job in identifying the best way to eliminate shoaling in the navigation channel, preventing ice jams, recognizing the limitations of the state of the art in modeling lake ice, and designing a channel safe for navigation in high wave conditions. Efforts should continue to improve the capability to model lake ice. This capability would increase the value of physical models where ice conditions must be considered.

q. There needs to be further investigation to identify the cause of stone cracking so the problem, which is becoming significant for structures on the Great Lakes, might be avoided in the future through better material specifications.

r. Experience in the Great Lakes appears to justify the use of 0.9 to 2.0 W stone weight range in design rather than that called for by the SPM (HQUSACE 1984). W is the weight in pounds of an individual armor unit in the primary cover layer. The SPM recommends 0.75 to 1.25 W, with 50 percent of the individual stones weighing more than W. It would be useful to further investigate the performance of structures using both stone weight criteria to identify which provides the most cost-effective design.

s. During the design of a project of this type, it is important to remember all the results of improved flows. Neither the increased transport of coarser material or the increase in debris on the north shore has been a problem, but it is important to be able to predict all the effects. For example, not long after construction of the project, coarser material appeared on the state beach a few miles to the north of the project site.

t. Localized effects such as wave refraction and diffraction near the structures must be considered when performing the design. These can offset potential sediment losses near these structures.

u. Experience at Cattaraugus Creek supports the lowering of the berm on the north side of the creek entrance, if permission can be obtained from the Seneca Nation.

v. The breakwaters at Cattaraugus Creek have performed admirably. This design solution should be considered at locations experiencing similar problems.

Cleveland Harbor, Cleveland, Ohio

Cleveland Harbor, Cleveland, OH (Figure 34), is located on the southern shore of Lake Erie 155 km (96 miles) east of Toledo, OH, and 285 km (176 miles) west of Buffalo, NY. Cleveland Harbor is situated at the mouth of the Cuyahoga River. It extends for a distance of about 7,620 m (25,000 ft) parallel to the shore. Cleveland Harbor is protected by a breakwater system which is over 9,150 m (30,000 ft) in total length. There are two entrances connecting the Harbor to Lake Erie. The west entrance is directly lakeward of the Cuyahoga River mouth, and the east entrance is at the open eastern end of the east breakwater. The east breakwater has had an extensive repair history, with storm damage occurring to cover stone especially on the lakeside.

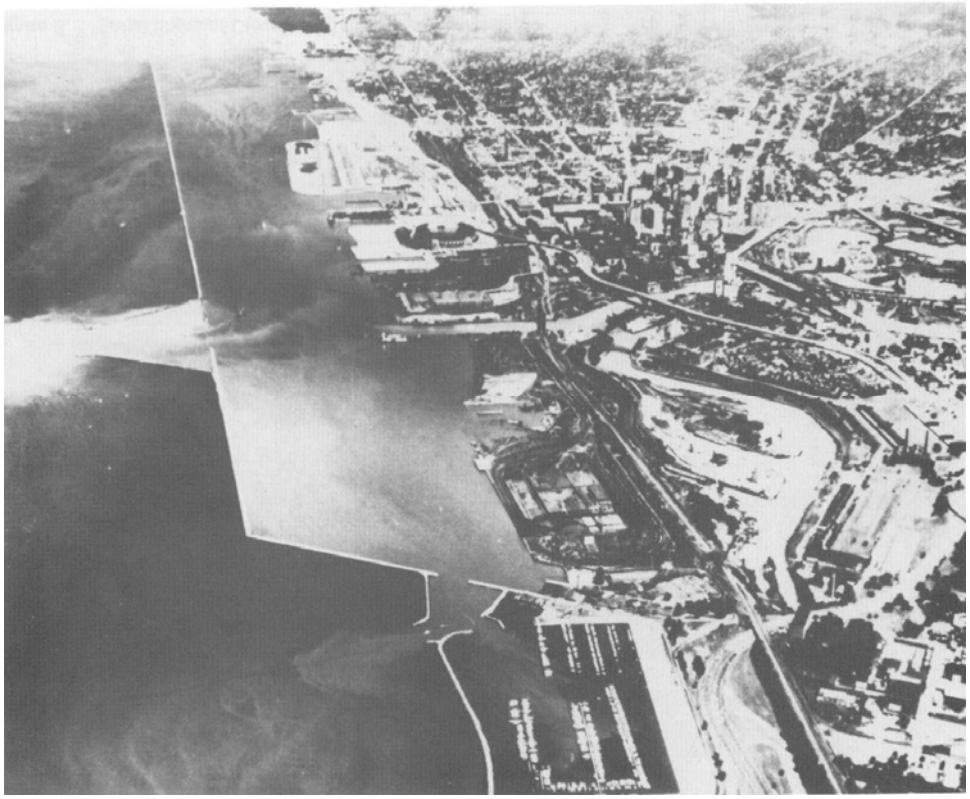

Figure 34. Cleveland Harbor, Cleveland, OH (after Pope et al. 1993)

Item monitored

East breakwater and breakwater stone.

Periods monitored

November 1980 – September 1985 monitoring, and 1995 periodic inspection.

Reason(s) for monitoring

The primary objective of the Cleveland Harbor east breakwater rehabilitation monitoring during time period November 1980 – September 1985 was to determine the stability of a dolos-armored unit cover. This was the first time dolosse were used by the United States in the Great Lakes environment. The monitoring program also evaluated the magnitude of armor unit breakage that could compromise the integrity of the structure. Additional objectives were to: (a) determine wave transmission by overtopping; and (b) document the effects of ice on the stability of dolos units. Under the Periodic Inspections work unit, base conditions were established in 1995 for above-water armor units. Periodic data sets will be obtained to improve knowledge in design, construction, and maintenance of the existing structure as well as proposed future coastal projects (Pope et al. 1993; Bottin et al. 1995).

Results of the November 1980 – September 1985 monitoring

Details of monitoring are:

a. Although the 1,816-kg (2-ton) dolos-armored layer has deteriorated over the years, the breakwater continues to provide the required level of shore protection. Maintenance of the dolos cover has been on an as-needed basis. Repairs, including repositioning and/or the installation of additional armor units, are required after major storms.

b. Wave reflection off the vertical concrete navigation light foundation at the breakwater head appears to contribute to the loss of armor units in that area. Dolos-armored units are very porous when a two-layer thickness is used. Wave energy transmits through the dolosse at Cleveland and reflects back upon them, apparently popping them out of place. Additional layers over reflective surfaces may be prudent for highly porous armor units.

c. As evidenced by significant movement and breakage, the 1,816-kg (2-ton) dolos appears to be under-designed for the Cleveland east breakwater. A 2-D model study also indicated that 3,632-kg (4-ton) armor units (as opposed to 1,816-kg (2-ton)) would decrease the probability of movement.

d. During the monitoring period, the 1,816-kg (2-ton) dolos cover continued to subside and lose elevation. Breakage of armor units also occurred throughout the monitoring period, but the rate of breakage appeared to decrease slightly toward the end of the monitoring period. Most breakage occurred along the waterline in the active wave zone. Little continued breakage was noted below the waterline during diving inspections.

e. Aerial photography of the dolos cover proved to be a useful tool during the monitoring program in spite of the fact that the photos were not completely rectified. Photos were used to evaluate qualitative changes in the armor cover. This photography served as the basis for planning maintenance and repair of damage zones during the monitoring period.

f. Wave gauges were not deployed at Cleveland during the winter months because of the concern that they would be lost to ice. Unfortunately, most severe storms during the monitoring period occurred during the winter. The wave data collected, therefore, were not representative of the most severe storm conditions.

g. Side-scan sonar surveys proved to be a valuable means for obtaining qualitative documentation of the condition of the structure toe and the consistency of the cover layer slope. Combined with diving surveys, the underwater condition of the dolos cover was determined to have several flaws from original construction, including zones of no armor and areas where the toe appears unstable.

h. Since dolos breakage can jeopardize the structure's integrity, dolosse should be designed for "no-rocking" criteria to minimize breakage resulting from movement. Consideration also should be given to reinforcement of dolosse in the active wave zone for a deepwater structure, since breakage appears to be concentrated in this area. In addition, dolosse should be placed over a stone underlayer rather than against a flat surface to prevent movement caused by wave reflection.

i. The 2-D model study investigation, conducted subsequent to prototype construction, indicated that the dolos cover at Cleveland would be unstable for wave conditions in excess of 3.2 m (10.5 ft). When new breakwater cover concepts are being considered, a model investigation, incorporated as part of the design, would help in selecting the optimum cover unit. Proper design will minimize repair and rehabilitation costs during the life of these projects.

j. Aerial photography targets and dolosse identified for armor unit surveys at Cleveland ranged from 90 to 275 m (300 to 900 ft) apart. More detail would have been useful in rectifying steropairs. For future monitoring efforts, it is recommended that controls be established to place at least three targets in each photo frame.

k. Photogrammetry could be an excellent means of mapping armor units above the waterline. The technique used at Cleveland, however, was qualitative. Emphasis should be placed on continued improvement of remote sensing methodology. With proper rectifying of stereopairs, photogrammetry can be used to quantify armor unit movement in the x, y, and z directions at relatively low cost.

l. Side-scan sonar should be considered during construction as an alternative to extensive and costly diver surveys, to inspect underwater placement of the structure. Sonar allows the inspection of large structures rapidly and economically. Annual records also could aid in identifying potential underwater problem areas as they evolve.

Results of the 1995 periodic inspection

Details of the periodic inspection are:

a. The winter of 1986-87, subsequent to the conclusion of the monitoring program, was characterized by higher-than-average lake levels, and several storms occurred during the period. In the spring of 1987, it was noted that most of the 1,816-kg (2-ton) dolosse around the head of the lighthouse on the eastern end of the structure were missing. The damage was evaluated and in May 1987, 234 dolos-armored units were placed around the head. These were 3,632-kg (4-ton) units as opposed to the 1,816-kg (2-ton) units previously used. Several 3,632-kg (4-ton) dolosse were also placed in low areas along the trunk to bring it back to the correct elevation. The 3,632-kg (4-ton) units appear to have remained stable around the head of the east breakwater since the 1987 rehabilitation.

b. Originally, data were obtained for the dolos-armored Cleveland Harbor East breakwater during the period 1980-1985. Armor unit breakage was documented, but limited quantitative data regarding armor unit movement were collected. Many of the units targeted during the effort were lost during storm wave conditions. Several stone rehabilitations of the East Breakwater were completed during the period 1985-1992. Walking inspections indicated extensive fracturing of armor stone. Progression of the stone breakage was documented periodically; however, armor unit movement data are nonexistent.

c. By means of limited ground-based surveys, low-level aerial photography, and photogrammetric analysis, very precise base level conditions have been established for portions of the Cleveland Harbor East Breakwater under the Periodic Inspections work unit. Accuracy of the photogrammetric analysis techniques were validated and defined through comparison of ground and aerial survey data on monuments and targeted armor units. A method using high resolution, stereo-pair aerial photos, a stereoplotter, and Intergraph based software has been developed to analyze the entire above-water armor unit fields and quantify armor unit movement. Detailed broken armor unit walking surveys have resulted in a well-documented data set that was compared with previous survey data.

d. Now that base (control) conditions have been defined at a point in time and a method has been developed to closely compare subsequent years of high resolution data for the Cleveland Harbor East Breakwater, the site will be revisited during future years under the Periodic Inspections work unit to gather data by which assessments can be made on the long-term response of the structure to its environment. The insight gathered from these efforts will allow engineering decisions to be made, based on sound data, as to whether or not closer surveillance and/or repair of the structure is required to reduce its chances of failing catastrophically. Also, the periodic inspection methods developed and validated for these breakwaters can be used to gain insight into other Corps structures.

St. Joseph Beach, St. Joseph, Michigan

St. Joseph Beach is located at St. Joseph, MI (Figure 35), on the southeastern side of Lake Michigan, about 42 km (26 miles) north of the Indiana/Michigan state line, and about 65 km (40 miles) west of Kalamazoo, MI.

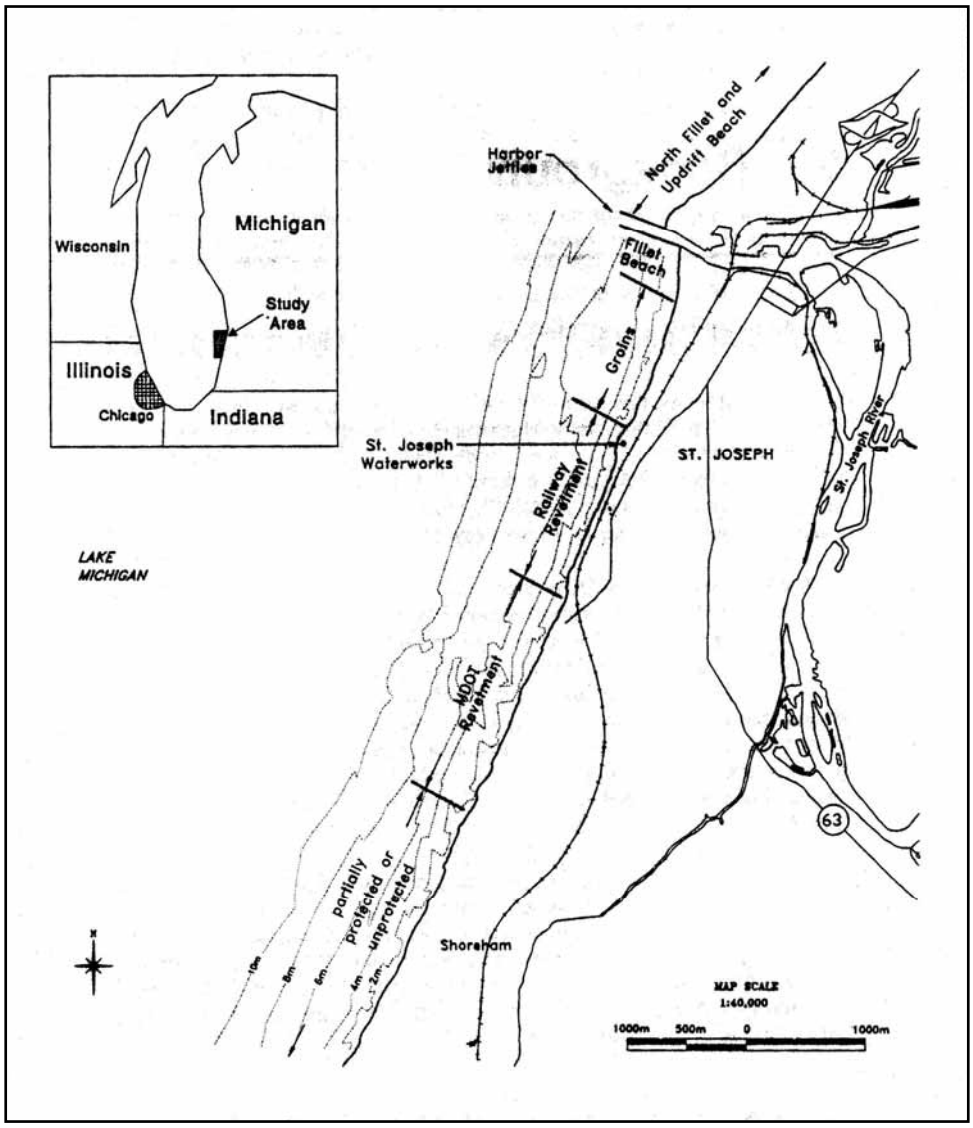

Figure 35. St. Joseph Beach, St. Joseph, MI (after Nairn et al. 1997)

Item monitored

Beach nourishment and sediment transport.

Period monitored

July 1991– June 1994 monitoring.

Reason(s) for monitoring

Jetties constructed at the mouth of the St. Joseph River in 1903 to stabilize the entrance have proven to be responsible for downdrift shoreline erosion. Monitoring was performed during the time period July 1991 – June 1994 to study native beach sediment characteristics and geology at the site and to evaluate the behavior of coarse-grained beach nourishment and sediment transport in the project area (Parson and Smith 1995; Parson et al. 1996; Nairn et al. 1997).

Results of the July 1991 – June 1994 monitoring

Details of monitoring are:

a. The shoreline in the vicinity of St. Joseph is one of many sites throughout the Great Lakes that exhibit highly irregular sedimentation zonations and wide ranges of sediment size gradation as opposed to classic sandy beach characteristics found on barrier island ocean coasts.

b. The validity of sampling techniques and methodologies used for sandy shorelines is questionable when used in areas similar to St. Joseph where highly irregular zonations and wide sediment gradations exist. To provide a realistic representation of native beach characteristics, sampling techniques should be based on unique sediment characteristics and natural variations in geology.

c. A cohesive sediment substratum at St. Joseph plays a dominant role in the change of the shoreline. Where the cohesive glacial till is exposed, downcutting is likely to occur during most wave conditions. Unlike unconsolidated sand and gravel, which may come and go under different energy regimes, fine-grained cohesive material, once eroded, cannot reconstitute itself and is removed from the beach system. The profile erosion that occurs during this process is permanent.

d. Erosion characteristics of cohesive shores are distinctively different when compared to sandy shores, a finding which has an impact on downdrift nourishment requirements. The analyses performed under this study suggest that the beach nourishment program at St. Joseph may provide at least partial protection to the underlying glacial till along and offshore of the feeder beach and the waterworks revetment section of shore. It is unclear whether the beach nourishment is having any negative or positive impact along the 3.5-km (2.2-mile) revetment section of shoreline south of the waterworks.

e. Cohesive shores have very different erosion characteristics from sandy shores, and this has a significant impact on the downdrift nourishment requirements. Additionally, there are varying degrees of cohesive shores (related to the extent and role of the overlying sand cover), which also have an important influence on the nourishment requirements.

f. In some cases, sections of cohesive shore on the Great Lakes (and elsewhere) will feature only a limited sand cover. As a possible defining variable, the sand cover between the 4-m (13.1-ft) depth contour and the

bluff would have a volume of less than 100 cu m per m (120 cu yd per yd) in these cases. Under these conditions, the underlying glacial till is either only thinly covered (i.e., with beach and bar thickness of less than 1 m (3.3 ft)) or entirely exposed. The till is frequently exposed over the entire profile to conditions of active downcutting. In these situations, it is not clear that the impoundment of sand in an updrift fillet beach and the deprivation of this sand from the downdrift beaches and lake bed will have any measurable impact on the rate of lake bed downcutting and the associated rate of shoreline recession. This hypothesis was successfully applied in the Port Burwell (north central shore of Lake Erie) litigation case where the Government of Canada successfully defended against a $30-million claim which alleged that the harbor structures at Port Burwell had caused accelerated recession for 40 km of downdrift cohesive shore.

g. The opposite extreme consists of a situation where the glacial till underneath the sand cover is rarely, if ever, exposed in the natural condition (prior to the construction of harbor jetties). This situation has been documented for the Illinois shoreline north of Chicago. In this case, the interception and impoundment of alongshore sediment by large shore-perpendicular structures has resulted in a reduction of sand cover from over 500 cu m per m (600 cu yd per yd) to less than 200 cu m per m (170 cu yd per yd) in places. The reduced sand cover resulting from the impoundment at the shore-perpendicular structures results in accelerated shoreline recession along the downdrift shore. Beach nourishment is required in these cases, not only to reinstate the historic sediment supply rate, but also to replenish the sand cover to its historic level. The latter requirement may be achieved through augmenting the sand cover volume to its natural level (this may not be practical or realistic owing to the large volumes required). Otherwise, the requirement may be relaxed if the effectiveness of the protective characteristics of the overlying sand cover can be augmented. The protectiveness of the sand cover could be improved through the provision of sediment that is coarser than the natural or native sediment. Specific grain size requirements should be determined based on the profile shape, properties of the underlying till, wave exposure, and sediment transport characteristics (both alongshore and cross-shore).

h. A special condition of cohesive shore that may be relatively common relates to cases where the natural profile shape is convex instead of concave. This type of cohesive shore exists at locations on the east shoreline of Lake Michigan north of St. Joseph. This condition is a result of the presence of a more erosion-resistant surface in the nearshore. The protected nearshore shelf may consist of some form of bedrock or glacial till that is armored by a boulder and cobble lag deposit. Shoreline (or bluff) recession on this type of cohesive shore is particularly sensitive to changes in lake level. While downdrift nourishment requirements for this type of cohesive shore may be less in volume (i.e., less than what might be determined based on potential transport rates), the timing and grain size characteristic requirements should be carefully considered.

i. Historical trends indicate the beach nourishment program has been successful in mitigating lake bed lowering rates south of the jetties between the period 1965 - 1991 (with a tenfold decrease in lake bed lowering rates in some areas). Between 1991- 1995, however, acceleration of lake bed erosion occurred (30 to 50 percent higher than earlier periods). During this period, annual nourishment volume decreased by 50 percent, which may partly explain the accelerated erosion rates.

j. Supplying downdrift areas with fill from a feeder beach is a complex process consisting of both cross-shore and longshore components. A comprehensive understanding of the amount of material being transported to the southern project limits is necessary for designing an effective nourishment program to provide protection to the vulnerable cohesive.

k. Effective downdrift nourishment requirements must be determined in light of changes to the lake bed that may have occurred as a result of the presence of the harbor structures prior to the initiation of a nourishment program. This is not necessarily the case for sandy shores downdrift of harbor structures.

l. Study results indicate that the fillet beach immediately south of the jetties would probably remain stable without beach nourishment. The area south of the fillet beach is definitely benefiting from the nourishment program with a stable shoreline being maintained. The coarse-grained sediment component of the fill protects the till under the upper beach from downcutting during storms. South of this sector, however, the beach nourishment program is not providing much benefit to the stability of a revetment or to the lake bed offshore of the revetment. In addition, about 50 percent of the beach fill is being deposited permanently on the lake bed in this sector due to a depression. The extreme southerly sector of the project is experiencing a deficit of material compared to historic supply rates. Accelerated offshore lake bed lowering as well as shoreline recession are occurring.

m. Based on the monitoring results, the study recommends the entire allotment of beach nourishment be placed on the extreme southerly sector of the project. Additional shoreline structures to the south of the area (to counteract erosion) may then be avoided. The shoreline between this sector and the jetties could be stabilized with other site-specific measures (i.e., rock headlands or breakwaters).

Burns Harbor, Indiana

Burns Harbor, Indiana, is a man-made harbor located on the southern shoreline of Lake Michigan, approximately 32.2 km (20 miles) southeast of Chicago, IL (Figure 36). The Harbor was constructed primarily to facilitate shipping from the steel industry in northern Indiana. Breakwater construction was completed in September1968, and harbor dredging was completed in August 1970.

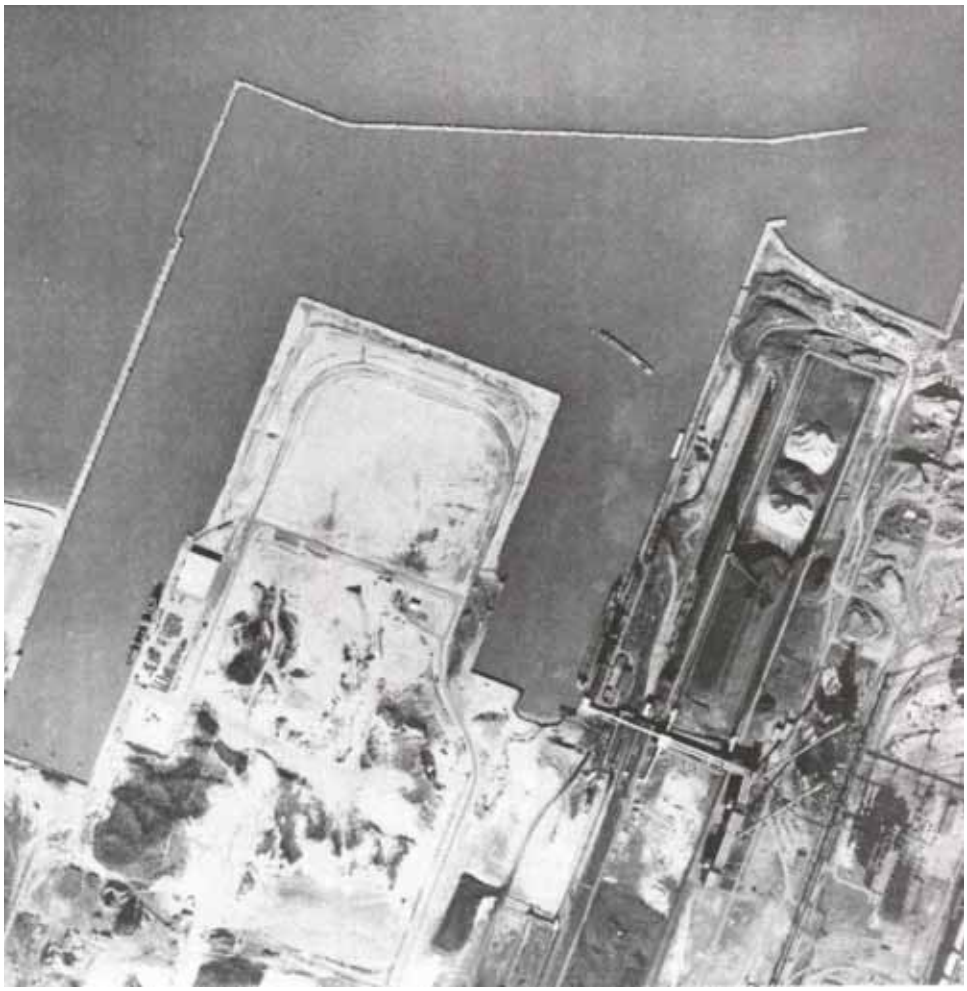

Figure 36. Burns Harbor, Indiana (after Bottin and Tibbetts 2000)

Items monitored

Harbor, breakwater, and breakwater stone.

Periods monitored

1985 – 1992 monitoring, July 1995 periodic inspection, and August 1999 periodic inspection.

Reason(s) for monitoring

Monitoring was conducted during the period 1985 – 1992 to determine the cause of loss of crest elevation of the breakwater and to evaluate excessive wave conditions in the harbor. This monitoring was also conducted to evaluate the design process, identify the causes of complaints of excessive wave energy by harbor users, and for frequent necessary maintenance requirements. Under the

Periodic Inspections work unit, targets and photo control points were determined during the period November 1994 – July 1995 to establish very precise base level conditions and conduct a broken armor survey. A periodic inspection was conducted during August 1999 to: (a) develop methods using limited land-based surveying, aerial photography, and photogrammetric analysis to assess the long-term stability response of the stone armor layer on the North Breakwater; (b) accurately define armor unit movement above the waterline; (c) determine and define changes that have occurred to the stone armor layer since last monitoring in 1992; (d) establish new baseline data since construction of the reef breakwaters lakeward of the existing North Breakwater; and (e) conduct a broken armor stone survey for comparison with data obtained in 1995. Subsequent monitoring will determine the effectiveness of the new reef breakwater structures relative to damages of the existing breakwater (Bottin and Mathews 1996; McGehee et al. 1997; McGehee et al. 1999; Bottin and Tibbets 2000).

Results of the 1985 – 1992 monitoring

Details of monitoring are:

a. Operational problems frequently occur in the harbor. Prototype wave gauging at the site revealed an approximate 30-percent transmission coefficient for the breakwater. This is attributed to the structure's high porosity. Therefore, when incident waves exceed 3 m (10 ft) (an annual occurrence), the 0.9-m (3-ft) operational criteria in the harbor are exceeded. The harbor was functioning, though not to the satisfaction of the users.

b. Analysis of design procedures used for Burns Harbor revealed that the design wave and water level were severely underestimated prior to original breakwater construction. In addition, a 3-D model investigation under-predicted wave heights throughout the harbor, because it used an impermeable breakwater (as opposed to a porous structure). A 2-D model also over-predicted armor stone stability and under-predicted transmission. These model investigations were performed in the early 1960's.

c. The structure was determined to be under-designed, principally due to underestimation of the wave climate in Lake Michigan. An improved hindcast, supplemented with wave data, produced an updated extremal analysis. The original 4-m (13-ft) design wave was determined to be a 2-year event.

d. The breakwater has experienced considerable damage over its life, but no single storm or specific event has caused loss of a section below the waterline. The loss of armor stones on the crest is assumed to be caused by high wave action. The structure has experienced waves larger than its design condition on numerous occasions. Harbor-side armor damage is assumed to be due primarily to overtopping and/or transmitted waves.

e. The crest elevation of the breakwater is 0.3 m (1.0 ft) below its design elevation on the average. There is evidence that the foundation may not have been constructed appropriately, thus causing greater settlement of the breakwater than anticipated. Excavation of clay and installation of

sand were included in design of the foundation. A trench was dug and clay was placed lakeward of it. Clay deposition piles noted during the geotechnical portion of the monitoring make it obvious that some of the clay was washed back into the trench prior to construction. In addition, there is evidence that a significant portion of the sand backfill material for the trench may have been placed lakeward of the proposed structure location.

f. The structure may have experienced greater than anticipated settlement, though the difficulty of evaluating historical survey data and variation in settlement along the structure hampers attempts to estimate the actual settlement. Both the original geotechnical design and a subsequent reanalysis predicted average settlement of about 0.3 m (1 ft). However, statistical analysis of the survey data suggests the structure has settled an average of about 0.6 m (2.0 ft). This settlement represents “loss” of armor stone on the order of 91 million kg (100 kt), roughly equivalent to the amount of repair stone placed on the structure over its life.

g. Alternatives for the reduction of maintenance of the breakwater are to: (1) add larger stone and/or increase the angle of the slopes; (2) add a concrete cap to the structure to improve stability of the crest; or (3) place a protective structure (reef-type structure well below the water level) in front of the existing breakwater. An economic analysis was conducted to determine which alternative(s) would result in reduced overall costs. (Alternative three was selected subsequent to monitoring of the site and was constructed in the prototype).

h. The cut stone armor used in the breakwater exhibited a wider variance in stability than associated with typical rubble mounds. The result is a highly variable pattern of damage on the structure. The stability of cut stone armor is more sensitive to placement technique than other types of armor. Weathering of the armor resulted in some breakage, but not a significant amount.

i. A 3-D model study used to plan the harbor resulted in an ineffective entrance design. This study did not accurately predict wave conditions in the harbor, because it assumed all wave energy would enter the harbor through the entrance (impermeable breakwater; no overtopping), and it underestimated the design wave.

j. A 2-D model study used to design the breakwater cross section underestimated wave transmission, possibly caused by settlement of the structure and subsequent repairs that resulted in a more porous structure. The 2-D model study appeared to predict the stability, but did not accurately predict the harbor side damage that was approximately equal to the lakeside damage over the life of the structure.

k. The functional requirements of the project have changed since design because of an increase in barge traffic in the harbor. Most of the user complaints regarding operations can be traced to one facility, the grain dock on the north wharf which is constructed with a vertical sheet-pile face. Measurements verify that reflection caused wave conditions in front of the dock to be twice the height of waves in the open area of the harbor. This facility was not anticipated in the design phase.

Results of the July 1995 periodic inspection

Details of the periodic inspection are:

a. Since construction of the Burns Harbor North Breakwater, extensive breakwater damage has occurred. Maintenance of the crest elevation and structure cross section required an average of about 6,937,000 kg (7,640 tons) of stone per year in the first 19 years of operation. The monitoring effort for Burns Harbor during the period 1985-1989, however, included little sound, quantifiable data relative to the positions of armor stones on the North Breakwater.

b. Now that base (control) conditions have been defined at a point in time, and methodology has been developed to closely compare subsequent years of high-resolution data for the Burns Harbor North Breakwater, the site will be revisited in the future to gather data with which assessments can be made on the long-term response of the structure to its environment. The insight gathered from these efforts will allow engineering decisions to be made, based on sound data, as to whether or not closer surveillance and/or repair of the structure might be required to reduce its chances of failing catastrophically. The periodic inspection methods developed and validated for these structures may be used to gain insight into other U.S. Army Corps of Engineers structures.

Results of the August 1999 periodic inspection

Details of the periodic inspection are:

a. Results of this monitoring effort indicated continued loss of structure elevation. Approximately 46 percent of the total length of the breakwater was below the design crest el of +4.3 m (+14 ft) versus 24 percent in 1995. Also, about 11 percent of the structure was below an el of +3.7 m (+12 ft) in 1999 versus 4.6 percent in 1995. Both surveys indicated crest width along the breakwater narrower than design and slopes on the harbor side of the structure steeper than design.

b. A total of 225 broken armor units were documented during the 1999 survey versus 165 during the previous survey. Data indicated the majority of additional stone breakage occurred on the harbor side of the structure, as opposed to the lakeside. As in the previous survey, higher concentrations of broken stone were noted on the eastern one-third of the breakwater during the current monitoring effort.

c. To reduce wave heights at the breakwater and minimize further damage, a submerged reef breakwater was constructed lakeward of the original structure during the construction seasons between June 1995 and August 1998. The photogrammetry conducted in 1999 not only quantified changes since 1995 but established new base conditions for the structure upon which the performance of the reef breakwater can be evaluated in future years.

Great Lakes Breakwater Stone

A survey of breakwater stone structures in the Great Lakes in 1990 indicated various degrees of excessive and premature stone deterioration (Figure 37). In some stones, the damage was significant and required costly rehabilitation of the structures far in advance of the expected economic life of the structures. Other stones were apparently performing well. The study initiated an effort to determine practical ways to reduce the amount of stone deterioration, lengthen the economic life of coastal structures, and to reduce the annual cost of maintenance. The study also compared changes in performance of the stone that resulted from inherent weaknesses such as diagenetic or depositional environment (facies) and fractures resulting from extraction methods at the quarries.

Figure 37. Great Lakes breakwater stone deterioration (after Richards et al. (in preparation))

Item monitored

Breakwater stone.

Period monitored

October 1995 – September 1998 monitoring.

Reason(s) for monitoring

Monitoring was conducted during the time period October 1995 – September 1998 because of concerns for the durability of breakwater stone used in the structures and their longevity in the Great Lakes area. It also included geological environmental evaluation of stone sources and laboratory testing related to a microstructural study in conjunction with aerial photographic surveys. Aerial surveys were conducted to determine movement of stones within the structures and shift of the structures. Structures at Chicago Harbor, Illinois; Calumet Harbor, Illinois; Burns Harbor, Indiana; and Cleveland Harbor, Ohio, were monitored (Richards et al. (in preparation)).

Results of the October 1995 – September 1998 monitoring

Details of monitoring are:

a. Three external factors contributing to stone deterioration are: (1) weathering environment (number of freeze/thaw and wet/dry cycles); (2) high-energy blasting; and (3) mishandling during placement causing cracks.

b. Three internal factors affecting rate of deterioration are: (1) depositional facies (environment of deposition influencing rock fabric and composition); (2) diagenesis (degree of interparticle suturing, cementation, and vugular porosity affecting induration and susceptibility to freeze/thaw action); and (3) in situ stress which after removal of confining pressure may cause cracks.

c. The durability performance of cut sedimentary stones is significantly better than blasted sedimentary stones. About 16.1 percent of 398 cut stones included in this study were categorized as failed stones, with an average age of 19.7 years. About 41.3 percent of 479 blasted stones were considered to have failed, with an average age of 7.8 years. Strongly indurated rocks such as unweathered granite and quartzite generally last longer. Date from the quartzite show 2.6 percent of 78 stones failed, with an average age of 2.2 years.

d. An immediate need exists for broadening the study to include stone deterioration on other structures in the Buffalo and Chicago Districts, and other regions on a national basis. A long-term evaluation on a national basis will result in significant cost savings and minimize replacement frequency of stone.

e. Laboratory testing of armor stone needs to be more correlative of the field conditions to determine durability of the stone in the environment it is placed.

8 Site-Specific Lessons Learned, Inland Navigation Sites

Marseilles Dam, Illinois River, Illinois

Marseilles Dam is located near the upstream end of the Marseilles Canal at river mile 247.0 on the Illinois River, near the city of Marseilles, IL, approximately 9.6 km (6 miles) southeast of the city of Ottawa, La Salle County, and 104.6 km (65 miles) southwest of Chicago, IL (Figure 38). The main dam is a gated structure consisting of a 168.2-m- (552-ft-) wide section containing eight 18.3-m- (60-ft-) wide submersible tainter gates. Prior to the 1988-89 installation of a remote operating system, Marseilles Dam had to be attended 24 hr/day (amounting to four full-time operator positions) because the lock and the dam are 3.9 km (2.4 miles) apart, too distant to reasonably work both sites in a single shift. The existing operation of the gates at the dam is performed remotely from the lock operation room. The gate operations can also be operated in a manual-local mode of operation where all controls for gate operation are done from the machinery bridge above each gate.

Item monitored

Navigation dam submersible gates.

Period monitored

June 1999 – June 2001.

Reason(s) for monitoring

Advantages of submersible gates are the capability of skimming loose ice with a minimum amount of flow and avoiding downstream scour often associated with large underflow gate openings at low tailwater. Also, submerged operation largely avoids the problem of freezing-in of side seals. Vibration of gates has been reported at some of the 33.5-m- (110-ft-) wide submergible gates on the

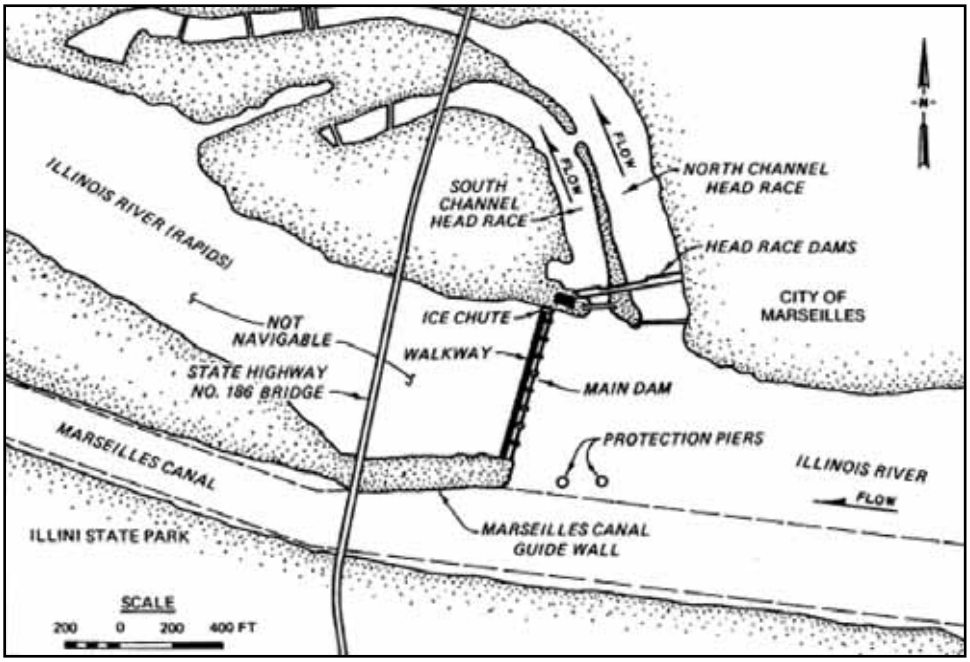

Figure 38. Marseilles Dam, Illinois River, IL (after Cooper et al. 2001)

Ohio River. On some of these projects, a retrofit to a sharp-crested bottom seal eliminated the problem, but at other projects, the submerged mode of operation was discontinued.

Hypotheses tested by the monitoring plan included: (a) the remote operating system increases the capability of the dam to maintain operation during extreme weather or river conditions; (b) the remote operating system meets the operating constraints previously identified; (c) the remote operating system is a reliable system that provides considerable cost savings over the previous onsite manual operation; (d) any operational limitations of the remote operating system will be identified, with proposals to minimize these limitations; (e) the submersible tainter gates are more effective in passing ice than the conventional counter-weighted tainter gates, and an operation schedule will be developed to enhance gate operation during adverse weather conditions, (f) adjustment of the submersible tainter gates in freezing conditions is less hazardous, less time-consuming, and more effective and efficient than the old steam method that was used on the counter weighted tainter gates; (g) the submersible tainter gates at Marseilles Dam do not significantly vibrate under normal operation for flows under and over the gate, or with passing ice; and (h) a previous model investigation accurately quantified vibration conditions for flows over and under the gates.

The most cost-effective methodology for Marseilles Dam operation will be determined from three alternatives: (a) maintain existing remote operation; (b) manual operation; and (c) replace remote operating system.

Results of the June 1999 – June 2001 monitoring

Details of monitoring are:

a. Based on results of this monitoring and a Life Cycle Cost analysis, the most cost-efficient alternative for Marseilles Dam operations is maintenance of the existing remote operation system. This will be the least cost plan for the short term (5-year period of analysis) and for the longer term, under the assumption that annual maintenance costs would not increase dramatically.

b. The submergible gates at Marseilles Dam have greatly improved winter operation of the project. Submerging the gates during cold low-flow periods with periodic cycling eliminates freezing in the gates and the need for personnel to be on site. The costs and hazards of chipping ice or thawing the gates with steam have been eliminated by the new gate design. The remote operation system allows operation of the Marseilles Dam from a control room at the Marseilles Lock, approximately 3.9 km (2.4 miles) away, eliminating the need for 24-hr shifts on the dam site and the costs associated with those shifts. The remote operating system was proven to be efficient and effective in maintaining the strict pool tolerance and improving winter operation of the dam.

c. At typical winter discharges, the gates effectively pass fragmented ice floes and loose brash in the submerged mode without loss of pool or scour damage to the downstream channel. To pass heavy brash however, it is still necessary to concentrate the flow by opening one or two gates nearest the canal in the raised mode. To draw ice beneath requires an opening of at least 1.5 m (5 ft), and it may be necessary to pull the gate clear of the water, similar to the practice with the old tainter gates.

d. The videotape analysis used to analyze ice passage was successful. The technique is relatively low cost, logistically simple, and provides a valuable visual record for analysis of the efficiency of the gates to pass ice in the submerged mode.

e. When passing light ice, measurable vibration in the 0.1-to 0.3-g range occurred above a background range of 0.006 to 0.02 g's. Unfortunately, because of the mildness of the winter of 2000, no data were obtained while passing moderate or heavy ice.

f. Mild winter weather conditions resulted in very light ice formation in the upper pool near the dam and the lock. The project operations log for the instrumented gates indicated a very short duration of gate submergence (-1.5 m (-5.0 ft) for a 10-min period) was used to initiate ice passage. The 10-min period was less than the data acquisition time delay (15 min) for steady flow to establish and the recording to be initiated. No vibration data were obtained for this operation.

g. The upper pool level elevations indicated a variation between 147.27 and 147.42 m (483.2 and 483.65 ft) during the 12-month data collection period. This indicates that the remote operation system meets the constraint of a tight pool tolerance.

h. A significant rise in river stage occurred between the periods April – July 2000 and required multiple gate operations to pass the inflows. These gate operations are characterized by larger raised gate openings, 1.5 - 2.1 m (5.0 - 7.0 ft), for longer periods of time (days) to maintain the upper pool water levels.

i. The vibration levels indicated that for these raised gate operation conditions, very insignificant gate movement is present. The maximum vibration level values and displacement observed during these operations were 0.20 g's and 0.005 cm (0.002 in.), respectively.

j. Flow releases from Marseilles Dam during the winter months of the year were generally performed with the gates operated in the submerged position. The gate operations during these seasonal periods are characterized as typically small gate openings for short durations (hours) to maintain the upper pool water levels. The majority of the submerged position gate openings for discharge of normal river inflows were recorded to be no greater than - 0.6 m (- 2.0 ft). In general, the vibration levels and displacements were extremely small (less than 0.3 g's and less than 0.002 cm (0.001 in.), respectively).

k. Vibration levels increased with a four-gate submerged operation and gate openings ranging from 0.4 - 0.6 m (1.5 - 2.0 ft). The maximum values of vibration level and displacement observed during these operations were 0.06 g's and 0.005 cm (0.002 in.), respectively. The values represent very insignificant movement for these submerged gate operation conditions.

l. Failure to continuously operate the gates in the submerged mode for periods exceeding 15 min had a negative impact on the collection and analysis of data for submerged operation during ice passage. Extended operation exceeding 15 min was required to activate remote collection of data to validate model results.

m. The absence of significant movement obtained during normal operation of the gates in the raised position appeared to validate the 2-D model study, that indicated only random vibrations of less than 1 percent of the gate's weight.

9 Structures Monitored and Generic Lessons Learned, Hawaii and the Pacific Islands

Wave Transformation

Structure monitored

Agat Harbor, Guam. Monitoring was performed during the period February 1991 – April 1994 to determine wave transformation across coral reefs, wave and surge levels behind coral reefs, and wave-induced circulation on a flat reef. Little engineering data exists relative to design guidance for wave characteristics and surge levels on coral reefs.

Generic lessons learned

Details are:

a. At a reef face, wind waves dissipate most of their energy in breaking. Wave energy propagates across reef flats as bores, moving water shoreward that returns seaward through breaks in the reef face.

b. Wave heights on a reef flat do not increase appreciably as wave height offshore increases, but the amplitude of seiche of the entire reef is effected by incident energy. Wave groups (surf beats) with periods near the principal seiche modes of a reef flat may induce harmonic coupling.

c. The combination of seiche, return flow from wave setup, and mass transport of bore-like waves can result in large currents running parallel to shore. For structures located on a reef flat, forces from the resulting currents may be of larger magnitude than forces due to the wind waves themselves.

d. Peak period on a reef flat bears little resemblance to the incident wave period. Long period waves (100 – 200 sec) dominate the signal.

e. A detached breakwater design at a reef environment promotes flushing of a harbor but can result in a significant influx of sediment during high-current events.

f. There is no indication that wind waves on a reef flat will exceed the depth-limited breaking criteria (0.78) used for sloping beaches. The height of the highest wind waves on a reef flat, a figure needed in calculating stone stability, will probably not even exceed one-half the water depth as long as the water depths are shallow. However, as the water depth increases because of surge, the breaking wave height limit will increase. Without verification of a lower breaking limit under typhoon conditions, the standard depth-limited criteria should be retained for design.

g. Estimates of surge from measurements in models of planar beaches are unlikely to apply to typhoon surge levels.

h. Wind waves propagating shoreward are not the only, and maybe not even the predominant, environmental loading for structures on reef flats. Forces on structures from currents associated with long waves should be considered as well as wind wave forces.

i. The shortest path (hydraulically) for return flow to take at a reef environment is toward the ends of the reef flat, where breaking and setup are not occurring. Since a harbor is connected to deep water by an entrance channel, the low water level is brought conveniently close (from the return flow's perspective). If just one-third of the return flow takes this shortcut through the harbor and entrance channel back to sea, velocities across a (for example) l00-m- (330-ft-) wide opening would be on the order of 1 m per sec (3.3 ft per sec). This is sufficient to balance the out-of-phase flow from a seiche, and double the in-phase flow, resulting in a pulsing flow of up to around 4 knots (2.5 mph). Highest velocities will occur where the gradient is steepest, which would be near the shoreward side of a harbor basin.

Harbors

Structure monitored

Barbers Point Harbor, Oahu, Hawaii. Monitoring was conducted during the period July 1986 – March 1990 to: (a) evaluate and validate results of model studies conducted for the harbor design; (b) perform wave gauging to measure wave climates in deep water and nearshore areas, and long-period oscillations of the harbor; (c) relate the conditions outside the harbor to surge found inside the harbor; (d) evaluate the effectiveness of the wave absorber; and (e) compare the measured data to the predictions of state-of-the-art physical and numerical model (HARBD) studies.

Generic lessons learned

Details of the ten (10) areas are:

a. The numerical model HARBD does well in predicting resonant modes of oscillation measured in prototype harbors. (A physical model will also accurately predict resonant modes occurring in a harbor.) HARBD is consistent with prototype measurements in predicting the shift of the Helmholtz mode and the appearance of additional peaks with inclusion of modifications inside harbors. Numerical model magnitudes of amplification are consistent with prototype amplifications if the numerical model is calibrated to measurements using bottom friction.

b. Sea-swell significant wave height in the nearshore can be accurately estimated with data from an offshore buoy.

c. Comparison of the infragravity significant wave heights measured inside a harbor with those measured at a slope array offshore shows a high correlation between significant wave height inside and outside the harbor. It can be concluded that an increase in harbor seiche is associated with an increase in swell energy outside the harbor. Therefore, nonlinear processes that transfer energy from swell waves to infragravity waves outside a harbor are clearly an important mechanism for harbor resonance forcing.

d. High correlation between harbor seiche and sea-swell wave heights rules out free long waves generated from distant sources as an important forcing mechanism, since such free waves are not necessarily coincident with energetic sea and swell.

e. A rubble-mound wave absorber effectively reduces the wave energy inside a harbor for wind wave periods of 20 sec or less. This type of wave absorber is less effective in decreasing wave energy for longer waves with periods of 50 sec or greater.

f. Lack of wave absorber will increase wave heights at locations inside a harbor by an estimated 125 percent. Wave absorber decreases the reflection coefficients up to 50 percent.

g. Numerical model strengths include: (1) ease of model setup and modifications; (2) availability of data throughout the modeled harbor grid which permits visualization of the wave response over the entire gridded region; (3) quick response time; and (4) less cost to run the model.

h. Numerical model limitations include users: (1) performing simulations with unidirectional regular waves without directional spreading effects; (2) neglecting nonlinear effects; and (3) having inadequate reflection coefficients and bottom friction data for accurately calibrating the model.

i. Physical model strengths include the ability to simulate: (1) directional wave spectra; (2) nonlinear wave-wave transformation as waves travel into harbors; (3) reflection, transmission, and overtopping of structures; (4) dissipation resulting from bottom friction within scale and depth limitations; (5) currents; and (6) navigation studies with model ships.

Limitations of physical models are mainly the result of the cost to construct and modify models and to collect data.

j. Long-period modes (resonance) cannot be effectively damped out once a harbor is constructed. A model investigation of resonant modes should be carried out before final project planning to ensure that the constructed harbor does not have unacceptable resonant modes of oscillation.

Breakwaters

Structures monitored

Ofu Harbor, American Samoa. Base conditions for future periodic inspections were determined in 1997, and a periodic inspection was conducted in 2002. Periodic inspections use limited land-based surveying, aerial photography, and photogrammetric analysis to assess long-term stability response of concrete armor units on the Ofu Harbor breakwater. Land surveys, armor unit inspection, aerial photography, and photogrammetric analyses will be used to define armor unit movement above the waterline.

The breakwater was constructed in 1994 by using various-sized concrete units for construction material instead of basalt stone. Unique concrete underlayer units consisting of 1,634-kg (1.8-ton) units with 0.4-m- (1.3-ft-) diam holes to dissipate wave energy were used. Concrete underlayer units weighing 454 and 2,270 kg (0.5 and 2.5 tons) were also formed by pumping high-strength fine-aggregate concrete into geotextile fabric bags. The breakwater armor consisted of a single layer of uniformly placed 4.086-lg (4.5-ton) concrete tribar units. To improve the stability of the tribars, work included the construction of a toe trench to stabilize the armor unit toe, and a concrete rib cap system on the breakwater crest to stabilize and buttress tribars at the upper sea-side and harbor-side slopes. The rib cap forms were fabricated and concrete poured into the top section of the tribars.

Ofu Harbor is subjected to severe storm conditions in the South Pacific, including tropical storms, hurricanes, and cyclones.

Nawiliwili Harbor, Kauai, Hawaii. Base conditions for future periodic inspections were determined in 1995, and a periodic inspection was conducted in 2001. Periodic inspections use limited land-based surveying, aerial photography, and photogrammetric analysis to assess long-term stability response of concrete armor units on the Nawiliwili breakwater. Land surveys, armor unit inspections, aerial photography, and photogrammetric analyses will be used to define armor unit movement above the waterline.

The Nawiliwili Harbor breakwater has been repeatedly subjected to major storm events, including three hurricanes, during its 70-year history. As a result, extensive breakwater damage has occurred. Major rehabilitations were completed in 1959, 1977, and 1987. The structure was originally armored with keyed-and-fitted stone, but now has several sizes of tribar and dolos concrete armor units. The Nawiliwili breakwater is one of the most complex rubb1e-mound structures

the Corps has constructed. No sound, quantifiable data relative to the movement or positions of the concrete armor units had been obtained for the structure prior to this study.

Kahului Harbor, Maui, Hawaii. Base conditions for future periodic inspections were determined in 1993, and a periodic inspection was conducted in 2001. Periodic inspections use limited land-based surveying, aerial photography, and photogrammetric analysis to assess long-term stability response of armor unit layers and concrete rib caps on the Kahului Harbor breakwaters. Land surveys, armor unit breakage inspections, aerial photography, and photogrammetric analyses will be used to define armor unit movement over the entire above-water armor unit fields.

The Kahului Harbor armor stone east breakwater was constructed in 1900. The west breakwater was constructed in 1919. In 1931, the east and west breakwaters were extended to their current lengths of 845 m (2,766 ft) and 705 m (2,315 ft), respectively. All original construction used a single layer of keyed and fitted 7,265-kg (8-ton) armor stone. Subsequent storms and rehabilitations have occurred since 1931. In 1966, both breakwater heads were armored with two layers of 31,780-kg (35-ton) tribars. A concrete rib cap was placed on the east breakwater. In 1969, a concrete rib cap and 260 reinforced tribars weighing 17,250 kg (19 tons) each were placed on the west breakwater. An inspection in 1973 revealed that 29,965-kg (33-ton) tetrapods on the seaside of both heads had sustained considerable damage and they, along with the 7,265-kg (8-ton) stone areas on both trunks, were in need of repair.

The most recent repairs were completed in 1984. This rehabilitation was carried out to eliminate the need for future “piecemeal” repairs. Five hundred and forty (540) tribars weighing 5,900 kg (6.5 tons) each, 755 tribars weighing 8,170 kg (9 tons) each, and 10 tribars weighing 22,700 kg (25 tons) each were placed during this rehabilitation.

Laupahoehoe Boat Launching Facility, Hawaii. Base conditions for future periodic inspections of the breakwater were determined in 1993, and a periodic inspection was conducted in 2001. Periodic inspections will use limited land-based surveying, aerial photography, and photogrammetric analysis to assess long-term stability response of armor unit layers and concrete rib caps on the Laupahoehoe breakwaters. Land surveys, armor unit breakage inspections, aerial photography, and photogrammetric analyses will be used to define armor unit movement over the entire above-water armor unit fields.

The initial design of the 75-m- (250-ft-) long Laupahoehoe rubble-mound breakwater called for the vertical placement of core stone to be armored with a 27,240-kg (30-ton) dolos, with the crest to be stabilized with a concrete rib cap. The rib cap increases crest stability, reduces wave overtopping, provides buttressing for crest armor units, allows ease of access for maintenance, and is less reflective than a solid concrete cap. The toe of the dolos was keyed into the hard basalt bottom by means of a trench excavated around the perimeter of the breakwater. However, the breakwater stability model study noted that the stone beneath the rib cap showed some displacement and consolidation during testing. The constructability review of the plans also noted that the vertical placement of

the breakwater core stone would be a formidable task in the area's year-round rough ocean conditions.

Generic lessons learned

Details are as follows:

a. A stable tribar breakwater core can be achieved through innovative design of a reinforced concrete pipe rib cage. Because of the interior geometry of such a structure, cylindrical reinforced concrete pipes should be placed on end and backfilled to provide a stable support for the rib cap. This unique design feature, along with a trenched toe for the dolos, will perform well structurally. Periodic photogrammetric surveys will provide a basis for long-term structural assessment of such a project.

b. Photogrammetric analysis of a breakwater is an excellent tool in mapping the above-water portion of the structure and quantifying changes in elevation.

c. Accuracy of the photogrammetric analysis techniques has been determined through comparison of ground and aerial survey data on armor units that had been specifically targeted and surveyed for this purpose. A method using high-resolution, stereo-pair aerial photographs, a stereo-plotter, and AutoCAD files has been developed and tested to analyze the entire above-water armor unit fields to quantify armor unit movement that exceeds a threshold value of 0.2 m (0.5 ft).

d. A walking inspection of a tribar breakwater revealed higher levels of armor breakage than those found by aerial photography studies. Areas in breakwaters where slight concentrations or cluster of breakage occur should be monitored more closely than other areas. Also, the land-based breakage survey revealed the accuracy of aerial breakage inspections can be questionable and that, for more accurate armor unit breakage counts, detailed walking inspections should be conducted over the armor unit fields.

10 Structures Monitored and Generic Lessons Learned, Alaska

Harbors

Structure monitored

St. Paul Harbor, Alaska. Monitoring was conducted during the period July 1993 – June 1996 to determine if the harbor and its breakwater structures were performing (both functionally and structurally) as predicted by model studies used for the project design.

Generic lessons learned

Details are:

a. When working in high-energy wave environments at remote locations, extra precautions should be taken to ensure that wave data can be collected. The loss of directional wave gauges outside the harbor significantly reduced the value of other data obtained. Devices hard-wired to shore to obtain real-time data, and/or other appropriate measures to improve the probability of success, should be included in project budgets. In-depth research of conditions should be conducted to assure success.

b. When working at remote sites, logistical problems may be a significant factor. In most cases, equipment and supplies required are not available locally and must be shipped from the mainland. Delivery times are uncertain and shipping costs are significantly higher.

c. Failure to obtain incident wave data outside the harbor will have a negative impact on analysis of other data collected during the monitoring effort. Incident wave data are required for correlation with wave data obtained inside a harbor, wave runup, and wave overtopping data to validate design methods and procedures.

Breakwaters

Structure monitored

St. Paul Harbor, Alaska. Monitoring was conducted during the period July 1993 – June 1996 to determine if the harbor and its breakwater structures were performing (both functionally and structurally) as predicted by model studies used for the project design. A periodic inspection of the breakwater was completed in 2000.

Generic lessons learned

Details are as follows:

a. Photogrammetric analysis of a breakwater is an excellent tool in mapping the above-water portion of the structure and quantifying changes in elevation.

b. Videotape analysis used to obtain wave runup data on a breakwater is very successful, except during periods of low visibility. The technique is relatively low in cost and logistically simple, and it provides relatively accurate measurements.

c. Trends in wave hindcast data obtained outside an arctic harbor to define incident wave conditions correlate reasonably well with runup data on breakwaters in a qualitative sense (i.e., larger wave heights correlate with higher runup, and smaller wave heights correlate with low runup). The absolute values of the hindcast significant wave heights, however, may be substantially lower than the waves experienced in the prototype based on runup values measured, overtopping observed, and local forecasts.

d. Deterioration of breakwater stone is predictable because of freeze-thaw and wet-dry cycles, large wave action, and sea ice effects. Structures in such environments should be monitored very closely, since the rate of deterioration can be expected to increase. The highest grade of geologically acceptable stone should be placed above the waterline in an extremely harsh arctic environment.

11 Structures Monitored and Generic Lessons Learned, Pacific Coast of the U.S. Mainland

Breakwaters

Structures monitored

Fisherman's Wharf, San Francisco, California. Monitoring was performed during the time period December 1982 – December 1989 to evaluate performance of the breakwater and to determine its impact on the adjacent harbor. Specific objectives included: (a) documenting wave attenuation of the structure compared to model studies and design criteria; (b) evaluating effect of the structure on surge within the harbor complex; (c) determining effect of the structure on water circulation within the harbor and surrounding areas and currents, especially at the entrance; (d) determining actual scour, measuring the scour, evaluating the cause, and comparing with predicted scour; (e) evaluating effect of the structure on littoral processes, including shoreline response and deposition within the harbor; and (f) determining integrity of the structure by investigating spalling, cracking, and settlement of the wall.

An impermeable vertical-wall detached breakwater structure forms the main element of the breakwater system. This 460-m- (1,509-ft-) long structure was built using driven prestressed/precast interlocking sheet piles. A cast-in-place reinforced cap beam ties the piles together. For most of its length, the detached breakwater is oriented approximately in the shore-parallel west-southwest to east-northeast direction. This alignment intercepts waves from the northwest, yet is essentially parallel to the prevailing tidal currents.

Spud Point, Bodega Bay, California. The concrete pile-supported breakwater structure was selected for monitoring during the period August 1985 – March 1988 because of its unusual baffled design. Openings in the breakwater below the mean lower low tide level permit relatively unimpeded marina flushing. The baffle panel submergence depth was chosen using theoretical wave height transmission results. A field study of wave transmission was conducted using boat wakes and pressure sensors to measure the generated waves.

Soundings of potential scour zones and a side-scan sonar survey were made. Circulation through the breakwater and marina were measured, and the breakwater was examined for structural integrity.

Crescent City, California. Monitoring was conducted during the time period 1986 – 1989 to define long-term trends in dolos movement, breakage, and static stresses (so these data could be used to further improve the structural dolos design procedure), and to observe the long-term response of the dolos portion of the Crescent City breakwater to its incident environment. The breakwater had been rehabilitated in 1986 using 38,135-kg (42-ton) dolosse. During the rehabilitation, 18,160-kg (20-ton) dolosse were instrumented to measure loading and armor unit motion. This monitoring was carried out as part of the Crescent City Prototype Dolos Study (CCPDS). Near the end of the CCPDS in 1989, it was noted that dolos movement was subsiding, but static loads were still showing increases. For this reason, additional monitoring data were obtained during the period November 1989 – October 1993 after conclusion of the CCPDS and were analyzed as a periodic inspection.

Morro Bay Harbor Entrance, Morro Bay, California. Monitoring was performed during the period January 1998 – August 2001 to evaluate the effectiveness of improvements to alleviate hazardous wave conditions at the entrance to the harbor. Prior to the improvements completed in December 1995, Morro Bay Harbor entrance was known as one of the most dangerous harbors in the United States. The latest improvements consisted of construction of a deepened, expanded entrance channel. The new channel doglegs westerly from the old entrance channel and flares open to a width of 290 m (950 ft). The authorized depth of the channel extension is -9.1 m (-30 ft). However, the plan provides for advanced maintenance by deepening the new channel to -12.2 m (-40 ft) and dredging an additional sand trap to a depth of -9.1 m (-30 ft) within the harbor entrance structures north of the head of the south breakwater.

Monitoring was conducted to determine that: (a) improvements would result in significantly improved navigation conditions in the harbor entrance; (b) improvements would have no negative impact on existing structures; (c) channel deepening can be effectively maintained with a 3-year dredging interval; (d) model investigations accurately quantified wave conditions in the entrance and correctly defined sediment patterns and deposition in a qualitative sense; and (e) methodology used in determining sedimentation rates in the harbor entrance are valid based on field data, model predictions, and sound engineering judgment.

Generic lessons learned

Details are as follows:

a. Vessel-generated waves may be the controlling design wave in small bodies of water. Predictions for vessel-generated waves are needed in addition to predictions for wind-generated waves.

b. In designing baffle openings in breakwaters to allow water circulation, consider natural circulation patterns. Openings (culverts or gaps) that are aligned parallel to the normal flow will be more effective. Thus, the openings for circulation should be placed in breakwater segments that are angled across the flow patterns.

c. Cast concrete breakwater caps may develop hairline shrinkage cracks. While small cracks may not affect structural integrity in warmer climates, expansion of freezing water can cause spalling of concrete in colder climates.

d. Side-scan surveys should be performed at extreme high tides to permit complete breakwater coverage and to lessen the risk of tow fish damage.

e. Under small tide and wind-induced current velocities, scour development is unlikely, except possibly during a prolonged high-wave event (from standing wave-induced bottom velocities along the outside of baffle breakwater walls and through baffle openings).

f. Brass disks should be installed in breakwaters with caps, and their 3-D position should be surveyed as part of a periodic inspection program after any major earthquake activity.

g. Visual inspections should pay careful attention to hairline cracks in caps, and these should be documented photographically according to a retrievable position identification system.

h. A baffle breakwater will meet its performance criteria with respect to currents if currents through this type breakwater are measurable and exchange takes place. A baffled design should be considered for lower energy environments where good circulation is critical to acceptability of the proposed structure.

i. Significant rolling of boats (docked so that they are broadside to a baffle breakwater) as a result of the largest wakes suggests that parameters other than wave height may be of interest for wake or wave transmission criteria. The baffled type of breakwater reduces vertical water particle motions and surface disturbances, yet allows enough horizontal motion to pass through the breakwater in the lower part of the water column to cause lateral motion in docked vessels. This emphasizes that for breakwaters in shielded locations, protection against wakes may govern the design more than protection against wind waves.

j. Storms that occur during the first postconstruction winter season will produce the largest dolos movements. Reduced movement during subsequent storms indicates that dolosse consolidate and nest into a more stable matrix.

k. Surges in dolos movement, where evident, have tended to follow peaks in the wave power record.

l. During nesting, the greatest movement of dolosse will occur on the upper slope of a breakwater and in the vicinity of the waterline. Movement on the upper slope will result because dolosse placed there have initial boundary conditions that do not inhibit sliding.

m. After initial nesting, dolos movement will slow significantly, but will continue to occur primarily near the waterline as well as on the upper slope.

n. The dominant direction of dolos movement has historically been upslope with slight settling plus rotation about the vertical axis (yaw). Upslope movement (i.e., a wave runup dominated movement) is thought to result, at least in part, when a breakwater has a mild slope.

o. Dolos breakage, while typically associated with some amount of movement, is not necessarily associated with significant movement, and vice versa. For large dolosse (which can have little residual strength), the extent to which movement causes a detrimental shift in boundary conditions appears more important than the absolute magnitude of the movement itself.

p. The most significant structural design parameter for large dolosse is static stress. Subtle movement in the dolos matrix can cause shifts in dolos boundary conditions which, in turn, produce a change in dolos static stress. Field data on dolos movement, static stress, and breakage should continue to be collected in order to better understand the long-term nesting behavior of large dolosse.

q. Photogrammetric analysis of a breakwater is an excellent tool in mapping the above-water portion of the structure and quantifying changes in elevation.

r. Aerial photography and subsequent photogrammetric analysis can provide very accurate data on movement of armor units located above the waterline. The methods require only minimal ground truthing to ensure accuracy of the data. Low-altitude helicopter surveys result in significant improvements in data accuracy and photo image resolution when compared to higher altitude, fixed-wing surveys.

s. Strain gauges positioned inside instrumented dolosse reveal that static stress loads in some of the units reach levels that leave little residual strength for pulsating wave loads and impact loads. The most significant structural design parameter for large dolosse is static stress.

t. Low-level helicopter inspections and 35-mm photography provide a good first indication of levels of armor unit breakage and give a basis for determining if an on-the-ground inspection is needed to gain more precision regarding armor unit breakage that is not captured by the aerial inspection.

u. The directionality of incident wave conditions should definitely be obtained. Wave monitoring should be planned and initiated as early as possible in the design process to allow definition of baseline conditions.

Floating Breakwaters

Structures monitored

Port of Friday Harbor Marina-Puget Sound, Friday Harbor, Washington. Onsite data were obtained during the time period January 1984 – July 1986 pertaining to the performance and durability of the floating breakwater. Operational experiences, such as recreational use, transient moorage difficulties/preferences, and wave/wake transmission, diffraction, and reflection problems, were documented.

The 580-boat marina at Friday Harbor is located on the eastern shore of San Juan Island on the inland waters of northwestern Washington State. The 488-m-(1,600-ft-) long floating breakwater was constructed and installed by the Corps in 1984. Tides at Friday Harbor are typical of those along the Pacific coast of North America, ranging from the lowest ever recorded at -1.2 m (-4 ft) mllw to +3.4 m (+11 ft) mllw. Water depth at the site varies between 12 and 15 m (40 and 50 ft). Maximum current velocities are northerly at less than 0.5 m per sec (1.5 ft per sec) during spring ebb tide. Currents are less than 0.3 m/sec (1.0 fps) during flood tide and are southerly. Winter storms can produce winds in excess of 80 knots (50 mph) from the northeast. Design wave conditions exhibit a significant wave height H_s of 1.0 m (3.2 ft) and period T of 3.2 sec from the northeast, and H_s of 0.8 m (2.7 ft) and T of 2.6 sec from the southeast.

The breakwater consists of five rectangular concrete pontoons, three of which are 100 m (330 ft) long by 6.4 m (21 ft) wide by 1.8 m (6 ft) high. Two pontoons are 4.9 m (16 ft) wide by 1.7 m (5.5 ft) high. Breakwater anchors are 52 steel H-piles embedded their full length. Anchor lines consist of 3.5-cm-(1-3/8-in.-) diam galvanized bridge rope with 9.1 m (30 ft) of 3.2-cm (1-1/4-in.) stud link chain at the upper end. Anchor-line lengths were sized to provide a scope of 4:1 to 5:1. A 908-kg (2,000-lb) concrete clump weight is attached approximately 15 m (50 ft) from the upper end of each anchor line. Anchor-line initial tension is approximately 4,540 kg (10,000 lb). Three large aluminum anodes were attached to each anchor line to prevent corrosion.

University of Washington Oceanographic Laboratory-Puget Sound, Friday Harbor, Washington. Onsite data were obtained during the time period 1979 – 1985 pertaining to the performance and durability of the floating breakwater. Operational experiences, such as recreational use, transient moorage difficulties/preferences, and wave/wake transmission, diffraction, and reflection problems, were documented.

The floating breakwater at the University of Washington Oceanographic Laboratory is about 0.8 km (0.5 miles) north of the Port of Friday Harbor. The site has an open fetch to the east of about 6.4 km (4 miles). Tide conditions are the same as for the floating breakwater at Friday Harbor, but the site is more exposed to the east. Design parameters were a 46-knot- (28-mph-) wind fetch-limited significant wave height of 0.8 m (3.0 ft), a period of 3.5 sec, and a current of 1.5 knots (0.9 mph). Boat wakes up to 0.6 m (2 ft) are common. Water depth varies between 3 and 18 m (10 and 60 ft).

Installed in 1979, the breakwater is a reinforced concrete caisson cast over a polystyrene foam core with a cross section of 1.4 by 4.6 m (4.5 by 15 ft) and a design freeboard of 0.5 m (1.5 ft). It is L-shaped with two 40-m (130-ft) sections on the long leg parallel to the east-west shore and a third 40-m (130-ft) section on the short north-south leg. The anchor system is laid out to maintain a 1.8-m (6-ft) space between the sections to avoid linkage and impact problems. Short gangways provide access between units. The breakwaters are used as staging areas for handling nets and other gear, as well as to provide a protected mooring area.

Each float is independently anchored by 2.54-cm- (1-in.-) diam stud-link chain anchor lines attached to the four corners of each section. Each corner line is oriented at a 45-deg angle to the breakwater. Clump weights (2,722-kg (3-ton)) are attached to the anchor lines except the landward line on the north-south leg. Because bottom conditions at the site consist of a shallow covering of sand over bedrock, only gravity anchors were considered. The main anchors are 2.4- by 2.4- by 1.8-m (8- by 8- by 6-ft) concrete blocks.

East Bay Marina-Puget Sound, Olympia, Washington. Onsite data were obtained 3 years following installation pertaining to the performance and durability of the floating breakwater. Operational experiences, such as recreational use, transient moorage difficulties/preferences, and wave/wake transmission, diffraction, and reflection problems, were documented.

The floating breakwater at East Bay, Olympia, WA, is located at the southernmost terminus of Puget Sound, approximately 145 km (90 miles) south of Seattle. Tidal range here varies from a lowest recorded -1.5 m (-5 ft) mllw to a highest recorded +5.5 m (+18 ft) mllw. The marina site is exposed to wind waves generated from the northwest through northeast directions. Design wave height at the breakwater is a 0.6-m (2.0-ft) significant wave with a period of 2.8 sec from the north-northwest.

The breakwater consists of seven rectangular concrete modules, 30 m (100 ft) long by 4.9 m (16 ft) wide by 1.7 m (5.5 ft) deep. Module walls are 12.7 cm (5.0 in.) thick with welded wire reinforcing, and each module is longitudinally posttensioned. The breakwater is held in place by timber anchor piles driven 6.1 m (20 ft) into the medium-dense sands below the bay muds. Modules are connected by large rubber fenders bolted between adjacent units. Dredging was required under the breakwater to a depth of -3.7 m (-12 ft) mllw to prevent the structure from striking bottom at extreme low tides and to provide keel clearance for boats at or near the breakwater.

Zittle's Marina-Puget Sound, Johnson Point, Washington . Onsite data were obtained during the time period 1983 – 1986 pertaining to the performance and durability of the floating breakwater. Operational experiences, such as recreational use, transient moorage difficulties/preferences, and wave/wake transmission, diffraction, and reflection problems, were documented.

The pipe-tire breakwater at Zittle's Marina near Johnson Point is a matrix of 40-cm- (16-in.-) diam pipes and truck tires held together with conveyor belting. It was constructed by the Seattle District as part of the Floating Breakwater Prototype Test Program. The breakwater was damaged as a result of faulty welds

during the test and was surplused at the end of the test program. A local marina operator salvaged the breakwater, towed it to the marina, and repaired it. The marina site is approximately 24 km (15 miles) south of the East Bay Marina, and tides at this location are essentially the same as those given for East Bay. It is completely protected from all directions except an open area to the north with a fetch of about 3.2 km (2 miles). No estimate of wave heights at the site has been made; but because of the limited exposure, wave heights probably do not exceed 0.9 m (3 ft.)

Since its installation at Johnson Point in 1983, the breakwater has sustained no damage; however, it has not been subjected to significant wave action (i.e., over 0.6 m (2 ft)). Even in this relatively mild environment, the marina operator feels that the breakwater performs a necessary function of providing protection from wave "chop" and boat wakes. The operator has made some progress in his attempt to refurbish the pipe-tire breakwater by repairing the damaged portions and adding several sections. Flotation of the breakwater is about the same as November 1983.

Port of Brownsville Marina-Puget Sound, Brownsville, Washington. Onsite data were obtained during the time period 1981 – 1983 pertaining to the performance and durability of the floating breakwater. Operational experiences, such as recreational use, transient moorage difficulties/preferences, and wave/ wake transmission, diffraction, and reflection problems, were documented.

The Brownsville Marina is located on the Kitsap Peninsula on the western margin of Puget Sound approximately 23 km (14 miles) west of Seattle, WA, with a maximum tide range of 6 m (19.5 ft). The breakwater, which provides protection from northerly waves (estimated H = 1.0 m (3.2 ft), T = 3.4 sec), was installed in 1981. It is a rectangular concrete pontoon 5.5 m (18 ft) wide and 1.5 m (5 ft) high and is composed of 24 units, each 4.6 m (15 ft) long. Units are posttensioned together to form a single 110-m- (360-ft-) long float. This float is moored in 3 to 6 m (10 to 20 ft) of water (at a 0.0-m (0.0-ft) tide) by stake piles, each attached to a 3.8-cm- (1.5-in.-) diam stud-link chain anchor line. No clump weights are attached to the anchor lines, but the oversized chain serves essentially the same purpose as clump weights. A north-south leg of the breakwater is exposed to much smaller waves from the south-east direction. It is composed of a series of 27 surplus U.S. Navy submarine net floats, each 3.7 m (12 ft) long and 1.8 m (6 ft) in diameter, and a 48-m- (157-ft-) long by 7-m- (23-ft-) wide landing craft ballasted to a 4.9-m (16-ft) draft. Floats and landing craft are ballasted with seawater. This makeshift portion of the breakwater is held in place by 7.6-cm- (3-in.-) diam nylon rope attached to the timber piles.

Semiahmoo Marina – Puget Sound, Drayton Harbor, Blaine, Washington. Onsite data were obtained during the time period 1981 – 1986 pertaining to the performance and durability of the floating breakwater. Operational experiences such as recreational use, transient moorage difficulties/ preferences, and wave/wake transmission, diffraction, and reflection problems were documented.

Since Drayton Harbor is shallow, the marina site had to be dredged to 3 m (-10 ft) mllw water. It is exposed only to the southerly quadrant with a fetch of

2.7 km (1.7 miles) to the south and 3.7 km (2.3 miles) to the southeast. Mean tide range is 1.7 m (5.7 ft), diurnal range is 2.9 m (9.5 ft), and maximum range is 5.2 m (17 ft). Wind waves used for design are not available but are probably in the 0.6- to 0.9-m (2- to 3-ft) range. Exposure to the south and southeast is likely to allow winds of 40+ knots (25+ mph) every winter, with 50-knot (30-mph) speeds on occasion.

The breakwater, constructed in 1981, is of the concrete caisson type. It was cast in 1.4- by 4.6- by 4.6-m (4.5- by 15- by 15-ft) units using polystyrene foam blocks as interior formwork and for positive flotation. The design draft was 0.9 m (3 ft). The total length of the breakwater, arranged in a U-shape, is approximately 1,065 m (3,500 ft). The marina eventually will have 840 slips for pleasure craft and fishing boats.

Each basic unit was truck-hauled to the site where four units were post-tensioned together to form 18.3-m (60-ft) modules. Next, the 18.3-m- (60-ft-) long modules were coupled by a chain-rubber fender connector. The anchor system uses clump weights on the anchor line consisting of a successive length of 2.54-cm- (1-in.-) diam nylon rope and stud-link chain to timber piles with a set of lines at each module connection.

Generic lessons learned

Details are as follows:

a. Floating breakwaters may become a popular fishing platform.

b. Considerable effort is required to adjust anchor-line tensions and clump-weight placement to align breakwater units. Interfloat connectors between units should be designed using large cylindrical rubber fenders to minimize destruction by severe storms. A corrosion protection system should be included in any design plan.

c. A floating breakwater detached from shore, with no boats moored to the breakwater itself, may become an excellent habitat for sea birds and seals.

d. Observations of pipe and scrap tire floating breakwaters made during the Puget Sound Prototype Test Program indicated that unfoamed tires tended to sink. Those same tires later appeared to have adequate flotation and were indistinguishable from the foam-filled tires. Several factors may contribute to this apparent contradiction. High tidal currents and resultant drag forces tend to pull the breakwater under and, once submerged, the tires may lose their entrapped air. Mild-wave climates probably leave trapped air undisturbed for longer periods of time, while large waves at the test sites may deform the tires enough to allow loss of some trapped air. Although the tires may still float at approximately the same level as they did originally, their ability to resist being submerged may be considerably less than when originally constructed.

e. After 4 years, tires between pipes will no longer support a person's weight. Apparently, without foam, the trapped air compresses as the tires

are submerged, resulting in decreased buoyancy. If marginally buoyant tires are submerged deeply enough, they will become negatively buoyant. Therefore, in areas where tidal currents are high or wave heights are greater than about 0.5 m (1.5 ft), including some type of incompressible flotation remains a requirement of conservative design.

f. Holes in pontoon floating breakwaters through which pilings pass should not be large enough to allow a child to fall between the piling and the float. As a temporary solution, plywood rings can be placed over the pilings.

g. During extreme cold weather conditions, waterlines may rupture because of differential expansion between the floats and polyvinyl-chloride waterlines or the freezing of trapped water. Waterlines on a floating breakwater should be enclosed within the float to avoid freeze damage.

h. Shackles used to connect stud-link chain to breakwater connection flanges should be designed adequately large to withstand storm conditions. Anchor lines should be inspected often and replaced as necessary.

i. Zinc anodes should be attached at various places along new anchor chains to reduce the rate of corrosion.

j. Collision by a large boat may severely damage floating breakwaters.

k. Loads for a floating breakwater design, and for the accompanying anchor system design, should include allowance for additional loading because of vessels moored on the seaward side of the breakwater, if such mooring is anticipated. Significant additional loads could be generated if large vessels are moored there. Adequate tieup facilities should be made available on the seaward side of the breakwater, which may prove to be a popular fishing pier.

l. Access and interfloat ramps should be wide enough to allow access to the breakwater by electrically powered vehicles. Such vehicles may be used to reduce travel time to the end of long breakwaters.

m. Stanchions, located on the breakwater to supply electrical service to transient boats, should be low enough to avoid vulnerability to being knocked over by bowsprits of docking boats.

n. Electrical junction boxes should be positioned where they do not fill with water; access plates should be carefully sealed. Hardware that provides mechanical support for the electrical wiring should be designed specifically for use in a marine environment.

Jetties

Structures monitored

Yaquina Bay North Jetty, Oregon. Monitoring was conducted during the time period October 1988 – September 1994 to determine the likely cause for chronic damage to the Yaquina Bay north jetty. This monitoring also offered the potential for increasing understanding of failure mechanisms associated with

rubble-mound structures and for improving methods of monitoring coastal structure performance in similar hostile wave and current environments.

Humboldt Bay, California. Base conditions for future periodic inspections of the jetties were determined in December 1996. Periodic inspections will use limited land-based surveying, aerial photography, and photogrammetric analysis to assess the long-term stability response of the concrete dolos-armored units on the heads of the Humboldt Bay jetties. Land surveys, broken armor unit inspections, aerial photography, and photogrammetric analyses will be used to define armor unit movement above the waterline.

The Humboldt Bay jetties have experienced a long history of damage and subsequent repairs since original construction was completed in 1899. Rehabilitations were completed in 1911, 1927, 1932, 1939, 1950, 1957, 1963, 1971, 1988, and 1995. These rehabilitations consisted of the construction and/or installation of concrete monoliths, parapet walls, mass concrete, stone, concrete blocks, tetrapods, and dolosse. Since the dolos rehabilitation of the heads of the jetties in 1971, damages have been primarily along the trunk (stone) reaches of the jetties. No extensive work has been required along the dolos fields since their construction. Prior to this study, no sound, quantifiable data relative to the movement or positions of the dolos concrete-armored units had been obtained for the jetties.

Umpqua River Mouth Training Jetty Extension, Oregon. Monitoring was conducted during the time period May 1983 – May 1984 to determine the effects of an extension to a third jetty that had been constructed inside previously completed arrowhead jetties at the mouth of the Umpqua River. The arrowhead jetties, constructed in 1938, were not satisfactory in eliminating shoaling of the entrance channel. A third, or training jetty, was constructed in 1951 on the south side of the entrance channel. This training jetty was 1,295 m (4,240 ft) long, and generally paralleled the entrance channel. The seaward terminus was about 0.8 km (0.5 miles) landward of the outer end of the old south arrowhead jetty. The training jetty might have caused a slight increase in channel shoaling, and a possible increase in wave activity in the entrance. A 790-m (2,600-ft) extension to the training jetty was recommended and completed in 1980 so that the training jetty now terminated at the same location as the old south arrowhead jetty.

Generic lessons learned

Details are as follows:

a. A sandy bottom in the vicinity of a breakwater or jetty has the potential to scour during storm events. A moveable-bed modeling effort should be used to determine if scour will lead to armored-layer instability.

b. Analysis of side-scan sonar images, collected as part of a geophysical survey, can be instrumental in determining the underwater configuration of jetty toes and their relationship to the surrounding sandy bottom. SEABAT track lines will provide sufficient data to detail a jetty's underwater configuration.

c. Deterioration and gradual armor displacement occurring during severe storm conditions is most likely not associated with liquefaction of a jetty foundation, but probably results from wave and current action on the structure units.

d. Through a semi-quantitative physical model that features a moveable-bed section, it can be determined whether waves alone will cause armor instability. Obliquely approaching waves modified by seaward flowing currents along a jetty and with a hard-bottom reef at a structure tip may cause waves to break directly onto the structure, resulting in extensive damage and ultimately eroding the jetty to an unsatisfactory crest elevation.

e. Current data acquired in the prototype with an Acoustic Doppler Current Profiler in the vicinity of a jetty indicate that, even in very mild wave conditions, the jetty redirects longshore-flowing currents to produce moderate seaward-flowing currents adjacent to the structure. This finding lends credence to the wave/current damage hypothesis that implies the combination of a current in the presence of low waves can induce greater structure stone damage than larger waves alone.

f. Jetties at tidal entrances should be constructed parallel to each other and to the navigation channel. Converging or arrowhead jetties often fail to provide for stable entrances and safe navigation.

g. Photogrammetric analysis of a jetty is an excellent tool in mapping the above-water portion of the structure and quantifying changes in elevation.

Jetty Spurs

Structures monitored

Siuslaw River Mouth Jetty Spurs, Oregon. Monitoring was conducted during the time period 1987 – 1990 to identify shoaling and current patterns and determine the effectiveness of jetty spurs in reducing maintenance dredging.

Generic lessons learned

Details are as follows:

a. Bathymetric data reveal that jetty spurs effectively deflect sediment away from entrance channels. Sediment either circulates back toward shore where it is reintroduced into the littoral system or is carried offshore away from the jetty by a jet of water parallel to the spur.

b. Drogues, dye studies, and aerial photographs used to determine current patterns are not adequate in delineating bottom currents. An Airborne Coastal Current Measurement (ACCM) system can be used to measure and establish bottom current patterns. The system is a very effective method for obtaining bottom currents in hostile wave environments

where boat operation is dangerous or where quick mobility is necessary. Current patterns correlate well with depositional patterns identified through bathymetric data.

c. The Scanning Hydrographic Observational Airborne LIDAR Survey (SHOALS) system (either helicopter or fixed-wing) is effective in measuring seabed bathymetry in hazardous regions where survey vessels cannot operate safely. Soundings can be taken quickly and are accurate and repeatable.

d. Current patterns and sediment depositional patterns can be predicted and verified by 3-D physical model laboratory experiments of spur jetties.

e. Navigation conditions improve at a spur-jettied entrance, as supported by analysis of shoaling and sediment volume accumulation, and by inspection of bathymetric data. Accumulation of material shifts offshore into deeper water as opposed to moving into the entrance channel. Vessels could then navigate an entrance year-round, barring storm events, and not be confined to periods of high tide.

f. Shoreline change upcoast and downcoast of jetties can be predicted by a numerical model using a wave energy littoral transport equation and an equilibrium shoreline concept.

Wave Transformation

Structures monitored

Redondo Beach, California. Monitoring was performed during the time period October 1992 – June 1994 to compare observed offshore wave transformation (as measured in the prototype) with theoretical wave propagation models for this area of steep, complex bathymetry. Field data measurements were compared with results from the Regional Coastal Processes Transformation Model (RCPWAVE) and from a spectral refraction model STeady WAVE (STWAVE) that treats the propagation of spectral waves rather than monochromatic waves as in RCPWAVE.

Columbia River Mouth, Washington/Oregon. Monitoring was conducted during the time period October 1994 – September 1999 at the Mouth of the Columbia River (MCR) to investigate dangerous wave transformation due to sandy dredged material being placed in Environmental Protection Agency (EPA) approved ocean dredged material disposal sites (ODMDS) and to estimate the rate of sediment transport from these ODMDS onto nearby beaches. Since 1986, dredged material placed within the designated ODMDS has accumulated at a rate much faster than the District had anticipated when the disposal sites were formally designated. Exceedence of ODMDS capacity at the MCR creates two operational problems for the District: (a) overall footprint of disposed dredged material extends beyond the existing ODMDS formally permitted boundaries by as much as 915 m (3,000 ft) in some cases; and (b) dredged material within the ODMDS has accumulated to such an areal and vertical extent that adverse sea conditions are created. In some cases, mounds rise 18.3 to 21.3 m (60 to 70 ft)

above the surrounding bathymetry. Mariners report that the ODMDS mounds cause waves to transform and steepen and/or break in the vicinity of the sites. This wave transformation is exceedingly hazardous to navigation.

The objectives of monitoring at the MCR were to assess the suitability of new USACE Dredging Research Program sediment fate models including Short-Term FATE (STFATE), Long-Term FATE (LTFATE), and Multiple-Dump FATE(MDFATE), Regional Coastal Processes WAVE (RCPWAVE) model, and synthetically generated input data from Height Period Direction PREliminary (HPDPRE) wave model, Height Period Direction SIMulation (HPDSIM) wave model, and Advanced CIRCulation (ADCIRC) hydrodynamic circulation model for predicting sediment dispersion in the environment of the MCR.

Generic lessons learned

Details are as follows:

a. Modeling wave transformation over a variable sea bottom remains a difficult task in most cases. Analytical solutions limit themselves only to simple geometry, and numerical treatments base their predictions on the fundamental assumption of slowly varying sea depth. Modeling difficulty is increased by the presence of deep submarine canyons offshore that affect waves from the predominant direction of attack. Neither RCPWAVE nor STWAVE was developed for application in complex steep topography and should not be applied in such locations. STWAVE is presently the numerical model being supported by the Corps.

b. RCPWAVE was used to compare wave climate resulting from the present ODMDS bathymetry with the wave climate resulting from past bathymetry before prominent mounds were formed at the ODMDS. In some cases, mounds rise 18.3 to 21.3 m (60 to 70 ft) above the surrounding bathymetry. The existing dredged material mounds increased the height of incident waves within or in proximity to ODMDS by 30 percent for 6-sec waves, 60 percent for 10-sec waves, and 80 percent for 16-sec waves. A 10-percent increase in wave height resulting from shoaling could cause a wave to break.

c. A technique for using helicopters to deploy and retrieve oceanographic instrumentation platforms for wave and other data collection under severe wave conditions was developed. Depending on the length of the desired measurement, the platform can be immediately withdrawn and repositioned, or released and subsequently recovered with the helicopter. This technique is exceedingly useful where safe navigation of a vessel and over-the-side research vessel operations for deploying instruments is not possible under severe wave climates.

Harbors

Structures monitored

Fisherman's Wharf, San Francisco, California. Monitoring was performed during the time period December 1982 – December 1989 to evaluate performance of the breakwater and to determine its impact on the adjacent harbor. Specific objectives were to: (a) document wave attenuation of the structure compared to model studies and design criteria; (b) evaluate effect of the structure on surge within the harbor complex; (c) determine effect of the structure on water circulation within the harbor and surrounding areas and currents, especially at the entrance; (d) determine the actual scour, measure the scour, evaluate the cause, and compare with predicted scour; (e) evaluate effect of the structure on littoral processes, including shoreline response and deposition within the harbor; and (f) determine integrity of the structure by investigating spalling, cracking, and settlement of the wall.

An impermeable vertical-wall detached breakwater structure forms the main element of the breakwater system. This 460-m- (1,509-ft-) long structure was built using driven prestressed/precast interlocking sheet piles. A cast-in-place reinforced cap beam ties the piles together. For most of its length, the detached breakwater is oriented approximately in the shore-parallel west-southwest to east-northeast direction. This alignment intercepts waves from the northwest, yet is essentially parallel to the prevailing tidal currents.

Morro Bay Harbor Entrance, Morro Bay, California. Monitoring was performed during the period January 1998 – August 2001 to evaluate the effectiveness of improvements to alleviate hazardous wave conditions at the entrance to the harbor. Prior to the improvements completed in December 1995, Morro Bay Harbor entrance was known as one of the most dangerous harbors in the United States. The latest improvements consisted of construction of a deepened, expanded entrance channel. The new channel doglegs westerly from the old entrance channel and flares open to a width of 290 m (950 ft). The authorized depth of the channel extension is -9.1 m (-30 ft). However, the plan provides for advanced maintenance by deepening the new channel to -12.2 m (-40 ft) and dredging an additional sand trap to a depth of -9.1 m (-30 ft) within the harbor entrance structures north of the head of the south breakwater.

Monitoring was conducted to determine that: (a) improvements would result in significantly improved navigation conditions in the harbor entrance; (b) improvements would have no negative impact on existing structures; (c) channel deepening can be effectively maintained with a 3-year dredging interval; (d) model investigations accurately quantified wave conditions in the entrance, and correctly defined sediment patterns and deposition in a qualitative sense; and (e) methodology used in determining sedimentation rates in the harbor entrance are valid based on field data, model predictions, and sound engineering judgment.

Generic lessons learned

Details are as follows:

a. Modifications to standard open-coast wave data processing and analysis procedures should be considered when monitoring sites that are subject to simultaneous ocean-generated and locally generated waves. Specifically, analysis should avoid overlapping frequency coverage between surge, ocean-generated (swell), and locally generated waves. Sampling rates (both frequency of gauge polling and frequency of pressure sampling within bursts) should be specifically tailored to the frequency regimes present. Sampling rate considerations and decisions about hard-wired versus self-recording gauge technology should also include examination of how fast wave conditions might change at the site.

b. Numerical model CGWAVE results compare much more favorably than numerical model HARBD results with physical model data within a harbor. This is partly attributable to CGWAVE being a more comprehensive model, and partly to CGWAVE being expressly configured to match physical model test conditions. CGWAVE also matches inner harbor prototype gauges remarkably well.

Inlets

Structures monitored

Morro Bay Harbor Entrance, Morro Bay, California. Monitoring was performed during the period January 1998 – August 2001 to evaluate the effectiveness of improvements to alleviate hazardous wave conditions at the entrance to the harbor. Prior to the improvements completed in December 1995, Morro Bay Harbor entrance was known as one of the most dangerous harbors in the United States. The latest improvements consisted of construction of a deepened, expanded entrance channel. The new channel doglegs westerly from the old entrance channel and flares open to a width of 290 m (950 ft). The authorized depth of the channel extension is -9.1 m (-30 ft). However, the plan provides for advanced maintenance by deepening the new channel to -12.2 m (-40 ft) and dredging an additional sand trap to a depth of -9.1 m (-30 ft) within the harbor entrance structures north of the head of the south breakwater.

Monitoring was conducted to determine that: (a) improvements would result in significantly improved navigation conditions in the harbor entrance; (b) improvements would have no negative impact on existing structures; (c) channel deepening can be effectively maintained with a 3-year dredging interval; (d) model investigations accurately quantified wave conditions in the entrance, and correctly defined sediment patterns and deposition in a qualitative sense; and (e) methodology used in determining sedimentation rates in the harbor entrance are valid based on field data, model predictions, and sound engineering judgment.

Umpqua River Mouth Training Jetty Extension, Oregon. Monitoring was conducted during the time period May 1983 – May 1984 to determine the effects of an extension to a third jetty that had been constructed inside previously completed arrowhead jetties at the mouth of the Umpqua River. The arrowhead jetties, constructed in 1938, were not satisfactory in eliminating shoaling of the entrance channel. A third, or training jetty, was constructed in 1951 on the south side of the entrance channel. This training jetty was 1,295 m (4,240 ft) long and generally paralleled the entrance channel. The seaward terminus was about 0.8 km (0.5 miles) landward of the outer end of the old south arrowhead jetty. The training jetty might have caused a slight increase in channel shoaling and a possible increase in wave activity in the entrance. A 790-m (2,600-ft) extension to the training jetty was recommended and completed in 1980 so that the training jetty now terminated at the same location as the old south arrowhead jetty.

Generic lessons learned

Details are as follows:

a. A deepened and widened entrance channel to a harbor will reduce wave heights previously existing there as the result of incident wave conditions and steepening of those incident waves by ebb currents.

b. A reduction of wave conditions at a harbor entrance by deepening and widening the entrance channel will have no adverse impact on breakwaters and jetties adjacent to the entrance channel.

c. The dredging cycle of an entrance channel to a harbor depends on episodic wave conditions, the availability of sediment for infilling of the channel, and whether or not over-dredging is deliberately performed to reduce the frequency of dredging.

d. Present-day physical and numerical modeling capabilities accurately quantify wave conditions in entrance channels, and correctly define sediment patterns and deposition in qualitative sense.

e. Regime theory or appropriate inlet stability analysis is important in tidal inlets on sandy coasts where maintenance dredging may be required.

f. Although wave-driven longshore sediment transport is certainly a key source of sediment arriving at a harbor entrance along a straight coastline, this source of sediment is not so directly related at unique coastline and harbor orientations. In unique situations, numerical computations of potential longshore sediment transport must be tempered with quality field data and sound engineering judgment.

Beach Nourishment and Sediment Transport

Structures monitored

Columbia River Mouth, Washington/Oregon. Monitoring was conducted during the time period October 1994 – September 1999 at the MCR to investigate

dangerous wave transformation resulting from sandy dredged material being placed in EPA approved ODMDS. Since 1986, dredged material placed within the designated ODMDS has accumulated at a rate much faster than the District had anticipated when the disposal sites were formally designated. Exceedence of ODMDS capacity at the MCR creates two operational problems for the District: (a) overall footprint of disposed dredged material extends beyond the existing ODMDS formally permitted boundaries by as much as 915 m (3,000 ft) in some cases; and (b) dredged material within the ODMDS has accumulated to such an areal and vertical extent that adverse sea conditions are created. In some cases, mounds rise 18.3 to 21.3 m (60 to 70 ft) above the surrounding bathymetry. Mariners report that the ODMDS mounds cause waves to transform and steepen and/or break in the vicinity of the sites. This wave transformation is exceedingly hazardous to navigation.

The objectives of monitoring at the MCR were to assess the suitability of new USACE Dredging Research Program sediment fate models including STFATE, LTFATE, and MDFATE, RCPWAVE, and synthetically generated input data from HPDPRE wave model, HPDSIM wave model, and ADCIRC hydrodynamic circulation model for predicting sediment dispersion in the environment of the MCR.

Generic lessons learned

Details are as follows:

a. Numerical simulation modeling has verified the applicability of the DRP numerical models for the evaluation of sediment transport, including STFATE, LTFATE, MDFATE, RECWAVE, HPDPRE, HPDSIM, and ADCIRC. Capabilities and limitations of the DRP models have been determined.

b. Predictive techniques for determining environmental conditions and sediment transport processes under both waves and currents have been developed. Three sediment transport methods were adapted to simulate time periods of data collection. The methods for simulating sediment transport by both waves and currents are those of van Rijn, Wikramanayake and Madsen , and Ackers and White. All methods performed reasonably well under most conditions. It was documented that long-term synthetic database of wave and currents could be used to estimate sediment transport. A 12-year-long synthetic database of wave and current conditions was developed from combined field measurements and numerical modeling. The sediment transport methods were then applied successfully to this 12-year period of the developed database.

12 Structures Monitored and Generic Lessons Learned, Gulf of Mexico

Weir-Jetties

Structures monitored

Colorado River Mouth, Texas. Monitoring was performed during the time period May 1990 – September 1992 to evaluate the design and efficiency of a weir jetty and adjacent impoundment basin at the mouth of the river.

East Pass Inlet, Destin, Florida. Monitoring of waves, currents, tidal elevations, bathymetry, and shoreline changes at East Pass Inlet was conducted during the time period 1983 – 1991 to better understand the inlet's behavior during the past 120 years: (a) pre-1928 – spit development and breaching, covering the period when the inlet was oriented northwest-southeast between Choctawhatchee Bay and the Gulf of Mexico; (b) 1928-1968 – stable throat position but main ebb channel that migrated over a developing ebb-tidal shoal, covering the time after the inlet breached Santa Rosa Island in a north-south direction and then migrated eastward; and (c) after rubble-mound arrowhead jetties with sand bypassing weir in west jetty were built – the throat and main ebb channel were stabilized while the ebb-tidal shoal grew. Because of uncertainties regarding its effectiveness, the weir was closed in 1986.

Generic lessons learned

Details are as follows:

a. A weir-jetty project should be maintained as designed unless long-term or overwhelming evidence indicates that changes are needed. If maintenance practices are frequently adjusted, it is almost impossible to determine how successful the project has performed and what lessons could be learned to improve future projects.

b. Scour at jetties can be minimized or eliminated by a number of engineering designs. A spur jetty can be built with extensive toe protection to prevent collapse. Any scour hole near the tip of a spur jetty should be

filled and then armored to prevent future scour. While use of concrete and rubble fill may provide temporary relief, an engineered approach employing precisely placed armor units may be more successful. A design using graded-stone layers may also be successful.

c. If the longshore transport rate at a project site is substantially underestimated during the design of the weir-jetty system, the impoundment basin and entrance channel will shoal substantially more rapidly than expected following construction. The creation of a safe, navigable inlet is the primary purpose of such construction, and shoaling of an inlet mouth will adversely impact navigation.

d. Good reliable estimates of longshore transport rate are needed prior to jetty and impoundment basin design. The current recommended method is to compute the longshore transport rate from at least 2 years of onsite wave data. Failure to do this will lead to uncertainties in anticipated dredging costs and may lead to poor choices in jetty and impoundment basin design.

e. As an impoundment basin fills, it may become less efficient at retaining sediments. This occurs because the bottom is subjected to increased wave and current forces as the basin fills.

f. When the principal management problem at a weir-jetty system is caused by inadequate size of the impoundment basin and its inefficiency in retaining sediments, one solution is to increase the frequency of the dredging schedule. This is an effective strategy, but other strategies may be more cost-effective and should be considered.

g. Total dredging costs may be decreased if an impoundment basin is enlarged, as the larger basin volume will delay the time required for shoaling to fill the basin.

h. Most weir-jetty systems are located at inlets that typically have minimal amount of inland-derived sediments. In designing weir-jetty systems at river mouths that carry large sediment loads, both beach and river sediments must be taken into consideration. If the riverborne sediments are expected to pass through the system without creating substantial shoaling problems, care should be taken to situate the impoundment basin so that minimal trapping of the riverborne sediments occurs. This could be done through the use of retaining dykes, by physically separating the basin from the river mouth, or by other creative approaches.

i. It is important for the project design to have flexibility to allow for modifications of the size and shape of the impoundment basin based on operational experience.

13 Structures Monitored and Generic Lessons Learned, Atlantic Coast of the U.S. Mainland

Jetties

Structures monitored

Manasquan Inlet, New Jersey. Monitoring was conducted from June 1982 – October 1984 to: (a) evaluate the performance of the dolos-armored units in maintaining structural stability of the jetties; (b) determine potential effects of the rehabilitated jetties on longshore sediment movement at the inlet; and (c) determine the effectiveness of the rehabilitated jetties in maintaining a stable inlet cross section. A periodic inspection was conducted in August 1994 to reexamine the dolos portions of the Manasquan Inlet jetties and determine changes that have occurred since prior monitoring ended in 1984. A second periodic inspection was conducted in October 1998 to determine dolos changes that might have occurred since the last inspection in 1994.

Barnegat Inlet, New Jersey. Monitoring was conducted during the period October 1992 – September 1997. Evaluation of the effectiveness of a new south jetty completed in June 1991, on the inlet system, needed to be ascertained to provide improved inlet and jetty system design guidance, to enhance construction of rubble-mound jetties, and to develop better maintenance techniques for tidal inlets. The new jetty was constructed parallel to the north jetty, and replaced an existing southern arrowhead jetty. The project performance was assessed with regard to providing a stable navigation channel and a stable rubble-mound jetty structure, should have no adverse impact on either tidal hydraulics or high tide levels, and would not adversely affect upcoast or downcoast beaches.

Generic lessons learned

Details are as follows:

a. When dolos jetty structures experience storms up to a design event, they perform successfully and may not require even the low level of maintenance anticipated by designers. Overall excellent performance of dolos jetties and the low percentage of dolosse broken during storms verify dolos design and construction procedures.

b. There is a threshold of breakage of a dolos-armored structure beyond which the structure is likely to fail.

c. Use of photogrammetric mapping of jetties allows a detailed evaluation of the motion of the armor units. This technique is cost-effective and accurate, providing accuracy comparable with standard leveling techniques.

d. Dolos-armored units on flatter slopes tend to be forced up the slope by forces associated with wave runup, while those on steeper slopes will be moved downslope by wave rundown.

e. Dolosse benefit from the use of steel reinforcement. Even units that crack remain intact by reinforcement. Reinforcing escalates the cost of casting dolosse, so the decision whether to reinforce the units is still one of cost versus benefits. At present, the largest dolosse are often designed for no impact; however, much of their unreinforced tensile strength is designed for supporting static loads. Smaller units will certainly be displaced and could benefit the most from reinforcement. The decision to reinforce dolos-armored units will continue to be based on engineering judgment until more information is acquired concerning the long-term effects of rust, the benefits associated with units maintaining their integrity even though cracked, and a better understanding of the relationship between impact load, static load, pulsating wave load, and dolos breakage.

f. The value of sand-tightening jetties is significant. Sand-tightening structures have little effect on the tidal prism.

g. Side-scan sonar is a cost-effective inspection tool for underwater portions jetties and breakwaters.

h. Photogrammetric mapping is equally applicable to structures with any type of natural (i.e., stone) or man-made armor (i.e., dolos, CORE-LOC, etc.). The accuracy of photogrammetry is more than adequate to evaluate armor unit movement. Periodic mapping of a coastal structure permits detection of incipient or progressive failure along any visible portion of the structure before such a problem is readily detectable by other means. This detection allows for early assessment and possible correction of the problem.

i. Photogrammetry offers several advantages over conventional land surveying techniques. First, it is possible to map armor units at or near the waterline of the structure, units that would be inaccessible or too hazardous to reach on foot. Second, photogrammetry is flexible in that all the information needed to perform the mapping can be obtained almost instantaneously, permanently, and at fixed cost with one aerial photographic flight. The mapping can then be performed at any time thereafter, depending on available resources, need for information, etc. In contrast, land survey methods capable of obtaining location, orientation,

and elevation data for mapping every visible armor unit are labor-intensive and require more time and expense than photogrammetry. It is unlikely that improvements in ground survey techniques will reduce costs enough to challenge the cost-effectiveness of photogrammetry.

j. Periodically (on the order of every 5 years) dolos jetties should be photogrammetrically mapped. Such mapping will provide additional useful information on the long-term stability of dolosse.

k. Resurveys indicate that dolos movement is less dynamic during later periods as opposed to earlier survey periods. Movement occurs in an asymptotic fashion as dolosse settle and nest into a stable position.

l. When a new stable rubble-mound jetty is constructed entirely within the confines of a previously existing jetty system with the same cross-sectional area, there will be no adverse impacts on either tidal elevation within the bay system, on navigation through the new inlet system, nor on the upcoast and downcoast beaches.

Inlets

Structures monitored

Manasquan Inlet, New Jersey. Monitoring was conducted from June 1982 – October 1984 to: (a) evaluate the performance of the dolos-armored units in maintaining structural stability of the jetties; (b) determine potential effects of the rehabilitated jetties on longshore sediment movement at the inlet; and (c) determine the effectiveness of the rehabilitated jetties in maintaining a stable inlet cross section. A periodic inspection was conducted in August 1994 to reexamine the dolos portions of the Manasquan Inlet jetties and determine changes that have occurred since prior monitoring ended in 1984. A second periodic inspection was conducted in October 1998 to determine dolos changes that might have occurred since the last inspection in 1994.

Ocean City, Maryland. Monitoring was conducted during the time period October 1986 – January 1989 to: (a) verify studies relating to the cause of the problem shoal; (b) evaluate the effectiveness of the rehabilitated jetty cross section as a littoral barrier; (c) evaluate the effectiveness of the shoreline stabilization on the northern shoreline of Assateague Island; (d) verify/calibrate the Shore Protection Manual Longshore Transport formula; (e) examine the distribution of longshore transport across the surf zone; (f) analyze the shoreline and profile response following rehabilitation of the jetty; (g) evaluate the ebb shoal equilibrium and northern Assateague Island growth; and (h) evaluate scour hole stabilization.

Barnegat Inlet, New Jersey. Monitoring was conducted during the period October 1992 – September 1997. Evaluation of the effectiveness of a new south jetty completed in June 1991, on the inlet system, needed to be ascertained to provide improved inlet and jetty system design guidance, to enhance construction of rubble-mound jetties, and to develop better maintenance techniques for tidal inlets. The new jetty was constructed parallel to the north jetty, and replaced an

existing southern arrowhead jetty. The project performance was assessed with regard to providing a stable navigation channel and a stable rubble-mound jetty structure, should have no adverse impact on either tidal hydraulics or high tide levels, and would not adversely affect upcoast or downcoast beaches.

Generic lessons learned

Details are as follows:

a. Construction of jetties causes establishment of a new equilibrium for the inlet ebb tidal delta system. Bathymetric measurements over shoals and surveys along adjacent shorelines are required over an extended time period to establish the new equilibrium. When an equilibrium state is reached, natural bypassing may resume via the ebb-tidal delta.

b. Sealing an updrift jetty to prevent passage of sand is effective in preventing shoaling in the inlet.

c. Sealing a jetty can result in erosion of a shoreline inside a jettied inlet, when that shoreline was previously nourished by sand passing through the jetty. Protective measures may be required for a shoreline inside a jettied inlet concurrent with sealing a jetty.

d. Average shoreline configuration between segmented “headland” breakwaters used to prevent shoreline erosion can be predicted based on empirical understandings, considering the combined effects of tidal currents and variations in wave conditions.

e. Filling a scour hole at the end of a jetty and covering the area with a layer of armor stone is effective in preventing further scouring.

f. Shorelines on the updrift side of a sealed jetty will advance oceanward as a result of the enhanced sand-trapping ability of the rehabilitated jetty. Accretion from the sand tightening will cause initial steepening of the profiles near the jetty. Later offshore transport of sand occurs with the subsequent flattening of the profiles.

g. The volume of the ebb-tidal delta increases rapidly after jetty construction, but the delta increase gradually tapers asymptotically as a state of equilibrium is approached. As the system moves toward equilibrium, more and more sediment will be bypassed.

h. Sand-tightening jetties will eliminate the need for some maintenance dredging of the navigation channel. In situations where porous structures contribute to shoaling of a channel, the economics of sand-sealing rehabilitation on the structures should be investigated.

i. Use of the equation relating critical inlet cross-sectional area and tidal prism is appropriate for inlets that have exhibited historic stability.

j. The use of Wave Information Study (WIS) data has the most potential for predicting sand transport with reasonable accuracy. Littoral Environmental Observations (LEO) data (formerly, but no longer

collected) should not be used for calculating sand transport because of the inherent inaccuracies involved in making the observations.

Beach Nourishment and Sediment Transport

Structures monitored

Oakland Beach, Rhode Island. Monitoring was conducted during the period April 1982 – April 1985 to perform an assessment of the Beach Erosion Control Project at Oakland Beach. Monitoring included hydrographic and topographic surveys of the beach and nearshore area, aerial and ground photographs, wind data, littoral environmental observations, and sediment sampling. Littoral transport, structure (groin) stability, and wind and wave data were evaluated.

Carolina Beach, North Carolina. Monitoring was conducted during the period April 1981 – September 1984 to determine the adequacy of a sediment trap to serve as a primary source of beach nourishment material for the project and to assess the impact of the trap on the inlet's ebb tide channel and delta.

Generic lessons learned

Details are as follows:

a. In computing the volume of material required to construct a beach fill having a specific width, the designer must assume that the improved beach profile will parallel the existing beach profile down to some depth of closure.

b. Once the design volume has been determined, the only practical way to construct the fill is to place the required quantity on the beach in the form of a sacrificial construction berm. The crest elevation of the construction berm should be equal to the natural berm elevation in the area. The width of the construction berm will depend on the slopes that the material assumes during placement and the volume of material to be placed. Since this slope will not generally be known beforehand, surveys should be conducted during placement to ensure that the correct volume of material is distributed along the beach. Once in place, the construction berm material will be displaced to the deeper portions of the active profile by wave action.

c. Beach fills should be designed with adequate transitions from the artificial beach back to the natural beach. If the transition is too sharp, material will be eroded from the ends of the fill at a rapid rate and could be transported out of the project area.

d. Sediment traps in tidal inlets should be located in areas removed from the concentrated tidal flows. For example, an ideal location for a sediment trap would be in the area of an existing interior shoal that is fed with littoral material moving off the inlet shoulders. The trap should also be

dredged as deep as possible but not deep enough to create problems with sloughing of the adjacent shorelines into the trap.

e. For beach fills, there is a trend of erosion (offshore movement) during storm conditions and accretion (onshore movement) during mild wave conditions.

f. Careful placement of profile lines is required for shorelines that are scalloped (e.g., where sand accumulates at groins). Linear interpolation between survey lines can give a misleading picture of the 3-D beachface.

g. Use of fill material coarser than the native material results in better retention of the beach fill in mild wave climates.

h. Presence of foreign material (e.g., glass fragments) on the beach can bias grain size analysis.

i. Beaches that have winter ice cover may be protected from erosion during the winter storm season.

j. The SPM method for adjusting winds measured over land to a site on the coast was developed for a situation in the Great Lakes. This SPM method should be used with care in areas not similar to the Great Lakes regimen where it was developed. Otherwise, winds at the coast may be over- or under-predicted, and the adjustment will produce information noticeably different from that measured at the site.

k. The use of depth-limited design wave conditions is a good choice.

l. The use of fill material coarser than the native material should be considered in areas where a low wave climate exists and where the coarser material would be acceptable to the users of a recreational beach.

Confined Aquatic Disposal (CAD) Cells

Structures monitored

Boston Harbor, Massachusetts. Monitoring was conducted during the period October 1998 – September 2001 and was composed of three primary activities: (a) water quality monitoring of suspended solids near the operation of two environmentally sensitive clamshell dredges and a normal clamshell to document the benefits of the special clamshell buckets; (b) monitoring contaminated dredged material consolidation and strength prior to and after placing the sand cap; and (c) calculating cap erosion predictions from both tidal currents and ship propeller to characterize the likely amount of cap damage to be expected from either source.

Generic lessons learned

Details are as follows:

a. The GLDD Enclosed clamshell bucket has lower overall turbidity and substantially less turbidity in the middle of the water column than does

the GLDD Conventional bucket or the Cable-Arm™. However, the Enclosed bucket adds additional water to already soft and weak sediments, possibly causing a further reduction of the bearing capacity of the sediments.

b. Natural cohesion and strength of sediments are altered by the dredging process, resulting in sediments in CAD cells that are unstable because of high water content and low shear strength.

c. Excess pore water is released not only through the cap but also is vented through diapir structures that served to breach the caps in discrete areas.

d. Projects should include an evaluation of in situ strength of the material to be capped and porosity and permeability of the CAD cell sediments.

e. Laboratory modeling of subaqueous sand capping processes indicates the sand cap is stable when placed on top of clay material having undrained shear strengths greater than 17 psf (0.8 kPa) and water contents below 100 percent.

f. The volume of sediments resuspended by liquid natural gas carrier tankers from capped and uncapped CAD cells is very small (well less than 1 cu m (1.3 cu yd)) for each vessel passage and settles to the seafloor within 1 hr of resuspension.

14 Structures Monitored and Generic Lessons Learned, Great Lakes

Breakwaters

Structures monitored

Cattaraugus Creek Harbor, New York. Monitoring was conducted during the time period May 1983 – December 1985 to evaluate waves, structure stability, sediment transport, channel stability, and ice-jam problems due to the construction of the project.

Burns Harbor, Indiana. Monitoring was conducted during the period 1985 – 1992 to determine the cause of loss of crest elevation of the breakwater and to evaluate wave conditions in the harbor. This monitoring was also conducted to evaluate the design process and identify the causes of complaints of excessive wave energy by harbor users and frequent necessary maintenance requirements. Under the Periodic Inspections work unit, targets and photo control points were determined during the period November 1994 – July 1995 to establish very precise base level conditions and conduct a broken armor survey. A periodic inspection was conducted during August 1999 to: (a) develop methods using limited land-based surveying, aerial photography, and photogrammetric analysis to assess the long-term stability response of the stone armor layer on the North Breakwater; (b) accurately define armor unit movement above the waterline; (c) determine and define changes that have occurred to the stone armor layer since last monitoring in 1992; (d) establish new baseline data since construction of the reef breakwaters lakeward of the existing North Breakwater; and (e) conduct a broken armor stone survey for comparison with data obtained in 1995. Subsequent monitoring will determine the effectiveness of the new reef breakwater structures relative to damages of the existing breakwater.

Cleveland Harbor, Ohio. The primary objective of the Cleveland Harbor east breakwater rehabilitation monitoring (1980 – 1985) was to determine the stability of a dolos-armored unit cover. This was the first time dolosse were used by the United States in the Great Lakes environment. The monitoring program was also to evaluate the magnitude of armor unit breakage that could compromise the integrity of the structure. Additional objectives were to: (a) determine wave transmission by overtopping; and (b) document the effects of ice on the

stability of dolos units. Under the Periodic Inspections work unit, base conditions were established in 1995 for above-water armor units. Periodic data sets will be obtained to improve knowledge in design, construction, and maintenance of the existing structure as well as proposed future coastal projects.

Generic lessons learned

Details are as follows:

a. Precautions should be undertaken during the design stage to protect structure toes. Based on experience with structures located on erodible material, additional toe protection is a technique that has repeatedly produced a stable toe.

b. A scour hole that appears off a breakwater head is probably the result of local wave effects and increased currents near the head. The reason it may fill later is because of natural bypassing of material around the breakwater as the fillet grows. Lower lake levels will increase this transport around the breakwater. The potential for scour near the head of coastal structures is justification for additional toe protection in the form of small stone or gravel mats to prevent a scour hole from developing.

c. Filter fabric has proven to be very successful for structure sealing. Sediment will be prevented from penetrating the structure and reaching the channel where filter fabric has been used. The use of filter fabric in this manner is very cost-effective in making a structure impermeable as it prevents sediment transport through structures at a relatively small cost. Periodic inspections should note its continued performance.

d. Experience in the Great Lakes justifies the use of 0.9 to 2.0 W stone weight range in design rather than that recommended by the SPM (HQUSACE 1984). The SPM recommends 0.75 to 1.25 W, with 50 percent of the individual stones weighing more than W. Further investigation should be conducted regarding the performance of structures using both these stone weight criteria to identify which one (Great Lakes experience or SPM recommendations) provides the most cost-effective design.

e. Use of toe protection where the lake bottom is susceptible to erosion prevents structure failure and should be used in such cases.

f. Alternatives for the reduction of maintenance of a breakwater are to: (1) add larger stone and/or increase the angle of the slopes; (2) add a concrete cap to the structure to improve stability of the crest; or (3) place a protective structure (reef-type structure well below the water level) in front of the existing breakwater. An economic analysis must be conducted to determine which alternative(s) would result in reduced overall costs.

g. When a submerged reef breakwater is constructed lakeward of the original structure, photogrammetry can be used to establish base conditions for the structure from which the performance of the reef breakwater can be evaluated in future years.

h. Dolos-armored units are very porous when a two-layer thickness is used. Wave energy transmits through the dolosse and reflects back upon them, possibly displacing them out of position. Additional layers over reflective surfaces may be prudent for highly porous armor units.

i. Most dolos breakage occurs along the waterline in the active wave zone. Little continued breakage occurs below the waterline.

j. Aerial photography of dolos cover has proven to be a useful tool. Photos have been used to evaluate qualitative changes in the armor cover. Such photography can serve as the basis for planning maintenance and repair of damage zones.

k. Since dolos breakage can jeopardize the structure's integrity, dolosse should be designed for "no-rocking" criteria to minimize breakage resulting from movement. Consideration also should be given to reinforcement of dolosse in the active wave zone for a deepwater structure, since breakage is concentrated in that area. Dolosse should be placed over a stone underlayer rather than against a flat surface to prevent movement caused by wave reflection.

l. When new breakwater cover concepts are being considered, a physical model investigation incorporated as part of the design would help in selecting the optimum cover unit. Proper design will minimize repair and rehabilitation costs during the life of these projects.

m. Emphasis should be placed on continued improvement of remote sensing methodology, although photogrammetry is an excellent means of mapping armor units above the waterline. With proper rectifying of stereo-pairs, photogrammetry can be used to quantify armor unit movement in the x, y, and z directions at relatively low cost. Controls should be established to place at least three aerial photography targets in each photo frame.

n. Side-scan sonar should be considered during construction as an alternative to extensive and costly diver surveys to inspect underwater placement of the structure. Sonar allows the inspection of large structures rapidly and economically. Annual records also would aid in identifying potential underwater problem areas as they evolve.

o. Photogrammetric analysis of a breakwater or jetty is an excellent tool in mapping the above-water portion of the structure and quantifying changes in elevation. A method using high resolution, stereo-pair aerial photos, a stereoplotter, and Intergraph based software has been developed to analyze the entire above-water armor unit fields and quantify armor unit movement.

p. Detailed broken armor unit walking surveys will result in a data set so well-documented that it can be compared with previous and/or subsequent survey data.

Breakwater Stone

Structures monitored

Great Lakes Breakwater Stone, Illinois, Indiana, and Ohio. Monitoring was conducted during the time period 1995 – 1997 because of concerns for the durability of stone used in the structures and their longevity in the Great Lakes area. It also included geological environmental evaluation of stone sources, and laboratory testing related to a microstructural study in conjunction with aerial photographic surveys. The aerial surveys were conducted to determine movement of stones within the structures, and shift of the structures. Structures at Chicago Harbor, Illinois; Calumet Harbor, Illinois; Burns Harbor, Indiana; and Cleveland Harbor, Ohio were monitored.

Generic lessons learned

Details are as follows:

a. Three internal factors affecting the rate of stone deterioration in the Great Lakes are: (1) depositional facies (environment of deposition influencing rock fabric and composition); (2) diagenesis (degree of interparticle suturing, cementation, and vugular porosity affecting induration and susceptibility to freeze/thaw action); and (3) in situ stress which may cause cracks after removal of confining pressure.

b. Three external factors contributing to stone deterioration are: (1) weathering environment (number of freeze/thaw and wet/dry cycles); (2) high-energy blasting; and (3) mishandling during placement causing cracks.

c. The primary cause of much rubble-mound breakwater stone damage is the result of stone cracking. The loss of a few stones by shattering causes adjacent stones to collapse into the void, resulting in a steepening of structure slope. The problem causes damage at many Great Lakes structures. The majority of cracked stones are located at or above the waterline. There should be further investigation to identify the cause of stone cracking so the problem can be avoided in the future through better material specifications.

d. Cut stone armor used for breakwaters exhibits a wider variance in stability than that associated with typical rubble-mound stone. The result is a highly variable pattern of damage on the structure. The stability of cut stone armor is more sensitive to placement technique than other types of armor. Weathering of cut stone armor results in some breakage, but not a significant amount.

e. The durability performance of cut sedimentary stones is significantly better than blasted sedimentary stones. About 15 percent of cut stones fail at an average age of about 20 years. About 40 percent of blasted stones fail at an average age of about 8 years. Strongly indurated rocks such as unweathered granite and quartzite generally last longer. Data

from quartzite show about 3 percent of stones fail at an average age of about 2 years.

f. An immediate need exists for a study of stone deterioration on other structures around the Great Lakes and other regions of the nation. A long-term evaluation on a national basis will result in significant cost savings and minimize replacement frequency of stone.

g. Laboratory testing of armor stone should be more correlative of field conditions to determine durability of stone in the environment into which it is to be placed.

Jetties

Structure monitored

Cattaraugus Creek Harbor, New York. Monitoring was conducted during the time period May 1983 – December 1985 to evaluate waves, structure stability, sediment transport, channel stability, and ice-jam problems caused by the construction of the project.

Generic lessons learned

The primary cause of much rubble-mound jetty stone damage is the result of stone cracking. The loss of a few stones by shattering causes adjacent stones to collapse into the void, resulting in a steepening of structure slope. The problem causes damage at many Great Lakes structures. The majority of cracked stones are located at or above the waterline. There should be further investigation to identify the cause of stone cracking so the problem can be avoided in the future through better material specifications.

Beach Nourishment and Sediment Transport

Structures monitored

Cattaraugus Creek Harbor, New York. Monitoring was conducted during the time period May 1983 – December 1985 to evaluate waves, structure stability, sediment transport, channel stability, and ice-jam problems caused by the construction of the project.

St. Joseph, Michigan. Jetties constructed at the mouth of the St. Joseph River in 1903 to stabilize the entrance have proven to be responsible for downdrift shoreline erosion. Monitoring was performed during the time period July 1991 – June 1994 to study native beach sediment characteristics and geology at the site and to evaluate the behavior of coarse-grained nourishment material in the project area.

Generic lessons learned

Details are as follows:

a. The designer should be careful when selecting the methodology for evaluating sediment transport. While no technique appears entirely perfect for every situation, some methodologies appear much less sophisticated than others. The selection of a technique must be tempered with experience and specific existing conditions.

b. Localized effects such as wave refraction and diffraction near structures must be considered when performing beach nourishment design. These local effects may compensate for potential sediment losses near structures.

c. Many shoreline regions throughout the Great Lakes exhibit highly irregular sedimentation zonations and wide ranges of sediment size gradation, as opposed to classic sandy beach characteristics found on barrier island ocean coasts.

d. The validity of routine sampling techniques and methodologies used for sandy shorelines is questionable when used in areas where highly irregular zonations and wide sediment gradations exist. To provide a realistic representation of native beach characteristics, sampling techniques should be based on unique sediment characteristics and natural variations in geology.

e. A cohesive sediment substratum plays a dominant role in the change of the shoreline. Where cohesive glacial till is exposed, downcutting is likely to occur during most wave conditions. Unlike unconsolidated sand and gravel which may come and go under different energy regimes, fine-grained cohesive material once eroded cannot reconstitute itself and is removed from the beach system. The profile erosion that occurs during this process is permanent.

f. Supplying downdrift areas with fill from a feeder beach is a complex process consisting of both cross-shore and longshore components. A comprehensive understanding of the amount of material being transported to the downdrift project limits is necessary for designing an effective nourishment program to provide protection to a vulnerable cohesive lake bed.

g. Cohesive shores have very different erosion characteristics than sandy shores, and this has a significant impact on downdrift nourishment requirements. Additionally, there are varying degrees of cohesive shores (related to the extent and role of the overlying sand cover), which also have an important influence on the nourishment requirements.

h. In some cases, sections of cohesive shore on the Great Lakes will feature only a limited sand cover. The underlying glacial till is either only thinly covered or entirely exposed. The till is frequently exposed over the entire profile to conditions of active downcutting. In these situations, it is not clear that the impoundment of sand in an updrift fillet beach and the deprivation of this sand from the downdrift beaches and lake bed will

have any measurable impact on the rate of lake bed downcutting and the associated rate of shoreline recession.

i. Where the sand cover is not limited, reduced sand cover resulting from impoundment at shore-perpendicular structures results in accelerated shoreline recession along the downdrift shore. Beach nourishment is required in those cases, not only to reinstate the historic sediment supply rate, but also to replenish the sand cover to its historic level. The latter requirement may be achieved through augmenting the sand cover volume to its natural level (this may not be practical or realistic because of the large volumes required). Otherwise, the requirement may be relaxed if the effectiveness of the protective characteristics of the overlying sand cover can be augmented. The protectiveness of the sand cover could be improved through the provision of sediment that is coarser than the natural or native sediment. Specific grain size requirements should be determined based on the profile shape, properties of the underlying till, wave exposure, and sediment transport characteristics (both alongshore and cross-shore).

j. A special condition of cohesive shores relates to cases where the natural profile shape is convex instead of concave. This condition is a result of the presence of a more erosion-resistant surface in the nearshore. The protected nearshore shelf may consist of some form of bedrock or glacial till that is armored by a boulder and cobble lag deposit. Shoreline (or bluff) recession on this type of cohesive shore is particularly sensitive to changes in lake level. While downdrift nourishment requirements for this type of cohesive shore may be less in volume (i.e., less than what might be determined based on potential transport rates), the timing and grain size characteristic requirements should be carefully considered.

Inlets

Structure monitored

Cattaraugus Creek Harbor, New York. Monitoring was conducted during the time period May 1983 – December 1985 to evaluate waves, structure stability, sediment transport, channel stability, and ice-jam problems caused by the construction of the project.

Generic lessons learned

Details are as follows:

a. Deposition occurs on the inside of bends in channels in inlets, and scour develops on the outside of those bends.

b. The inability to model lake ice prevents physical model reproduction of flooding when lake ice stops ice flows from exiting inlets by stream.

c. The use of ice-breaking equipment to break up harbor ice in inlets helps prevent flooding.

d. The natural relocation of channels in inlets is evidence that consideration should be given to accommodating natural scour at the outside of bends when designing a channel alignment. Dredging requirements could be significantly reduced.

e. As part of the design process, consideration should be given to whether, and under what conditions, ice-breaking equipment could be used to advantage in inlets.

f. A physical model for evaluating design alternatives at inlets is an excellent tool for identifying the best ways for eliminating shoaling in an inlet navigation channel, preventing ice jams, recognizing the limitations of the state-of-the-art in modeling lake ice, and designing a channel safe for navigation in high wave conditions. Efforts should be continued to improve the capability to model lake ice. This capability would increase the value of physical models where ice conditions must be considered at inlets.

Harbors

Structure monitored

Burns Harbor, Indiana. Monitoring was conducted during the period 1985 – 1992 to determine the cause of loss of crest elevation of the breakwater and to evaluate wave conditions in the harbor. This monitoring was also conducted to evaluate the design process, identify the causes of complaints of excessive wave energy by harbor users, and frequent necessary maintenance requirements. Under the Periodic Inspections work unit, targets and photo control points were determined during the period November 1994 – July 1995 to establish very precise base level conditions and conduct a broken armor survey. A periodic inspection was conducted during August 1999 to: (a) develop methods using limited land-based surveying, aerial photography, and photogrammetric analysis to assess the long-term stability response of the stone armor layer on the North Breakwater; (b) accurately define armor unit movement above the waterline; (c) determine and define changes that have occurred to the stone armor layer since last monitoring in 1992; (d) establish new baseline data since construction of the reef breakwaters lakeward of the existing North Breakwater; and (e) conduct a broken armor stone survey for comparison with data obtained in 1995. Subsequent monitoring will determine the effectiveness of the new reef breakwater structures relative to damages of the existing breakwater.

Generic lessons learned

Details are as follows:

a. Rubble-mound breakwaters with high porosity allow large transmission of incident wave energy through the structures and into harbors. Sealing

of such structures may be required for the harbor facility to function satisfactorily.

b. Design wave and water levels should be accurate for estimating the amount of wave energy that will penetrate into a harbor. Reliable wave hindcasts, supplemented by field wave data, should be used in the analysis.

c. Three-dimensional (3-D) physical models should be constructed with the appropriate scaled transmission coefficients, since not all wave energy propagation into a harbor passes through the entrance channel.

15 Structure Monitored and Generic Lessons Learned, Inland Navigation Sites

Navigation Dam Submersible Gates

Structure monitored

Marseilles Dam, Illinois River, Illinois. Monitoring was conducted during the period June 1999 – June 2001 to determined if: (a) a remote operating system increases the capability of a navigation dam to maintain operation during extreme weather or river conditions; (b) submersible tainter gates are more effective in passing ice than conventional counter weighted tainter gates; and (c) adjustment of submersible tainter gates in freezing conditions is less hazardous, less time-consuming, and more effective and efficient than the old steam method previously used on counter weighted tainter gates,

Generic lessons learned

Details are as follows:

a. Submerged gates during cold low-flow periods with periodic cycling eliminates freezing in the gates and the need for personnel to be onsite.

b. The costs and hazards of chipping ice or thawing gates with steam will be eliminated by submerged gates.

c. At typical winter discharges, submerged gates effectively pass fragmented ice floes and loose brash in the submerged mode without loss of pool or scour damage to the downstream channel. To pass heavy brash, however, it may be necessary to concentrate the flow by opening one or two gates. To draw ice beneath requires an opening of at least 1.5 m (5 ft), and it may be necessary to pull the gate clear of the water, similar to the practice with tainter gates.

d. Videotape analysis to analyze ice passage is very successful. The technique is relatively low cost, logistically simple, and provides a valuable visual record for analysis of the efficiency of gates to pass ice in the submerged mode.

e. A remote operating system is efficient and effective in maintaining strict pool tolerance and improving winter operation of a navigation dam.

16 Guidance from Generic Lessons Learned

The generic lessons learned from the seven distinct geographic navigation regions of the United States (Hawaii and the Pacific Islands, Alaska, Pacific Coast of the U.S. Mainland, Gulf of Mexico, Atlantic Coast of the U.S. Mainland, Great Lakes, and inland navigation sites) were composited by 12 structure feature categories to provide guidance applicable to all regions of U.S. coastlines. The structure feature categories included: (a) breakwaters, (b) floating breakwaters, (c) beach nourishment and sediment transport, (d) jetties, (e) jetty spurs, (f) weir-jetties, (g) inlets, (h) wave transformation, (i) harbors, (j) confined aquatic disposal (CAD) cells, (k) breakwater stone deterioration, and (l) navigation dam submersible gates.

Breakwaters

Remote sensing

Details of remote sensing are:

a. Photogrammetric analysis of a breakwater is an excellent tool for mapping the above-water portion of the structure and quantifying changes in elevation, although emphasis should be placed on continued improvement of remote sensing methodology. Videotape analysis used to obtain wave runup data on a breakwater is very successful, except during periods of low visibility. The technique is relatively low cost, logistically simple, and provides relatively accurate measurements. A method using high resolution, stereo-pair aerial photos, a stereoplotter, and Intergraph based software has been developed to analyze the entire above-water armor unit fields. With proper rectifying of stereopairs, photogrammetry can be used to quantify armor unit movement in the x, y, and z directions at relatively low cost. Controls should be established to place at least three aerial photography targets in each photo frame. Detailed broken armor unit walking surveys will result in a well-documented data set that can be compared with previous and/or subsequent survey data. Accuracy of the photogrammetric analysis techniques has been determined through comparison of ground and aerial survey data on armor units where movement exceeds a threshold value of 0.2 m (0.5 ft).

b. Side-scan sonar should be considered during construction as an alternative to extensive and costly diver surveys to inspect underwater placement of the structure. Side-scan sonar allows the horizontal inspection of large structures rapidly and economically. Annual records also would aid in identifying potential underwater problem areas as they evolve. Side-scan surveys should be performed at extreme high tides to permit complete breakwater coverage and to lessen the risk of tow fish damage. Some side-scan sonar systems incorporate echo-sounders for vertical surveying to determine seafloor bathymetry or navigation channel topography. (e.g., SEABAT).

c. Brass disks should be installed in breakwaters with caps, and their 3-D position should be surveyed as part of a periodic inspection program and after any major earthquake activity. Visual inspections should pay careful attention to hairline cracks in caps, and these should be documented photographically according to a retrievable position identification system.

d. Low-altitude helicopter surveys result in significant improvements in data accuracy and photo image resolution when compared to higher altitude, fixed-wing surveys. Low-level helicopter inspections and 35-mm photography provide a good first indication of levels of armor unit breakage and give a basis for determining if an on-the-ground inspection is needed to gain more precision regarding armor unit breakage that is not captured by the aerial inspection.

Waves

Trends in wave hindcast data obtained outside a harbor to define incident wave conditions correlate reasonably well with runup data on breakwaters in a qualitative sense (i.e., larger wave heights correlate with higher runup, and smaller wave heights correlate with lower runup). The absolute values of hindcast significant wave heights, however, may be substantially different than waves experienced in the prototype based on runup values measured, overtopping observed, and local forecasts. Vessel-generated waves may be the controlling design wave in small bodies of water. Predictions of vessel-generated waves are required, as well as predictions of wind-generated waves.

Stone deterioration

Details of deterioration are:

a. The primary cause of much rubble-mound breakwater stone damage is the result of stone cracking. The loss of a few stones by shattering causes adjacent stones to collapse into the void, resulting in a steepening of structure slope. This problem causes damage at many structures. The majority of cracked stones are located at or above the waterline. The rate of deterioration of breakwater stone is unpredictable because of freeze-thaw and wet-dry cycles, large wave action, and sea ice effects. Structures in such environments should be monitored very closely since the

rate of deterioration can be expected to increase with age. The highest grade of geologically acceptable stone should be placed above the waterline in an extremely harsh arctic environment.

b. Three internal factors affecting the rate of stone deterioration are: (1) depositional facies (environment of deposition influencing rock fabric and composition); (b) diagenesis (degree of interparticle suturing, cementation, and vugular porosity affecting induration and susceptibility to freeze/thaw action); and (c) in situ stress which may cause cracks after removal of confining pressure. Three external factors contributing to stone deterioration are weathering environment (number of freeze-thaw and wet-dry cycles), high-energy blasting, and mishandling during placement causing cracks.

c. An immediate need exists for a study of stone deterioration to identify the cause of stone cracking so the problem can be avoided in the future through better material specifications. Long-term evaluation on a national basis will result in significant cost savings and minimize replacement frequency of stone. Laboratory testing of armor stone needs to be more correlative of field conditions to determine durability of stone in the environment into which it is to be placed.

Cut stone

Details of cut stone are:

a. Cut stone armor used for breakwaters exhibits a wider variance in stability than that associated with typical rubble-mound stone. The result is a highly variable pattern of damage on the structure. The stability of cut stone armor is more sensitive to placement technique than other types of armor. Weathering of cut stone armor results in some breakage, but not a significant amount.

b. The durability performance of cut sedimentary stones is significantly better than blasted sedimentary stones. About 15 percent of cut stones fail at an average age of about 20 years. About 40 percent of blasted stones fail at an average age of about 8 years. Strongly indurated rocks such as unweathered granite and quartzite generally last longer. About 3 percent of quartzite stones fail at an average age of about 2 years.

Scour

Precautions and details of scour are:

a. Precautions should be undertaken during the design stage to protect breakwater structure toes from scouring. Based on experience with structures located on erodible material, additional toe protection is a technique that has repeatedly produced a stable toe.

b. A scour hole that appears off a breakwater head probably results from local wave effects and increased currents near the head. It may fill later

because of natural bypassing of material around the breakwater as the fillet grows. Low-water levels may increase transport around the breakwater. Under small tide and wind-induced current velocities, scour development is unlikely except possibly during a prolonged high-wave event (from standing wave-induced bottom velocities along the outside of baffle breakwater walls and through baffle openings). The potential for scour near the head of coastal structures is justification for additional toe protection in the form of small stone or gravel mats to prevent a scour hole from developing.

Sealing breakwaters

Filter fabric has proven to be very successful for structure sealing. Sediment will be prevented from penetrating the structure and reaching the channel where filter fabric has been used. The use of filter fabric in this manner is very cost-effective in making a structure impermeable as it prevents sediment transport through structures at a relatively small cost. Periodic inspections should note its continued performance.

Breakwater design considerations

Details of such considerations are:

a. Alternatives for the reduction of maintenance of a breakwater are to: (1) add larger stone and/or increase the angle of the slopes; (2) add a concrete cap to the structure to improve stability of the crest; or (3) place a protective structure (reef-type structure well below the water level) in front of the existing breakwater. An economic analysis must be conducted to determine which alternative(s) would result in reduced overall costs.

b. Experience in the Great Lakes justifies the use of 0.9 to 2.0 W stone weight range in design rather than that recommended by the SPM (HQUSACE 1984). The SPM recommends 0.75 to 1.25 W, with 50 percent of the individual stones weighing more than W. Further investigation should be conducted regarding the performance of structures using both these stone weight criteria to identify which one (Great Lakes experience or SPM recommendations) provides the most cost-effective design.

c. Cast concrete breakwater caps may develop hairline shrinkage cracks. While small cracks may not affect structural integrity in warmer climates, expansion of freezing water can cause spalling of concrete in colder climates. When new breakwater cover concepts are being considered, a physical model investigation incorporated as part of the design would help in selecting the optimum cover unit. Proper design will minimize repair and rehabilitation costs during the life of these projects.

Dolosse

Specifics concerning dolosse are as follows:

a. Dolos-armored units are very porous when a two-layer thickness is used. Wave energy transmits through the dolosse and reflects back upon them, possibly displacing them out of position. Dolosse should be placed over a stone underlayer rather than against a flat surface to prevent movement caused by wave reflection. Additional layers over reflective surfaces may be prudent for highly porous armor units.

b. Most dolos breakage occurs along the waterline in the active wave zone. Little continued breakage occurs below the waterline. Since dolos breakage can jeopardize the structure's integrity, dolosse should be designed for "no-rocking" criteria to minimize breakage because of movement. Consideration also should be given to reinforcement of dolosse in the active wave zone for a deepwater structure, since breakage is concentrated in that area. Aerial photography of dolos cover has proven to be a useful tool. Photos have been used to evaluate qualitative changes in the armor cover. Such photography can serve as the basis for planning maintenance and repair of damage zones.

c. Storms that occur during the first postconstruction winter season will produce the largest dolos movements. Reduced movement during subsequent storms indicates that dolosse consolidate and nest into a more stable matrix. Surges in dolos movement, where evident, tend to follow peaks in the wave power record.

d. During nesting, the greatest movement of dolos will occur on the upper slope of a breakwater and in the vicinity of the waterline. Movement on the upper slope will result because dolosse placed there have initial boundary conditions that do not inhibit sliding. After initial nesting, dolos movement will slow significantly but will continue to occur primarily near the waterline as well as on the upper slope. The dominant direction of dolos movement has historically been upslope with slight settling plus rotation about the vertical axis (yaw). Upslope movement (i.e., a wave runup dominated movement) is thought to result, at least in part, when a breakwater has a mild slope.

e. The most significant structural design parameter for large dolosse is static stress. Strain gauges positioned inside instrumented dolosse reveal that static stress loads in some of the units reach levels that leave little residual strength for pulsating wave loads and impact loads. Breakage, while typically associated with some amount of movement, is not necessarily associated with significant movement, and vice versa. For large dolosse (which can have little residual strength), the extent to which movement causes a detrimental shift in boundary conditions appears more important than the absolute magnitude of the movement itself. Subtle movement in the dolos matrix can cause shifts in dolos boundary conditions that, in turn, produce a change in dolos static stress. Field data on dolos movement, static stress, and breakage should continue to be collected in order to better understand the long-term nesting behavior of large dolosse.

Baffle openings

Details of baffle openings are:

a. In designing baffle openings in breakwaters to allow water circulation, natural circulation patterns should be considered. Openings (culverts or gaps) that are aligned parallel to the normal flow will be more effective. Thus, the openings for circulation should be placed in breakwater segments that are angled across the flow patterns.

b. A baffle breakwater will meet its performance criteria with respect to currents if currents through this type breakwater are measurable and exchange takes place. A baffled design should be considered for lower-energy environments where good circulation is critical to acceptability of the proposed structure.

c. Significant rolling of boats (docked so that they are broadside to a baffle breakwater) resulting from the largest wakes suggests that parameters other than wave height may be of interest for wake or wave transmission criteria. The baffled type of breakwater reduces vertical water particle motions and surface disturbances, yet allows enough horizontal motion to pass through the breakwater in the lower part of the water column to cause lateral motion in docked vessels. This emphasizes that for breakwaters in shielded locations, protection against wakes may govern the design more than protection against wind waves.

Tribars

Details of tribars are:

a. A stable tribar breakwater core can be achieved through innovative design of a reinforced concrete pipe rib cage. Because of the interior geometry of such a structure, cylindrical reinforced concrete pipes should be placed on end and backfilled to provide a stable support for the rib cap. This unique design feature, along with a trenched toe for the dolos, will perform well structurally.

b. Periodic photogrammetric surveys may provide a basis for long-term structural assessment of such a project. However, walking inspections of tribar breakwaters reveal higher levels of armor breakage than found by aerial studies. While the level of breakage is still minimal, the area at the confluence of the sea side of the head and trunk of the breakwaters may show a slight concentration or cluster of breakage. Such an area should be monitored more closely than other areas. Also, land-based breakage surveys reveal the accuracy of aerial breakage inspections can be questionable and that, for more accurate armor unit breakage counts, detailed walking inspections should be conducted over the armor unit fields.

Floating Breakwaters

Pontoon floating breakwaters

Details of these breakwaters are:

a. Pontoon floating breakwaters may become a popular fishing platform. Loads for a floating breakwater design, and for the accompanying anchor system design, should include allowance for additional loading because of vessels moored on the seaward side of the breakwater, if such mooring is anticipated. Significant additional loads could be generated if large vessels are moored there. Adequate tieup facilities should be made available on the seaward side of the breakwater. A floating breakwater detached from shore, with no boats moored to the breakwater itself, will become an excellent habitat for sea birds and seals. Collision by a large boat will severely damage floating breakwaters.

b. Considerable effort is required to adjust anchor-line tensions and clump-weight placement to align breakwater units. Interfloat connectors between units should be designed using large cylindrical rubber fenders to minimize destruction by severe storms. A corrosion protection system should be included in any design plan. Holes in pontoon floating breakwaters through which pilings pass should not be large enough to allow a child to fall between the piling and the float. As a temporary solution, plywood rings can be placed over the pilings. During extreme cold weather conditions, waterlines may rupture because of either differential expansion between the floats and polyvinyl-chloride waterlines or the freezing of trapped water. Waterlines on a floating breakwater should be enclosed within the float to avoid freeze damage. Shackles used to connect stud-link chain to breakwater connection flanges should be designed adequately large to withstand storm conditions. Anchor lines should be inspected often and replaced as necessary. Zinc anodes should be attached at various places along new anchor chains to reduce the rate of corrosion. Access and interfloat ramps should be wide enough to allow access to the breakwater by electrically powered vehicles. Such vehicles can be used to reduce travel time to the end of long breakwaters. Stanchions, located on the breakwater to supply electrical service to transient boats, should be low enough to avoid vulnerability to being knocked over by bowsprits of docking boats. Electrical junction boxes should not fill with water; access plates should be carefully sealed. Hardware which provides mechanical support for the electrical wiring should be designed specifically for use in a marine environment.

Scrap tire floating breakwaters

Details of these breakwaters are:

a. Observations of pipe and scrap tire floating breakwaters indicate that unfoamed tires tend to sink. Those same tires later appear to have adequate flotation and were indistinguishable from the foam-filled tires.

Several factors contribute to this apparent contradiction. High tidal currents and resultant drag forces tend to pull the breakwater under and, once submerged, the tires lose their entrapped air. Mild wave climates probably leave trapped air undisturbed for longer periods of time, while large waves at the test sites deform the tires enough to allow loss of some trapped air. Although the tires may still float at approximately the same level as they did originally, their ability to resist being submerged will be considerably less than when originally constructed.

b. After 4 years, tires between pipes will no longer support a person's weight. Without foam, the trapped air compresses as the tires become submerged, resulting in decreased buoyancy. If marginally buoyant tires are submerged deeply enough, they will become negatively buoyant. Therefore, in areas where tidal currents are high or wave heights are greater than about 0.5 m (1.5 ft), including some type of incompressible flotation remains a requirement of conservative design.

Beach Nourishment and Sediment Transport

Design considerations

Several points must be considered concerning proper construction of a beach fill.

a. In computing the volume of material required to construct a beach fill having a specific width, the designer must assume that the improved beach profile will parallel the existing beach profile down to some depth of closure. Once the design volume has been determined, the only practical way to construct the fill is to place the required quantity on the beach in the form of a sacrificial construction berm. The crest elevation of the construction berm should be equal to the natural berm elevation in the area. The width of the construction berm will depend on the slopes that the material assumes during placement and the volume of material to be placed. Since this slope will not generally be known beforehand, surveys should be conducted during placement to ensure that the correct volume of material is distributed along the beach. Once in place, the construction berm material will be displaced to the deeper portions of the active profile by wave action.

b. Beach fills should be designed with adequate transitions from the artificial beach back to the natural beach. If the transition is too sharp, material will be eroded from the ends of the fill at a rapid rate and could be transported out of the project area.

c. Careful placement of profile lines is required for shorelines that are scalloped (e.g., where sand accumulates at groins). Linear interpolation between survey lines can give a misleading picture of the 3-D beachface. Localized effects such as wave refraction and diffraction near structures must be considered when performing beach nourishment design.

d. Beaches that have winter ice cover may be protected from erosion during the winter storm season.

e. Many shoreline regions throughout the Great Lakes exhibit highly irregular sedimentation zonations and wide ranges of sediment size gradation, as opposed to classic sandy beach characteristics found on barrier island ocean coasts. The validity of routine sampling techniques and methodologies used for sandy shorelines is questionable when used in areas where highly irregular zonations and wide sediment gradations exist. To provide a realistic representation of native beach characteristics, sampling techniques should be based on unique sediment characteristics and natural variations in geology.

f. Supplying downdrift areas with fill from a feeder beach is a complex process consisting of both cross-shore and longshore components. A comprehensive understanding of the amount of material being transported to the downdrift project limits is necessary for designing an effective nourishment program to provide protection to a vulnerable shoreline.

Fill material

Use of fill material coarser than the native material results in better retention of the beach fill in mild wave climates. The use of fill material coarser than the native material should be considered in areas where a low wave climate exists, and where the coarser material would be acceptable to the users of a recreational beach. Presence of foreign material (e.g., glass fragments) on the beach can bias grain size analysis.

Waves

Options of methods for choosing design wave conditions are:

a. The SPM method (HQUSACE 1984) for adjusting winds measured over land to a site on the coast was developed for a situation in the Great Lakes. This SPM method should be used with care in areas not similar to the Great Lakes regimen where it was developed. Otherwise, winds at the coast may be over- or under-predicted, and the adjustment will produce information noticeably different from that measured at the site, resulting in unexpected beach processes.

b. The use of depth-limited design wave conditions is a good choice.

Sediment transport

The designer should be careful when selecting the methodology for evaluating sediment transport. While no technique appears entirely perfect for every situation, some methodologies appear much less sophisticated than others. The

selection of a technique must be tempered with experience and specific existing conditions.

a. Numerical simulation modeling has verified the applicability of the Dredging Research Program numerical models for the evaluation of sediment transport, including STFATE, LTFATE, MDFATE, RECWAVE, HPDPRE, HPDSIM, and ADCIRC. Capabilities and limitations of the DRP models have been determined.

b. Predictive techniques for determining sediment transport under both waves and currents have been developed. Three sediment transport methods were adapted to simulate time periods of data collection. The methods for simulating sediment transport by both waves and currents are those of van Rijn, Wikramanayake and Madsen, and Ackers and White. All methods performed reasonably well under most conditions. It was documented that long-term synthetic database of wave and currents could be used to estimate sediment transport.

Cohesive sediments

A cohesive sediment substratum plays a dominant role in the change of the shoreline. Where cohesive glacial till is exposed, downcutting is likely to occur during most wave conditions. Unlike unconsolidated sand and gravel which may come and go under different energy regimes, fine-grained cohesive material once eroded cannot reconstitute itself and is removed from the beach system. The profile erosion that occurs during this process is permanent. Characteristics of cohesive shores are:

a. Cohesive shores have very different erosion characteristics from sandy shores, and this has a significant impact on downdrift nourishment requirements. Additionally, there are varying degrees of cohesive shores (related to the extent and role of the overlying sand cover), which also have an important influence on the nourishment requirements.

b. In some cases, sections of cohesive shore on the Great Lakes will feature only a limited sand cover. The underlying glacial till is either only thinly covered or entirely exposed. The till is frequently exposed over the entire profile to conditions of active downcutting. In these situations, it is not clear that the impoundment of sand in an updrift fillet beach and the deprivation of this sand from the downdrift beaches and lake bed will have any measurable impact on the rate of lake bed downcutting and the associated rate of shoreline recession.

c. Where the sand cover is not limited, reduced sand cover resulting from impoundment at shore-perpendicular structures results in accelerated shoreline recession along the downdrift shore. Beach nourishment is required in those cases, not only to reinstate the historic sediment supply rate but also to replenish the sand cover to its historic level. The latter requirement may be achieved through augmenting the sand cover volume to its natural level (this may not be practical nor realistic, however, because of the large volumes required). Otherwise, the requirement may

be relaxed if the effectiveness of the protective characteristics of the overlying sand cover can be augmented. The protectiveness of the sand cover could be improved through the provision of sediment that is coarser than the natural or native sediment. Specific grain size requirements should be determined based on the profile shape, properties of the underlying till, wave exposure, and sediment transport characteristics (both alongshore and cross-shore).

d. A special condition of cohesive shores relates to cases where the natural profile shape is convex instead of concave. This condition is a result of the presence of a more erosion-resistant surface in the nearshore. The protected nearshore shelf may consist of some form of bedrock or glacial till that is armored by a boulder and cobble lag deposit. Shoreline (or bluff) recession on this type of cohesive shore is particularly sensitive to changes in lake level. While downdrift nourishment requirements for this type of cohesive shore may be less in volume (i.e., less than what might be determined based on potential transport rates), the timing and grain size characteristic requirements should be carefully considered.

Jetties

Scour

A sandy bottom in the vicinity of a breakwater or jetty has the potential to scour during storm events. A moveable-bed modeling effort should be used to determine if scour will lead to armor layer instability. Deterioration and gradual armor displacement occurring during severe storm conditions is most likely not associated with liquefaction of a jetty foundation, but probably results from wave and current action on the structure units.

Surveying

Methods of surveying are discussed:

a. Side-scan sonar is a cost-effective inspection tool for jetties and breakwaters. Analysis of side-scan sonar images, collected as part of a geophysical survey, can be instrumental in determining the underwater configuration of jetty toes and their relationship to the surrounding sandy bottom.

b. Use of photogrammetric mapping of jetties allows a detailed evaluation of the motion of the armor units. This technique is cost-effective and accurate, providing accuracy comparable with standard leveling techniques. Photogrammetric mapping is equally applicable to structures with any type of natural (i.e., stone) or man-made armor (i.e., dolos, CORELOC, etc). The accuracy of photogrammetry is more than adequate to evaluate armor unit movement. Periodic mapping of a coastal structure permits detection of incipient or progressive failure along any visible portion of the structure before such a problem is readily detect-

able by other means. This detection allows for early assessment and possible correction of the problem.

c. Photogrammetry offers several advantages over conventional land surveying techniques. First, it is possible to map armor units at or near the waterline of the structure, units that would be inaccessible or too hazardous to reach on foot. Second, photogrammetry is flexible in that all the information needed to perform the mapping can be obtained almost instantaneously, permanently, and at fixed cost with one aerial photographic flight. The mapping can then be performed at any time thereafter, depending on available resources, need for information, etc. In contrast, land survey methods capable of obtaining location, orientation, and elevation data for mapping every visible armor unit are labor-intensive and requires more time and expense than photogrammetry. It is unlikely that improvements in ground survey techniques will reduce costs enough to challenge the cost-effectiveness of photogrammetry.

Armor stability

Details on armor stability are:

a. Through a semi-quantitative physical model that features a moveable-bed section, it can be determined whether waves alone will cause armor instability. Obliquely approaching waves modified by seaward flowing currents along a jetty and with a hard-bottom reef at a structure tip may cause waves to break directly onto the structure, resulting in extensive damage and ultimately eroding the jetty to an unsatisfactory crest elevation.

b. Current data acquired in a prototype with an Acoustic Doppler Current Profiler in the vicinity of a jetty indicate that, even in very mild wave conditions, the jetty redirects longshore-flowing currents to produce moderate seaward-flowing currents adjacent to the structure. This finding lends credence to the wave/current damage hypothesis that implies the combination of a current in the presence of low waves can induce greater structure stone damage than larger waves alone.

Arrowhead jetties

Jetties at tidal entrances should be constructed parallel to each other and to the navigation channel. Converging or arrowhead jetties often fail to provide for stable entrances and safe navigation.

Currents

Current data acquired in a prototype with an Acoustic Doppler Current Profiler in the vicinity of a jetty indicate that, even in very mild wave conditions, the jetty redirects longshore-flowing currents to produce moderate seaward-flowing currents adjacent to the structure. This finding lends credence to the

wave/current damage hypothesis which implies that the combination of a current in the presence of low waves can induce greater structure stone damage than larger waves alone.

Dolosse

Details concerning dolos/jetty structures are:

a. When dolos/jetty structures experience storms up to a design event, they perform successfully and may not require even the low level of maintenance anticipated by designers. Overall excellent performance of dolosse/jetties and the low percentage of dolosse broken during storms verify dolos design and construction procedures. However, there is a threshold of breakage of a dolos-armored structure beyond which the structure is likely to fail.

b. Dolos-armored units on flatter slopes tend to be forced up the slope by forces associated with wave runup, while those on steeper slopes will be moved downslope by wave rundown.

c. Dolosse benefit from the use of steel reinforcement. Even units that crack remain intact by reinforcement. Reinforcing escalates the cost of casting dolosse, so the decision whether to reinforce the units is still one of cost versus benefits. At present, the largest dolosse are often designed for no impact; however, much of their unreinforced tensile strength is designed for supporting static loads. Smaller units will certainly be displaced and could benefit the most from reinforcement. The decision to reinforce dolos-armored units will continue to be based on engineering judgment until more information is acquired concerning the long-term effects of rust, the benefits associated with units maintaining their integrity even though cracked, and a better understanding of the relationship between impact load, static load, pulsating wave load, and dolos breakage.

d. Periodically (on the order of every 5 years), dolosse/jetties should be photogrammetrically mapped. Such mapping will provide additional useful information on the long-term stability of dolosse. Resurveys indicate that dolos movement is less dynamic during later periods as opposed to earlier survey periods. Movement occurs in an asymptotic fashion as dolosse settle and nest into a stable position on the jetties.

Sealing jetties

The value of sand-tightening jetties is significant. Sand-tightening jetty structures has insignificant effect on the tidal prism.

Jetty Spurs

Details of the data concerning jetty spurs are:

a. Bathymetric data reveal that jetty spurs effectively deflect sediment away from entrance channels. Sediment either circulates back toward shore where it is reintroduced into the littoral system or is carried offshore away from the jetty by a jet of water parallel to the spur.

b. Drogues, dye studies, and aerial photographs used to determine current patterns are not adequate in delineating bottom currents. The Airborne Coastal Current Measurement (ACCM) system can be used to measure and establish bottom current patterns. The system is a very effective method for obtaining bottom currents in hostile wave environments where boat operation is dangerous or where quick mobility is necessary. Current patterns correlate well with depositional patterns identified through bathymetric data. Current patterns and sediment depositional patterns can be predicted and verified by 3-D physical model laboratory experiments of jetty spurs.

c. The Scanning Hydrographic Observational Airborne LIDAR Survey (SHOALS) system (either helicopter or fixed-wing) is effective in measuring seabed bathymetry in hazardous regions where other survey vessels cannot operate safely. Soundings can be taken quickly and are accurate and repeatable.

d. Navigation conditions improve at a spur-jettied entrance, as supported by analysis of shoaling and sediment volume accumulation, and by inspection of bathymetric data. Accumulation of material shifts offshore into deeper water as opposed to moving into the entrance channel. Vessels could then navigate an entrance year-round, barring storm events, and not be confined to periods of high tide.

Weir-jetties

Design considerations

Specifics concerning design considerations are:

a. Most weir-jetty systems are located at inlets that typically have minimal amount of inland-derived sediments. In designing weir-jetty systems at river mouths that carry large sediment loads, both beach and river sediments must be taken into consideration. If the riverborne sediments are expected to pass through the system without creating substantial shoaling problems, care should be taken to situate the impoundment basin so that minimal trapping of the riverborne sediments occurs. This could be done through the use of retaining dykes, by physically separating the basin from the river mouth, or by other creative approaches.

b. A weir-jetty project should be maintained as designed unless long-term or overwhelming evidence indicates that changes are needed. If maintenance practices are frequently adjusted, it is almost impossible to determine how successful the project has performed and what lessons could be learned to improve future projects.

Scour

Scour at jetties can be minimized or eliminated by a number of engineering designs. A spur jetty can be built with extensive toe protection to prevent collapse. Any scour hole near the tip of a spur jetty should be filled and then armored to prevent future scour. While use of concrete and rubble fill may provide temporary relief, an engineered approach employing precisely placed armor units would be more successful. A design using graded-stone layers would also be successful.

Longshore transport

Details are as follows:

a. If the longshore transport rate at a project site is substantially under-estimated during the design of the weir-jetty system, the impoundment basin and entrance channel will shoal substantially more rapidly than expected following construction. The creation of a safe, navigable inlet is the primary purpose of such construction, and shoaling of an inlet mouth will adversely impact navigation.

b. Good reliable estimates of longshore transport rate are needed prior to jetty and impoundment basin design. The current recommended method is to compute the longshore transport rate from at least 2 years of onsite wave data. Failure to do this will lead to uncertainties in anticipated dredging costs and may lead to poor choices in jetty and impoundment basin design.

Impoundment basin

Details are as follows:

a. It is important for the project design to have flexibility to allow for modifications of the size and shape of the impoundment basin based on operational experience. As an impoundment basin fills, it became less efficient at retaining sediments. This occurs because the bottom is subjected to increased wave and current forces as the basin fills.

b. When the principal management problem at a weir-jetty system is caused by inadequate size of the impoundment basin and its inefficiency in retaining sediments, one solution is to increase the frequency of the dredging schedule. This is an effective strategy, but other strategies may be more cost-effective and should be considered. For example, total dredging costs may be decreased if an impoundment basin is enlarged, as the larger basin volume will delay the time required for shoaling to fill the basin.

Inlets

Scouring and shoaling

Details are as follows:

a. Deposition occurs on the inside of bends in channels in inlets, and scour develops on the outside of those bends. The natural relocation of channels in inlets is evidence that consideration should be given to accommodating natural scour at the outside of bends when designing a channel alignment. Dredging requirements could be significantly reduced.

b. Construction of jetties causes establishment of a new equilibrium for the inlet ebb tidal delta system. Bathymetric measurements over shoals and surveys along adjacent shorelines are required over an extended time period to establish the new equilibrium. When an equilibrium state is reached, natural bypassing may resume via the ebb-tidal delta. The volume of the ebb-tidal delta increases rapidly after jetty construction, but the delta increase gradually tapers asymptotically as a state of equilibrium is approached. As the system moves toward equilibrium, more and more sediment will be bypassed.

c. Filling a scour hole at the end of a jetty and covering the area with a layer of armor stone is effective in preventing further scouring. Use of the equation relating critical inlet cross-sectional area and tidal prism is appropriate for inlets that have exhibited historic stability.

Sediment traps

Sediment traps in tidal inlets should be located in areas removed from the concentrated tidal flows. For example, an ideal location for a sediment trap would be in the area of an existing interior shoal that is fed with littoral material moving off the inlet shoulders. The trap should also be dredged as deep as possible, but not deep enough to create problems with sloughing of the adjacent shorelines into the trap.

Ice conditions

Details are as follows:

a. The inability to model lake ice prevents physical model reproduction of flooding when lake ice stops ice flows from exiting inlets by stream.

b. The use of ice-breaking equipment to break up harbor ice in inlets helps prevent flooding. As part of the design process, consideration should be given to whether, and under what conditions, ice-breaking equipment could be used to advantage in inlets.

Physical modeling

A physical model for evaluating design alternatives at inlets is an excellent tool for identifying the best ways for eliminating shoaling in an inlet navigation channel, preventing ice jams, recognizing the limitations of the state of the art in modeling lake ice, and designing a channel safe for navigation in high wave conditions. Efforts should be continued to improve the capability to model lake ice. This capability would increase the value of physical models where ice conditions must be considered at inlets.

Jetty sealing

Details are as follows:

a. Sealing an updrift jetty to prevent passage of sand is effective in preventing shoaling in the inlet. However, sealing a jetty can result in erosion of a shoreline inside that inlet when that shoreline was previously nourished by sand passing through the jetty. Protective measures may be required for a shoreline inside a jettied inlet concurrent with sealing that jetty.

b. Shorelines on the updrift side of a jetty will advance oceanward as a result of the enhanced sand-trapping ability of the rehabilitated jetty. Accretion from the sand tightening will cause initial steepening of the profiles near the jetty. Later, offshore transport of sand occurs with the subsequent flattening of the profiles.

c. Sand-tightening jetties will eliminate the need for some maintenance dredging of the navigation channel. In situations where porous structures contribute to shoaling of a channel, the economics of sand sealing rehabilitation on the structures should be investigated.

Wave Transformation

Submarine canyons

Modeling wave transformation over a variable sea bottom remains a difficult task in most cases. Analytical solutions limit themselves only to simple geometry, and numerical treatments base their predictions on the fundamental assumption of slowly varying sea depth. Modeling difficulty is increased by the presence of deep submarine canyons offshore that affect waves from the predominant direction of attack. STWAVE is presently the numerical model being supported by the Corps; however, STWAVE was not developed for application in complex steep topography and should not be applied in such locations.

Severe wave conditions

A technique for using helicopters to deploy and retrieve oceanographic instrumentation platforms for wave and other data collection under severe wave

conditions was developed. Depending on the length of the desired measurement, the platform can be immediately withdrawn and repositioned, or released and subsequently recovered with the helicopter. This technique is exceedingly useful where safe navigation of a vessel and over-the-side research vessel operations for deploying instruments is not possible under severe wave climates.

Reefs

Details are as follows:

a. At a reef face, wind waves dissipate most of their energy in breaking. Wave energy propagates across reef flats as bores, moving water shoreward that returns seaward through breaks in the reef face. Wave heights on a reef flat do not increase appreciably as wave height offshore increases, but the amplitude of seiche of the entire reef is affected by incident energy. Wave groups (surf beats) with periods near the principal seiche modes of a reef flat may induce harmonic coupling. Peak period on a reef flat bears little resemblance to the incident wave period. Long period waves (100 – 200 sec) dominate the signal.

b. Wind waves propagating shoreward are not the only, and maybe not even the predominant, environmental loading for structures on reef flats. The combination of seiche, return flow from wave setup, and mass transport of bore-like waves can result in large currents running parallel to shore. For structures located on a reef flat, forces from the resulting currents may be of larger magnitude than forces from the wind waves themselves. Forces on structures resulting from currents associated with long waves should be considered as well as wind wave forces.

c. There is no indication that wind waves on a reef flat will exceed the depth-limited breaking criteria (0.78) used for sloping beaches. The height of the highest wind waves on a reef flat, a figure needed in calculating stone stability, will probably not even exceed one-half the water depth as long as the water depths are shallow. However, as the water depth increases because of surge, the breaking wave height limit will increase. Estimates of surge from measurements in models of planar beaches are unlikely to apply to typhoon surge levels. Without verification of a lower breaking limit under typhoon conditions, the standard depth-limited criteria should be retained for design.

d. A detached breakwater design at a reef environment promotes flushing of a harbor but can result in a significant influx of sediment during high-current events. The shortest path (hydraulically) for return flow to take at a reef environment is toward the ends of the reef flat, where breaking and setup are not occurring. Since a harbor is connected to deep water by an entrance channel, the low water level is brought conveniently close (from the return flow’s perspective). If just one-third of the return flow takes this shortcut through the harbor and entrance channel back to sea, velocities across a (for example) l00-m- (330-ft-) wide opening would be on the order of 1 m per sec (3.3 ft per sec). This is sufficient to balance the out-of-phase flow from a seiche, and double the in-phase flow, resulting

in a pulsing flow of up to around 4 knots (2.5 mph). Highest velocities will occur where the gradient is steepest, which will be near the shoreward side of a harbor basin.

Harbors

Data collection

Details are as follows:

a. When monitoring in high-energy wave environments at remote locations, extra precautions should be taken to ensure that wave data can actually be collected. The loss of directional wave gauges outside the harbor would significantly reduce the value of other data obtained. Devices hard-wired to shore to obtain real-time data and/or other appropriate measures to improve the probability of success should be included in project budgets. In-depth research of conditions should be conducted to assure success.

b. When working at remote sites, logistical problems may be a significant factor. In most cases, equipment and supplies required are not available locally and must be shipped from distance sources. Delivery times are uncertain and shipping costs are significantly higher.

c. Failure to obtain adequate incident wave data outside the harbor may have a negative impact on analysis of other data collected during the monitoring effort. Incident wave data are required for correlation with wave data obtained inside a harbor, wave runup, and wave overtopping data to validate design methods and procedures.

Data processing

Modifications to standard open-coast wave data processing and analysis procedures should be considered when monitoring at sites that are subject to simultaneous ocean-generated and locally generated waves. Specifically, analysis should avoid overlapping frequency coverage between surge, ocean-generated (swell), and locally generated waves. Sampling rates (both frequency of gauge polling and frequency of pressure sampling within bursts) should be specifically tailored to the frequency regimes present. Sampling rate considerations and decisions about hard-wired versus self-recording gauge technology should also include examination of how fast wave conditions might change at the site. The directionality of incident wave conditions should definitely be obtained. Wave monitoring should be planned and initiated as early as possible in the design process to allow definition of baseline conditions. Sea-swell significant wave height in the nearshore can be accurately estimated with data from an offshore buoy.

Models

Details are as follows:

a. The numerical model HARBD does well in predicting resonant modes of oscillation measured in prototype harbors. (A physical model will also accurately predict resonant modes occurring in a harbor.) HARBD is consistent with prototype measurements in predicting the shift of the Helmholtz mode and the appearance of additional peaks with inclusion of modifications inside harbors. Numerical model magnitudes of amplification are consistent with prototype amplifications if the numerical model is calibrated to measurements using bottom friction.

b. Numerical model CGWAVE results compare much more favorably than numerical model HARBD results with physical model data within a harbor. This is partly attributable to the fact that CGWAVE is a more comprehensive model and that CGWAVE is expressly configured to match physical model test conditions. CGWAVE also matches inner harbor prototype gauges remarkably well.

c. Numerical model strengths include: (1) ease of model setup and modifications; (2) availability of data throughout the modeled harbor grid which permits visualization of the wave response over the entire gridded region; (3) quick response time; and (4) less cost to run the model.

d. Numerical model limitations include users: (1) performing simulations with unidirectional regular waves without directional spreading effects; (2) neglecting nonlinear effects; and (3) having inadequate reflection coefficients and bottom friction data for accurately calibrating the model.

e. Physical model strengths include the ability to simulate: (1) directional wave spectra; (2) nonlinear wave-wave transformation as waves travel into harbors; (3) reflection, transmission, and overtopping of structures; (4) dissipation resulting from bottom friction within scale and depth limitations; (5) currents; and (6) navigation studies with model ships. Limitations of physical models are mainly the result of cost to construct and modify models and to collect data.

Waves

Details are as follows:

a. Comparison of infragravity significant wave heights measured inside a harbor with those measured at a slope array offshore shows a high correlation between significant wave height inside and outside the harbor. An increase in harbor seiche is associated with an increase in swell energy outside the harbor. Therefore, nonlinear processes that transfer energy from swell waves to infragravity waves outside a harbor are clearly an important mechanism for harbor resonance forcing.

b. High correlation between harbor seiche and sea-swell wave heights rules out free long waves generated from distant sources as an important

forcing mechanism, since such free waves are not necessarily coincident with energetic sea and swell.

c. Long-period modes (resonance) cannot be effectively damped out once a harbor is constructed. A model investigation of resonant modes should be carried out before final project planning to ensure that the constructed harbor does not have unacceptable resonant modes of oscillation.

Wave absorbers

Rubble-mound wave absorbers effectively reduce the wave energy inside a harbor for wind-wave periods of 20 sec or less. This type of wave absorber is less effective in decreasing wave energy for longer waves with periods of 50 sec or greater. Conversely, lack of wave absorbers will increase wave heights at locations inside a harbor by up to 125 percent. Wave absorbers decrease the reflection coefficients up to 50 percent.

Confined Aquatic Disposal (CAD) Cells

Details are as follows:

a. The GLDD Enclosed clamshell bucket has lower overall turbidity and substantially less turbidity in the middle of the water column than does the GLDD Conventional bucket or the Cable-Arm™. However, the GLDD Enclosed bucket adds additional water to already soft and weak sediments, possibly causing a further reduction of the bearing capacity of the sediments.

b. Natural cohesion and strength of sediments are altered by the dredging process, resulting in sediments in CAD cells that are unstable because of high water content and low shear strength.

c. Excess pore water is released not only through the cap but also is vented through diapir structures that served to breach the caps in discrete areas.

d. Projects should include an evaluation of in situ strength of the material to be capped and porosity and permeability of the CAD cell sediments.

e. Laboratory modeling of subaqueous sand capping processes indicates the sand cap is stable when placed on top of clay material having undrained shear strengths greater than 17 psf (0.8 kPa) and water contents below 100 percent.

Breakwater Stone Deterioration

Details are as follows:

a. Three internal factors affecting the rate of stone deterioration are: (1) depositional facies (environment of deposition influencing rock fabric and composition); (2) diagenesis (degree of interparticle suturing,

cementation, and vugular porosity affecting induration and susceptibility to freeze/thaw action); and (3) in situ stress which may cause cracks after removal of confining pressure.

b. Three external factors contributing to stone deterioration are: (1) weathering environment (number of freeze/thaw and wet/dry cycles); (2) high-energy blasting; and (3) mishandling during placement causing cracks.

c. The primary cause of much rubble-mound breakwater stone damage is the result of stone cracking. The loss of a few stones by shattering causes adjacent stones to collapse into the void, resulting in a steepening of structure slope. The majority of cracked stones are located at or above the waterline. There needs to be further investigation to identify the cause of stone cracking so the problem can be avoided in the future through better material specifications.

d. Cut stone armor used for breakwaters exhibits a wider variance in stability than that associated with typical rubble-mound stone. The result is a highly variable pattern of damage on the structure. The stability of cut stone armor is more sensitive to placement technique than other types of armor. Weathering of cut stone armor results in some breakage, but not a significant amount.

e. An immediate need exists for a study of stone deterioration on structures around the nation. A long-term evaluation on a national basis will result in significant cost savings and minimize replacement frequency of stone.

f. Laboratory testing of armor stone needs to be more correlative of field conditions to determine durability of stone in the environment into which it is to be placed.

Navigation Dam Submersible Gates

Details are as follows:

a. Submerged gates during cold low-flow periods with periodic cycling eliminates freezing in the gates and the need for personnel to be onsite.

b. The costs and hazards of chipping ice or thawing gates with steam will be eliminated by submerged gates.

c. At typical winter discharges, submerged gates effectively pass fragmented ice floes and loose brash in the submerged mode without loss of pool or scour damage to the downstream channel. To pass heavy brash, however, it may be necessary to concentrate the flow by opening one or two gates. To draw ice beneath requires an opening of at least 1.5 m (5 ft), and it may be necessary to pull the gate clear of the water, similar to the practice with tainter gates.

d. Videotape analysis to analyze ice passage is very successful. The technique is relatively low cost, logistically simple, and provides a valuable

visual record for analysis of the efficiency of gates to pass ice in the submerged mode.

e. A remote operating system is efficient and effective in maintaining strict pool tolerance, and improving winter operation of a navigation dam.

References

Bass, G. P., Fulford, E. T., Underwood, S. G., and Parson, L. E. (1994). "Rehabilitation of the south jetty--Ocean City, Maryland," Miscellaneous Publication CERC-94-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr. (2003a). "Periodic inspections of concrete-armored coastal structures," ERDC/CHL CHETN-III-66, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

_____. (2003b). "Periodic inspections of coastal structures with stone armor," ERDC/CHL CHETN-III-65, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Bottin, R. R., Jr., and Appleton, W. S. (1997). "Periodic inspection of Humboldt Bay jetties, Eureka, California; Report 1, Base conditions," Technical Report CHL-97-17, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr., and Boc, S. J., Jr. (1996). "Periodic inspection of Nawiliwili Harbor breakwater, Kauai, Hawaii; Report 1: Base conditions," Technical Report CERC-95-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

_____. (1997). "Periodic inspection of Ofu Harbor breakwater, American Samoa; Report 1, Base conditions," Technical Report CHL-97-32, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr., and Eisses, K. J. (1997). "Monitoring of harbor improvements at St. Paul Harbor, St. Paul Island, Alaska," Technical Report CHL-97-13, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr., and Gebert, J. A. (1995). "Periodic inspection of jetties at Manasquan Inlet, New Jersey; Report 1: Armor unit monitoring for period 1984-1994," Technical Report CERC-95-19, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr., and Jeffries, A. C. (2001). "Periodic inspection of St. Paul Harbor breakwater, Alaska," ERDC/CHL TR-01-2, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Bottin, R. R., Marcus, D. W., and Mohr, M. C. (1995). “Periodic inspection of Cleveland Harbor east breakwater, Ohio; Report 1: Base conditions,” Technical Report CERC-95-3, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr., and Matthews, E. W. (1996). “Periodic inspection of Burns Harbor north breakwater, Indiana; Report 1: Base conditions,” Technical Report CERC-96-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr., and Meyers, D. T. (2002a). “Periodic inspection of Nawiliwili Harbor breakwater, Kauai, Hawaii; Armor unit monitoring for period 1995-2001,” ERDC/CHL TR-02-7, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

_____. (2002b). “Periodic inspections of Kahului and Laupahoehoe breakwaters, Hawaii; Armor unit monitoring for period 1992/93-2001,” ERDC/CHL TR-02-11, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

_____. (2003). “Periodic inspection of Ofu Harbor breakwater, American Samoa; Armor unit monitoring for period 1996-2002,” ERDC/CHL TR-03-1, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Bottin, R. R., Jr., and Mize, M. G. (1990). “Redondo Beach King Harbor, California, design for wave protection,” Technical Report CERC-90-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr., and Rothert, W. F. (1999). “Periodic inspection of jetties at Manasquan Inlet, New Jersey; Armor unit monitoring for period 1994-1998,” Technical Report CHL-99-16, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Bottin, R. R., Jr., and Thompson, E. F. (2002). “Comparisons of physical and numerical model wave predictions with prototype data at Morro Bay Harbor entrance, California,” ERDC/CHL CHETN-I-65, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Bottin, R. R., Jr., and Tibbets, M. K. (2000). “Periodic inspection of Burns Harbor north breakwater, Indiana; armor unit monitoring for period 1994-1999,” ERDC/CHL TR-00-12, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Cooper, D. R., Fagerburg, T. L., Waller, T. N., Guy, S. W., and Tuthill, A. M. (2001). “Monitoring of Marseilles Dam submersible gates, Illinois River, Illinois,” Technical Report ERDC TR-01-15, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Gailani, J. Z., Smith, J. W., Kraus, N. C., McGehee, D. D., Hands, E. B., Mayers, C. J., Moritz, H. R., Moritz, H. P., Siipola, M. D., Slocum, D. B., Byrnes, M. R., Li, F., Dibble, T. L., Hollings, W. H., Lund, C. R., Sollitt, C. K., and Standley, D. R. (2003). "Monitoring dredged material disposal at the mouth of the Columbia River, Washington/Oregon, USA," U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Gebert, J. A., and Hemsley, J. M. (1991). "Monitoring of jetty rehabilitation at Manasquan Inlet, New Jersey," Miscellaneous Paper CERC-91-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hales, L. Z. (2001). "Monitoring of Boston Harbor confined aquatic disposal cells," ERDC/CHL TR-01-27, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Headquarters, U.S. Army Corps of Engineers. (1984). *Shore Protection Manual*, Vol II, Washington, DC.

_____. (1986). "Engineering and design: Design of breakwaters and jetties," Engineer Manual EM 1110-2-2904, Washington, DC.

_____. (1997). "Engineering and design: Monitoring completed navigation projects," Engineer Regulation ER 1110-2-8151, Washington, DC.

Hemsley, J. M., Bottin, R. R., Jr., and Mohr, M. C. (1991). "Monitoring of completed breakwaters at Cattaraugus Creek Harbor, New York," Miscellaneous Paper CERC-91-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Herndon, H. D., Andrew, M. E., Hemsley, J. M., and Bottin, R. R., Jr. (1992). "Monitoring of jetty improvements at Umpqua River, Oregon," Miscellaneous Paper CERC-92-1, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Hughes, S. A., Prickett, T. L., Tubman, M. W., and Corson, W. D. (1995). "Monitoring of the Yaquina Bay entrance north jetty at Newport, Oregon; Summary and results," Technical Report CERC-95-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Jarrett, J. T., and Hemsley, J. M. (1988). "Beach fill and sediment trap at Carolina Beach, North Carolina," Technical Report CERC-88-7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

King, D. B., and Prickett, T. L. (1998). "Mouth of the Colorado River, Texas, monitoring program," Technical Report CHL-98-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

LeBlanc, C., and Bottin, R. R., Jr. (1992). "Monitoring of the beach erosion control project at Oakland Beach, Rhode Island," Miscellaneous Paper CERC-92-7, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Lillycrop, L. S., Briggs, M. J., Harkins, G. S., Boc, S. J., and Okihiro, M. S. (1993). “Barbers Point Harbor, Oahu, Hawaii, monitoring study,” Technical Report CERC-93-18, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Lott, J. W. (1991). “Spud Point Marina breakwater, Bodega Bay, Sonoma County, California,” Miscellaneous Paper CERC-91-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

_____. (1994). “Fisherman’s wharf breakwater monitoring study, San Francisco, California,” Miscellaneous Paper CERC-94-8, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Markle, D. G., and Boc, S. J., Jr. (1994). “Periodic inspections of Kahului and Laupahoehoe breakwaters, Hawaii; Report 1: Base conditions,” Technical Report CERC-94-12, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Markle, D. G., Melby, J. A., and Kendall, T. R. (1995). “Periodic inspections of dolosse on Crescent City breakwater, Crescent City, California; Report 1: Monitoring data for period 1989-1993,” Technical Report CERC-95-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McGehee, D. D., and Boc, S., Jr. (1997). “Results of monitoring study of Agat Harbor, Guam,” Technical Report CHL-97-22, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McGehee, D. D., Moritz, H.P., Prickett, T., and Shirley, J. (1997). “Burns Harbor, Indiana, monitoring study: Volume I; Overview of approach and results,” Technical Report CHL-97-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

McGehee, D. D., Prickett, T., and Moritz, H. P. (1999). “Burns Harbor, Indiana, monitoring study; Volume II: Detailed approach and results,” Technical Report CHL-97-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Morang, A. (1992). “A study of geologic and hydraulic processes at East Pass, Destin, Florida,” Technical Report CERC-92-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Nairn, R. B., Zuzek, P., Morang, A., and Parson, L. E. (1997). “Effectiveness of beach nourishment on cohesive shores, St. Joseph, Lake Michigan,” Technical Report CHL-97-15, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Nelson, E. E., and Hemsley, J. M. (1988). “Monitoring completed coastal projects: Operational assessment of floating breakwaters, Puget Sound, Washington,” Miscellaneous Paper CERC-88-6, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Parson, L. E., Morang, A., and Nairn, R. B. (1996). “Geologic effects on behavior of beach fill and shoreline stability for southeast Lake Michigan,” Technical Report CERC-96-10, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Parson, L. E., and Smith, J. B. (1995). “Assessing the native beach characteristics for St. Joseph, Michigan-southeastern Lake Michigan,” Miscellaneous Paper CERC-95-2, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Pollock, C. E. (1995). “Effectiveness of spur jetties at Siuslaw River, Oregon; Report 2: Localized current flow patterns induced by spur jetties, Airborne current measurement system and prototype/physical model correlation,” Technical Report CERC-95-14, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Pollock, C. E., McGehee, D. D., Neihaus, R. W., Jr., Chesser, S. A., and Livingston, C. (1995). “Effectiveness of spur jetties at Siuslaw River, Oregon; Report 1: Prototype monitoring study,” Technical Report CERC-95-14, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Pope, J., Bottin, R. R., Jr., and Rowan, D. (1993). “Monitoring of east breakwater rehabilitation at Cleveland Harbor, Ohio,” Miscellaneous Paper CERC-93-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Rhee, J. P., and Corson, W. D. (1998). “Wave refraction at Redondo Beach, California; Comparison of field measurements with models,” Technical Report CHL-98-16, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Richards, K., Marcus, D. W., Baig, M. M., and Ward, D. L. “Stone degradation on Great Lakes coastal structures,” (technical report in preparation), U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Sabol, M. A. (1996). “Redondo Beach, California, 1992-1994 wave data,” Technical Report CERC-96-4, U.S. Army Engineer Waterways Experiment Station, Vicksburg, MS.

Seabergh, W. C., Cialone, M. A., McCormick, J., Watson, K., and Chasten, M. (2003). “Monitoring Barnegat Inlet, New Jersey, south jetty realignment,” Technical Report ERDC/CHL TR-03-9, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

Thompson, E. F., Bottin, R. R., Jr., and Shak, A. T. (2002). “Monitoring of entrance channel navigation improvements at Morro Bay Harbor, Morro Bay, California,” ERDC/CHL TR-02-18, U.S. Army Engineer Research and Development Center, Vicksburg, MS.

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14. ABSTRACT

The purpose of this report is to provide comprehensive site-specific and generic lessons learned from intensive monitoring of 12 different project features at each of 38 navigation projects located in 16 U.S. Army Corps of Engineers Districts around the continental United States, Alaska, Hawaii, and other Pacific islands. Generic lessons learned from seven geographic regions (Hawaii and the Pacific Islands, Alaska, Pacific coast of the U. S. mainland, Gulf of Mexico, Atlantic coast of the U. S. mainland, the Great Lakes, and inland navigation sites) have been deduced from the site-specific lessons learned for each of these seven geographic regions. From these generic lessons learned after several years of monitoring and/or periodic inspection, data collection, and data analyses at each of the 38 navigation projects, guidance has been developed for planning and design of 12 navigation project features evaluated by the MCNP program to the present time. The 12 navigation project features for which guidance has been developed include: (1) breakwaters, (2) floating breakwaters, (3) beach nourishment and sediment transport, (4) jetties, (5) jetty spurs, (6) weir-jetties, (7) inlets, (8) wave transformation, (9) harbors, (10) confined aquatic disposal (CAD) cells, (11) breakwater stone deterioration, and (12) inland navigation dam submersible gates.

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Beach nourishment
Breakwaters
Confined aquatic disposal cells
Floating breakwaters
Harbors
Inland navigation dams
Inlets
Jetties
Sediment transport
Spur jetties
Stone deterioration
Submersible gates
Wave transformation
Weir-jetties